2023

Manual on Drainage Design

for Highways

Approved by: William Cass Date: January 27, 2023

PROPER DESIGN

Bureau of Highway Design

SPECIALTY SECTION - HYDRAULICS GROUP

HIGH 1 Manual 1

POLICY MEMO FOR HIGHWAY DRAINAGE DESIGN

The Manual on Drainage Design for Highways (HDM) has been developed to provide uniform guidelines and procedures to Department and consultant personnel performing drainage investigations and preparing hydraulic designs for culverts measuring < 10 feet for the New Hampshire Department of Transportation (NHDOT) and local agency projects. The Manual is intended to comply with all State and Federal laws, statutes and regulations, and it presents NHDOT criteria, practices and procedures on the hydraulic design of drainage appurtenances. This manual pertains specifically to highway drainage, and it contains limited content for bridge and large open channel hydraulics. As practical, the user should follow the guidance presented in the Manual.

The scope of the Manual is neither that of a textbook nor is it a substitute for engineering knowledge, experience or judgment. It includes techniques as well as graphs, tables, and appendices not ordinarily found in textbooks. These are intended as aids in the solution of routine field and office problems.

In accordance with NH statutes, a “bridge” is a structure with a clear span greater than or equal to 10-ft. measured along the centerline of the roadway or a combination of culverts constructed to provide drainage for a public highway with: (I.) An overall combined span of 10-ft. or more; and (II.) A distance between culverts of ½ the diameter or less of the smallest culvert. The hydrology and hydraulics for bridge structures shall be in accordance with the NHDOT Bridge Design Manual, Chapter 2 Bridge Hydraulic Study. The federal government and other state DOTs have different definitions for culverts.

This Manual in its entirety is hereby approved and declared effective superseding all previous manuals.

Comments regarding applicable practices, rules and regulations, and/or changes thereof affecting the contents of this manual should be addressed to the **Specialty Section Chief**, New Hampshire Department of Transportation, PO Box 483, Concord, New Hampshire, 03302-0483 (drainmanual@dot.nh.gov). Those wishing to view the contents of the Manual on Drainage Design for Highways may do so at: http://www.nh.gov/dot/org/projectdevelopment/highwaydesign/documents.htm.

Commissioner **Date**
New Hampshire Department of Transportation

REVISION STATEMENT

This Highway Design Drainage Manual is a revision of the NHDOT Manual on Drainage Design for Highways, previously dated: April 1998 with interim revisions dated January 2015. Prior versions of the manual include a 1980s version when the Department changed from the Bureau of Public Roads to the Department of Transportation. The 1970s version updated a similar manual dated 1959. Contact the Specialty Hydraulics Group for information on these earlier manuals. Versions earlier than 1998 do not have exact dates of publication. This revision is necessary to provide updated practices, clarity, and an organized structure for practices, principles and procedures. Drainage practices are always evolving. This guidance manual supplements Chapter 6 of the Highway Design Manual and Bridge Manual Section 2.7, Bridge Hydraulic Study. The intent of this revision is to create a foundation for a "living" document while retaining the same concise format of prior manuals. This edition of the manual contains the following changes:

1.) Updated introduction.
2.) Updated references including: AASHTO, FHWA, HEC/HDS, & State DOTs.
3.) Provided new hyperlinks including NHI videos, software, and manuals.
4.) Glossary updated and moved to the end.
5.) Revised principles and practices guidance.
6.) Revised content in the runoff hydrology Section 3, such as Tc content.
7.) Added information on precipitation records (Atlas 14 & NRCC).
8.) Added & revised hydraulic subject matter in Section 4.
9.) Provided guidance for two dimensional hydraulics, including project scope
10.) Provided guidance on resilient design.
11.) Added discussion for culvert inspection.
12.) Added content for structural condition.
13.) Moved tables to appendices
14.) Added appendices
15.) Removed historical background on the evolution of regression methods.
16.) Removed the dated New England High & Low (NEHL-AWM) method.

The Hydraulics Group of the Specialty Section, in the Bureau of Highway Design, has prepared this guidance document with significant assistance from other NHDOT staff. Users of this manual are encouraged to share comments and suggestions for improvement with the Hydraulics Group through the Specialty Section Chief. Please direct all communication to:

New Hampshire Department of Transportation
Highway Design – **Specialty Section Chief**
7 Hazen Drive
Concord, NH 03302-0483
drainmanual@dot.nh.gov

NHDOT anticipates future revisions to this manual due to technological advances, data densification and improvement, climate trends, flood management, and other reasons.

TABLE OF CONTENTS

Section 3

Section 4

LIST OF FIGURES

LIST OF TABLES

I PURPOSE AND SCOPE

The purpose of highway drainage is to intercept flow coming to a highway facility and transport it as nearly as practical to its natural destination in a safe, practical, economic, environmentally suitable, and reasonably maintainable manner while minimizing detrimental effects to upstream and downstream property owners. All of these components are considered in a time continuum and not merely when the drainage installation is done, but for the design life of the system.

The purpose of the manual is to establish uniform and concise practices and procedures for the New Hampshire Department of Transportation (NHDOT). This manual provides hydrologic and hydraulic (H&H) guidance for most situations encountered in highway design. Drainage design of projects should be completed in conformance with the practices and procedures contained within this manual, including the documentation of any necessary creative discretionary decisions. Exceptions are permissible where existing conditions strongly indicate that an alternative solution deviating from standard practice is acceptable. These alternative solutions must be based on the results of a thorough engineering study. The designer will insure proper review with NHDOT and other professionals for approval of alternative solutions prior to execution. This manual does not function as a legal standard. This manual is published solely for the information and guidance of the designers and other employees of the NHDOT. Drainage infrastructure must be economical and efficient both to construct and to maintain. Particular attention needs to focus on reducing damage to highways and maintaining public safety. Careful evaluation of existing conditions, judicious application of the principles, practices, and procedures described or referenced in this manual will result in designs that are functional and cost effective. The manual is not meant to be an exhaustive reference for all possible scenarios.

The NHDOT Bureau of Materials and Research coordinates and continually reviews drainage problem statements for research consideration. The NCHRP and TRB committees evaluate the merits of problem statements for potential overlaps with other research projects. The Hydraulics Section encourages individuals to present research ideas that will improve future designs. This manual is intended to be a “living” document in that the content will be updated from time to time or transferred to other manuals as appropriate.

Any updates, additions, or Standard Details should be immediately referenced in the “living” manual to avoid conflict and confusion.

II USER INSTRUCTIONS

The Manual on Drainage Design for Highways is to be used in conjunction with other NHDOT design standards such as: NHDOT Highway Design Manual, Standard Specifications, Standard Details, and Highway and Bridge Standard Plans.

The designer is responsible for identifying standards that are appropriate for a particular project or situation, in addition to determining the aspects needed to achieve a proper design.

Users of the manual must keep in mind the legal and ethical obligations of the facility owner concerning hydraulic function. The design should be carefully examined to assure that stormwater quality requirements are met while avoiding significant changes in flow conditions that could be detrimental to upstream or downstream property and water bodies. Concerns or existing issues must be documented in order to clarify facts.

Environmental regulations must be adhered to in design, construction, and maintenance. Documenting the rationale behind a decision is key to providing the necessary understanding of the capacity and limitations of a drainage design.

Interagency Cooperation is an essential element for serving the public interests.

The NHDOT Highway Design Manual provides drainage guidelines in Chapter 6. The Highway Drainage Guidelines published by the American Association of State Highway and Transportation Officials (AASHTO) and the AASHTO Model Drainage Manual will also provide useful guidance. Software tools such as the FHWA sponsored culvert hydraulic analysis program (HY8), the FHWA Hydraulic Toolbox, the FHWA sponsored SMS/SRH2d modelling engine & GUI developed by Aquaveo, the ACOE Hydrologic Engineering Centers River Analysis System (HEC-RAS), and proprietary stormwater modeling programs, such as HydroCAD are available to the NHDOT designer. Geographic Information Systems (GIS) are also available to the designer within the various software and online viewers and by contacting the Bureau of Planning & Community Assistance.

A principle, practice, and procedure format is used for much of the guidance in this manual to help identify conceptual aspects and to provide an organized means for future revision and additions.

III ACKNOWLEDGMENTS

Documentation was obtained from Highway Drainage Manuals from other States. Documentation was also provided from the American Association of State Highway and Transportation Officials (AASHTO), the Federal Highway Administration (FHWA), the U.S. Geological Survey (USGS), the National Oceanic and Atmospheric Administration (NOAA), the Natural Resources Conservation Service (NRCS), and others. The use of references from others does not constitute adherence to each and every aspect of the referenced document. The contribution of material from all sources is gratefully acknowledged; the Department is especially appreciative of the FHWA Hydraulic Engineering Discipline, and affiliates who published the HEC & HDS series guidance documents (see list below). We respectfully acknowledge any content derived from other state DOTs, such as Vermont & Maine. The reference manuals supplement the NHDOT Manual on Drainage Design for Highways in New Hampshire by providing technical guidance that would not be possible with Department resources alone. The intent is to use the latest edition of the referenced documents.

HDS 1 Hydraulics of Bridge Waterways
HDS 2 Highway Hydrology
HDS 3 Design Charts for Open Channel Flow
HDS 4 Introduction to Highway Hydraulics
HDS 5 Hydraulic Design of Highway Culverts
HDS 6 River Engineering for Highway Encroachments
HDS 7 Hydraulic Design of Safe Bridges

HEC 5 Hydraulic Charts for the Selection of Highway Culverts
HEC 9 Debris Control Structures Evaluation & Control Measures
HEC 10 Capacity Charts for the Design of Highway Culverts
HEC 12 Drainage of Highway Pavements (**superseded by HEC 22**)
HEC 13 Hydraulic Design of Improved Inlets for Culverts
HEC 14 Hydraulic Design of Energy Dissipaters for Channels & Culverts
HEC 15 Design of Roadside Channels with Flexible Linings
HEC 16 Addendum to Highways in the River Environment/Hydraulic & Environment…
HEC 17 Highways in the River Environment: Extreme Events, Risk and Resilience
HEC 18 Evaluating Scour at Bridges
HEC 19 Hydrology, circa 1984 (**superseded by HDS 2**)
HEC 20 Stream Stability at Highway Structures 4th Ed.
HEC 21 Design of Bridge Deck Drainage
HEC 22 Urban Drainage Design Manual
HEC 23 Bridge Scour and Stream Instability Countermeasures, Experience, Selection &…
HEC 24 Highway Stormwater Pump Design
HEC 25 Highways in the Coastal Environment
HEC 26 Culvert Design for Aquatic Organism Passage

Refer to the following link for additional FHWA/NHI documents:
http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm

IV AASHTO

The **AASHTO Highway Drainage Guidelines,** circa 2007 & the "Model" **Drainage Manual,** circa 2014, or the most current revisions of these manuals are used by the Department. Volume 1 of the Drainage Manual covers policy, whereas Volume 2 covers procedures. The Department uses the most recent AASHTO manuals for reference. An electronic copy of the individual chapters is available from the Specialty Section - Hydraulics Group.

AASHTO Table of Contents (common to both manuals unless noted otherwise):

Chapter 1	Introduction	
Chapter 2	Legal Aspects (Vol. 1)	Permits and Certifications (Vol. 2)
Chapter 3	Data Collection	
Chapter 4	Documentation	
Chapter 5	Software	
Chapter 6	Planning and Location	
Chapter 7	Surface Water Environment	
Chapter 8	Wetland Creation and Restoration	
Chapter 9	Hydrology	
Chapter 10	Channels	
Chapter 11	Culverts	
Chapter 12	Energy Dissipation	
Chapter 13	Storm Drainage Systems	
Chapter 14	Storage Facilities	
Chapter 15	Pump Stations	
Chapter 16	Stream Stability	
Chapter 17	Bridges	
Chapter 18	Channel and Stream Bank Stabilization	
Chapter 19	Coastal Zone	
Chapter 20	Erosion and Sediment Control	
Chapter 21	Construction	
Chapter 22	Maintenance	

V ACRONYMS

AASHTO: American Association of State Highway and Transportation Officials

BMP: Best Management Practice

FHWA: Federal Highway Administration

GIS: Geographical Information System

HDS: FHWA Hydraulic Design Series

HEC: FHWA Hydraulic Engineering Circular

HEC-RAS: Hydrologic Engineering Centers River Analysis System

HY8: Culvert Hydraulic Analysis Program sponsored by FHWA

NAVD 88: North American Vertical Datum of 1988, an elevation reference to the primary tidal benchmark. This datum replaced NGVD 29. Elevations in this datum are usually less than 1 ft lower than corresponding NGVD 29 values in New Hampshire, units are U.S. Survey Feet.

NGVD 29: National Geodetic Vertical Datum (1929 reference to sea level), units are U.S. Survey Feet.

NGS: National Geodetic Survey

NOAA: National Oceanic and Atmospheric Administration

NRCS: Natural Resources Conservation Service

NRCC: Northeast Regional Climate Center

UNHSC: University of New Hampshire Stormwater Center

Note: *Refer to HEC manuals for additional Acronyms*

VI ENGINEERING SYMBOLS

A	Area
b	Bottom width of channel
C	A coefficient of discharge
CMP	Corrugated metal pipe
cfs	Cubic feet per second
D	Diameter
DMH	Drainage manhole
d	Depth of flow
dc	Critical depth
dn	Normal depth
E	Energy
Fn	Froude number
f	Friction loss coefficient
g	Acceleration due to the force of gravity
h	Head loss
HDPE	High density polyethylene (plastic pipe)
i	Rainfall intensity as used for IDF curves with the Rational Method
IDF	Intensity, Duration, Frequency
k	Conveyance factor in Manning’s formula
L	Length (horizontal)
m	Mass
n	Manning’s roughness coefficient

P	Pressure
Pw	Wetted perimeter
Q	Discharge
qu	Discharge per unit width of channel
Rh	Hydraulic radius
RCP	Reinforced concrete pipe
Rn	Reynolds number
r	Radius
s	Slope in percent or feet per feet
sc	Critical slope
sf	Friction slope
TW	Tailwater
t_c	Time of concentration
V	Velocity
Vol.	Volume
W	Width
w	Unit weight
Z	A height above or below a specific datum

VII UNITS OF MEASUREMENT

All equations, tables, and figures provided in this manual are in English units. Many of the Federal Highway Administration's (FHWA) Hydraulic Engineering Circulars (HEC) and Hydraulic Design Series (HDS) references are available in dual units.

Section 1

Principles, Practices, and Procedures

General

Principle: Principles and Practices are not Policy. Policy comes from a variety of sources within the Department.

Principle: Uncertainty is greater in hydrologic methods than it is with hydraulic design.

> **Practice:** Research efforts to evaluate standard hydrologic engineering methods and reduce uncertainty are supported and in some cases financed by NHDOT.

Principle: Cost is a consideration for all drainage infrastructure.

> **Practice:** When cost is a driver in sizing drainage infrastructure it shall be so identified in the design documents.

> **Practice:** Often times "alternative designs" are developed for wetland permitting when it is not practical to build the road to be hydrologically transparent.

When planning the rehabilitation of an existing drainage system, the condition of the drainage system including any measures for the improvement of the system, will be evaluated. Past design, performance, and construction should be researched. Evaluation and research is also necessary for the construction of a new drainage system. Stakeholders must be consulted where significant changes or construction will occur.

> **Practice:** The design of hydraulic structures and roadside channels will include an analysis of any adverse effects they may have, for the annual exceedance percentage (aka design storm frequency) on upstream and downstream public and private lands or facilities. Some examples of potential impacts are:

- *Contamination of public and private water supplies, ponds, pools, or wells*
- *Increased flows in existing drainage channels that may influence stability*
- *New drainage outlets on public or private property*
- *Disruption of existing public or private surface or subsurface drainage*
- *Increased duration or extent of temporary or permanent ponding on public or private property*
- *Erosion or sediment deposition on public or private property*
- *Increase in flood elevation to existing upstream or downstream structures*
- *Changes to wetland function or type*

Principle: Drainage system planning involves evaluating the performance of the existing stormwater drainage system, and the design of a new or upgraded stormwater system that considers the potential impact of the proposed changes.

> **Practice:** Drainage structures will be designed according to **Table 0.a** design frequencies; individual locations may be subject to less frequent (more stringent) storm event criteria due to environmental rules or permitting.

Table 0.a – Design Frequencies for New Construction

Structure	Interstate Projects	Federal Aid Primary & Secondary and Major State Aid & Urban Highways	Minor State Aid Highways and Betterments
Bridges	Refer to the latest Bridge Design Manual	Refer to the latest Bridge Design Manual	Refer to the latest Bridge Design Manual
Culverts [4]	Min. 50 yr. Check for 100 yr.	Min. 50 yr. Check for 100 yr.	Min. 25 yr. [1] Check for 50 yr.
Storm sewers for depressed sections & underpasses	Check for 50 yr.	Check for 50 yr.	Check for 25 yr.
Curbed Roadway & Roadside Ditches	10 yr.	10 yr.	10 yr.
Storm Sewers [2]	10 yr.	10 yr.	10 yr.
Stormwater Ponds [3]	Refer to DES Stormwater Manuals	Refer to DES Stormwater Manuals	Refer to DES Stormwater Manuals

[1] *Environmental considerations such as: wetland function, permits, and stream crossing rules routinely increase the size of culverts above hydraulic capacity requirements.*

[2] *Whenever a culvert greater than or equal to 15" diameter is introduced into a closed system, the system should be analyzed for the design frequency of a culvert from said pipe inlet to the outlet of the closed system. This will help avoid unexpected collateral impact to downstream pipe capacity.*

[3] *Should be reviewed to ensure that there are no unexpected flood hazards.*

[4] *When a culvert warrants design for AOP (Aquatic Organism Passage) design to low flow in StreamStats or use Regression Equations such as those referenced in HEC 26.*

Practice: The intent of a "check" frequency is to review to ensure that there are no unexpected flood hazards. The use of this terminology for bridge scour is different.

Practice: Use a low flow from regression equations, such as StreamStats, for evaluation of Aquatic Organism Passage (AOP). A good starting point is ~ 1 cfs.

Practice: Use engineering discretion to apply factors, indexes, or percentages to the results obtained using **Table 0.a**. Regional guidance can be found with sources such as: Tools & Resources at the TheIcnet.org, or reports from nhcrhc.org. Nationwide guidance is available through The Center for Environmental Excellence at AASHTO and the FHWA

Office of Planning Environment and Realty. Links are provided below. However, it is understood that these links may need to be updated from time to time.

- https://theicnet.org/?page_id=44 (Tools & Resources)
- https://www.nhcrhc.org/ (NH Coastal Risks and Hazards Commission)
- https://environment.transportation.org/center/rsts/ (AASHTO)
- https://www.fhwa.dot.gov/environment/sustainability/resilience/ (FHWA)

Practice: Downscaling of climate modelling can be effective and might be considered as part of engineering discretion.

Principle: Highway facilities are affected by site development. For site development, the approach in stormwater design is to avoid a net increase in peak flows resulting from development. The reason for this is to avoid damage to existing hydraulic structures, infrastructure, and property.

Practice: No net increase in peak flow on a project is mandatory.

Practice: The outlet velocity of designed drainage structures should be determined and proper precautions must be taken against predictable and preventable downstream erosion and sediment deposition. Energy dissipation analysis using HY8 provides sound guidance for the selection of proven structural techniques based on Froude numbers, scour data and other hydraulic parameters.

Principle: Hydrologic changes can introduce differences in the volume and timing of runoff. The differences in volume often produce a widening of the peak flow hydrographs that can cause flooding for hours or days, or environmental degradation downstream.

Practice: Changes in the volume of runoff are normally addressed as part of the project permitting or in substantial compliance therewith.

The precipitation frequency estimates in NOAA's National Weather Service (NWS) Atlas 14 Volume 10 supersede the estimates for New Hampshire contained in the NWS HYDRO-35, and the Weather Bureau Technical Papers 40 and 49 (all references that were created by NOAA or the earlier National Weather Bureau).

Principle: Atlas 14 published by NOAA is essentially data. The potential effects of climate change are not included in the precipitation data.

Some years ago NHDES adopted the use of the Northeast Regional Climate Center's (NRCC) extreme precipitation estimates for use in substantial compliance with Alteration of Terrain (AoT) permit rules, wetland permits, and Dam studies. In recent years, NHDOT has used the NRCC data, including the Intensity Duration Frequency (IDF) curves for extreme precipitation data products. With the availability of NOAA's National Weather Service (NWS) Atlas 14, designers should continue to use and compare NRCC data products with Atlas 14 data for NHDOT projects. Ultimately, NHDOT will adopt the use of Atlas 14 as the best available data. Until such time, choosing precipitation data products from the NRCC and/or Atlas 14 should be compared and used appropriately. This means documenting the data that was used for the project, and explaining why the data was used. The choice of precipitation data for some projects could contribute to less resilient designs, or unnecessary increases in cost. The two sources of data are:

1) https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html
2) http://precip.eas.cornell.edu/

The New England State DOTs, including NHDOT, helped finance Atlas 14 Volume 10 work which was coordinated by FHWA. The data can be obtained for any geographic point from the NWS Precipitation Frequency Data Server (PFDS) Precipitation frequency estimates have been computed for a range of frequencies and durations. The statistical methods used for Atlas 14 are different than those used by the NRCC at Cornell University. There is also more data included within Atlas 14. Climate trends are built into the NRCC extreme precipitation reports to some degree. Greater detail on the approaches used in each study and the precipitation data deliverables available are available on the respective web sites. Both the NWS and the NRCC have provided a “point and click” application for using the data and methods similar to Thiessen polygons taught in civil engineering curriculums. Search online for the Precipitation Frequency Data Server (PFDS), or use the link above. Tables and charts can be generated using the PFDS data server for any geographic point. Atlas 14 Volume 10 is published under the Documents Section of the NOAA Hydrometeorological Design Studies Center web site. For those seeking more information there are studies referenced by both the NWS & the NRCC, such as the IDF reports by the U.S. Department of Energy Argonne National Laboratory for the Casco Bay Region.

Principle: In general, different statistical methods were used in Atlas 14 than those methods used by the NRCC. Modern statistical methods were used in both studies, however, Atlas 14 used more data and Parameter-elevation Regressions on Independent Slopes Model (PRISM) gridded estimates. NRCC uses Beta distributions to capture trends.

Practice: Precipitation records from the following sources can be used or considered to add confidence in the design of culverts and stormwater systems:

- ✓ The NOAA’s NWS Atlas 14 “point and click” data server application.
- ✓ Data products from the NRCC at Cornell University.
- ✓ Local precipitation gages such as those used for forecasting by the Transportation Management Center TMC (not calibrated to NWS standards).
- ✓ NOAA’S certified data or the volunteer Community Collaborative Rain, Hail & Snow Network program gages (CoCoRaHS).

Practice: Designers should continue to use and compare NRCC data products with Atlas 14 data for projects requiring DES permits or for design that must meet substantial compliance with environmental rules.

Practice: Designers may factor in climate trends for Atlas 14 precipitation data for resilient design.

Practice: Design flow estimates for sizing culverts determined using rainfall-runoff modelling with a unit hydrograph method, such as HydroCAD, shall be compared to estimates obtained from regression equations if the watershed size is valid for each method. The Rational Method is acceptable for sizing drainage systems with surface runoff areas less than 200 acres.

Practice: Design flow estimates taken from FEMA FIS / NFIP studies are sometimes used after careful consideration of changes in the watershed since the study.

Principle: Using modern precipitation data in old methods such as the NRCS TR-20, TR-55, it’s surrogates, or the Rational Method does not automatically make the method better. Precipitation is only one parameter influencing runoff. Watershed characteristics need to be considered.

Impervious Area & Urbanization

Impervious area in small watersheds has been the subject and continues to be the focus of hydrologic research efforts by the Transportation Research Board (TRB), USGS, educational institutions, and non-profit groups. In the 1950s, the Bureau of Public Roads recognized urbanization as a key factor in increased runoff. Indeed, increased runoff was the driver for creating the Rational Method in the 1800s. Many studies have evaluated the environmental impacts of watersheds with increasing impervious area, in part to evaluate if a threshold of ecological degradation could be identified. The Senator George J. Mitchell Center for Environmental and Watershed research at the University of Maine: (Impervious Threshold Research) published **Table 0.a** that points to a range of values that vary mainly due to different ecologic conditions for a given watershed. Across the country there is wide agreement that 10% impervious area is a reasonable average threshold where degradation of stream stability and ecology is evident. Many urban watersheds in NH are close to, or have exceeded 10 % impervious area.

Table 0.a – Impervious Thresholds of Degradation

Researchers	State	PTA Threshold
C. May (1997)	Washington	5-10%
R.D. Klein (1979)	Maryland	10%
E.J. Shaver, G.C. Maxted, D. Carter (1995)	Delaware	8-15%
T.R. Schueler & A. Gali (1992)	Maryland	15%
G.C. Maxted (1996)	Delaware	10-15%
R.C. Jones & C.C. Clark	Virginia	15-25%

(Source: Schueler, T.R. 1994. The Importance of Imperviousness. Watershed Protection Techniques 1 (3): 100-111.)

Principle: The effects of increased impervious area is greater in smaller catchments than it is in larger catchments because the loss of natural hydrologic processes is not buffered to the degree that it is in larger watersheds, in other words: the land cover change in a small runoff area influences the peak flow and the water quality and the ecology more than the same amount of change would for a larger runoff area.

> **Practice:** Estimates of peak runoff may need to be increased in catchments that have foreseeable potential for impervious area to exceed 20% of the total area and are less than 200 acres or at the discretion of the designer. Highway designers comply with permitting to mitigate for the potential increase in impervious area within roadway right-of-way. Whereas, designers for private development evaluate pre and post construction in addition to the performance of drainage conveyance systems with similar objectives as highway designers.

Principle: Water runs off wetlands similar to impervious area when the wetland storage is exceeded.

> **Practice:** The existing percentage of wetland within a catchment will be quantified using available resources and the result will be stored with the project documents. LIDAR is useful for approximate quantification of wetland storage. NH StreamStats also provides a percent of wetlands based on national inventory data (https://fws.gov/wetlands/)

Principle: Urban areas have expanded and in some cases have relied on attenuation (to permit controlled release of stored runoff), either in sizing the original crossing or in later site development upstream or downstream. The designer should carefully evaluate the potential changes that are likely with a significant change in attenuation, storage, or timing of runoff.

Principle: In the early days of stormwater system design the approach was geared toward channeling water to low areas without regard for the benefits of natural phenomena such as: infiltration, evaporation, and transpiration.

> **Practice:** For proposed crossings, impervious area and the potential for downstream flooding and/or backwater at critical stream crossing structures should be studied during preliminary design.

> **Practice:** New construction or modification of existing urban stream crossings shall evaluate culverts upstream and downstream of the project. Simple upsizing of a single culvert in a "system" of culverts could have system wide detrimental effects.

Dams & Highways

Principle: Man-made dams are often an integral part of urbanized and rural areas. Economic and flood control benefits are provided by dams by attenuating flow similar to natural ponds. Dams have a designed stage discharge.

Principle: A dam is classified as such by NHDES either by the culvert rule, the stormwater pond rule, and/or because a structure such as a bridge or culvert controls the water surface to a depth higher than the natural stream channel invert elevation.

> **Practice:** Constructing roads that meet dam classification criteria will be avoided wherever possible.

> **Practice:** Lowering an existing dam hazard class and/or entirely declassifying a dam is a Department goal in any dam reconstruction where it is feasible.

Environmental Hydraulics

Environmental Hydraulics consists of stabilizing, restoring or otherwise altering any portion of a drainage catchment to dually meet infrastructure and environmental objectives. Categories of Environmental Hydraulics include best management practices (BMPs) for water quality improvement, streambed and bank protection measures, fish (and other aquatic organism passage designs), and stream restoration projects.

Stream Restoration: is the re-establishment of the general structure, function, and self-sustaining behavior of the stream system that existed prior to disturbance. Stream restoration is an option as might be a payment to the Aquatic Resource Mitigation (ARM) Fund. *A natural stream forms in such a way as to effectively transport a wide range of flows and sediments through the watershed while maintaining a state of natural equilibrium. When this state of equilibrium is interrupted or altered the stream tends to alter itself for the new condition. The stream does this by redefining its physical characteristics, which can impact adjacent properties requiring additional work and impacts on the stream. Many of these problems can be avoided by designing structures to mimic the natural form of a stream in order to minimize impact.*[1,0 or other Ballestero]
Determining equilibrium and natural stream forms can be difficult in some cases, it is desirable to use uniform procedures with results that can be repeated by various designers in order to accomplish the goals cost effectively. The UNH Stream Crossing Guidelines and HEC 26 should be reviewed prior to undertaking a stream restoration or Aquatic Organism Passage (AOP) crossing. NHI has made an AOP video by Jim Schall.

Stream Bed and Bank Stabilization: *is typically done in order to curb or prevent a stream from incising, redefining an existing channel, to dissipate energy by reducing head, and to protect structures and/or infrastructure at risk.*[10] Armoring a stream bed and banks helps to prevent erosion upstream which can result in adjacent structures being undermined as well as subsequent deposition downstream resulting in flooding. By armoring a stream and preventing erosion this transfers energy downstream where it still has the potential for erosion, but can be managed through the use of an energy dissipating structure or natural means.

Principle: Armoring a stream bank or embankment will likely transfer energy downstream where problems could develop if a natural or manmade energy dissipation is not provided.

Aquatic Organism Passage (AOP):

Methods of fish passage design can be separated into four categories:

- No Impedance – simply spans the entire stream channel and floodplain, generally a bridge is assumed to offer no impedance.
- Geomorphic Simulation – creates fish passage by matching natural channel dimensional characteristics and conditions within the culvert crossing.
- Bed Stability – replicates hydraulic diversity found in the natural channel through the use of natural and oversized substrate.

- Hydraulic design for new construction commonly utilizes embedded culverts that lower velocity and increase internal depth of flow. Retrofits to existing culverts might use roughness elements, such as baffles and weirs, to meet species-specific passage criteria during periods of fish movement. Design flow for AOP is typically on the order of ~ 25% of the 2 yr. event.

- The NHDOT Bureau of Environment coordinates elements of continuous improvement in environmental permitting for hydraulic design with other agencies. The Highway Design Specialty Section (includes hydraulics and applied surface hydrology group) provides guidance and design as appropriate for Betterment, Project Development, and a degree of inter-agency collaboration. Certain documents are uniformly recognized references in New England, however, it is not intended to adopt each and every aspect of the references for highway design or maintenance purposes.

  Practice: Analytic aspects of Environmental Hydraulics such as shear stress simulation can be applied using the HY8 culvert program, or SMS/SRH2d.

For guidance pertaining to stream crossing design, the designer will find a solid foundation in the New Hampshire Stream Crossing Guidelines. Ongoing improvement to the design and assessment of stream crossings is coordinated by the Bureau of Environment (BOE) in collaboration with the New Hampshire Geological Survey (NHGS) at NHDES. NHDOT specific Stream Assessments are available with a Department viewer application. Additional information that may be considered for design is available through the NHDES Aquatic Restoration Web Mapper. The designer can obtain assistance from the BOE with appropriate advance notice. The Specialty Section can provide additional support on technical aspects for specific crossings. Experience is the greatest teacher when designing for aquatic organism passage.

Principle: Stream crossings require designers to balance the environmental objectives with the infrastructure objectives. The goal is to meet both sets of objectives at reasonable cost. Roadway ecology is a term sometimes used to describe the system of transportation assets and ecological services.

Principle: Stream crossings often influence attenuation in a watershed. Therefore, one must carefully evaluate the hydrologic significance of changing impoundment conditions, including the alteration of wetland function. The natural patterns and dimensions of a stream may not be obvious if impoundments (not necessarily caused by roadways) are present in the watershed. i.e., remnants of breached dams, natural sediment deposits, or beaver activity.

> **Practice:** Refer to guidance in the HEC 26 publication, or site specific data for developing design flow for aquatic organism passage.

Web Document References:

UNH Stream Crossing Guidelines

Hydrologic Change

On average, spring comes earlier than it did in the last century. This has led to adjustments in road operations to reduce damage due to frost heaves and heavy truck traffic. In recent years, snow melt has been replaced with a more runoff based hydrology that has shifted the timing of spring flows, with flows that are predicted to be lower in late summer. This is especially true for larger watersheds. The apparent increase in the proportion of very intense storms since 1970 is much larger than models from that period predicted.[10] Pros and Cons of this phenomena are:

- Less winter ice/snow maintenance
- Longer mowing season
- Longer construction season

There are consequences to the existing infrastructure such as: the inability to pass the original design flow at the same design headwater because of increased peak flow, more frequent road overtopping, hanging infrastructure caused by an increasing frequency of extreme daily rainfall and the associated erosion and sediment transport, loss of floodplain utility (as streams incise), FEMA flood mapping now represents something less than 100 year and generally becomes less accurate with the increasing frequency and intensity of storms, stability of streams, and roadside ditches and hills are more susceptible to erosion yielding more sediment. [10]

Also, longer spans of consecutive dry days are predicted along with an increase in the average temperature by 1 or 2 degrees Fahrenheit, and an increase in the number or days above 90 degrees. Warmer runoff means less dissolved oxygen in the water and the potential for higher pollutant Total Mass Daily Loads (TMDLs). [10]

Principle: Prior to recent years, most highway infrastructure was designed, constructed, and maintained on the premise that the overall climate is static.

Principle: Culverts, bridges, and closed pipe stormwater systems are among the transportation infrastructure that is most susceptible to even a low climate change scenario.

Principle: Records of precipitation and stream flow are used by hydraulic and hydrologic practitioners for the design of stormwater infrastructure. Records do not exist for future events, modeled forecasts of climatic trends may be a factor in the engineering judgment of a design.

> **Practice:** The Department uses the NWS and the NRCC precipitation data products.

Principle: Hydrologic change is real and already occurring.

Resilient Design / Adaptation & Preparedness

Drainage infrastructure is only one component of transportation infrastructure assets; however, in recent decades New England States have experienced costly destruction from weather events more frequently than in past decades. Vulnerability and risk assessments have been completed in Washington State using the FHWA template for risk assessment of transportation assets. Other states have similar projects. A strategic management approach using asset data for prioritizing vulnerabilities caused by severe weather can be used for long term changes in site conditions. Rating assets State-wide or by District, low-moderate-critical is another technique that has been used. Categorizing according to the level of potential damage is another approach (Temporary, Operational, or Catastrophic). The Department's Asset Management Section within Bureau 10, is part of a strategic management plan for performance, inventories, data, and systems for transportation drainage infrastructure.

Principle: Existing transportation infrastructure is aging and in many cases was not designed to handle the severe weather occurring at a higher frequency in the last few decades. Most of the hydraulic failures have been to structures that were already in a deteriorating state.

Principle: The objective is not to be caught off guard but not to over design either.

Principle: Build resilience to the extent practical.

> **Practice:** Armor the downstream embankment slope of roads that have an overtopping risk identified by maintenance staff. LIDAR can provide the approximate weir geometry for predicting road overtopping.
>
> **Practice:** Provide appropriate underdrain systems in wash out areas that have failed because of, or partly due to, soil conditions.
>
> **Practice:** Provide more than the absolute minimum requirements in pipe capacity, channel/outlet/inlet protection, freeboard etc. to accommodate greater than anticipated events, and/or the potential for debris. For example, a surge pipe, or larger culvert, may be designed to offset the potential for higher peak flow from increased impervious land cover or climate predications.
>
> **Practice:** Continue the design technique of reducing concentrated flow where practicable in lieu of closed stormwater systems. This design strategy reduces concentrated flows by handling runoff without closed stormwater systems wherever practical.
>
> **Practice:** Maintain up to date information related to climate science.

Strength Requirements

The two basic functions of buried pipe are hydraulic and structural capacity.[4] Reference manuals for the designer when evaluating or specifying strength aspects of drainage facilities include Section 12 of the AASHTO LRFD Bridge Design Specifications *Buried Structures and Tunnel Liners*, the most current AASHTO LRFD *Bridge Construction Specifications*, and the FHWA publication No. FHWA Hl-98-032 for NHI course No. 132068, or any future versions. The method of installation and bedding during construction is an important factor in the subsequent performance of culverts. Refer to Section 603 of the NHDOT Construction Manual.

"With regard to construction practices and control, both good bedding and good backfilling adjacent to the pipe are important to the performance of both flexible and rigid pipes; however, these requirements become more significant as the pipe diameter and height of fill increase to the point where structural considerations outweigh handling and durability considerations." [33]

Principle: Structural design and analysis of culverts is predominately either rigid or flexible in relation to the composite materials and soil interaction problems.[33]

Principle: New culvert products are providing pipe material technology employing a flexible structural shell with a rigid liner to resist abrasion and corrosion. The differentiation between flexible and rigid culverts is becoming blurred.[33]

Principle: Rigid culvert designs are becoming more dependent upon soil support than designs based solely upon the rigid body three-edge bearing model.[33]

> **Practice:** The project personnel need to check the condition of existing pipes. However, the actual condition of a pipe may not be completely evident until excavation occurs.
>
> **Practice:** The minimum cover is taken from the bottom of pavement because only flexible pavement is use by NHDOT.

Table 0.a – Minimum Pipe Cover Requirements

Application	Cover Depth (feet unless otherwise specified)
CLOSED SYSTEM	
Under pavement	4 or Standard Spec. (SS)
Other locations	2 or AASHTO Ref.*
CULVERTS	
RCP, under pavement	4 or SS
RCP, under grass	2 or SS
RCP, driveways	1
CMP, under pavement	4 or SS
CMP, under grass	2 or SS
CMP, under driveways	1
Thermoplastic, under pavement	SS or ID/2 ≥ 24” (AASHTO)
Thermoplastic, under grass	SS or ID/8 ≥ 12” (AASHTO)
Steel Reinforced Thermoplastic	SS or S/5 ≥ 12” (AASHTO)

* AASHTO Ref. is typically less stringent, see AASHTO LRFD Bridge Design Specifications, Section 12, Buried Structures & Tunnel Liners

S = pipe diameter (inches)

ID = inside diameter (inches)

Table 0.b – Minimum Spacing Requirements for Round Pipes

Diameter, D (ft.)	Minimum Distance Between Pipes (ft.)
< 2.0	1
2.0 – 6.0	D/2
> 6.0	3

***Note**: This minimum distance is for sufficient haunch compaction, bedding & backfill.*

Utilities and Drainage Systems

Principle: Underground utilities must be considered when setting location and grade of drainage structures.

Practice: Alteration of utilities must be done in accordance with the NHDOT Utility Accommodation Manual (UAM).[1]

Page intentionally left blank

Section 2

Documentation & Pre-Design

Documentation

Principle: Adequate documentation of the design of any highway hydraulic system is essential for:

- Public safety, i.e. minimizing road overtopping, stable embankment slopes, etc.
- Justification and transparency of public fund expenditure & providing public info
- Future reference by designers when needed for rehabilitation, improvement, or assessment of performance.
- Reference by others such as developers, property owners, municipal agents, etc. who may be interested in the capacities of the drainage facility.
- Compliance with environmental rules and permits.
- Preparation of a defense in the event of litigation, and
- Information that aids the designer in discretionary design functions.

Providing good documentation details the design procedures and shows how the final design and decisions were determined. Most documents should be electronic.

> **Practice:** The compilation and preservation of pertinent hydrologic and hydraulic documentation shall include water subcatchment maps, field survey information, photographs, engineering calculations, design correspondence and flood history including narratives from newspapers and individuals, such as: District highway maintenance personal and local residents. Municipal Hazard Mitigation plans should be scanned for identified flooding hazards. The amount of documentation for each design shall be commensurate with the risk and the importance of the facility.

> **Practice:** Drainage study files are saved in the project directory. These files include any supplemental narratives, models, spreadsheets, or outputs that support the hydrologic design estimates or hydraulic design.

> **Practice:** Depending on the complexity and expertise needed, Peer review from a member of the design team or from another Section, or Bureau such as the Specialty Section, Bridge Design, or Materials and Research is encouraged when needed.

Generally, if problems are evident, more documentation is required. The designer will likely discover aspects of a watershed or drainage system that were not previously known, and the only way for the learned knowledge to be passed on is through documentation. The designer determines the necessary level of documentation. Electronic files have replaced paper copies for new projects. Documentation of built designs for archiving should only include the data for those designs, not alternatives, unless these were developed for legal considerations. Check in with the Specialty Section for guidance or examples of standard reports or forms

Pre-Design Evaluation and Data Collection

The extent of the drainage analysis and design should be established before undertaking a detailed investigation, evaluation may include:

- Condition of the transportation drainage asset, such as, inlets, outlets & inverts.
- Whether or not there is a flooding problem with the existing system.
- Adequacy of existing system.
- Visual or historic evidence of plugging with debris or sediment.
- Evidence that indicates the existing storm drain system may be inadequate.

In addition, environmental or flood hazard priorities may have been identified previously by scientific study or zoning of a watershed, such as Stream Crossing or Watershed Assessments, local Watershed Overlay Zones, or Hazard Mitigation Plans. This information should be considered as part of the design process when appropriate.

Information available to the designer includes, but is not limited to, the following:

- Condition and expected life of existing culvert (estimate).
- Reports by other agencies.
- LIDAR terrain data sets, such as those available at http://lidar.unh.edu/map/
- USGS reports concerning floods, and storms of record.
- Photogrammetry (Aerial photography and associated digital features);
- Historic USGS topographic maps.
- FEMA Floodplain maps, floodways determined by detailed methods, and supporting "background data" from the FEMA Engineering Library, if available. (Detailed hydrologic and hydraulic background information from the FEMA library typical costs a few hundred dollars). Free info. is available from the Map Service Center (https://msc.fema.gov/portal/home). The modeling is only available by requesting background data at this time.
- Old project plans are usually filed in the Highway Design Records Section, Bridge Design or in Planning and Community Assistance. District Offices may have plans that are not available in Concord. Many legacy projects are scanned and stored electronically and GIS applications provide project numbers.
- Soil maps are available online and in CAD format. This information may be obtained directly from sources such as: NH Granit and the NRCS Soils web soil survey link Web Soil Survey (.shp files can be used to define the area of interest). Soils maps in Microstation format are available on the Department intranet.
- Watershed boundaries for the state are shown on a .pdf published by NHDES; https://www.des.nh.gov/organization/divisions/water/wmb/documents/nh-watersheds.pdf
- High water elevations and evidence of scour, erosion, and corrosion.

- Flooding problems and history obtained through personal interviews with local residents, state and municipal highway department personnel, and public record information.
- Estimated velocity and depth of flow in channels for conditions observed (record in a note to file, or comment in a model calibration, the weather, season, and other pertinent information).
- Description of channel upstream & downstream of structure including condition, bottom, side slope characteristics, profile, typical section, and debris marks and /or depth of water if encountered (typical peak high water is above debris line).
- Hydraulic controls, such as pond elevations, dams, confining embankments.
- Review existing contours to determine drainage flow paths and erosion areas.
- Coarse screening type 2d hydraulic models have been found useful for identifying potential areas of instability.
- Identify seasonal flooded areas, wetlands, and evidence of high ground water.
- Identify visible evidence (paint or other) of existing underground utilities.
- Overhead wires (typically proposed work needs to be at least 10 ft. distant).
- Identify septic systems and wells to the extent possible and
- Check with District(s) for history of drainage problems in the area.

2.0.1 Initial Field Review

The designer should evaluate the following list of drainage characteristics during the field review of the project and determine how much additional field information is needed: Accuracy of watershed delineations, type(s) of ground cover, condition of existing culverts and/or closed drainage system, water elevation marks or stains at existing drainage structures; description of culvert structural condition, any significant corrosion, the material type, diameter or size (should be checked with asset management data).

2.0.2 Culvert Inspection

In 2020, AASHTO published the 1st edition of the Culvert & Storm Drain System Inspection Guide. It is intended to update the FHWA Culvert Inspection Manual published in 1986. The FHWA guide was a supplement to the Bridge Inspector's Training Manual 70. Designers rely on the guide and staff experience for culvert inspection. The NHDOT Culvert Management Committee responsibilities are continuous, whereas, Task Forces are created from time to time for temporary purposes.

Practice: Data dictionaries are kept lean to remain effective for priority metrics.

Principle(s): The 2020 AASHTO guide along with other Department systems and protocols are intended to address the need to inventory, quantify, and rate the condition of in-service culverts and to update the inspection and rating criteria. The need for standardized inspections, structural integrity, hydraulic performance, and roadside compatibility are recognized as common criteria across the Department Bureaus.

> **Practice:** Highway Designers will request expertise from the Existing Bridge Section staff and/or Bridge Maintenance for culvert inspections from time to time.

For typical installations, it is usually convenient to begin the field inspection with general observations of the overall condition of the structure and inspection of the approach roadway. The following sequence is applicable to all culvert inspections:

- Review available information and safety concerns
- Observe overall condition of the site
- Inspect approach roadway and embankment
- Inspect waterway
- Inspect end treatments
- Inspect barrel
- Identify any culverts with debris passage or sediment transport impediment

General observations of the condition of the culvert or storm drain site should be made while approaching the area. The purpose of these initial observations is to familiarize the inspector with the structure. They may also point out a need to modify the inspection sequence or indicate areas requiring special attention. The inspector should also be alert for changes in the drainage area that might affect runoff characteristics. It is recognized that accessibility, confined space issues, and traffic control can present serious challenges to culvert inspections.

Most defects are first detected by visual inspection. If possible, a close-up, hands-on inspection is preferred. The types of defects to look for when inspecting the barrel will depend upon the type of structure being inspected. In general, barrels should be inspected for cross-sectional shape and barrel defects such as joint defects, seam defects, plate buckling, lateral shifting, differential movement, missing or loose bolts, corrosion, excessive abrasion, material defects, and localized construction damage.

Drainage Aspects of Project Development

Principle: Drainage requirements will likely be design constraints for most projects and, the Highway Design Manual provides guidance on the project development process. The purpose of this section is to identify the areas where drainage influences project development and to aid the hydraulic designer in coordinating with the appropriate Bureaus and Agencies.

Permits: Coordinate with the Bureau of Environment (BOE) Refer to Chapter 6 of the Highway Design Manual [2] and Guidance provided by the BOE for a general overview of the various permits and authorizing agencies. Necessary permits are typically one or more of the following:

NHDES:

- Wetlands (riparian, coastal Tier 4 crossings & project specific).
- Shoreland Protection.
- Substantial compliance to Alteration of Terrain (AoT) rules.
- Stream Assessments & Stream Crossings including the associated data.
- NHDES Dam Permits for NHDOT owned dams typically operated by District staff.

EPA:

- EPA, National Pollutant Discharge Elimination System (NPDES) & Storm Water Pollution Prevention Plan (SWPPP).
- Water Quality Certificates (Projects that have a federal permit).

MS4 compliance which are conveyance or system of conveyances that are:

- Owned by a state, city, town, village, or other public entity that discharges to waters of the U.S.
- Designed or used to collect or convey stormwater (including storm drains, pipes, ditches, etc.).
- Not a combined sewer; and not part of a Publicly Owned Treatment Works (sewage treatment plant).

ACOE:

- U.S. Army Corps of Engineers, State Program General Permit (SPGP)

In areas within jurisdiction of the Coastal Zone of NHDES, additional regulations apply to tidal stream crossings (Tier 4) wetland permits. The United States Coast Guard also has regulatory requirements in navigable waters. For projects of specific scope and/or characteristics, the US Army Corps of Engineers may require an Individual Permit.

Projects that have a federal permit may also require a Water Quality Certification, issued by the New Hampshire Department of Environmental Services (NHDES), in fulfillment of Section 401 of the U.S. Clean Water Act, and the State of New Hampshire's Surface Water Quality Regulations (Env-Wq 1700). The conditions included in a Water Quality Certification require Inspection and Maintenance, and Turbidity Sampling and Analysis Plans that are an important part of construction.

Department coordination:

Green sheets are used to alert the BOE of a project and the need for environmental review, however, many projects are initiated in Highway Design and designers should be aware of the need to process these requests:

- Green Sheet (no longer color coded) – Typically used by Final Design

The Bureau of Bridge Design, Bureau of Highway Design, and the Operations Division share the responsibility of drainage design. In general, Highway Design is responsible for culverts less than 10 ft. open span, and Bridge Design is responsible for structures over 10 ft. Refer to the Chapter 6 of the Highway Design Manual for further details.

Inter-Agency coordination: Refer to the Highway Design Manual.

Principle(s): Drainage construction frequently impacts other project aspects, such as; wetland impacts, utility relocation, constructability, maintenance of traffic, right of way requirements, cost, etc. Drainage construction can also cause the need to comply with multiple and varying environmental regulations. Early consideration in the design process is essential to promote efficiency regarding both temporary and permanent impacts. The earlier this is done the less risk of requiring rework, schedule delay, and/or costlier design. Inter-Agency collaboration is integral with serving the public interest.

Drainage Reviews for Driveways and Other Accesses

Refer to the document titled POLICY FOR THE PERMITTING OF DRIVEWAYS AND OTHER ACCESSES TO THE STATE HIGHWAY SYSTEM latest version published by the NHDOT Bureau of Highway Maintenance. The information contained here is not intended to supersede the policy document. Driveway permits will often follow or be concurrent with site development applications reviewed by NHDES, sometimes land disturbance area is below the threshold requiring a permit from NHDES. The following information shall be required before initiating drainage reviews for new or reconstructed access to highways:

- Location of relevant culverts by survey measurement including inverts, lengths, culvert size, and type of material.
- Digital photos of inlet and outlet conditions.
- Depth of cover over the culvert.
- Analysis of culvert capacity and low point where overtopping may occur.
- Notification of debris, freeboard, or erosion issues, if any.
- Electronic copies of any approved for use models, such as HydroCAD or HY8.
- Identification of surface flows that cross proposed or existing access points.
- Runoff concerns related to sloped driveways.
- Identification of tailwater constraints.
- Calculated capacity of stormwater systems that will receive runoff from the site.

Principle: Driveway Access Permits accompanied with the appropriate supporting information will allow for a timely review. Applicants can avoid unnecessary engineering costs associated with concept designs by providing the basic information required to complete a review, such as the pre & post-performance of a highway culvert in the Right of Way if needed.

Practice: No net increase of peak flow will be allowed into the existing roadway drainage system and substantial conformance with State and Local stormwater rules is required. An increase in peak flow will not be conveyed through the roadway drainage system to a downstream property.

Practice: An increase in the volume or timing of runoff to downstream properties is not allowed by approval of a permit from NHDOT.

Practice: Requiring models, such a HydroCAD, StormCAD, SMS, HEC RAS or HY8 should be submitted for review for timely reviews rather than just the results from print-outs in .pdf format. Making model submittals part of the contract scope, or even Department policy will avoid the extra time in requesting them. From time to time a consultant may use modelling software not available to Department engineers. Typically, these types of studies will involve more detailed presentations.

Bureau of Environment Support and Guidance

Environmental permit applications required for NHDOT projects are prepared by Highway Design or District Operations Offices and reviewed by the BOE prior to submittal to the regulatory agency. The designer is responsible for ensuring that the permit conditions are incorporated into a project. The BOE does not normally provide design work for a project. The BOE does publish example permits, BMP manuals and other guidance on the Bureau's website.

Page intentionally left blank

Section 3

Surface Hydrology

"*To be able to examine a topographic map, perhaps walk over the ground, and then predict the flood regimen of an ungaged drainage area - that is at once the grand dream and the despair of hydrologists*."

Don Johnstone & William P. Cross [18]
Co-authors, Elements of Applied Hydrology, 1949

Analysis Methods for Applied Surface Hydrology

Uncertainty exists in varying degrees in every aspect of hydrology. In general, hydrologic design is less certain than hydraulic design. For the purpose of this manual, a hydrologic study is an approximation of the complex relationship between precipitation that falls on the drainage basin and the surface water that runs off the basin. Approx. 96.5 % of the earth's water is in the oceans. Of the remainder, 1.7 % is in the polar ice, 1.7 % in the groundwater, and 0.1 % in the surface and atmospheric water systems. The driving component of surface water hydrology is a small portion of all the earth's water. Although the freshwater system is relatively small at any given moment, immense quantities of water pass through the atmospheric and groundwater systems on an annual basis. [USGS] The hydrologic cycle is depicted in Figure 0.a. The designer applies surface hydrology with a collection of empirical and statistical procedures and methods for transportation drainage infrastructure.

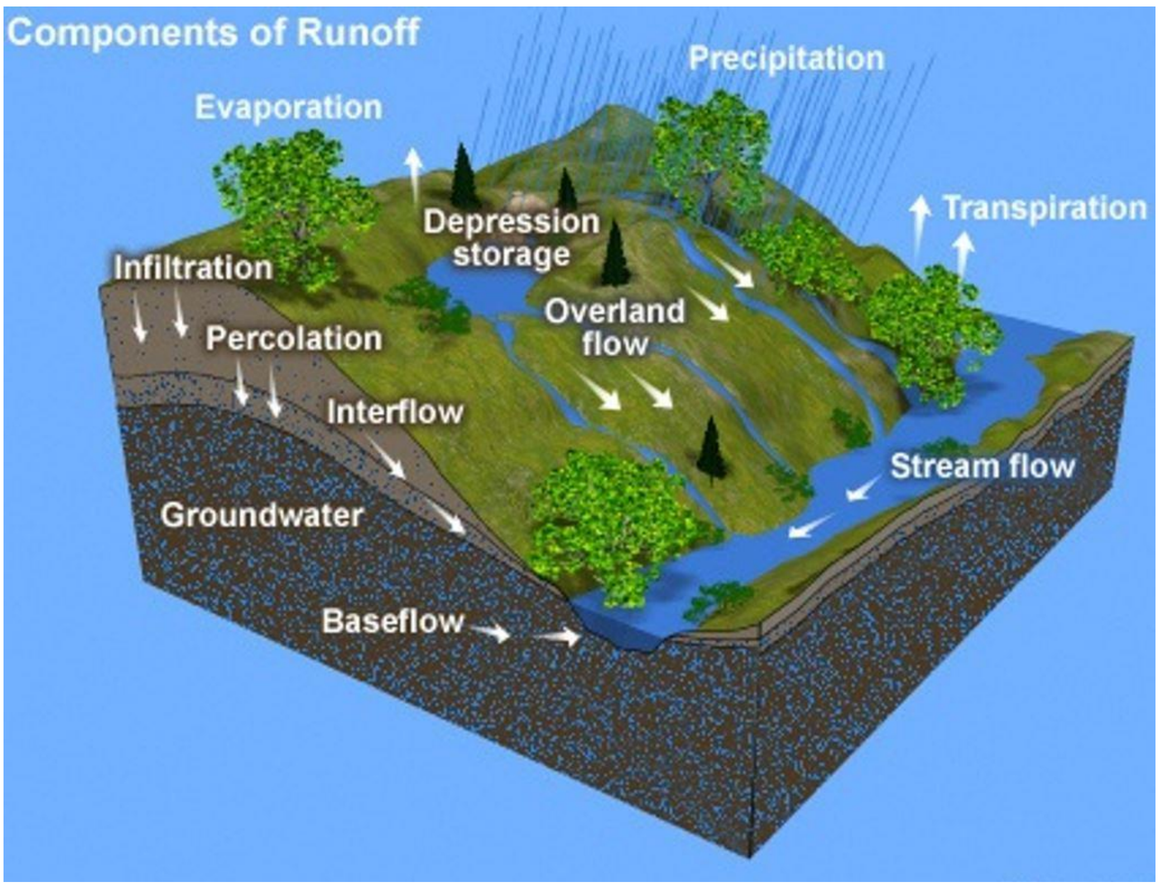


Figure 0.a Components of surface runoff courtesy of the Comet Program

The highway project hydrologic components will emerge after evaluating basin characteristics, obtaining rainfall data, and utilizing acceptable methodologies. Drainage design starts with runoff that considers the watershed characteristics, including amount of storage and percent of wetlands, impervious areas, average basin slope, primary channel slope, land use, and soils. The purpose of this section is to provide a reference for common methods used for runoff analysis by the Department. This manual is not necessarily the only reference that may be used. In particular, the FHWA HDS 2 publication, available internally and from the documents section of the FHWA Resource Center website, should be consulted for more scientific material.

Principle: Runoff is typically defined by three characteristics (1) peak rate (2) total volume (3) and distribution over time.

Principle: The peak flow, pertinent environmental aspects, and in some cases the volume and timing of runoff will be used in the hydraulic analysis for determining the required size of highway drainage facilities.

Principle: The changes in land use and design of stormwater mitigation for no-net-increase peak flows have increased the "peak width" (see glossary) runoff and increased the volume of water that moves further downstream in many developed watersheds. This is a major factor in the more frequent flooding in the lower elevations of watersheds in recent years.

The calculations used to determine peak flow will not necessarily be the same calculations used for pollutant loading or water quality evaluation. Refer to the NHDOT Highway Design Manual and the NHDES Stormwater Manuals for BMP sizing and digital worksheets. Hydraulic improvements to a watershed should start at the low point and move upstream; however, this approach is not always economical and/or technically practical.

Hydrologic methods can be termed deterministic or statistical (aka stochastic, see glossary).

Principle: Deterministic methods often require a large amount of judgment and experience to be used effectively and the accuracy is hard to quantify.[21] These methods depend heavily on the approach used, and it is not uncommon for two different designers to arrive at different estimates of runoff for the same watershed. Unit hydrograph methods such as the NRCS TR-20/TR-55 curve number method and the Corps of Engineers HEC-HMS program and other hydrograph routing methods are deterministic.

> **Practice:** Deterministic methods are based on a cause-effect consideration of the rainfall runoff processes.[21] Such methods are used when:
>
> - There are limited streamflow records for frequency analysis;
> - The rainfall-runoff response has changed due to land use, drainage improvements or other reason that has upset the homogeneity of the streamflow data, causing complications to statistical analysis of the data;
> - A runoff hydrograph is required for volumes, timing or routing.

Principle: Statistical methods of runoff estimation can give good results if the stream flow records are long enough and, unlike deterministic methods, a level of error prediction can be calculated for stream flow records. Runoff may be subject to changes from urbanization and drainage improvements. Such changes can be better estimated by deterministic methods. [21]

> **Practice:** More complex hydrology and hydraulic computer simulation models may be applied in project specific cases. However, these methods typically require costly field measurements, large amounts of data preparation, model setup, sensitivity analysis and calibration. Careful scoping of tasks with expertise from staff and consultants is often necessary.

> **Practice:** The use of state-wide LIDAR datasets is often merged with Department survey.

NHDOT uses several methods to estimate peak flows and run-off volumes, primarily the five methods listed below. Experience indicates these methods are practical, and attainable in terms of accuracy and cost. Rational and regression methods are the simplest methods to apply and are preferred. While curve number event modeling (TR-20/TR-55, HydroCAD) is moderately more data intensive, the increased amount of data used in the model does not necessarily translate into a greater accuracy of prediction than the Rational Method or regression equations.[21] The highway designer should rely on methods that provide reasonable accuracy and methods that can be efficiently applied. For small homogeneous areas ranging between 10,000 – 50,000 square feet more or less (~0.25 to 1 acre), that encompass the majority of routine highway runoff estimates, the Rational Method will agree closely with estimates the designer obtains from the Curve Number Method. Contact the NHDOT Specialty Section if other methods will be used.

1. Rational Method (peak runoff only, not used for volume) **see Section 3.0.2.**
2. NH StreamStats (usgs.gov) regression equations with limitations set forth in SIR-2008-5206. The Rural Equation can be used in USGS National Urban Equation, see HDS 2 chp.5. Regression equation confidence limits can be used to resolve significant differences between other methods. [AASHTO]
3. The FHWA Method that has roots in the Potter Method, aka, Runoff Estimates for Small Rural Watersheds and Development of a Sound Method, Vol. I & II, Report No. FHWA-RD-77-159, October 1977
4. Rainfall-Runoff unit hydrograph hydrologic routing, aka, the Curve Number Method including, but not limited to the NRCS methods: TR-55, TR-20, HydroCAD;
5. Published Flow Records & Reports, such as: USGS storm specific reports, consultant or university reports, stream gage records. The USGS | National Water Dashboard is continuously evolving and the agency coordinates w/ FHWA, FEMA, and others.

The Log-Pearson Type III flood frequency distribution is an additional method used for larger watersheds. Refer to USGS Guidelines for determining flood flow frequency (Bulletin 17C).

The designer may choose to rely on the Curve Number Method for the following situations:

- Rational Method and StreamStats regression equations are outside their limitations.
- The catchment area is heavily developed.
- Hydrograph routing is required (as in pond design, LID & BMPs, and/or some culvert designs).
- The catchment area has significant storage not accounted for with another method.
- Assumptions and/or parameters may be improved with better data, such as LIDAR.

Principle: In using the Curve Number Method, the lack of flow and process-based hydrologic data available in most highway design situations requires major assumptions about parameter values and distributions.[21]

> **Practice:** Use of HydroCAD, a propriety software and user friendly surrogate for TR-20 TR-55 methodology, may be used to support the design of stormwater ponds and the sizing of culverts or closed drainage systems. Modeling synthetic rainfall hydrographs with HydroCAD is widely practiced by consulting engineers in New Hampshire and elsewhere. The peak flow design estimates must be checked with a second method when sizing culverts.

Table 0.a – Ungaged Runoff Estimation Methods

ESTIMATION METHOD	ASSUMPTIONS	DATA NEEDS	WHEN USED
Rational Method	• Small catchment (< 200 acres) • T_c < 1 hr. • Storm duration > or = to the T_c. • Rainfall uniformly distributed spatially and over time. • Runoff is primarily overland flow. • Negligible channel storage.	• Tc • Drainage area • Runoff Coefficient 'C' • Rainfall intensity 'I'	• For runoff from watersheds < 200 acres & paved or homogenous surfaces. • Sizing small pipes, e.g. stormwater systems, driveway culverts; • Quick check on other methods (even slightly larger watersheds).
USGS Regional Regression Equations: Scientific Investigations Report (SIR): 2008-5206	• Peak discharge value for flow under natural conditions unaffected by urban development and little or no regulation by lakes or reservoirs • Ungaged channel • Basin Development Factor (BDF) can be applied to the Rural StreamStats Equation obtained from the web application.	• Provided in StreamStats	• For drainage areas 0.7 – 1290 sq. miles; • For slopes from 5 – 543 ft./mile;
Runoff Estimates for Small Rural Watersheds and Development of a Sound Method, Vol I & II, Report No. FHWA-RD-77-159, October 1977, (The FHWA Method). AKA the FHWA Method	• Catchments <50 sq. miles • Significant amounts of storage can be estimated with a 'C' factor.	• Drainage area used is in acres. • Iso-erodent value 'R'. • Watershed slope. • "Off-line" storage area.	• Used as a replacement for Potter's Method (aka the Bureau of Public Roads Method).
NRCS TR 55	• Runoff is overland and channel flow. • Negligible channel storage.	• Drainage area. • 24 hr. rainfall distribution. • Tc. • Soil information (HSG). • Land use (CN).	• Small or midsized catchment (<2000 acres); • T_c range from 0.1 – 10 hr.;
Unit Hydrograph • NRCS Unit Hydrograph • Synthetic Unit Hydrograph • Snyder * (NRCS & Synthetic Unit Hydrographs found in surrogate programs such as HydroCad)	• Uniformity of rainfall intensity and duration. • Duration of direct runoff constant for all uniform-intensity storms of same duration, regardless of differences in the total volume of the direct runoff. • Time distribution of direct runoff from a given storm duration is independent of concurrent runoff from preceding storms • Channel-routing techniques used to connect streamflows	• Rainfall hyetograph and direct runoff hydrograph for one or more storm events • Drainage area and lengths along main channel to point on watershed divide and opposite watershed centroid (Synthetic Unit Hydrograph)	• Used for detailed hydrologic studies when the model will be calibrated to a storm. • Pre & Post condition comparisons.
Bulletin 17C	Guidelines for determining flood flow frequency—Bulletin 17C		•

Table 0.b – Gaged Runoff Estimation Methods

Estimation Method	Assumptions	Data Needs
Log Pearson Distribution Bulletin 17c	1. Midsized and large catchments with stream gage data 2. Appropriate station and/or generalized skew coefficient relationship 3. Channel storage	10 or more years of gaged flood records; Estimates should be limited to twice the years of record (e.g. 25 years of record for 50-yr discharge estimates)
Basin Transfer of Gage Data	1. Similar hydrologic characteristics 2. Channel storage	Discharge and area for gaged watershed area for ungaged watershed

Principle: Past performance is not a guarantee of future results, no matter how good the statistical correlation, or modelling.

Practice: Compare the results of at least two runoff methods for design flows, preferably more.

Practice: Field inspections should be made to observe actual conditions and to compare design assumptions and calculations.

Hydrologic Soil Groups (HSG)

In 1955 George W. Musgrave described a hydrologic classification of soils that is based on infiltration rate.[13] Four basic soil groups depend on the minimum infiltration capacity, and were based on laboratory tests and soil texture. The four groups are: A, B, C, and D, with sands in group A, and clays in group D. Soils for most of the United States have been assigned to Hydrologic Soil Groups (not within National Parks in NH). The HSG, along with land cover are major parameters for the Curve Number method. Soil scientists normally must be consulted to interpret criteria that are used to assign a soil to an HSG group if the published NRCS classification is questioned. Geotechnical engineers within the Bureau of Materials and Research can perform infiltration tests when requested. Often times the designer can use NRCS tables for reasonable values. Generally, the HSG is based on a soil's long term infiltration rate which is a function of a soil's composition, depth and slope. This leads to a soil's runoff potential when a soil is thoroughly wet, not protected by vegetation, and experiencing precipitation for long duration.

Group A: This group of soils has a *high* infiltration rate when thoroughly wet. These soils are deep and well drained to excessively drained sands or gravely sands. These soils typically have less than 10% clay, and have 90% sand or gravel and have gravel or sand textures. Some soils having loamy sand, sandy loam, or silt loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35% rock fragments.

Group B: This group of soils has a *moderate* infiltration rate when thoroughly wet. These soils consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine to moderately coarse texture. These soils are between 10% and 20% clay, 50% to 90% sand, and have loamy sand or sandy loam textures. Some soils having loam, silt loam, silt or sandy clay loam textures may be added to this group if they are well aggregated, of low bulk density, or contain greater than 35% rock fragments.

Group C: This group of soils has a *slow* infiltration rate when thoroughly wet, and has a somewhat restricted water transmission rate. This group consists chiefly of soils containing 20% to 40% clay, and less than 50% sand and has loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures. Some soils having clay, silty clay, or sandy clay textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35% rock fragments.

Group D: This group of soils has a *very slow* infiltration rate when thoroughly wet, and the water transmission rate is restricted to very restricted. These soils typically have greater than 40% clay, less than 50% sand, and clayey textures. In some areas they also have a high shrink-swell potential. These soils may also have a high water table, may have a clay pan or a layer of clay at or near the surface, or are shallow over a nearly impervious area. Most wetland soils are Group D.

Certain wet soils are placed in HSG D solely on the presence of a water table within 24 inches of the surface even though the saturated hydraulic conductivity may be favorable for water transmission. If these soils can be adequately drained, then they are assigned to dual hydraulic soil groups. The dual HSG's are: A/D, B/D, and C/D. The first letter applies for the drained condition, and the second letter for the natural undrained condition. For the purpose of Hydrologic Soil Group, adequately drained means that the seasonal high water table is kept at least 24 inches below the surface in a soil where it would be higher in a natural state. Refer to NRCS guidance for further discussion.

Principle: More granular soils are non-cohesive and susceptible to transport by incipient motion, whereas, the cohesive soils exhibit plastic or sticky characteristics. Clays will absorb water if exposed or striped of vegetation and can break off in larger pieces. All soils are susceptible to erosion.

Web Soil Survey

Society of Soil Scientists of Northern New England

New England Soil

Table 0.a – Hydrologic Soil Group Descriptions

Group	Minimum Infiltration Rate (inch/hour)	Soil Description
A	0.30 - 0.45	Deep sand; deep; loess; aggregated silts
B	0.15 - 0.30	Shallow loess; sandy loam
C	0.05 - 0.15	Clay loams; shallow sandy loam; soils low in organic content; usually high in clay
D	0.00 - 0.05	Soils that swell significantly when wet; heavy plastic clays; certain saline soils

Note*: These values are approximate references. If needed, contact the NHDOT Geotechnical Section for site specific design infiltration rates.*

(Source: McCuen, 2004)

Methods Used by NHDOT for Estimating Peak Discharge

Common methods used for highway drainage purposes are included in this manual. The current state of practice at NHDOT for determining peak flow rates, volumes, and timing are, in part, provided in this manual. The methods used for determining design flows should be archived with the design documents. Design flows are typically tabulated in reports or spreadsheets; paper charts may also be a part of the documentation. There are important similarities between methods e.g., time of concentration (t_c), physical area, slope, land cover, and equally important distinctions such as appropriate range of area for which the method should be used, watershed characteristics, and stochastic vs. deterministic methods. Contact the Specialty Section for runoff estimation methods less commonly used.

3.0.2 Rational Method

The Rational Method has been in use since the middle of the 19th century. Thomas James Mulvaney, an Irish Engineer, was the first to publish the principles on which it is based. The original study was for combined sewer design and the upper area limit of the study was 200 acres. Emil Kuichling is recognized as the founder in America. He published a paper entitled "The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts", in 1889. In recent years, studies continue to be published on applications of the Rational Method to paved and small homogenous areas, usually less than approximately 200 acres.

Principle: The Rational Method is normally used to estimate peak runoff, but the method cannot rout hydrographs. The modified Rational Method could be used to calculate a volume of runoff (this is a triangular hydrograph with an asymmetrical receding side).

> **Practice:** The Department does not use triangular hydrographs to calculate the volume of runoff for sizing drainage practices. Curved hydrographs are used.

Principle: The Rational Method can be as good as the more complicated methods when used for small homogeneous subcatchments. This method yields conservative peak flows and despite valid criticisms, it is still one of the most widely used methods for stormwater drainage design because of its simplicity.[21]

> **Practice:** This method is used in areas that can be independently classified and averaged. It is also used for site design and pavement runoff of small drainage areas. While best suited to small urban drainages, it is also used for rural subcatchments. Greater emphasis should be given to other methods as the subcatchment area exceeds 200 acres.

The general formula is **Q=CiA,** where: Q = peak runoff (cfs), C= runoff coefficient (dimensionless; $0<C<1$), I = avg. rainfall intensity lasting for critical time T_c (in/hr), & A = contributing drainage area (subcatchment) in acres.

Principle: This formula, although dimensionally incorrect, is considered "rational" because an inch depth of rainfall applied at a uniform rate for 1 hour on an area of one acre of impervious surface will yield 1.000833 cfs of runoff barring any losses. The runoff coefficient is considered dimensionless because 1.00833 acre-inch/hour is equivalent to 1.0 cfs.

Principle: The Rational Method is based on the thesis that if a uniform rainfall of intensity (i) is falling on an impervious area (A), the maximum rate of runoff at the outlet to the drainage area would be reached when all portions of the drainage area are contributing; then the runoff rate becomes constant. The principle time required for runoff from the hydraulically most remote point of the drainage area to arrive at the outlet is called the time of concentration (time when the entire catchment is assumed to contribute to runoff), see separate Section 3.2.6 for T_c discussion.

Rational Method Parameters C, i, A & T_c

Table 3.0.2.a is taken from McCuen (1989) and provides guidance in choosing C values for use in Rational Method calculations. This table correlates C values to land use, slope, and hydrologic soil group (HSG) discussed earlier in Section 3.1. It can be used to provide some consistency between application of the Rational Method and the NRCS Curve Number Method. Coefficients by land use that were more commonly used in past designs are included in **Table 3.0.2.b**.

Principle: The <u>*runoff coefficient*</u> C is interpreted as the ratio of peak runoff rate over the rate of rainfall at an ***average*** intensity when all the drainage area is contributing. The coefficient C varies from storm to storm, but studies have shown[21] that when rainfall intensity and runoff are considered separately, the ratio is reasonably constant for the various frequencies.

$$C = \frac{\text{Peak runoff rate of a given frequency}}{\text{Average rainfall intensity of the same frequency}}$$

Principle: The C coefficient represents the fraction of rainfall converted to runoff.

C = Total depth of runoff / Total depth of precipitation

"Calibration of the runoff coefficient has almost always depended on comparing the total depth of runoff with the total depth of precipitation." David B. Thompson

* The range in C values permits some allowance for land slope, soil type, vegetal cover, surface storage, urban development and rain event magnitude. The C coefficient is the least precise variable in the Rational Method.

Practice: NHDOT uses a C value of 0.95 for all paved landcover.

- The Meadow C values are used in lieu of pasture when fields are not grazed by livestock.
- It is not intended for **Table 3.0.2.a** to be in complete agreement with **Table 3.0.2.b**. Both tables are provided for a wider range of C values to be available to the practitioner to suit specific circumstances, such as historic designs.
- Historic practice included the use of customized cardboard slide rules – the Specialty Section has some of these early devices used in applying the Rational Method.

Table 3.0.2.a – 'C' Value Guidance for Use in Rational Method

	Hydrologic Soil Group											
	A			**B**			**C**			**D**		
Land Use*	**0 - 2%**	**2 - 6%**	**> 6%**	**0 - 2%**	**2 - 6%**	**> 6%**	**0 - 2%**	**2 - 6%**	**> 6%**	**0 - 2%**	**2 - 6%**	**> 6%**
Cultivated Land	0.08	0.13	0.16	0.11	0.15	0.21	0.14	0.19	0.26	0.18	0.23	0.31
	0.14	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Pasture	0.12	0.20	0.30	0.18	0.28	0.37	0.24	0.34	0.44	0.30	0.40	0.50
	0.15	0.25	0.37	0.23	0.34	0.45	0.30	0.42	0.52	0.37	0.50	0.62
Meadow	0.10	0.16	0.25	0.14	0.22	0.30	0.20	0.28	0.36	0.24	0.30	0.40
	0.14	0.22	0.30	0.20	0.28	0.37	0.26	0.35	0.44	0.30	0.40	0.50
Forest	0.05	0.08	0.11	0.08	0.11	0.14	0.10	0.13	0.13	0.12	0.16	0.20
	0.08	0.11	0.14	0.10	0.14	0.18	0.12	0.16	0.20	0.15	0.20	0.25
Residential **1/8** acre lot	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Residential **1/4** acre lot	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Residential **1/3** acre lot	0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.29
	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.39	0.40	0.50
Residential **1/2** acre lot	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.48
Residential **1** acre lot	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Industrial	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Streets	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78
	0.76	0.77	0.79	0.80	0.82	0.84	0.84	0.85	0.89	0.89	0.91	0.95
Open Space	0.05	0.10	0.14	0.08	0.18	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39
Parking**	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87
	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97

* For each land use, the first row is for $T < 25$ years, and the second row is for $T \geq 25$ years.

** Department policy is to use 0.95 for all paved land cover.

(Source: McCuen, 1989)

Table 3.0.2.b – Rational 'C' Coefficients by Land Use

Land Use	Range of Runoff Coefficients, C	Recommended Value
Business		
Downtown	0.70 – 0.95	0.85
Neighborhood	0.50 – 0.70	0.60
Residential		
Single-family	0.30 – 0.50	0.40
Multiunit, detached	0.40 – 0.60	0.50
Multiunit, attached	0.60 – 0.75	0.70
Residential (suburban)	0.25 – 0.40	0.35
Industrial		
Light	0.50 – 0.80	0.65
Heavy	0.60 – 0.90	0.75
Parks, cemeteries	0.10 – 0.25	0.20
Playgrounds	0.20 – 0.35	0.30
Railroad yards	0.20 – 0.35	0.30
Unimproved natural conditions	0.10 – 0.30	0.20
Pavement		
Asphalt and Concrete	use permeable guidance	0.95
Brick	-	0.95
Roofs	-	0.95
Lawns, sandy soil		
Flat, < 2%	0.05 – 0.10	0.08
Average, 2-7%	0.10 – 0.15	0.13
Steep, > 7%	0.15 – 0.20	0.18
Lawns, heavy soil		
Flat, < 2%	0.13 – 0.17	0.15
Average, 2-7 %	0.18 – 0.22	0.20
Steep, > 7%	0.25 – 0.35	0.30

Note: See Table 3.2.1a for more recommended C values.

Principle: Rainfall intensity **(i)** is the average rainfall intensity selected on the basis of the design rainfall duration and return period. The design rainfall duration is taken as the time of concentration, $\mathbf{t_c}$.

Principle: Rainfall intensity **(i)** is assumed uniform over the catchment and constant for the storm duration **($\mathbf{t_r}$)**. This assumption is necessary to make the Rational Method practical. *"The essence of the Rational Method (as opposed to the Rational Formula) is the decision to set design storm duration t_r equal to the time of concentration t_c. Choosing an appropriate design rainfall is a matter of determining t_c, once t_c is known, intensity **(i)** follows directly from the IDF curve, or by formula. In highway hydrology t_c is usually taken as a deterministic subcatchment parameter. Smaller durations equal to t_c correspond to higher intensities for a given return period."*[21]

Intensity, duration, frequency or IDF curves can be obtained from charts or by computer applications. Charts for durations from 5 minutes to 24 hours and return periods from 1 to 100 years were created by the U.S. Weather Bureau in the early 1960s. Charts used in prior versions of this manual were created for NH counties with different frequency storms **(**legacy manual reference Figures 2-3, 2-4, 2-5**)**.[36] These figures were created from data prior to 1960. These charts can be used for comparing rainfall intensity with the more modern references such as the data products available from the NRCC & the National Weather Service NWS-NOAA Atlas 14.

> **Practice:** Designers should use IDF curves from NOAA 1st, and NRCC 2nd.
>
> **Practice:** Custom IDF curves can be created with the FHWA's Hydraulic Tool Box and Hydro-35 (circa 1977), or other data products if needed (typically no longer needed).

Principle: Area is the easiest parameter to determine with the highest degree of accuracy in the Rational Formula. Area can be challenging to quantify in flat watersheds that have undergone significant development. It is sometimes difficult to determine the runoff direction with the information available. It has been made easier to determine runoff direction and area in flatter watersheds with LIDAR data sets. High relief watersheds can also change significantly with the higher resolution topography provided by LIDAR.

> **Practice:** Field checks are necessary to determine the direction of flow in flat areas and areas with closed drainage systems.
>
> **Practice:** Use HydroCAD, the FHWA Hydraulics Toolbox, or equivalent to calculate T_c.

A brief discussion on sheet flow is included on the next page. See guidance for $\mathbf{T_c}$ in Section 3.0.7 , pg. 3-25 further on.

Sheet Flow is flow over planar surfaces such as paved areas and fields. The standard assumption is that it occurs primarily in the subcatchment upland areas. The Manning's "n" is an effective roughness coefficient that includes the depth effect of flow, raindrop impact, drag over the surface, obstacles to flow eg: crop ridges, rocks, leaf litter, erosion and transport of sediment. Appropriate n values for sheet flow are readily available in NRCS references, HydroCAD, and elsewhere, see **Table 3.0.2.c**.

> **Practice:** Flow depth (effectively hydraulic radius R_h for wide flow paths) should ordinarily not exceed 0.1 ft for sheet flow. Assessment of reasonable sheet flow lengths (across planer surfaces) should be part of the hydrologic site inspection if possible.

> **Practice:** A **maximum of 100 ft** is recognized in the most current research[12] (ASCE, Curve Number Hydrology *State of the Practice*). The designer will find that a maximum of 300 ft was used on many earlier studies. The average is ~ 80 ft. according to HDS 2. One would expect shorter times for smooth and steep surfaces and longer sheet flow periods for rough textured flat areas.

Table 3.0.2.c – Manning's n for Sheet Flow

Surface Description	**n**
Smooth surfaces (concrete, asphalt, gravel, or base soil)	0.011
Fallow (no residue)	0.05
Cultivated soils (residue cover less than 20%)	0.06
Cultivated soils (residue cover greater than 20%)	0.17
Short grasses	0.15
Dense grasses	0.24
Woods, light underbrush	0.4
Woods, dense underbrush	0.8

Note: *When selecting n, consider cover to a height of ~ 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.*

(Source: Adapted from USDA SCS)

Principle: The discharge may only be a function of depth when precipitation is the only source of water and there is no ponding or backwater, and no storage effects. This is true for *sheet flow* in the most distant parts of a watershed.

Principle: The equation above is the Manning's equation with the added assumption that there is a relationship between hydraulic radius and the roughness coefficient.

Principle: By fitting the conveyance equation to observations, lines of velocity vs. slope for various land cover can be plotted to estimate velocity such as figures available in the National Engineering Handbook (NEH).

Open Channel Flow:

Principle: The Manning's Equation is used to determine average velocities for the **TR-55** channel component of the $\mathbf{t_c}$ path.

$$\mathbf{V = 1.49/n\ R^{2/3}\ S^{1/2},\ t_c = (L_f / V)\ (1\ hr\ /\ 3600\ s)}$$

Principle: The assumed flow depth for calculations should be the "low flow" range, namely the 2 yr. storm, because the $\mathbf{t_c}$ is intended to represent the beginning stages of a runoff event when the entire watershed theoretically begins to contribute. If the NRCS method is not used for t_c estimation one could use a different storm frequency than the 2 yr. storm.

> **Practice:** Often times the blue hydrologic stream lines on USGS quadrangle sheets or within StreamStats can be used to determine the approximate limits of open channel flow. Due to increased sinuosity of the actual length of flow, the distances scaled from a map are normally shorter than actual lengths of open channel flow. Aerial photos or 3D lines can be used to calculate a more accurate length.

Kerby-Hathaway Equation:

The time of concentration for small watersheds with predominately overland flow can be calculated using the Kirby-Hathaway: $t_{ov} = 0.828(L \times N)^{0.467}\ S^{-0.235}$

Table 3.0.2.d – Kerby Equation Retardance Coefficients

Land Cover	Retardance 'N' factor
Pavement	0.02
Smooth, bare, packed soil	0.10
Poor grass, row crops, or moderately rough packed surfaces	0.20
Pasture, average grass	0.40
Deciduous forest	0.60
Dense grass, coniferous forest, or deciduous forest w/deep litter	0.80

(Source: ?)

Kirpich (Pennsylvania) Formula:

Discussion here is mainly for context on the origin of the method. This formula is applicable for small watersheds with fairly steep slopes (3 -10 %) between 1 and 112 acres where overland flow (defined as sheet flow for this formula) is minimal. This method for calculating sheet flow is not commonly used in New England. Z.P. Kirpich developed this equation in 1940 using agricultural watersheds with bare soil. He recorded the rise and fall of outlet stream depths after rain events.

$$t_c = 0.00002167 L_f^{0.77} S_f^{-0.5} M_t \text{ (Bentley Storm CAD)}$$

L_f = Flow length in the channel in feet

Roussel (2005) concluded that the Kerby – Kirpich approach is straight forward to apply and produces readily interpretable results.

Kerby – Kirpich Method:

Older method available in HydroCAD. However, not typically used.

$$\mathbf{t_c} = t_{ov} + t_{ch}$$

$t_{ov} = K(L \times N)^{0.467} S^{-0.235}$ (K = 0.828 for imperial units)

$t_{ch} = K(L_{ch})^{0.770}(S)^{-0.385}$ (K = 0.0078 for imperial units)

3.0.3 Regression Equations-StreamStats & The FHWA Method

A cooperative effort between USGS and ESRI has resulted in the stream statistics, aka "StreamStats.", GIS application that provides regression equation estimates of flows across most of the nation. The equations are developed in each State by USGS and available in the National StreamStats program (NSS), formerly known as the National Flood Frequency regression equations (NFF). The NFF program was a compilation of all the current statewide and metropolitan area regression equations. The regression equations are a product of years of research by the USGS and others to develop regional regression equations for estimating flood magnitude and frequency of ungaged watersheds. The USGS, in cooperation with the Federal Highway Administration (FHWA) and the Federal Emergency Management Agency (FEMA) compiled all the regression equations into a single database file. This database file was the basis of the NFF program. More recently the USGS New Hampshire office, in cooperation with NHDOT & NHDES, produced regression equations now implemented in StreamStats that replace the NFF equations.[26] The Potter Method equations that were used as a starting point for the FHWA Method equations have not seen the continued development that the USGS equations have, moreover, as stated above FHWA worked with USGS to compile prior regression equations in the 1980s. Contact the Specialty Section for more on the history of the FHWA, Bureau of Public Roads, and/or the Potter's Method. The Department primarily uses the StreamStats rural equations and the associated national urban equation. Design guidance on early methods is removed from the Manual on Drainage Design for Highways starting in the 2021 version.

Principle: Regression equations attempt to relate runoff from selected gaged watersheds to ungaged watersheds by statistical analysis of selected watershed parameters such as drainage area, watershed slope, and average precipitation. The accuracy of the resulting equation depends on the degree of similarity between the watershed being studied and the watersheds used in the statistical analysis. Accuracy is also dependent on the number and quality of characteristic watershed parameters used. The limits of use are also dependent on the watersheds that were studied.

Principle: It is useful to compare the results of statistical and deterministic models. Significant disagreement should be a reason for more in-depth investigation.

> **Practice:** Contact the Specialty Section when there are differences of more than 50% between NH StreamStats. & HydroCAD estimates for peak flows. Check the calculations for any errors first.

Principle: The existence of impervious area is a constraint to using regression equations because of a lack of historical data regarding impervious area that can be applied to stream gage data and applying impervious area as a basin characteristic within the statistical analysis. Impervious area was not considered in the NH USGS equations; therefore, it is a matter of engineering judgment when the percent of impervious area is significant enough to require a correction factor.

Principle: The nature of stochastic modeling (regression in this case) can be a constraint to evaluating the effects of impervious area, which is a physically based parameter more suitable to deterministic models.

> **Practice:** A Basin Development Factor (BDF) can be applied to the StreamStats "Rural" equation by using the national "Urban" equation also developed by USGS. The procedure is described in the National Highway Institute HDS-2, 2nd edition document, circa 2002.

USGS StreamStats

The primary modern regression equations used by the Department were developed for New Hampshire by USGS. (Olson, S.A., 2009, Flynn, Robert H. 2003). The NH StreamStats equations have a lower percentage of error than prior NFF equations. Results from the USGS "Rural" equation can be applied to the USGS "National Urban" equation as detailed in HDS 2.

3.0.4 NRCS Curve Number Method

SCS, now NRCS, developed four dimensionless rainfall distributions using the Weather Bureau's Rainfall Frequency Atlases (Circa 1960). The rainfall frequency data was for areas less than 405 sq. miles, for durations to 24 hours, and for frequencies from 1 to 100 years. Data analysis indicated that New Hampshire contains Type II & Type III rainfall distributions. Even though none of the study's watersheds were in New Hampshire, the data was thought to adequately represent regional conditions and the curve number method has been widely applied in applications worldwide that are far beyond the original intent. Refer to references at the end of this manual for additional information on the Curve Number Method. The method can provide reasonable peak flow and volume estimates.

Principle: Computers have made the NRCS and surrogate methods easier to apply, however, the designer of highway facilities will likely be faced with decisions using only limited information in order to build hydrologic and hydraulic models.[20]

Principle: Analysis of pre and post conditions is useful for design purposes even if the model is not calibrated using stream gages, precipitation gages, other scientific means and the modeling is only judged to be reasonable. For example, tests to validate reasonableness may include appraisal of practical culvert capacities or observed road overtopping depths, or other similar factual evaluations. The exercise of model development often helps the designer recognize important aspects of runoff, attenuation from storage and other subcatchment characteristics. *Tests of reasonableness are not calibrations.*

Principle: The curve number method, when used for analysis of catchments less than 200 acres does not necessarily produce more accurate results than the Rational Method. Both methods are empirical and error prediction is not obtained as it is with regression equations. Application of the curve number method, without calibration using gages, may lead to a better understanding of the parameters or basin characteristics; however, this does not mean that the deterministic values of flow are more accurate than other methods. The fact that the curve number method has been widely used and accepted does not mean that the method is necessarily accurate for deterministic flows without calibration of the model.

Technical Release 55 (TR-55) presents graphical and tabular procedures for estimating runoff and peak discharges in small watersheds originally intended to be less than 2000 acres and more than 5 acres. The original development of the TR-55 method grew out of the need to apply TR-20 methods that were used in agriculture to urban areas that contain substantial impervious areas. There is really no need to use TR 55 when computer versions of TR-20 or surrogates such as HydroCAD programs are available. For those who wish to further their knowledge on these methods, the ASCE publication Curve Number Hydrology *State of the Practice* provides discussion on the history, limitations, advantages, and challenges latent in the use of the curve number method. A copy of Curve Number Hydrology is available in the Specialty Section. Fact sheets detailing planned developments of WinTR-55, Win TR-20, and EFH 2 were published in June 2010. The following is a list of planned enhancements:

- Combine features of Win TR-20, Win TR-55, and EFH 2 into one interface

- GIS capacity
- Import Northeast Regional Climate Center (NRCC) data
- Import NOAA Atlas 14 data

Some limitations of the curve number method are:

- Narrow range of curve numbers for soil and land cover;
- Generic initial abstraction, or curve numbers may require adjustments for new applications of the method, such as for use in sizing costly water treatment practices;
- Extreme precipitation values can produce excessive peak flows and volumes of runoff compared to statistically based estimates that are better for application of risk based assessments associated with highway design;
- Coding of curve number models with more physically based detail does not directly translate into increased hydrologic certainty and may in fact lead to false complacency;
- Applications of the method are well beyond the original intent.

Curve number tables are provided in **Appendix B**. Other sources such as HydroCAD, and the NEH contain identical CNs. A comprehensive discussion on TR-55 is provided in the June 1986 publication by NRCS entitled "Urban Hydrology for Small Watersheds".

Table 3.0.4.a – Coefficient of Velocity (K_v) Values for Overland Flow

Surface Description	**K_v**
Paved area & small upland gullies ([1] & [2])	20.3282
Unpaved ([2])	16.1345
Grassed waterway ([1])	15
Nearly bare and untilled ([1])	10
Short grass pasture ([1])	7
Woodland ([1])	5
Forest with heavy ground litter ([1])	2.5

(Source: [1] NEH Fig. 15.2 & [2] TR-55 Appendix F)

Practice: The open flow equation (Manning's) should be used downstream of where the shallow concentrated flow (or sheet flow) enters a defined channel.

Formula that can be used to calculated overland travel times for shallow concentrated flow:

Velocity $\mathbf{V = K_v(S)^{1/2}}$ & Travel time $\mathbf{t_t = (L)/3600V}$ in hours

Where:
K_v = coefficient of velocity
S = avg. slope of the flow path in ft/ft
L = length in ft.

3.0.5 Unit-Hydrograph Methods in General

The designer is applying a unit hydrograph method whenever hydrographs are routed, such as with HydroCAD (synthetic hydrographs). The unit hydrograph concept was first presented by L.K. Sherman in the April 7, 1932 Engineering News-Record.[19] Although the use of empirical relations is similar to other hydrologic methods, the unit hydrograph distribution does not provide a statistical measure of error. There were three basic propositions listed by Sherman:

I. For a given drainage basin, the duration of surface runoff is essentially constant for all uniform-intensity storms of the same length, regardless of the differences in the total volume of surface runoff;

II. For a given drainage basin, if two uniform-intensity storms of the same length produce different total volumes of surface runoff, then the rates of surface runoff at corresponding times *t*, after the beginning of two storms, are in the same proportion to each other as the total volumes of surface runoff;

III. The time distribution of surface runoff from a given storm period is independent of concurrent runoff from antecedent storm periods.

All these propositions are empirical; it is not possible to prove them mathematically. In fact, not a single one of them is mathematically accurate [19] (Johnstone & Cross).

"A common empirical approach to a solution involves (1) selecting drainage basin characteristics (e.g., shape, slope) that seem likely to have an effect on the shape of the hydrograph; (2) selecting a number of gaged drainage basins possessing these characteristics in varying degree; (3) looking for correlations between these characteristics, on the one hand, and the observed flood regimens of the various basins, on the other; and (4) expressing the most significant correlations mathematically so that they can be used to predict the flood regimen of ungaged basins. The shape of any actual flood hydrograph is determined not only by drainage basin characteristics but also by the pattern of the storm that produces it. *The effect of the storm pattern must be eliminated before the correlations in step (3) can be undertaken.* Discovery of the unit-hydrograph principle made it possible to reduce flood hydrographs for a wide variety of streams to functions of drainage basin characteristics alone. The reason that the unit hydrograph distribution is useful (even though it is not mathematically accurate) is that once an empirical relation is found by correlation, the distribution can be synthesized for basins not included in the study."

Principle: Rainfall is related to runoff for a given watershed using unit hydrographs.

Principle: A unit hydrograph is the mathematical tool that is used to determine how a single burst of runoff is distributed over time. The hydrograph procedure provides a volume distribution over time, and therefore affords the opportunity to better manage the runoff in a cost effective manner. [20] (Urban Storm Drainage Management, pg. 105.)

The unit hydrograph results from one inch of runoff over the drainage area ***not*** one inch of rainfall. For more in-depth scientific discussion on the creation of design hydrographs and their limitations refer to Highway Hydrology, HDS No. 2, Second Ed. published by FHWA in 2002.

3.0.6 Reservoir Routing Methods in General

The net rainfall-direct runoff routing can be looked at as a reservoir routing process with the inflow (I) due to the net rainfall falling on the catchment and the outflow (Q) as the direct runoff from the catchment. The flood storage volume (S) in the catchment is assumed to be a function of the outflow. Linearity of this function determines whether the reservoir is linear or non-linear (not all runoff is linear in relation the area of the watershed). Moreover, the reservoir can either be single or a series of reservoirs in cascade.

Dams and Ponds in New Hampshire cause some watersheds to be "regulated", meaning that the hydrology is affected by manmade structures (the term regulated as used here has nothing to do with Administrative Codes). For some of these instances it may be necessary to analyze the catchments using reservoir routing methods. This type of empirical analysis may have been completed by other agencies or consultants in New Hampshire. The Dam Bureau has records of thousands of registered dams throughout the State. Typically, there are stage discharge curves and inlet and outlet hydrographs to consider if flood storage volumes could influence the runoff estimate. Depending on the complexity it may or may not be necessary to contact the Dam Bureau or the Specialty Section.

3.0.7 Time of Concentration (t_c)

The FHWA definition for time of concentration ($\mathbf{t_c}$) is the time required for a particle of water to flow from the hydraulically most distant point in the watershed to the outlet or design point. Travel time is also sometimes referred to as time of concentration or storm duration. This section summarizes the common $\mathbf{t_c}$ methods used for surface runoff calculations. Flow (Q) is a function of volume and time. In addition, the intensity of rainfall runoff is a function of depth and time. Duration and depth impact the flow magnitude. Most drainage basins consist of different types of ground covers and conveyance systems that flow segments must navigate. It is common for a basin to have overland and open-channel flow segments. Urban drainage basins often have flow segments that flow through a storm sewer pipe in addition to overland and open-channel flow segments. A travel time (the amount of time required for flow to move through a flow segment) must be computed for each flow segment. The time of concentration is equal to the sum of all the flow segment travel times.

"Factors that affect the time of concentration are the length of flow, the slope of the flow path, and the roughness of the flow path. For flow at the upper reaches of a watershed, rainfall characteristics, most notably the intensity, may influence the velocity of the runoff." [HDS 2, pg. 59]

Travel time (**t_t**) is the time it takes water to travel from one location to another in a watershed. **t_t** is a component of **t_c**, which is computed by summing all the travel times for consecutive components of the drainage flow path. [#WSDOT]

t_t =length(L)/velocity (V) **$V=1.49/nR^{2/3}S^{1/2}$**

$1.49/nR^{2/3}$ could be replaced by a conveyance constant k.

Various methods can be used to estimate the **t_c** of a watershed. When selecting a method to use in design, it is important to select a method that is appropriate for the watershed being studied. Some estimation methods were designed and can be classified as "lumped" in that they were designed and calibrated to be used for an entire watershed; the SCS lag formula is an example of this method. Other methods are intended to model the velocity along an individual segment of flow path that can be used with the length of that segment of the flow path to compute the travel time for that segment. With this method, the time of concentration equals the sum of the travel times on each segment of the principal flow path.[HDS 2, pg. 59]

The **t_c** for the NRCS method assumes that rainfall is applied at a constant rate over a drainage basin, which would eventually produce a constant peak rate of runoff. Actual precipitation does not fall at a constant rate. A precipitation event usually begins with less rainfall intensity, builds to peak intensity, and eventually tapers down to no rainfall. The maximum depth of peak flow for a given rainfall event can be observed. However, the observed time to maximum depth may not be the same as the calculated value arrived at.

Principle: Because rainfall intensity is variable, the time of concentration is included in the Rational Method to apply the proper rainfall intensity across the basin. The intensity that should be used for designing is the highest intensity that will occur with the entire basin contributing flow to the flow rate location being studied. This may be a much lower intensity than the maximum intensity due to it taking several minutes before the entire basin is contributing flow; the maximum intensity lasts for a much shorter time, so the rainfall intensity that creates the greatest runoff is less than the maximum by the time the entire basin is contributing flow.

Practice: Describe typical data source and format for rainfall intensity.

In rare cases, the time of concentration that produces the largest amount of runoff is less than the time of concentration for the entire basin. This can occur when two or more sub-basins have dramatically different types of cover (i.e., different runoff coefficients). The most common case would be a large paved area together with a long, narrow strip of more natural area, such as a highway median. In this case, runoff from the paved area alone should be compared to the scenario when both land segments are contributing to flow. The goal is to determine which scenario would cause a greater peak runoff rate. The flow based on a shorter time of concentration for the pavement-only area, or the more common scenario for the entire drainage area contributing. Use the greatest peak runoff rate even if the entire basin is not contributing to flow.

The time of concentration was initially estimated with empirical formulas. Empirical formulas are only useful in a narrow set of circumstances.

Principle: Empirical formulas for $\mathbf{t_c}$ should be avoided unless the specific characteristics of the research that developed the formula are pertinent to the catchment. Empirical formulas for travel time such as the Kirby Method for parking lots, or the Kirpich kinematic wave method deal only with water flowing as a "sheet". Additionally, $\mathbf{t_c}$ for the entire hydraulic path is not accounted for with older empirical methods. For example, the FAA equation is not included in this manual because its use is only applicable to airports, or large flat or similar land cover. There are many empirical methods mostly derived from different forms of the Mannings equation for determining $\mathbf{t_c}$ for specific types of watersheds. The Kirby-Hathaway, and the Kirpich empirical equations (Tennessee & Pennsylvania) for small watersheds with primarily overland flow are discussed in HDS 2. These methods for calculating T_c have recently been added to HydroCAD with the release of version 10.1 (November 2020). However, the methods may not be suitable for many watersheds in NH. The TR-55 Lag CN method is discussed further on.

Velocity Method, (aka the Upland Method)

The procedure was developed by the Natural Resources Conservation Service (NRCS; formerly known as the Soil Conservation Service, aka SCS). This procedure that uses sheet, shallow concentrated, and open channel segments of flow for determining the $\mathbf{T_c}$ can be used with the Curve Number Method and the Rational Method. It is sensitive to slope, type of ground cover, and channel geometry.

> **Practice:** The NRCS Velocity Method is the most common procedure used by NHDOT for determining the time of concentration. Tc by this method is defined by the relationship:
>
> $$\mathbf{t_c} = \mathbf{t_t}\ \text{(sheet)} + \mathbf{t_t}\ \text{(concentrated)} + \mathbf{t_t}\ \text{(channel)}$$
>
> **Practice:** If the total **Tc** is less than six minutes (0.1 hr., which is the smallest interval available for most precipitation gage records), a minimum of six minutes shall be used as the duration. A minimum of 5 min. is cited in some documents. It is possible to have $\mathbf{T_c}$ less than 5 min., but evidence of such an occurrence should be provided in the drainage report if less than the minimum is used.

Principle: All three types of flow need not be present in every catchment. This method indicates that **Tc** is a function of length and velocity.

Principle: Underestimating $\mathbf{T_c}$ leads to overestimating Q and thus to over design of hydraulic structures. Conversely, overestimating $\mathbf{T_c}$ leads to lower flow estimates and under design.

Sheet Flow is flow over planar surfaces such as paved areas and fields. The standard assumption is that it occurs primarily in the subcatchment upland areas. The Manning's *"**n**" is an effective roughness coefficient that includes the depth effect of flow, raindrop impact, drag over the surface, obstacles to flow eg: crop ridges, rocks, leaf litter, erosion and transport of sediment. Generally, one would expect shorter times for smooth and steep surfaces and longer sheet flow periods for rough textured flat areas. Another factor is vegetative cover which can significantly increase time of travel. Look up tables for appropriate **n** values for sheet flow are available in the Appendix A of this manual.*

Principle: Sheet flow is usually the slowest flow component followed by a shorter time for shallow concentrated flow and an even shorter time for open channel flow. The definition of velocity is used with the Manning's equation to calculate sheet flow.

The 2 yr. 24 hr. precipitation event is used when applying the NRCS Velocity method. Other storm frequencies and intensities can be evaluated if needed. The Rational Method would most commonly be used for such evaluations. Changes in intensity relate to sheet flow rather than open channel flow.

***Principle*:** Manning's **n** for sheet flow is much greater than the typical **n** for open channel flow. Roughness is greater for shallow flow depths for sheet flow of ~ 0.1 – 0.15 ft. or less.

- Use Appendix A for Mannings **n** values.

Principle: The steeper the slope and the smoother the surface the longer the sheet flow can go before concentrating into gullies. The rougher the surface the shorter the path before concentrating into gullies.

> **Practice:** Flow depth (effectively hydraulic radius R_h for wide flow paths) should ordinarily not exceed 0.1 ft for sheet flow. Assessment of reasonable sheet flow lengths (across planer surfaces) should be part of the hydrologic site inspection if possible.
>
> **Practice:** A maximum of 100 ft shall be used. This max. sheet flow length is cited in modern research and the ASCE, Curve Number Hydrology *State of the Practice* publication. The designer may find old drainage studies that used a maximum of 300 ft. (typ. more than a decade old).

Shallow Concentrated Flow (SCF) Sheet flow tends to concentrate in terrain geometry such as rills, gullies, or in pavement gutters, SCF depths normally occur between approximately 0.1 ft and less than 0.5 ft. A good approach to determining the length of SCF is to establish the lengths of sheet flow and open channel flow first, leaving the remainder as SCF. Shallow concentrated flow should not be use where open channel flow is evident. Excessive lengths of SCF of several thousand feet likely contain sections more suitably represented by open channel flow. Mannings n is not needed to calculate SCF. For paved surfaces the following formula can be used to estimate TR-55 SCF: $V = 20.3282\ S_f^{\,0.5}$

Table 3.0.7.a – Intercept Coefficients for Velocity vs. Slope Relationship

Land Cover/Flow Regime	k
Forest with heavy ground litter; hay meadow (overland flow)	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)	0.152
Short grass pasture (overland flow)	0.213
Cultivated straight row (overland flow)	0.274
Nearly bare and untilled (overland flow); alluvial fans in western mountain regions	0.305
Grassed waterway (shallow concentrated flow)	0.457
Unpaved (shallow concentrated flow)	0.491
Paved area (shallow concentrated flow); small upland gullies	0.619

Note: *The intercept coefficient, k, is a dimensionless function of land cover.*

(Source: McCuen, 1989)

$$\mathbf{V = 33\, k\, S^{0.5}}$$

Principle: The equation above is the Manning's equation with the added assumption that there is a relationship between hydraulic radius and the roughness coefficient.

Open Channel Flow:

Principle: The Manning's Equation is used to determine average velocities for the open channel component of the $\mathbf{T_c}$ path.

$\mathbf{V = 1.49/n\, R^{2/3}\, S^{1/2}}$ **,** $\mathbf{t_c = (L_f /V)\ (1\ hr\ /\ 3600\ s)}$

Principle: The assumed flow depth for calculations should be the "low flow" range, namely the 2 yr. storm or less, because the $\mathbf{T_c}$ is intended to represent the beginning stages of a runoff event when the entire watershed theoretically begins to contribute. If the NRCS method is not used for $\mathbf{T_c}$ estimation one could use a different storm frequency than the 2 yr. storm.

> **Practice:** Often times the blue hydrologic stream lines on USGS quadrangle sheets or within StreamStats can be used to determine the approximate limits of open channel flow. Due to increased sinuosity of the actual length of flow, the distances scaled from a map are normally shorter than actual lengths of open channel flow. Aerial photos or LIDAR can be used to calculate more accurate flow lengths than used prior to having this data.

LAG CN method

The SCS LAG CN method is a means to quantify mathematical predictions pertaining to the entire watershed rather than only surface flow prior to any shallow concentrated flow. The flow paths can be much longer with SCS LAG CN. The duration of rain events cannot be changed, but rather the equations for rainfall "excess" runoff after losses have been developed to predict hydrographs (runoff vs. time graph), specifically, unit hydrographs that take a shape based on the watershed characteristics. The time in hours from the center of mass of rainfall excess to peak discharge is known as the time lag $\mathbf{t_L}$ of a rainfall excess volume defined by the equation:

$$t_L = L^{0.8}(S+1)^{0.7}/1900Y^{0.5} \qquad (t_c = 5/3\ t_L)$$

L= Hydraulic length in feet from the furthest place in the watershed to the outlet. The furthest away is not necessarily the furthest distance.

Y= average slope of the watershed which can be acquired from GIS data or CAD. The slope value is used as a direct percent, so 1% is 1 not 0.01 ft/ft.

> **Practice:** The NH StreamStats report can be used for an average watershed slope based on a 30 m DEM.

S = retention in inches. S=(1000/CN-10)

The time of concentration is 5/3 (1.6667) times the lag time with this method (t_c = 5/3 t_l), and it can also be calculated with the SCS Lag equation below:

$$\mathbf{t_c}=0.00526L^{0.8}(1000/CN-9)^{0.7}S^{-0.5}$$

Time to peak is 0.67 x the t_c. Different peak factors can be used. The typical factor is 484, but calibration can be performed for watershed characteristics such as urbanization or rural runoff characteristics. The SCS Lag formula is intended to be used in non-urban watersheds.

HydroCAD & WMS can be used to calculate the time of concentration and travel time. The terms discussed above are shown graphically below. More detailed guidance is available from sources such as the **Office of Hydrology and Hydrologic Research.** at NOAA.

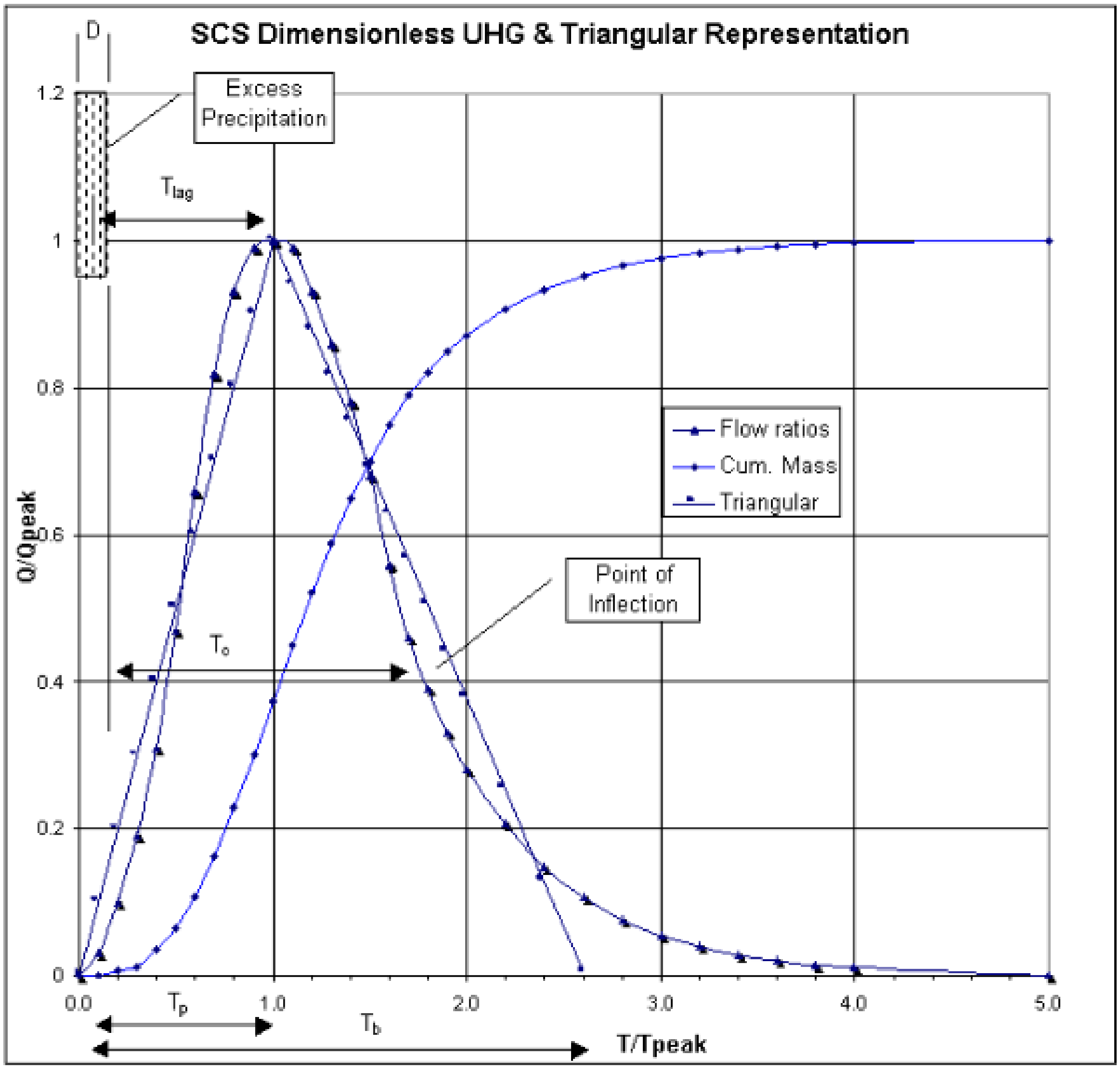


Figure 3.0.7.a Dimensionless curvilinear unit hydrograph and equivalent triangular hydrograph used with the Rational Method (NWS-NOA Office of Hydrology)

Principle: Time of concentration is a real stochastic value, a real phenomenon that can be estimated. In some watersheds when conditions are adequate after a rain storm physical observations can be made. It is a lumped parameter that contains aspects that can be measurable.

Principle: For an impervious watershed, all precipitation within the drainage area is considered run-off once the t_c has occurred.

Principle: The t_c for a specific rain event can be observed by measuring a hydrograph or by measuring the depth of flow at a point in the channel at the bottom of the catchment. This does not mean that the t_c for a future event will be identical.

Calibration and Observation-Based Estimates [21]

Historic rainstorms are used in actual storm event simulations, which are carried out in conjunction mostly with the calibration/verification of hydrological/hydraulic models, and to a lesser extent, with flood-forecast and post-event flood evaluations.

Principle: Rainfall and runoff data are rarely available for calibration of highway drainage calculations and simulations. Very often the best available site-specific data consists of anecdotal information about high water marks, inundation levels, and their relative frequency. Such information is not suitable for numerical calibration of calculations, but it can be used to judge or validate the calculations.

Every effort should be made to identify specific problem areas prone to flooding and to collect anecdotal information about flooding. This information can be used with backwater analysis and culvert analysis to independently estimate extreme flows.

> **Practice:** In the case of flood estimates at locations with existing culverts, the designer should compare calculations against the existing pipe capacity in the context of the pipe history. For example, if calculations indicate flooding while experience suggests acceptable culvert performance, the runoff model should be re-evaluated before recommending a design flow.
>
> **Practice:** The span or size of a new or replacement highway structure is based on more than the hydraulic capacity as determined by the equations of a computer model. Natural and cultural resource protection and stream assessment procedures are usually evaluated. One exception is an inlet that discharges into a closed stormwater system.
>
> **Practice:** Scenarios in hydraulic modelling with a two dimensional mesh are used evaluate the sensitivity to changes in Mannings roughness, boundary conditions, and topographic connectivity.in some areas.

Wetland Hydrology

Principle: Wetlands are complex requiring comprehensive evaluation to adequately understand processes for design purposes.

Practice: Experts, including a Wetland Scientist and possibly a Professional Hydrologist (PH) may be relied on when designing a wetland facility. Often times this will require consulting with professionals outside of the NHDOT.

Snow Melt

The designer should recognize high relief catchments such as mountainous areas in NH that are more influenced by rapid snow melt than lower floodplains without large rivers. Lower floodplains with rivers that convey snow melt from mountainous areas experience peak stages many hours or days after a combined snow melt and precipitation event. A warm period that includes nights with temperatures above freezing can cause continuous snow melt producing a hydrograph that will be superimposed on a simultaneously occurring storm hydrograph.

Principle: Snow melt has an important influence on seasonal ground water levels, stream flows, ice break up, and pond depths. Mild winter temperatures can replace snow melt periods with a more sudden runoff hydrology resulting in higher short term peaks and seasonal drought conditions.

> **Practice:** The Department will review research problem statements and/or reports associated with changes in annual snow fall that pertain to watershed runoff methods.

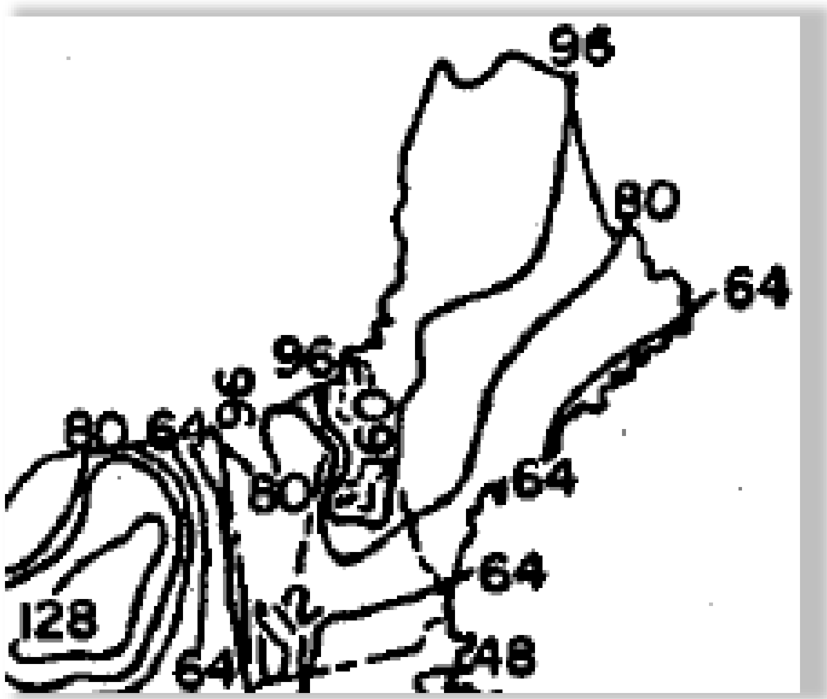


Figure 0.a Mean annual snow fall in the NE from 1931-1952
(US Weather Bureau 1960)

Table 0.a – Average Annual Snowfall from 1981-2010

	Town	Average Annual Snowfall (inches)
Southern NH	Concord	60.8
	Portsmouth	59.9
	Laconia	56.1
	Nashua	54.9
	Keene	54.6
	Durham	44.7
Northern NH	Mt. Washington	281.2
	Bethlehem	101.1
	Colebrook	80.3
	North Conway	80
	Berlin	78.3
	Tamworth	77.6
	Plymouth	67.9

(Source: Northeast Regional Climate Center)

Regarding future revisions

Engineers and other hydrologic professionals will continue to use the best hydrologic methods that are available to design hydraulic structures for transportation infrastructure. New methods might come in the form of advanced statistical analysis, GIS algorithms, in down scaling regional models, or in simpler forms such as indexes applied as coefficients. Some changes are anticipated for time tested methods, such as different initial abstraction for the NRCS Curve Number Method, and more stream flow data for USGS regression equations. Other likely advances include continued development of the Army Corp. HEC HMS software for rainfall on grid, evapotranspiration, and other grid based GIS layers in lieu of unit hydrograph methods. Data indicating hydrologic change will likely be available for more resilient designs. Reducing the uncertainty with runoff calculations will continue to challenge designers. It will be important to stay current with research and the potential for improvements to methods used.

Page intentionally left blank

Section 4

Highway Hydraulics

Highway Hydraulics

The hydraulic design process involves analyzing existing conditions and alternatives judged to meet site conditions and flood flows for the selected design frequency. Hydraulics focuses on the behavior of fluid flow using the principles of energy conservation, continuity, and momentum (including energy loss from friction and minor losses caused by convergence, divergence and changes in flow direction). Hydraulic energy is a function of elevation, velocity and pressure. A change in pressure or energy is needed to cause flow. Highway engineers analyze stormwater pipes, culverts, pavement drainage, slope drainage, and the performance of hydraulic structure inlets, open channels, and ditch flow. A wealth of information is available from federal, and state agencies, educational institutions, and other reputable sources.

Environmental Hydraulics consists of stabilizing, restoring or otherwise altering any portion of a drainage catchment to dually meet infrastructure and environmental objectives. Categories of Environmental Hydraulics include best management practices (BMPs) for water treatment, streambed and bank protection measures, aquatic organism passage, and stream restoration. The Department works closely with federal and state resource agencies and the NEPA process to design and operate transportation infrastructure for resilience.

The applications, criteria for design, principles, practices, and most common hydraulic design references used by NHDOT are provided in this section and in chapter 6 of the Highway Design Manual.

Principle: Sediment transport and the associated shear force is considered an element of hydraulic science for the purposes of highway design.

Open Channel "Ditch" Conveyance

NHDOT uses the following types of open channels and ditches for runoff conveyance:

Roadside Ditches are used adjacent to and parallel with the highway and include inlet, outlet, and lateral ditches. They remove storm runoff from the highway section and may provide drainage for part of the subbase material depending on the ditch depth. The purpose is mainly to intercept surface flow not groundwater.

Median Ditches are located in depressed areas between multilane and divided highways.

Berm Ditches are provided longitudinally at the top of a cut to intercept runoff.

Outlet channels provide conveyance from the right of way to a natural channel or river. These ditches may have been constructed prior to modern wetland regulations and permits and maintaining these ditches is expensive. In most cases NHDOT is responsible for the drainage facilities in the right of way and easements are required to maintain ditches across private property. Consider AASHTO guidance provided in Chapter 11 of the Model Drainage Manual for aspects such as adequate freeboard, debris & ice passage.

4.0.1 Hydraulic Design of Open Channels

"Design analysis of both natural and artificial channels proceeds according to the basic principles of open channel flow (see Chow, 1970; Henderson, 1966). The principles of fluid mechanics (continuity, momentum, and energy) can be applied to open channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the principle problems of open channel analysis and it depends on quantification of the flow resistance. Natural channels display a much wider range of roughness values than artificial channels."[22]

Open channels are defined as natural or artificial conveyances for water where the water is exposed to the atmosphere, and the motion of water is driven by gravity. The designer will routinely perform hydraulic analysis of open channels. Useful software and publications are available from Hydraulics | FHWA (dot.gov) for a productive means to analyze & design open channels. Open channel charts are good independent checks on computer reports. FHWA references available to the designer include:

FHWA Hydraulic Design Series (HDS 3) Design Charts for Open Channel Flow

FHWA Hydraulic Design Series (HDS 4) Introduction to Highway Hydraulics

Two-Dimensional Hydraulic Modeling for Highways in the River Environment

Instructional videos associated with NHI training, such as the mobile flume series by James Schall are useful in reviewing Open Channel Principles.

4.0.2 Design Criteria for Open Channels and Ditches

Channel analysis for transportation drainage systems is necessary in order to ensure that the proposed design meets BMP practice for capacity, flow distribution, water surface profiles, AOP, and stability against erosive forces.

Principle: Stream channels are typically 1) Natural channels where the size and shape are defined by natural forces. 2) Consist in a main channel for conveying low flows and a floodplain for conveying less frequent flows. 3) Shaped geomorphically by the long term history of sediment load and flow

Cross Sections

NHDOT specifies standard cross sections in the Highway Design Manual. Design dimensions include front slope, back slope, and ditch width. The designer must check that the standard ditch dimensions provide adequate conveyance of flow for the application. The cross sections of a re-routed natural channel will be based on hydraulic analysis and fluvial geomorphologic considerations.

Channel Lining

For extensive coverage consult the FHWA Hydraulic Engineering Circular (HEC 15) and manufactures literature.

Drainage of Roadside Channels with Flexible Linings

Practice: All channels will be checked for possible erosion and subsequent siltation of streams. Because depth and slope are easier to measure than channel velocity the tractive force method is preferred over the permissible velocity method. **Table 4.0.2.a** is provided to summarize more commonly encountered channel material (Refer to HEC 15 for additional guidance).

Practice: Acceptable methods of stabilization include matting and stone for erosion control, stone fill, and riprap.

Practice: Berm ditches should be at the top of cut slopes when excessive off site runoff could damage slopes and/or overtax on-site systems. The designer should review berm ditches during the Slope & Drain design phase. The recommendation to construct such ditches will commonly be made by the Bureau of Materials and Research. It may be necessary to make them aware of the situation. A Geotechnical report may be available.

Table 4.0.2.a – Permissible Shear Stress (τ_p)

Protective Cover	**Description**	**Permissible Shear Stress, τ_p (lb/ft^2)**
Class B, native grass	dense growth uncut > 12”	2.1
Class C, native grass	good stand, mowed, 6”- 12”	1.0
Class D, native grass	good stand, 3” - 6”	0.6
Class E, “Bermuda” grass	good stand cut to 1.5”	0.35
Gravel	D50 = 2”	0.8
Stone	D50 = 6”	2.4
Stone	D50 = 12”	4.8

Note: *See HEC 15 for additional permissible shear stress values in cohesive & non-cohesive soils.*

(Source: ???)

The mean boundary shear stress (τ_o) applied to the wetted perimeter was formerly calculated from the relationship $\tau_o = \gamma R S_o$, where, R= hydraulic radius in ft., S = slope, & gamma = 62.4 lb/ft^3. The latest HEC 15 methodology uses maximum shear stress on the bottom of the channel (water depth instead of hydraulic radius). The permissible shear stress values shown in **Table 4.0.2.a** should be checked with the latest HEC 15 methodology. Channel analysis can be performed using the FHWA Sponsored Hydraulics Toolbox or by other means.

Practice: A maximum length of 400 ft for a ditch to a cross pipe, catch basin or drop inlet is desirable. Local conditions may require variations.

Practice: In order to keep the ditch self-cleaning, a minimum grade of 0.5% (0.005 ft/ft) will be employed where possible. However, there are circumstances where this may not be practical, primarily in urbanized areas. The absolute minimum grade is 0.25 %. Ditch grades constructed to the absolute minimum will likely not be self-cleaning, thereby requiring maintenance.

Practice: Channels along aquifers and water supply areas will be designed in accordance with governing environmental agencies.

4.0.3 Open Channel Treatment Practices

Water Quality Treatment Swales

Treatment swales are designed to promote sedimentation and infiltration by providing a minimum hydraulic residence time within the channel under design flow conditions. These swales are permanent water treatment features and are/may be subject to specific NHDES regulations. Guidance from operation maintenance manuals (O&M) may be provided with consultant designs, and the NHDES Stormwater Manuals are used for design considerations and maintenance requirements. The NH Stormwater Manuals are published by NHDES.

Stream Restoration Channels

In general, these channels are naturally occurring pathways for water. In some circumstances a channel may be designed where a natural channel does not already exist. Hydraulic design of channels and maintenance work in natural channels must be coordinated with the Bureau of Environment. Channel design requires execution by a qualified Engineer and perhaps a Fluvial Geomorphologist.

4.0.4 Manning's Roughness Values

Refer to **Appendix A** for recommended Manning's n design values. The FHWA third edition of **HDS 5** contains an **Appendix B** for guidance on the roughness of culvert barrels. Actual values for older pipelines may vary depending on abrasion, corrosion, deflection, and joint conditions. However, existing pipes are usually analyzed as originally constructed. Correcting n for the actual condition could have a significant effect on pre vs. post capacity and velocity calculations. Concrete pipe with poor joints and deteriorated walls may have higher n values than the typical 0.012 used for new pipes. Corrugated metal pipe (CMP) with joint and wall problems may also have higher n values. Smaller diameter CMP has a Mannings of 0.024-0.025. However, n values for CMP vary by size, type, and corrugation profile. Refer to chart for corrugated metal conduits in **HDS 5 Appendix B**. Also, CMP pipes may have shape changes that could adversely affect the general hydraulic characteristics. Additional guidance for Mannings values applied to 2d hydraulics will be available once the NCHRP 24-49 research project is completed. **Appendix A** has further discussion and contains additional guidance.

Principle: The roughness coefficient, n, varies with the type of vegetative cover and flow depth. At very shallow depths, where the vegetation height is equal to or greater than the flow depth, the n value should be approximately 0.15 (typical value for depths up to 4"). For higher flow rates and depths, the n value decreases to a minimum of 0.03 for grass channels at a depth of approximately 12".

Practice: The n value must be adjusted for varying flow depths between 4"- 12" (Figure 4.0.4.a).

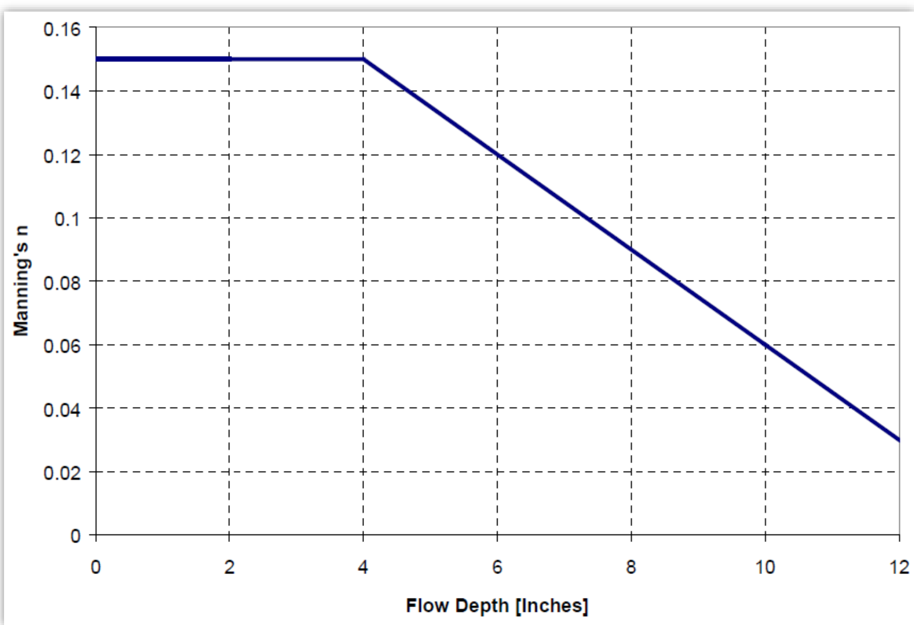


Figure 4.0.4.a Manning's n chart for water treatment swales (Clayton & Schuster, 1986)

4.0.5 Specific Energy & Froude Number

Principle: Specific energy in a channel is the energy head relative to the channel bottom. If the channel is not too steep (less than 10%) and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy becomes the sum of the depth and velocity head. Many practical problems of open channel flow are solved by application of the energy principle using the bottom of the channel as a datum as indicated in Figure 4.0.5.a [HDS 7] (Bernoulli's equation). The flow represented in Figure 4.0.5.a has smooth parallel streamlines.

The specific energy E is given by the total energy head consisting of the depth of flow and the velocity head. If head losses are negligible the specific energy can be written as:

$$\mathbf{E = y + v^2 / 2g}$$

The velocity head of an open channel is usually greater than the average velocity head computed as $(Q/A_t)^2/2g$, a velocity distribution coefficient can be applied to the velocity head if needed.

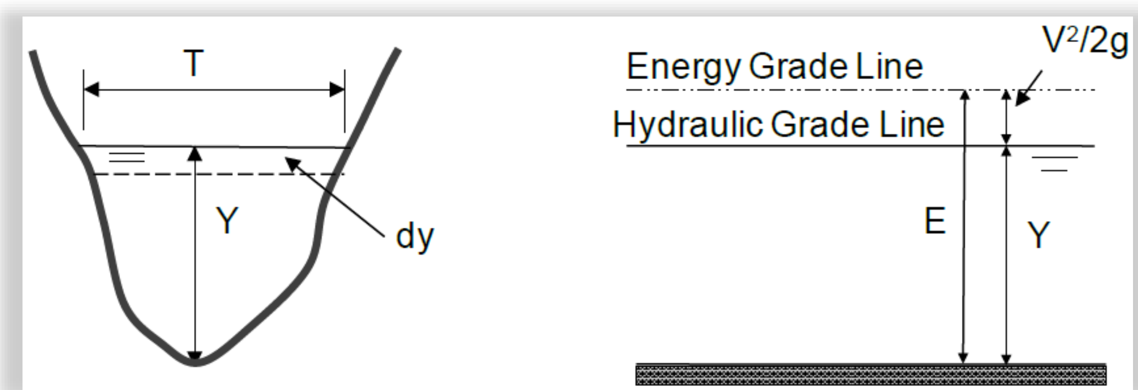


Figure 4.0.5.a Description of specific energy (HDS 7)

Principle: If the channel has a slope less than ~ 10% and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds) then the specific energy becomes the sum of the depth and velocity head.

Principle: Figure 4.0.5.b shows that there are two depths for the same energy except at minimum energy where critical depth occurs.

> **Practice:** The designer uses the concept of specific energy because it is built into the charts and software that are used for hydraulic analysis. Refer to an open channel text, such as "Open Channel Flow" by Ven T. Chow for detailed discussion. The FHWA HDS 7 publication provides further discussion and a derivation that relates specific energy, the continuity equation (Q = VA) and the Froude number. An in-depth discussion of specific energy is beyond the scope of this manual.

Principle: With the introduction of 2d hydraulic modelling into mainstream engineering design applications, and the inclusion of momentum equations, methods of routine analysis for open channels are evolving. Graphical distribution of velocity vectors and color coded shear force can be output from 2d modelled areas.

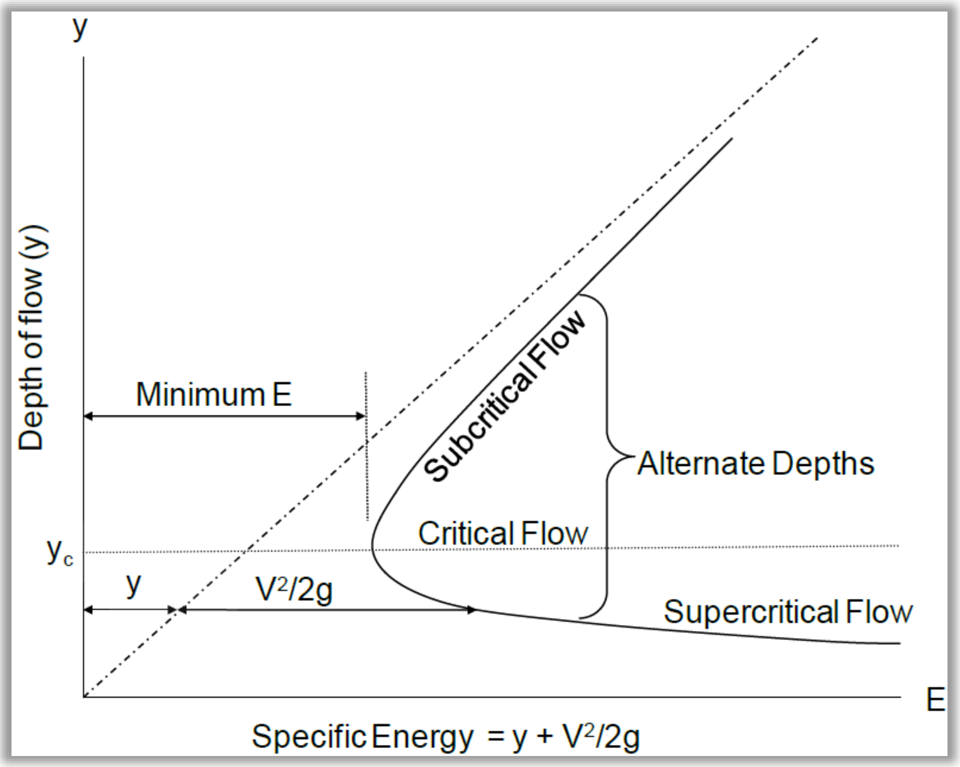


Figure 4.0.5.b Specific energy curve for a constant discharge (HDS 7)

Principle: For a channel of small slope that produces uniform average velocity $E = y + v^2/2g$, transitions occur between conjugate depths at hydraulic jumps, hydraulic drops and flow over a dam. The specific energy curve also illustrates subcritical and supercritical flow ranges according to depth of flow. Critical depth, where the Froude number is unity, occurs at minimum specific energy.

The dimensionless Froude number for rectangular channels is $Fr = {}^*v/(g \bullet y)^{1/2}$, for non-rectangular channels the depth of flow, y, is replaced with cross sectional area divided by surface width. The Froude number is a ratio of inertial forces over gravitational forces.

Fr > 1 for supercritical flow
Fr = 1 for critical flow
Fr < 1 for subcritical flow
* v = average velocity

Practice: Avoid hydraulic jumps in stormwater systems.

Practice: Hydraulic jumps can be effective for energy dissipation for Froude Numbers less than one at the outlet of culverts and even inside culverts (broken back slope) provided the joints and/or bottom of an open bottom culvert can remain stable in profile. Tailwater is required.

Practice: Froude number is the most common means of identifying appropriate energy dissipation structures and techniques. Refer to **Table 4.0.22.a**.

Practice: Avoid pressure flow in closed stormwater systems.

4.0.6 Total Energy Head

Principle: The total energy head is the specific energy head plus the elevation of the channel bottom with respect to some datum. The energy head from one cross section to the next defines the energy grade line.

4.0.7 Steady & Unsteady Flow

Principle: A steady flow is when the discharge passing a given cross section is constant with respect to time. The maintenance of steady flow requires that the rates of inflow and outflow be constant and equal. When the discharge varies with time, the flow is unsteady. Unsteady flow is commonly represented with a hydrograph.

4.0.8 Uniform Flow & Non-uniform Flow

Principle: A non-uniform flow is when the velocity and depth vary in the direction of motion, while they remain constant in uniform flow. Uniform flow can only occur in a prismatic channel, which is a channel of constant cross section, roughness, and slope in the flow direction. However, non-uniform flow can occur either in a prismatic channel or in a natural channel with variable properties.[23]

4.0.9 Gradually Varied & Rapidly Varied Flow

Principle: A non-uniform flow when the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected, is referred to as gradually-varied flow; otherwise, it is considered to be rapidly-varied.

4.0.10 Flow Classification

In summary of Sections 4.1.5 – 4.1.9, whether a particular flow can be considered steady or unsteady can depend on the frame of reference. The simplest steady flow is uniform flow, in which no variable (such as depth or velocity) changes with distance. If a flow is not uniform, then it is classified as gradually varied or rapidly varied. In gradually varied flow the flow variables may change with distance but are constant with respect to time. In rapidly varied flow, substantial variations are present in the vertical and/or transverse flow. Examples of rapidly varied flow are hydraulic jumps, flow through culverts and over weirs.

Table 4.0.10.a – Summary of Flow Profiles

Channel Slope	Designation	Relation of y to yn & yc	General Type of Curve	Type of Flow
Mild Slope	M1	$y > y_n > y_c$	Backwater	Subcritical
	M2	$y_n > y > y_c$	Drawdown	Subcritical
	M3	$y_n > y_c > y$	Backwater	Supercritical
Critical Slope	C1	$y > yc = y_n$	Backwater	Subcritical
	None	$y_c = y_c = y_n$	None	None
	C3	$yc = y_n > yc$	Backwater	Supercritical
Steep Slope	S1	$y > y_c > y_n$	Backwater	Subcritical
	S2	$y_c > y > y_n$	Drawdown	Supercritical
	S3	$y_c > y_n > y$	Backwater	Supercritical
Horizontal Slope	None	$y > y_n > y_c$	None	None
	H2	$y_n > y > y_c$	Drawdown	Subcritical
	H3	$y_n > y_c > y$	Backwater	Supercritical
Adverse Slope	None	Not applicable	None	None
	A2	$y > y_c$	Drawdown	Subcritical
	A3	$y_c > y$	Backwater	Supercritical

***Note**: yc = critical depth & yn = normal depth*

(Source: FHWA HY8)

4.0.11 Critical Flow

Principle: Critical flow occurs when the specific energy is a minimum.[18] It is very unstable, a slight change in specific energy can produce a significant rise or fall in the depth of flow. Critical depth is also the dividing point between the subcritical flow regime (tranquil flow), where the normal depth is greater than critical depth, and the supercritical flow regime (rapid flow), where normal depth is less than critical depth. For a given value of specific energy, the critical depth gives the greatest discharge (but it is an unstable condition). One can determine the slope through a critical section that will not cause any backwater effect. After the critical depth is determined, the critical slope can be calculated by combining the continuity equation with the Manning's equation:
(The equations in this manual use English units exclusively).

$$\mathbf{S_c = (Qn / 1.486\ A_c R_{hc}^{2/3})^2}$$

(A_c = the area for critical flow depth, R_{hc} = the hydraulic radius for the critical flow depth)

Examples of when critical flow occurs:

- Passing through a sudden contraction, either vertical or horizontal, before the water outlets into an unrestricted channel, culvert, bridge, pond;
- Near the brink of an over fall, such as at a dam;
- At certain stream flows that normally are supercritical (characteristic of critical depth is often a series of surface undulations over a very short stretch of channel);
- When a hydraulic jump occurs.

Examples of what critical depth or critical slope are useful for:

Determining inlet or outlet control because critical depth occurring at the outlet of a culvert typically indicates outlet control, whereas critical depth that occurs at the inlet indicates a culvert operating under inlet control.

Determining if increasing culvert barrel size is useful because as long as the critical depth is higher than the crown of a culvert, outlet velocity can be reduced by increasing the barrel size [HEC 14] (today environmental aspects often control rather than capacity).

Critical slope is the minimal slope for maximum discharge to occur without a box culvert flowing full, so culverts with slopes less than critical slope tend to flow full at lower headwater than culverts with slopes greater than critical slope.

> **Practice:** Usually the designer can provide a stable stream profile by matching the culvert slope with natural stream slope.

Observation: For almost full pipe flows, the critical slope is approximately 0.01 ft/ft. Drainage pipe constructed at 1% slope will be predominately supercritical. Critical slope is typically less for partially full pipes.

Critical depth can be calculated using the following formulas that use imperial units for various channel shapes:

Rectangular:

An example of critical flow is a rectangular box culvert that discharges a pond. When the upstream pond surface reaches the top of the inside of the box and considering friction as negligible and discharge to the atmosphere; there will be a drop down curve from the pond elevation to critical depth as illustrated in **Figure 4.0.11.a**. The velocity is increasing from the pond condition to critical velocity.

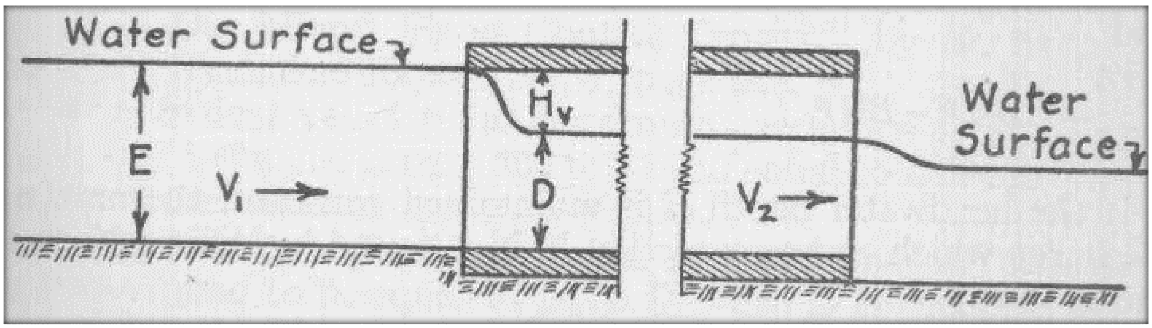


Figure 4.0.11.a Critical flow through rectangular box culvert schematic (D in the Figure is D_c)

$D_c = (0.176\ Q/b)^{2/3}$ where b=width of the box in ft. & Q is the flow in cfs.

Weir discharge can be derived from the equation $Q = \left(\frac{2}{3}\right) C\text{w}\ b(2g)^{1/2}\ H_w^{3/2}$
where b is the length of the weir perpendicular to the flow, and H_w is the depth of water above the weir crest (aka head). Cw is the discharge coefficient that incorporates the effects on discharge of the approach velocity, head loss, and contraction. $Cw = 0.602 + 0.075(\frac{Hw}{y})$ with y being the height above the bottom of the pond

Triangular:

$D_c = (0.757\ Q/Z_1+Z_2)^{2/5}$

Trapezoidal:

Trial and error solution using $Q=(gA^3/T)^{1/2}$

Circular:

In lieu of a trial and error solution, an approximate solution can be obtained using: $D_c=0.42Q^{1/2}/D^{1/4}$ (English units, dia. D in ft, Q in cfs). By substituting the critical velocity in the Mannings equation, the critical slope is equal to $2.04/D^{1/3}$.

Critical depth is inevitably linked with specific energy and therefore the concept is important when evaluating hydraulic jumps, hydraulic drops, pipe flow performance, and sediment transport. Many of the charts and software algorithms are made possible due to an understanding of critical flow and specific energy.

> **Practice:** Critical depth is not used for the design flow conditions; designers use software that evaluate critical depths for a range of conditions. Open channel charts and the equations provided above are good checks for questionable computer generated reports.

4.0.12 Normal Depth

Principle: Normal depth and velocity: The depth of flow using Manning's Equation is referred to as the normal depth and the velocity is referred to as the normal velocity. The slope of the water surface and the channel slope is the same and the depth remains constant. Normal depth occurs when the gravitational force of the water equals the frictional drag along the culvert or channel and there is no acceleration of flow.[18] It is the depth for uniform and steady flow.

Practice: The geometry involved in a direct mathematical solution of Manning's equation can be complex for some channel shapes. Instead, a trial and error approach is necessary. Tables and computer programs, such as **HY8,** are available to assist in determining normal depths for various geometries.

Flow at normal depth in culverts often presents the highest ***average*** velocities and the shallowest depths at that flow. Normal depth is a mathematical means used to determine if the natural flow is subcritical or supercritical. It is typically used in the following circumstances:

- For depth of flow at unsubmerged outlets on steep slopes;
- For the development of rating curves using the Manning's equation, and;
- For boundary conditions in hydraulic models.

Principle: The designer would not use normal depth if tailwater can be determined by other means, such as a fixed water body (pond), recorded water depths for various flow rates, or a backwater calculation. For example, one would not use normal depth to represent the downstream condition at a culvert discharging into a pond.

4.0.13 Weirs & Baffles

Roadway overtopping, pond outlet structures (such as emergency spillways), grate inlets, baffles for Aquatic Organism Passage (AOP) and energy dissipation are the primary applications where the designer uses weir equations and flume research. The variables are geometric dimensions and head. The length of overtopping along a roadway is a key dimension for understanding the hydraulic relationship between a series of roadway crossings. The availability of state-wide LIDAR allows us to determine overtopping dimensions accurately in most cases.

Principle: Weir flow can transition to orifice flow at inlets with higher headwater. In such cases, orifice flow capacity is more conservative than the weir equation.

Practice: The length and depth of overtopping during flooding conditions should be recorded when observed and documented for future design work.

Weir Coefficients

Flow over a weir always includes a discharge coefficient. Some care is required when using published coefficients, since their units may not be clearly stated. Discharge coefficients will often increase as depths increase and flow becomes more efficient. The coefficients used in modelling calculations may need to be manually changed for vary flow profiles. As a general rule-of-thumb, metric coefficients are in the range of 1 to 2, while English coefficients range from 1.8 to 3.6. If necessary, coefficients may be manually converted as follows:

English Coefficient = Metric Coefficient * 1.81
(The factor 1.81 is the square root of the number of feet per meter.)

The primary function of a constructed weir is to measure or control flow. Equations for the four primary types of weirs that highway designers may use are:

Broad Crested Rectangular Weir (long crest compared to crest width)

The FHWA **HDS 5** publication provides coefficient charts for application of the weir formula to roadway overtopping conditions. However, overtopping is not typically part of the design. The basic broad crested weir formula is $Q = CLH^{3/2}$ Refer 'H' is the same as HW in the FHWA figures shown on the next page, **HDS 5 Section 3.1.5** for additional analysis guidance on roadway overtopping. Weir coefficients can be specified as a function of head for increased accuracy. The width of the crest does not influence the coefficient as much as head once the broad crest is established at over approximately 0.5 ft width.

C_d = The overtopping discharge coefficient from **Figure 4.0.13.b**[16] for shallow, deep or the downstream submergence condition. Refer to HY8 help files or HDS 5 for more in-depth discussion.

HW_r = Upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown.

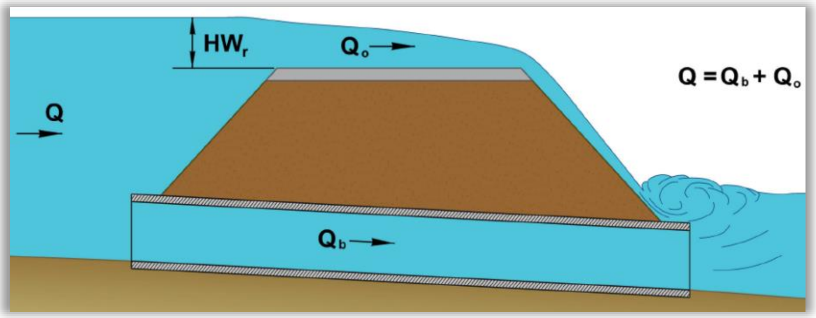


Figure 4.0.13.a Overtopping is not typically a design intent

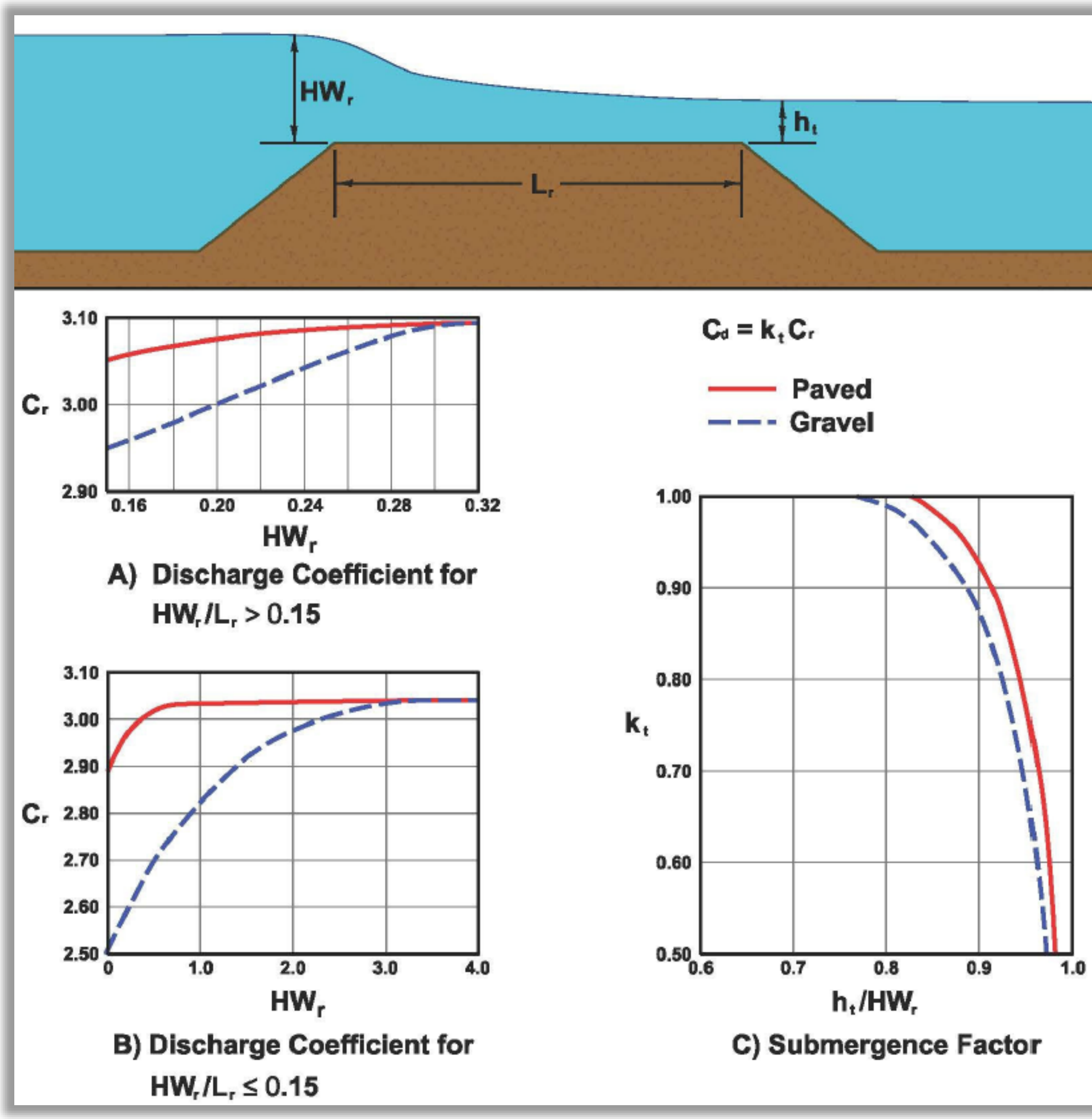


Figure 4.0.13.b Broad crested weir discharge coefficients

Sharp Crested, aka, Thin Plate, Rectangular Weir

Water flows cleanly over the crest, example is flow over dam stop logs, see **Figure 4.0.13.c**. The falling sheet of water below the plate is called the nappe, the weir discharge coefficient changes with the curvature of the nappe. Above the plate the water "draws down" because of acceleration and the velocity head can be measured.

$$Q = C[L-0.1(i)H]\, H^{3/2}$$

*H = Depth of water over the crest in feet
i = 0 for a suppressed rectangular weir
i = 1 for a contracted rectangular weir
Typical C value for this type of weir is 3.33 (which is much more efficient than broad crested)

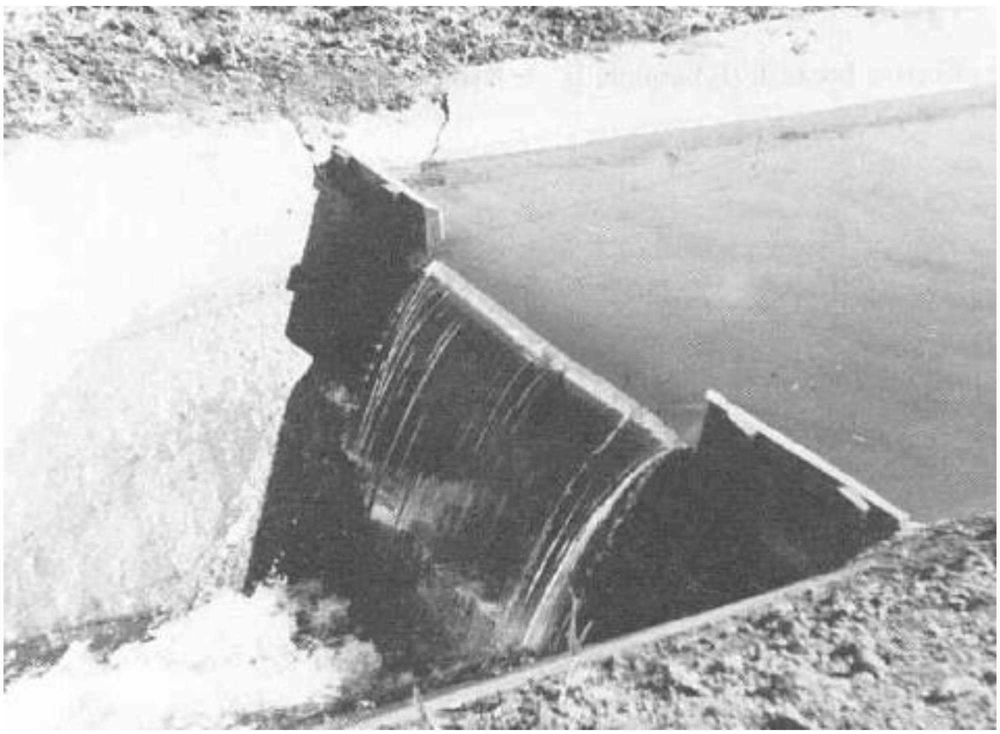


Figure 4.0.13.c Sharp crested weir

When approach conditions allow full contractions at both the bottom and the ends, it is a contracted weir. Whereas, when the sides of the channel match the ends, it is a suppressed weir, such as shown in **Figure 4.0.13.d.**

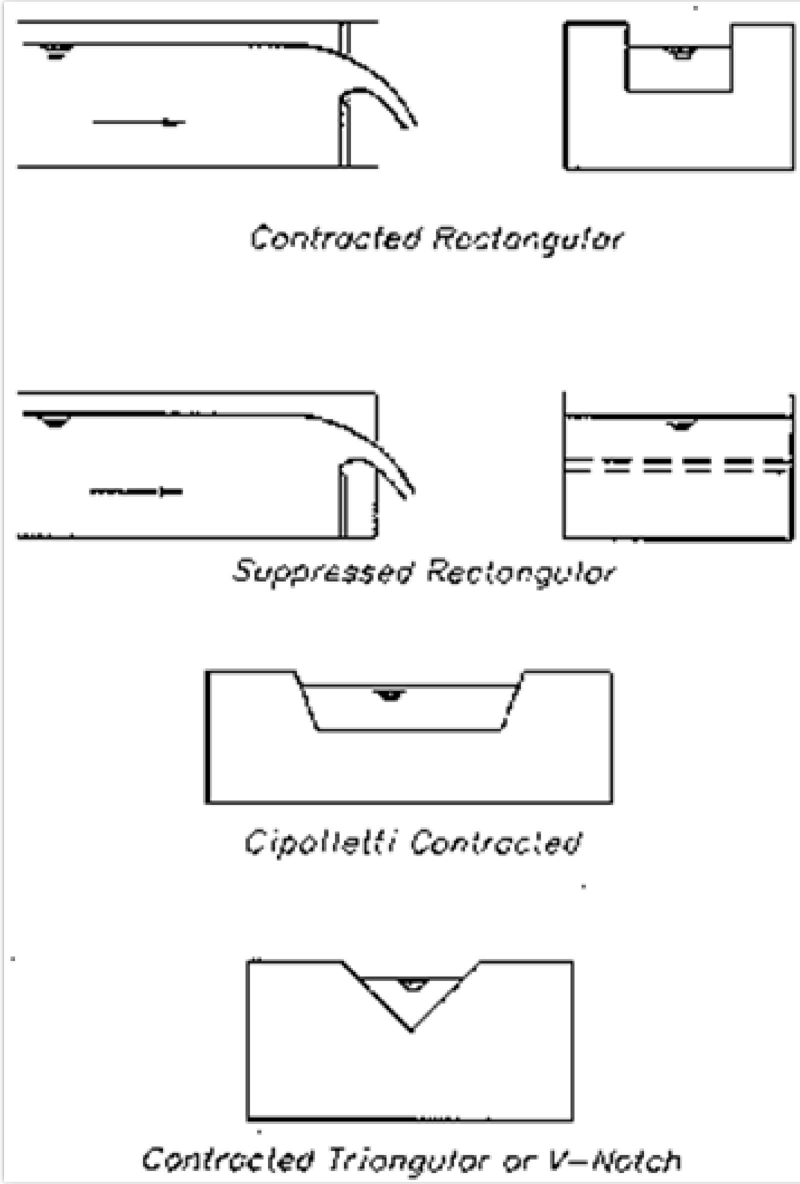


Figure 4.0.13.d Different types of sharp crested weirs
(USDA Water Measurement Manual)

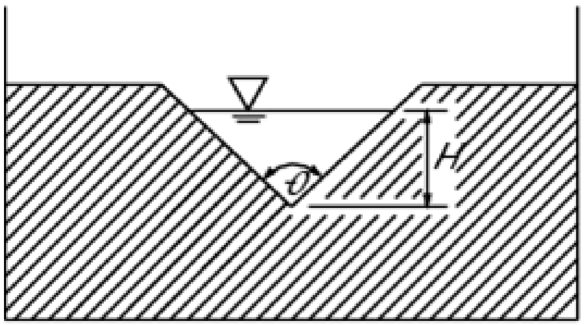
<u>V notch, aka, triangular sharp crested weir</u>

Preferred for measuring low flows, such as for outlet structures in a stormwater pond, because the head is sensitive to changes in flow.

$$\mathbf{Q = C(8/15)2g^{1/2} \tan \theta/2\ H^{2.5}}$$

where theta is the notch angle in decimal degrees and gravity is 32.2 fps.

C= 0.58 typically used for a 90 degree V-notch



Head (feet)	Weir Angle (degrees)					
	22.5	30	45	60	90	120
0.5	.611	.605	.596	.590	.584	.581
1.0	.593	.590	.583	.580	.576	.575
1.5	.586	.583	.578	.575	.572	.672
2.0	.583	.580	.576	.573	.571	.571
2.5	.580	.578	.574	.572	.570	.570
3.0	.579	.577	.574	.571	.570	.570

Figure 4.0.13.e V notch (triangular) weir

<u>Cipolletti weir (trapezoidal - similar to V notch only with L)</u>

$Q = 3.367LH^{3/2}$

L is the crest length (ft.) at the base of the trapezoid, H is the head on the weir crest (ft.) 1:4 side slopes are used to compensate for end contraction losses; typical C = 3.367.

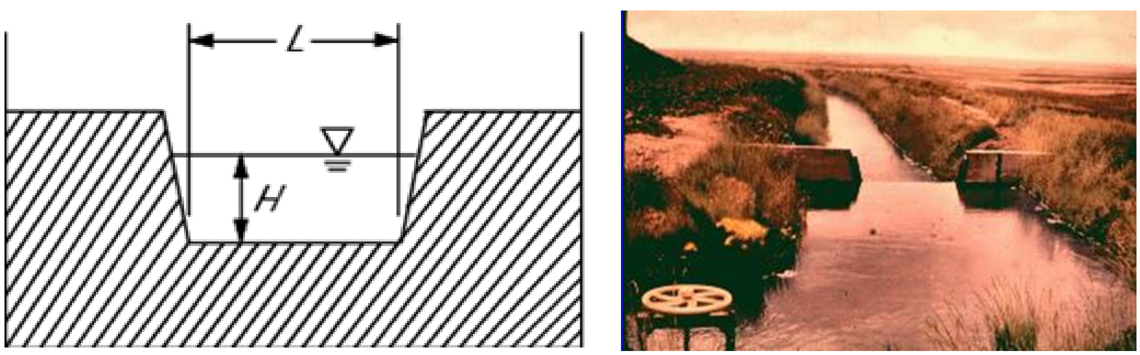


Figure 4.0.13.f USDA Manual on Flow Measurement

*HydroCAD & Bentley help files contain good information for weir flow.

<u>Baffle Design</u>

Principle: The predominate use of baffles is for retrofits in existing culverts, they can improve AOP potential if it is not practical to provide a similar function by natural means, or if used as a weir at the end of the tailwater pool for adequate backwater depth.

> **Practice:** In general, when setting baffle spacing and determining optimal dimensions for the weir it is preferred to use guidance from flume research and field tests. It is possible to model the flow through a culvert with baffles in the one dimensional flow HEC RAS program using in-line weirs (but this approach is not recommended).

Practice: Baffles will not be used in culverts less than 36" diameter.

Practice: Guidance from research is used for baffle design. Baffles are more commonly used for retrofits in culverts rather than new construction.

Baffle designs typically rely on physical model tests and scaling, when modeling with HEC RAS (using in-line weirs) one is moving further away from the actual research guidance, even if HEC-RAS is used in the research such as with the procedure detailed in CALTrans "Fish Passage Design for Road Crossings", Appendix F [25]. Comprehensive working documents were published by Chris Katapodis, P.Eng. and others in the early 1990s. Since hydraulic efficiency and optimum fish passage designs are mutually exclusive objectives, compromises must be effected that permit adequate fish protection with maximum economy.[31]

There is a large body of literature dating back 3 or 4 decades that documents the success and failure of AOP installations. There are different opinions about what is appropriate for new construction vs. retrofits and whether or not to design an AOP system for specific species. Brook Trout are a common species in New Hampshire that may serve as a target species. Consult with the Specialty Section, or an engineer with prior experience, and the BOE for more guidance and research references for baffle system designs. Consult with Fish and Game. Wetland rules in NH have criteria for critter crossings above and beyond the need for fish passage.

Practice: Minimum depth of flow for fish passage should be no less than 0.6 ft for the estimated annual migrating flow which can be determined by various means, refer to HEC 26, regional regression equations including StreamStats, or other scientific reference.

Practice: The distance between the inlet and the first downstream baffle should be between 0.5 W – 1.4 W for box culverts or 0.5D – 1.4D for circular culverts to have the least effect on HW (otherwise significant debris can accumulate at the inlet).

Pavement Drainage

HEC 22 is the primary FHWA publication for pavement drainage. These references contain charts, and example problems. Other sources for geometry, design curves, and research data are manufactures and universities.

Principle: The purpose of pavement drainage is to remove stormwater from the pavement and to minimize water and subsequent hydroplaning on the pavement.

Principle: The intensity of rainfall events, classification of highway, and traffic volumes are factors in selecting the design frequency for pavement drainage. Each revision of this manual should consider these criteria, capital costs, and nuisances/hazards for design frequency choice.

Principle: Increases in rainfall intensity are predicted by climate models and to some degree by observed hydrologic change.

If analysis using the Curve Number method is used, the results should be similar to that obtained using the Rational formula for small uniform pavement runoff areas. Geometric and structural details of the pavement drainage system are specified in the NHDOT Standard Detail Sheets. Design for runoff to gutters and through grate inlets is for relatively small flows. System components include the following: curb and gutter, grate inlets, catch basins, manholes and pipes.

Practice:

- The Rational Method is preferred for estimation of pavement runoff;
- Time of concentration should be a minimum of 6 minutes for consistency with rainfall data sampling intervals. Reasons for using less should be documented.
- Pavement drainage is designed for allowable spread for the 10 yr. storm frequency. Refer to design criteria in **Section 1, Table 0.a (Page 1-3)**.
- Off-pavement runoff should not mix with pavement runoff before entering the closed system. Divert off-pavement runoff into alternative drainage paths when practical, this is particularly important for cut slopes to minimize the amount of water carried by the gutter.
- Treatment practices should be placed, for pavement runoff, when off-pavement runoff has been separated.
- Shallow swale sections at the edge of the roadway pavement or shoulder should be utilized where curbs are not needed for traffic control or other reasons.

4.0.14 Hydraulic Analysis of Gutters

Principle: A composite triangular gutter typically ends at a vertical curb wall. Flow is controlled by the roughness **n**, transverse, and longitudinal slope components along the curb length, allowable water spread width T, and the distance T_b from the curb to the break in cross section slope. Inlets of width W intercept flow at determined locations.

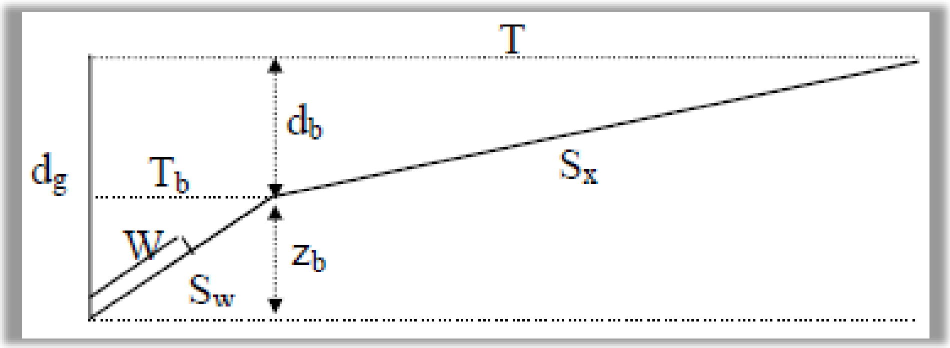


Figure 4.0.14.a Triangular gutter geometry schematic

Designers should refer to FHWA HEC 22 Urban Drainage Design Manual for background and techniques for possible configurations. The designer may choose to use the Curb and Gutter calculator within the Hydraulic Toolbox application available from FHWA. Calculations should be saved with drainage documentation in design project folders.

Principle: Modification of the Manning equation is necessary for use in computing flow in triangular channels because the hydraulic radius in the equation does not adequately describe the gutter cross section.

The capacity of the section with a depressed gutter is approximately 50 % greater than that of the section with a straight cross slope with all other parameters held constant. The effects of spread on gutter capacity are greater than the effects of cross slope and longitudinal slope. The relative effects of spread, cross slope, and longitudinal slope can be analyzed with the FHWA Hydraulics Toolbox.

In general, more than permissible spread should not be substantially greater in sags than it is at inlets on grade. Other alternatives for mitigation of grate clogging or snow curb include a high capacity type grate or a double grate. If an alternative solution is developed that is not covered in AASHTO or other DOT guidance, then the Specialty Section and Maintenance will need to review prior to final design and implementation.

Practice:

- Do not allow more than the permissible spread at sags and within critical areas of the pavement. These areas include, but are not limited to: intersections, slip ramps, lane merges, acceleration/deceleration areas, superelevation locations, ADA curb ramps pedestrian areas, and uphill from bridges. At least one CB will be located at the bottom of a sag. Depending on roadway classification and design considerations, an additional CB on either /both sides could be necessary. The spacing between the three CBs will set to prevent ponding of ½ the travelled way for roadway typical involving no shoulders.
- The permissible limit of spread is fully within the paved shoulders where 8 to 10 feet wide shoulders exist.
- In sections with no paved shoulders or paved shoulders less than 8 feet wide, spread limits must not encroach beyond one-half the traveled way in either direction of travel.
- In sag locations where greater than permissible spread may occur, such as where snow curb or grate clogging is likely, place flanking basins on either side of the main inlet.
- The AASHTO recommended transition guidance should be maintained. It is critical to intercept runoff prior to flat areas to prevent hydroplaning. A careful check of designs should be made to minimize the length of flat pavement sections in cross slope transitional areas, e.g. superelevation profiles.
- Consider increasing cross slope in sag and crest vertical curves and in flat longitudinal grades. Exceeding recommended cross slope by more than 0.005 ft/ft is not recommended because changes could be noticeable to drivers.
- Gutter flow from roadways should be intercepted before it reaches a bridge deck.
- Gutter grades should not be less than 0.3% for curbed sections, and not less than 0.2% in very flat profiles. Minimum grades can be maintained in flat terrain by use of a rolling profile or by warping the cross slope to achieve a rolling gutter profile.
- Inlets must be spaced in the gutter to avoid exceeding the design spread (T value).

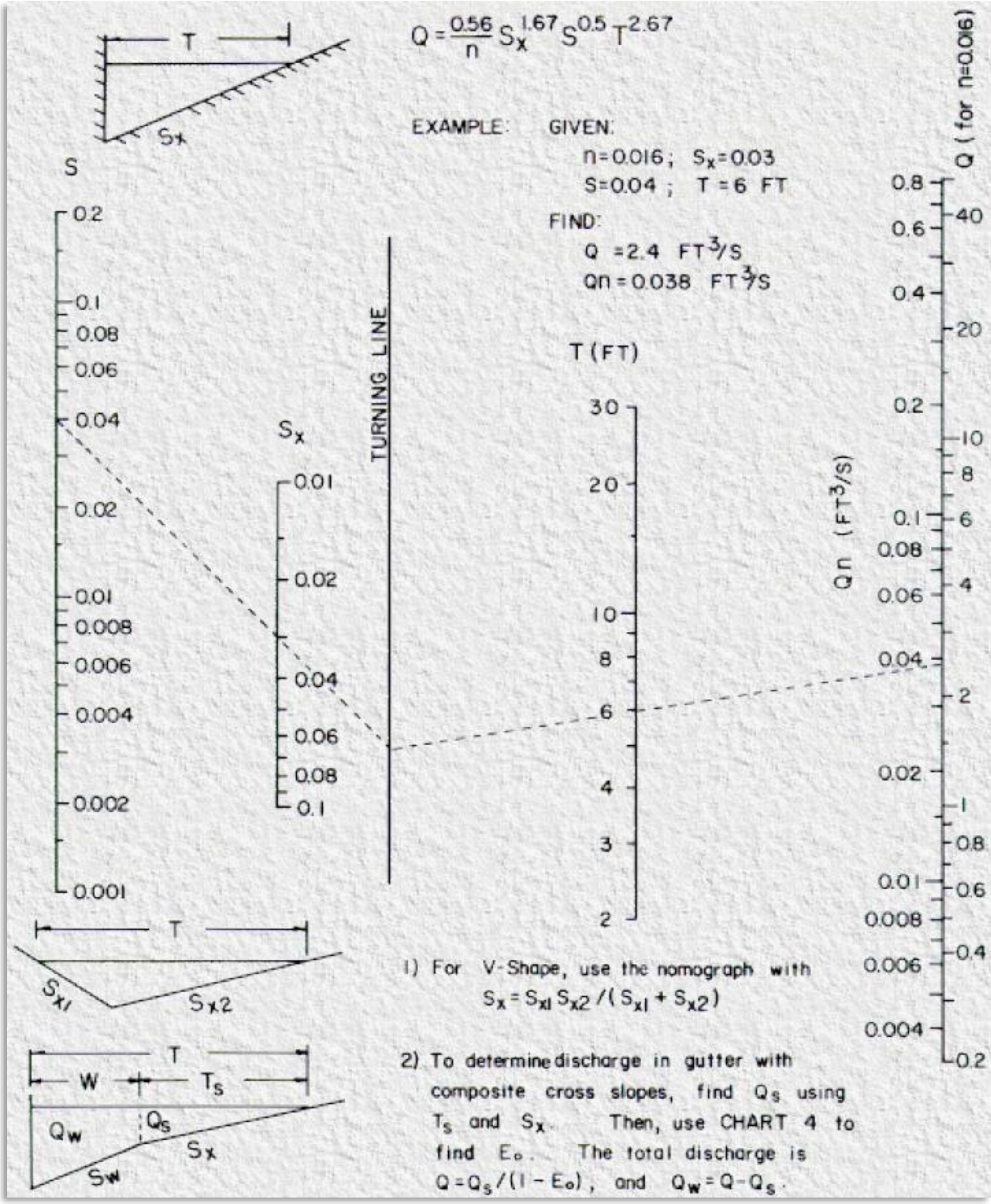


Figure 4.0.14.b Flow in triangular gutter sections

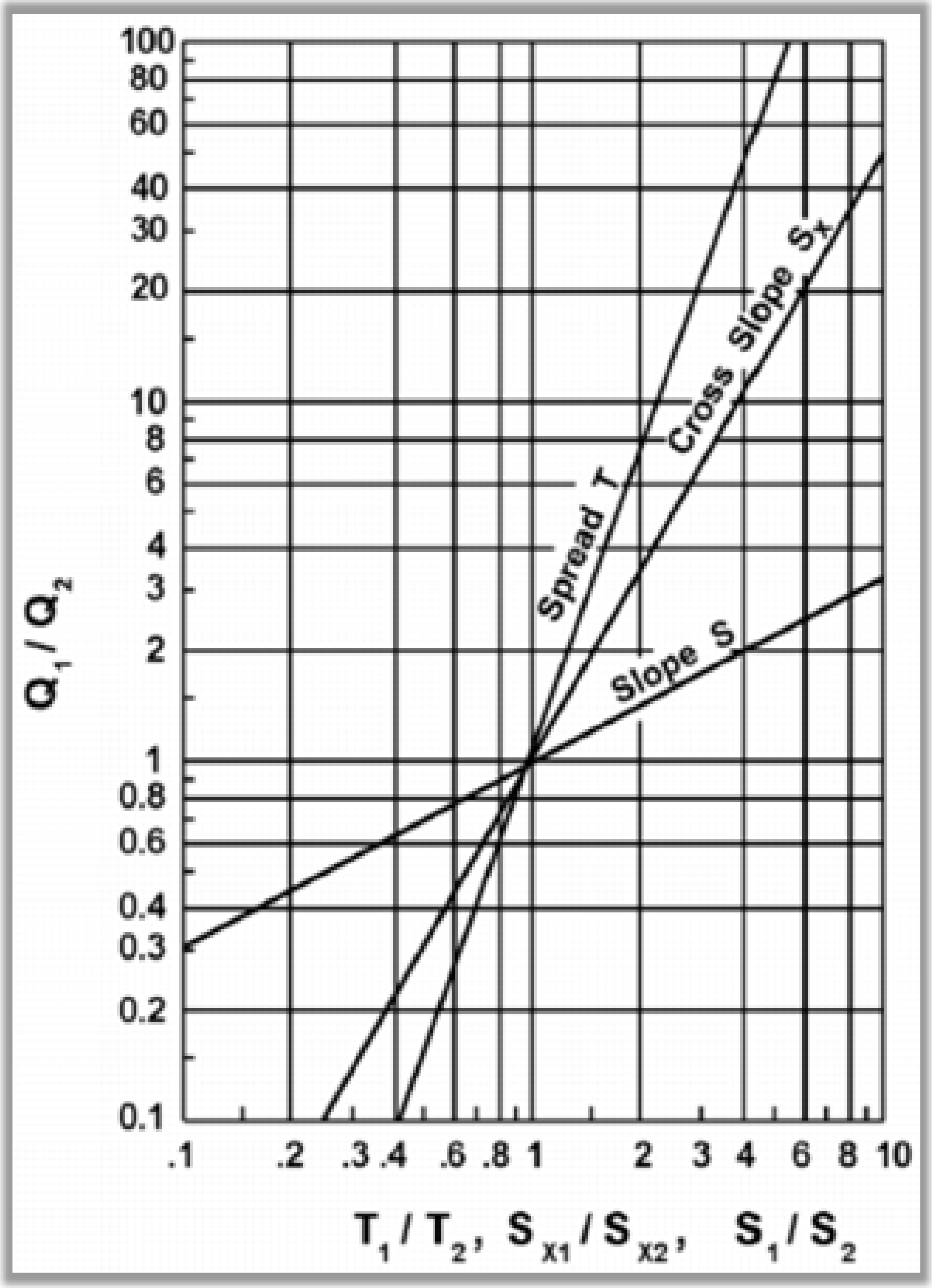


Figure 4.0.14.c Relative capacity factors

4.0.15 Hydraulic Analysis of Pavement Inlets

Standard gutter inlets are 2 ft x 2 ft square Type A and B. High capacity median grates are either 'Beehive' type or Type C. Curved vane inlets are sometimes used where high one direction flow is expected (Type E in **Table 4.0.15.a**). Type F curved vane grates are used where bicycle traffic is expected and there is a high flow in one direction (the open area is safe in all directions).

Table 4.0.15.a – Standard NH Grates by Type and Typical Location Used

Grate Standard (Type)	Utilization
A	On limited access roadways, ramps, and medians where bicycle traffic is prohibited.
B	In roadways, ditches, and medians where bicycle and pedestrian traffic is anticipated. Used for low, multi-directional flow on grades ~ < 3%.
C	Used in berm channels and at locations inaccessible to vehicle traffic, not widely used today. A Type G grate may be a better choice.
E	Where high, one-directional flow is expected (such as on a gutter slope >3%) and bicycle or pedestrian traffic is not anticipated.
F	Where high grate capacity is required on slopes ~ > 3% and bicycle or pedestrian traffic is anticipated.
G	In ditches and in other appropriate locations such as on top of outlet structures.

NHDOT designers assume that frontal flow is fully captured by grates. This assumption may not hold for grates on steep slopes greater than 6% or where splash over can occur. Accurate hydraulic performance curves for grate inlets improve designs and productivity to utilize said curves within stormwater software such as StormCAD or the FHWA Hydraulics Toolbox or similar. Mathematically, inlets are spaced so that design runoff spread matches the T value at each inlet. Optimization of design dictates that a certain amount of flow is allowed to bypass an inlet and flow to the next inlet. For example, it has been determined by research that at a slope of 6%, velocities are such that splash-over occurs on all except curved vane and parallel bar grates. Refer to the latest version of the Urban Drainage Design Manual (HEC 22). Most grate inlets are used in applications where less than 0.5 ft of head is anticipated and the depth on grate is rarely ever more than 3 ft. in the most extreme cases. Interception efficiency, $\mathbf{E_0}$ equals the ratio of captured flow $\mathbf{Q_w}$ over the total gutter flow, Q_g. $(\mathbf{E_o = Q_w / Q_g})$. Some of the capture width may bypass the inlet when velocity is high. A conservative estimate of 5%, meaning a 95% interception efficiency $\mathbf{R_f}$ may be used although this could be addressed in more detail if necessary. The resulting capture efficiency becomes $\mathbf{E=0.95E_0}$ and the resulting bypass flows are calculated as follows:

$\mathbf{Q_g=Q_i+Q_b = EQ_g+Q_b}$ $\mathbf{Q_b=(1-E)Qg}$ $\mathbf{Q_i=EQ_g}$

Principle: Grate inlets will intercept all of the frontal flow (gutter flow passing over the grate) if the grate is sufficiently long and flow velocity is low (or if in a sag location where there is no other route for water to flow).

As the gutter flow velocity increases, the grate capacity will decrease.

Principle: The location of a grate is either in a sag or on grade. For a sag location and a certain ponding depth all of the flow will eventually enter the grate if there is no alternative flow path. Conversely, an inlet on grade will have a flow-bypass component depending on the intensity of runoff, slope, spread, and grate length and open area type.

Practice: The type of grate chosen is based on the use restrictions in **Table 4.0.15.a**.

Practice: Model the actual open area and grate geometry with HydroCAD or similar if necessary.

Principle: Grate inlets act as weirs up to a certain water depth and are dependent upon the perimeter of the inlet. At greater depths the inlets will act as an orifice and capacity will be controlled by the inlet area.

Capacity in weir situation: $Q = C_wPd^{1.5}$

Practice: Reduce perimeter (P) by appropriate percent for the application, if any, to allow for clogging;

P = perimeter of the grate;
C_w = 3.3 in English units;
d = depth of water at the curb;

Capacity in orifice situation: $Q = C_oA(2gd)^{0.5}$

Practice: Reduce area (A) by appropriate percent for the application, if any, to allow for clogging;

A = clear opening of grate (ft^2);
C_o= orifice coefficient = 0.67;
g = 32.2 ft/s^2

The FHWA Hydraulics Toolbox or HydroCAD can be used for these calculations.
See also the NHI mobile flume video by James Schall: Grate Inlets.

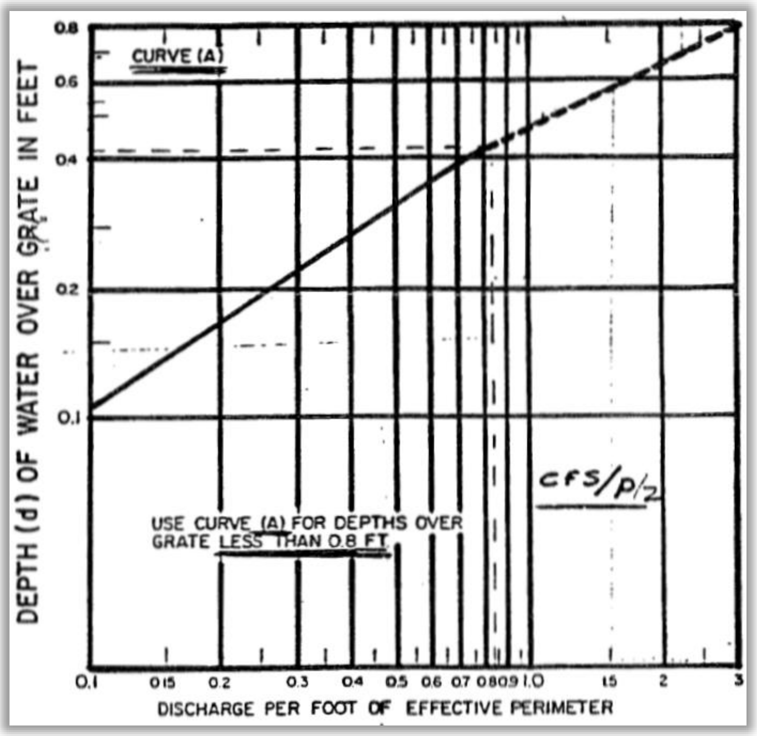


Figure 4.0.15.a Discharge Per Foot of Effective Perimeter

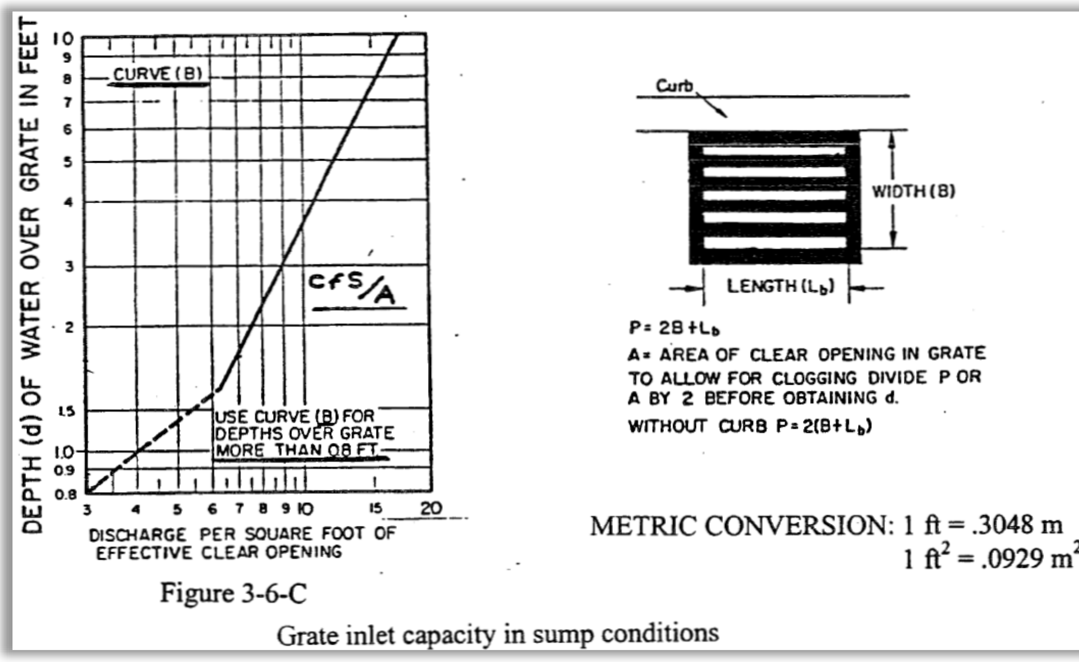
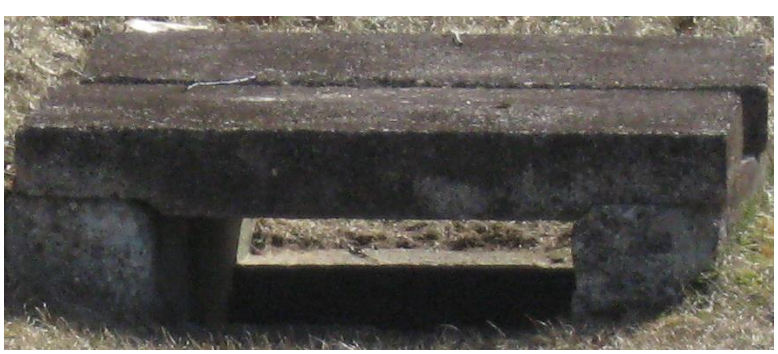
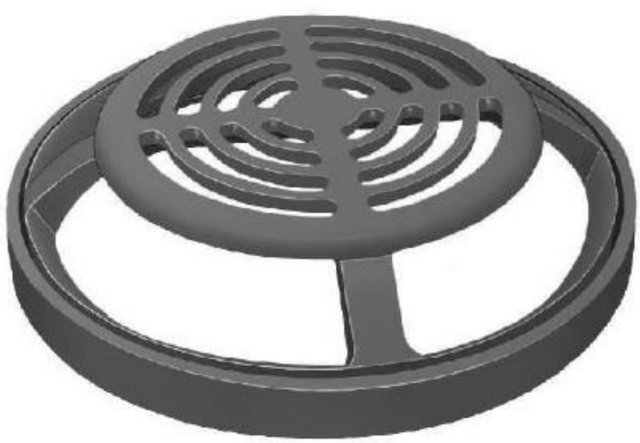


Figure 4.0.15.b Grate Inlet Capacity in Sump Conditions

Figure 4.0.15.a & **Figure 4.0.15.b** were created from the weir and orifice equations as a tool for sizing grates. The FHWA Hydraulics Toolbox is available for iterative calculations. HydroCAD is another effective tool.

Table 4.0.15.b – Grate Types and Images

Image of Grate	**Grate Type** Description
	**Type A** Nominal Dimensions = 2 ft x 2 ft Free Open Area = 1.73 ft^2 Perimeter = 8.0 ft. Capacity will decrease if the long opening is not installed parallel to the flow (most dangerous direction for bicycles). Debris capture and clogging will increase for grates installed in the wrong direction.
	**Type B** Nominal Dimensions = 1.9 ft x 1.8 ft Free Open Area = 2.55 ft^2 Perimeter = 7.4 ft.
	**Type C** Weir Flow

	**Type E** Free Open Area = 1.8 ft^2 Weir Perimeter = 7.3 ft
	**Type F** Free Open Area = 1.5 ft^2 Weir Perimeter = 7.9 ft
	**Type G** Typical high capacity ‘Beehive’ grate Free Open Area = 5.4 ft^2 Weir Perimeter ~10 ft

Notes:

- There is no ‘D’ grate in New Hampshire.
- Use of Reticuline Type B grates have been discontinued by NHDOT because of light weight and durability compared to conventional B grates. However, hydraulic performance curves for reticule type grates may continue to be useful to some degree.
- Type A, B, E, F, & G grates can be used within the clear zone because they are less than 4” high.

Practice:

- Use "Flanking Inlets" to intercept debris as the slope decreases and to act in relief of the inlet at the low point.
- Clogging factors for grates are not typically part of routine design calculations.
- Due to clogging potential, practical depth, and the expected velocity, grate inlet capacity for the 10 yr. design storm should be limited to 2 cfs.
- Design and placement of a grate must be suitable for use in areas where handicapped persons, bicycles, and concern for pedestrian safety may be present.
- Design storm drains on the assumption that each inlet intercepts all runoff that contributes to it, provided the inlet capacity is equal to or greater than the design runoff, and the spread does not exceed the grate width. Where this cannot be accomplished, by-pass flow is added to the flow entering the next downstream inlet.
- Where stormwater can be temporarily impounded safely, such as open space areas, and some parking lot areas, or LID rain gardens; an inlet grate can be used effectively to slowly release water and lessen the burden on a stormwater system.

4.0.16 Design Criteria and Control

Practice:

- Catch basins shall be placed at the bottom of sag curves with flanking catch basins upgrade from areas where concentrated flows need mitigation to minimize cross-pavement flow.
- The minimum grade of closed system pipes will maintain a minimum 10-year design flow velocity of 2.5 ft/s. where practical. Request the Specialty Section & Maintenance to review if the criteria are not met.
- Catch basin outlet pipes should be at least 3 inches lower than the lowest inlet pipe. Transitions in pipe size occurring within a drainage structure should be accomplished by keeping the crowns of the pipes equal. The objective is to maintain an energy grade line across the structures. In cases where significant tailwater develops greater than a 3 inch drop between inverts may be used. Utility conflicts may dictate adjustments to the basic transition guidance.
- A catch basin or manhole shall be placed wherever a change in grade or alignment of a storm drain occurs. A storm drain normally will have less than 300 feet separating a manhole, catch basin, or drop inlet.
- Drop inlets are typically only used where no pipes inlet to the structure (except underdrain) and where soils and other debris are not liable to wash in, typically at ditches or gutters for slope drain inlets.
- Minimum stormwater pipe sizes under roadways are as follows. These minimum criteria replace same in the 1998 manual on Page 1-3 (revised 3/19/02):
  - Perpendicular to pavement – 18 inches (3” greater) *
  - Parallel to roadway – 12 inches (same as 1998 manual) *
  - Slope drainage – 12 inches (same as 1998 manual) *

  **Bridge decks and existing low profiles might be exceptions to this guidance*
- Whenever a culvert greater than or equal to 15” diameter is introduced into a closed system, the system should be analyzed for the design frequency of a culvert from the point of introduction of the pipe inlet downstream to the outlet.

4.0.17 Catch Basins and Manholes

Principle: Modern catch basins (CBs) are typically modular structures, separate from the grate inlet which convey runoff into the closed stormwater system. If the outlet is higher than bottom of the cylinder, sump volume is formed that serves to collect sediment, debris and provide a degree of pretreatment for pollutant removal and water quality improvement. Routine maintenance is needed to maintain the sump.

Practice:

- Material and construction requirements for catch basins and manholes is covered under *Standard Specification Section 604* Standard Specifications;
- Regardless of the pavement drainage analysis, catch basins shall be spaced no more than 300 ft. apart for ease of maintenance and inspection.
- If the location falls in a cross walk, sidewalk ramp, intersection, driveway entrance, curb-cut or other similar location, the catch basin should be placed on the upstream side of the feature.
- Catch basins should be placed to capture the flows before they reach the intersections;
- On superelevated curves, catch basins should be placed to intercept storm runoff before allowing it to sheet across the highway; Placement of catch basins in curbed roadway sags shall conform to the following:
- At least one catch basin will be located at the bottom of a sag curve. An additional "flanked" catch basin on either or both sides could be necessary depending on roadway classification and design considerations, such as curbing or inlet geometry.
- Catch basins shall be placed in sags where snow curb is anticipated.
- Catch basins should be placed uphill of bridges to prevent freezing on the bridge.
- Outlet pipes from manholes and catch basins will not be smaller than the largest inlet pipe diameter.
- Minimum cover for closed system pipes is as follows:
    - Under Pavement 4 ft. (for avoidance of freezing not only structural integrity)
    - Other 2 ft.
    - 1 ft under driveways
    - If the minimum cover cannot be met, the required strength of the pipe should be increased by sleeving or other material enhancement such as specifying a manufacturer designed plastic pipe, using a higher class of concrete, or ductile iron pipe.
-

- The maximum pipe size for a standard 4 ft. dia. DI, CB or DMH is 30” for corrugated pipe or 24” for smooth pipe. For larger pipes and multiple pipes refer to **Table 4.0.17.a**.
- Structures with multiple inlet pipes must be checked for separation distances, **Table 4.0.17.b** is a general guide for core diameters, ASTM testing specs. for precast manufactures offer additional guidance based on pipe material and joining method. Typically, CAD is used to determine the clearance:
  - The horizontal distance between pipe coring limits shall be no less than 12”, measured along the inside of the structure.
  - The vertical distance between pipe coring limits shall be no less than 12”, measured along the inside of the structure.
  - The diagonal distance between pipe coring limits shall be no less than 12”, measured along the inside of the structure.
  - The minimum horizontal, vertical, and diagonal distance between pipe coring limits to an underdrain pipe shall be no less than 6”, measured along the inside of the structure.

Table 4.0.17.a – Sizing Catch Basin Information; Structure Dimensions

Structure Diameter	Wall Thickness	Base Thickness	Maximum Pipe Size	
			RCP	CMP
4 ft (CB or DI)	5”	6”	24”	30”
5 ft (CB, MH, DI)	6”	7”	36”	42”
6 ft (CB, MH, DI)	7”	8”	42”	48”
7 ft (MH & DI)	8”	9”	48”	54”
8 ft (CB, MH, DI)	9”	10”	54”	60”

Note: Structures greater than 8 ft require site specific design.

Table 4.0.17.b – Sizing Catch Basin Information; Inner and Core Diameter

Pipe Inner Diameter (inches)	RCP Core Diameter feet (inches)	Plastic Pipe Core Diameter feet (inches)
12	18 (1.5)	18 (1.5)
15	22 (1.8)	20 (1.7)
18	26 (2.2)	24 (2.0)
24	34 (2.8)	32 (2.7)
30	42 (3.5)	42 (3.5)
36	48 (4.0)	48 (4.0)
42	54 (4.5)	54 (4.5)
48	64 (5.3)	64 (5.3)
54	72 (6.0)	N/A
60	78 (6.5)	N/A

4.0.18 Closed Stormwater Systems

Closed drainage systems are primarily used in urban areas where roadside ditches are not practical. A storm drain can be a closed-conduit, open-conduit, or a combination of the two. These systems are organized into components for analysis by software or by hand calculations. Generally, computations are done using the FHWA Hydraulics Toolbox, HydroCAD, StormCAD, and similar, or with hand calculations for checks and unique circumstances. The components of a closed stormwater system include: pipe runs, a.k.a. "links" (main trunk line & collectors), catch basins, manholes (nodes), gutter inlets, and subcatchments used to quantify surface runoff (refer to Section 3, Surface Hydrology). References used by NHDOT designers for closed systems include:

- FHWA Urban Drainage Design Manual, HEC 22, 3rd edition, Sept. 2009 or latest rev. (HEC 22 replaced HEC 12).
- FHWA-RD-79-106, Bicycle – Safe Grate Inlets Study, Vol. 4, February 1980
- Grate Manufactures
- AASHTO Model Drainage Manual Chapter 13, 2014 or latest;
- AASHTO Drainage Guidelines Chapter 9, 2007 or latest.

Three types of flow conditions can occur in closed conduits:

- Open channel flow
- Gravity full-flow
- Pressure flow (not a design option for new construction)

Practice:

- Typically closed systems have limited capacity and are designed for the 10 yr. event.
- The gravity full flow condition is usually assumed for storm drain design.

NHDOT uses open channel flow to design pipe runs in closed systems. **Figure 4.0.18.a** shows that design efficiencies can be realized by sizing pipes for open channel flow using the fact that maximum capacity in a circular pipe actually occurs at a depth equal to **0.938D**, where D equals depth of pipe. The condition corresponding to a flow depth equal to the diameter of the pipe has a capacity of approximately **0.83D**.

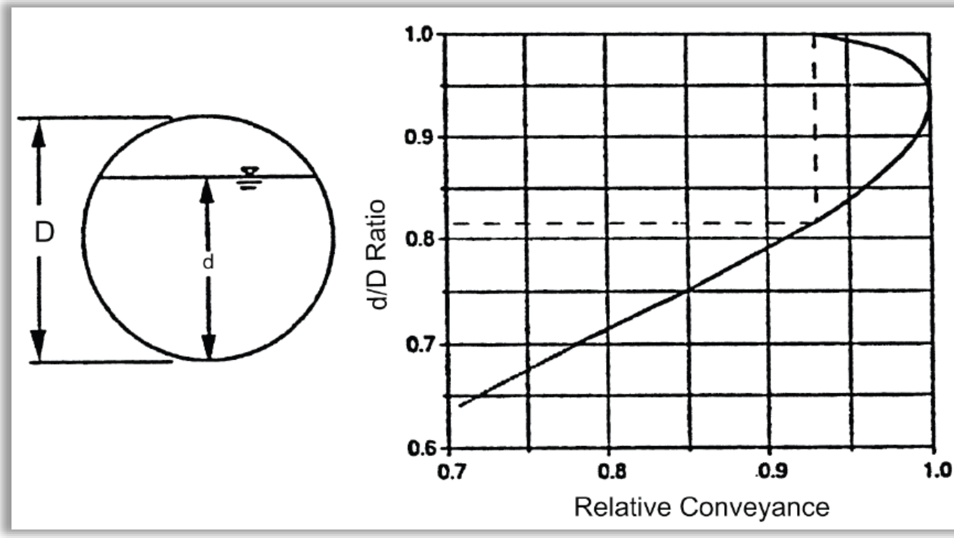


Figure 4.0.18.a Relationship between relative conveyance and d/D ratio

Peak flow occurs between 89 to 94% of the height of a circular conduit, and the average velocity flowing one-half full is the same as gravity full flow.[14] If the extra capacity is not significant because adequate elevation drop exists to maintain free surface flow, then designing for full flow conditions can be done.

For Manning's equation in circular pipe flowing full, the 1.486 constant (λ) converts the formula to English units (ft). The 2/3 exponent for R_h is Robert Manning's approximation of Henri-Emile Bazins' experimental work on artificial channels:

$v = \lambda R_h^{2/3} S^{1/2}/n$	$A = \pi D^2/4$
$k = (0.46/n)D^{8/3}$	$P = \pi D$
0.46 is derived from 1.486 $(\pi/4^{5/3})$	$R_h = D/4$
D = pipe diameter in feet	S=Slope

$$\mathbf{Q = kS^{1/2}}$$

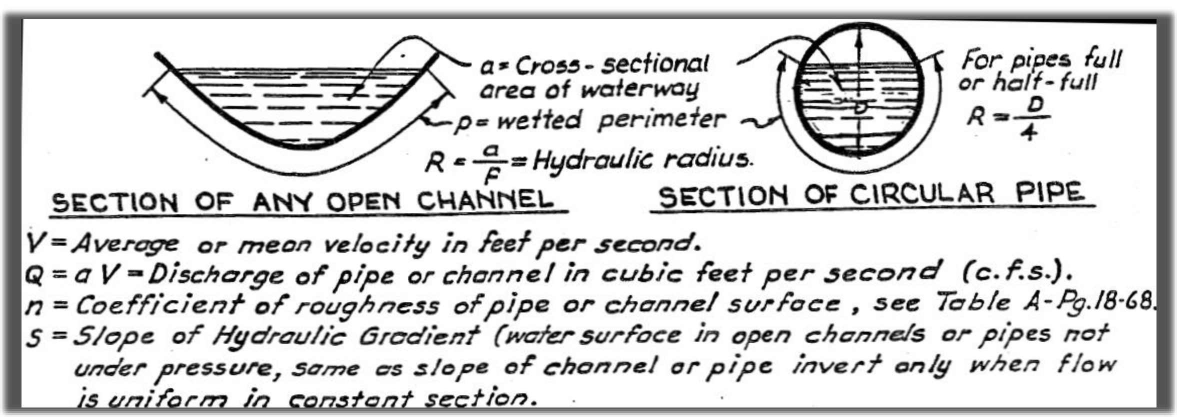


Figure 4.0.18.b Cross-sectional parameters of open channel flow and circular pipe

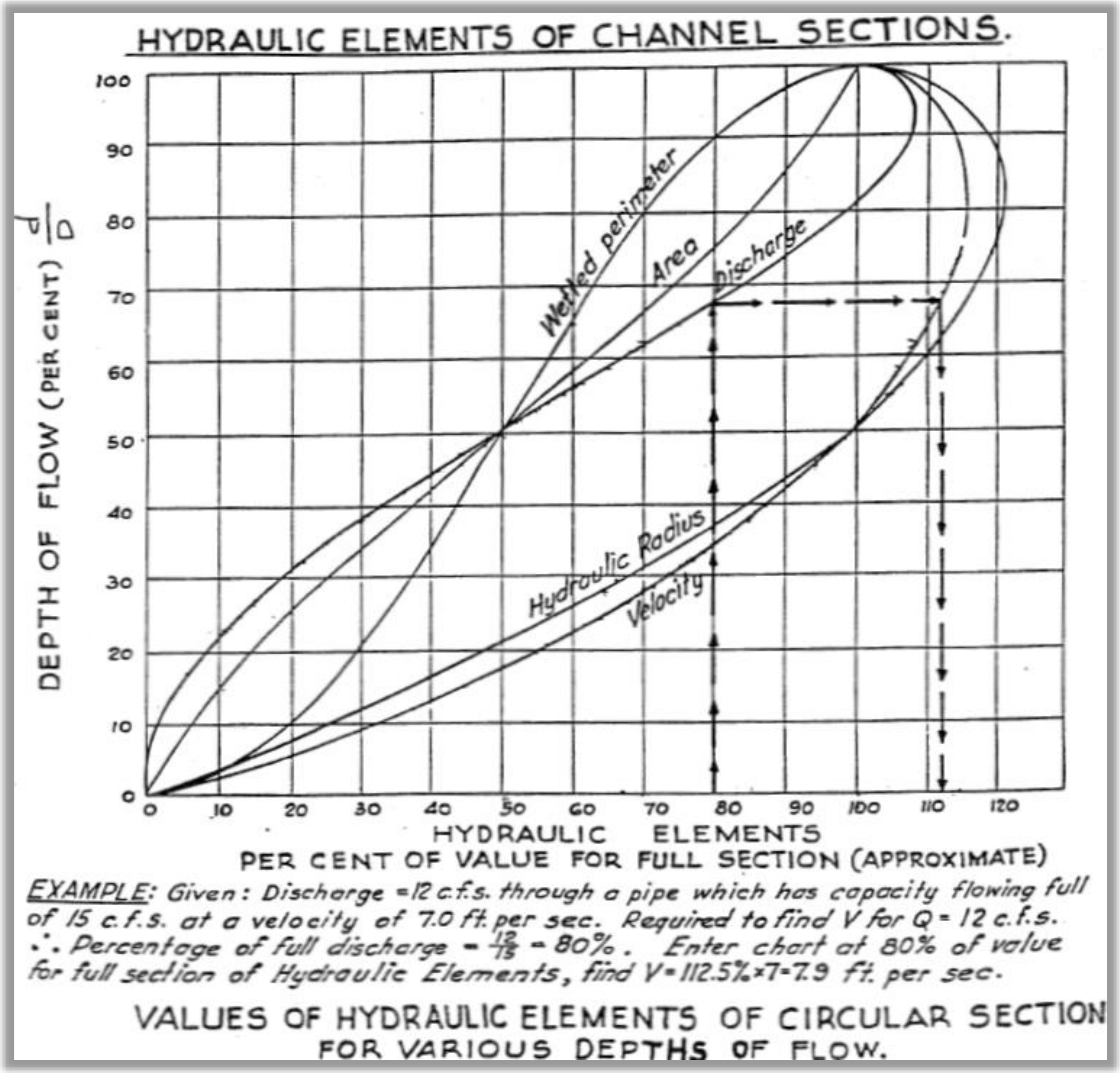


Figure 4.0.18.c Hydraulic elements of channel sections (1998 NHDOT manual)

4.0.19 Energy Grade Line (EGL) & Hydraulic Grade Line (HGL)

The HGL is used to determine the acceptability of the system by establishing elevations along a gradient that the water elevations will rise to for the evaluated flood frequency (usually the 10 or 25 yr. design storm). Pressure flow exists when the HGL rises above the crown of the pipe, otherwise open channel flow calculations are used. Typically, stormwater systems are evaluated and designed with CAD programs. It is becoming more common to design and model systems with 3d stormwater design applications. The intent of this section is to provide general guidance that supplements more in-depth manuals such as, the HEC 22, or software help files.

Principle: Prior to undertaking EGL/HGL analysis the designer has already completed sizing of pipe runs, or an existing system is being studied. To size a closed system, the designer starts from the upstream side and proceeds downstream, once the system is initially sized based on the upstream to downstream analysis, the task becomes one of determining the effects of tailwater (backwater), friction and pressure. The designer starts from the downstream end and works upstream for the tailwater part of the EGL/HGL analysis.

Usually it is helpful to calculate the EGL first then subtract the velocity head (V^2~~/**2g**)~~ to obtain the HGL

> **Practice:** Prior to computing the HGL, all losses in the pipe runs and junctions must be estimated.

Use HEC 22 methodology. If more guidance is needed refer to the most current AASHTO Model Drainage Manual.

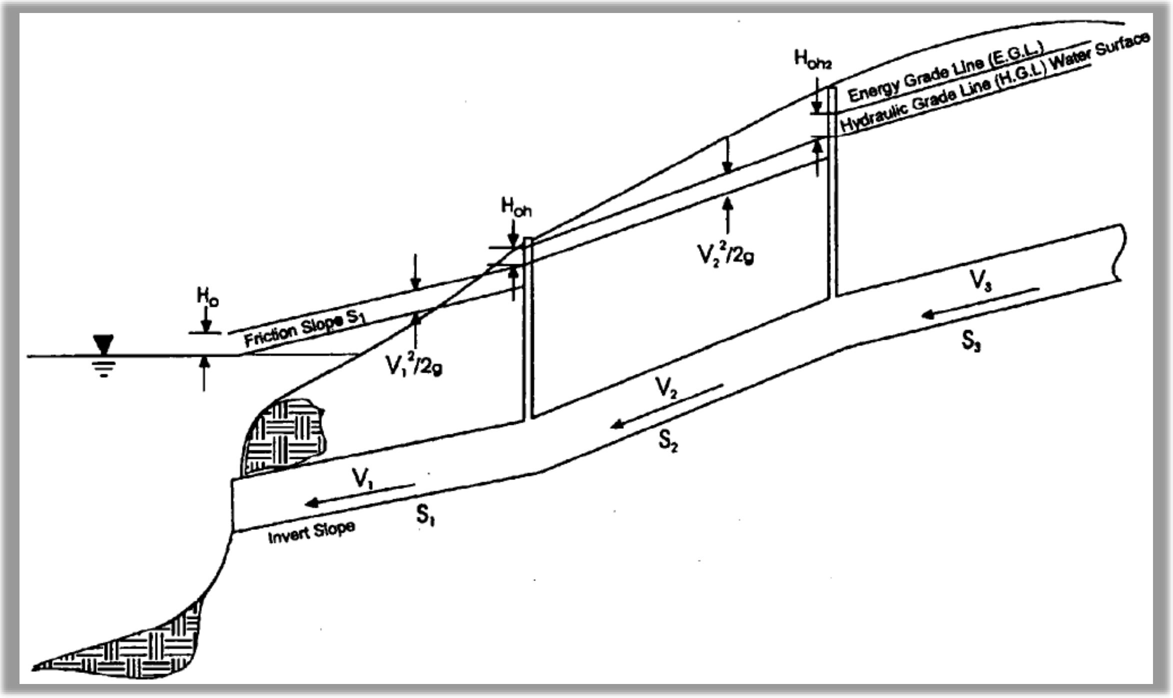


Figure 4.0.19.a EGL/ HGL diagram

Principle: Pipe friction losses are calculated using the equation $H_f = S_f L$, where S_f is the friction slope which is also the slope of the HGL for a particular pipe.

Principle: Pressure is equivalent to depth of flow for free surface conditions whereas it is above the pipe when the system is under pressure. Flow energy to produce pipe conveyance is from a pressure greater than atmospheric pressure when there is no free surface within the pipe runs. Pipes under pressure are very sensitive to increases in flow and the HGL is more important than in systems that have a free surface in the pipe. Blow-outs or surging of inlets can occur in pipes that are under pressure.

> **Practice:** NHDOT designs closed systems conservatively to operate with a free surface for the 10-year storm. Existing systems that do not have a free surface within all the pipe runs for the 10-year storm have generally been affected by urbanization, groundwater, or the systems are constructed on a nearly flat gradient in areas that did not present an economically viable alternative.

Procedure: The designer begins EGL and HGL calculations at the downstream end of the system and then methodically works upstream. The purpose is to ensure the storm drain system does not inundate, adversely affect inlets, manholes, or other appurtenances. The design procedure is based on the assumption of a uniform HGL within a conduit.

- Tailwater depth must be established first. For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth of the outlet. The tailwater may also occur below the critical depth, however, the recommended starting point for the HGL is either the tailwater elevation produced by the downstream channel or the average of the critical depth and the height of the conduit, $(d_c+D)/2$, whichever is greater.[14] Note that frictional resistance can be caused by tailwater external from the conduit or by roughness elements or length of the conduit.

**Tip:* *If the slope of the upstream pipe run is greater than 2% it is likely that influence from downstream does not continue through the upstream pipe because the upstream pipe will contain supercritical flow, which would normally start the process over with a new gradient controlled by the free flowing upstream pipe.*

Principle: In subcritical flow, pipe and juncture losses are summed to determine the upstream EGL levels. If supercritical flow occurs, pipe and access losses are not carried upstream. Reaches are defined between hydraulic structures and/or grade breaks. The designer works upstream on a reach-by-reach basis. Many storm drain systems are designed to function in a subcritical flow regime. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream conduit to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime.

Principle: If the friction slope is steeper than the pipe slope, the potential for pressure flow exists. The Department does not design for pressure flow with new construction. If the friction slope is less than the pipe slope, partial flow may occur (depends on downstream tailwater).

Practice:

- Friction losses are calculated using the Manning equation and by assuming a design flow.
- At the location where the pipe transitions to partial flow, normal depth calculations may be used to estimate hydraulic conditions.
- Form losses for each hydraulic structure should be calculated using the methods described by HEC-22

After completing the HGL/EGL calculations, the designer should evaluate the system using a higher check flood, such as the 25 yr. storm, and adjust to reduce cost and risk as appropriate. If the HGL is too high in a given reach, the pipe size or slope will have to be increased which will require recalculation of the HGL. The designed system will operate near gravity full flow, *(may not be near-full due to minimum pipe size requirement)* however, if surcharging (pressure flow) is acceptable (typically it is not), the pipe sizes can be reduced and the system reanalyzed. Contact the Specialty Section if needed.

Culvert Design

In accordance with NH statutes, the term "culvert" generally includes crossing structures that have a clear span or inside diameter less than 10-ft., measured along the centerline of the roadway.

However, for *multiple* culvert structures, each of which have a clear span less than 10-ft. measured along the centerline of the roadway, and the distance between the culverts is one-half the diameter or less of the smallest culvert, and an overall combined span of 10-ft. or more, then the total combined structure is considered a bridge, not a culvert.

The FHWA sponsored HY8 program and/or the methods defined in HDS 5 are used for typical culvert analysis using headwater and tailwater. Example calculations are provided in FHWA publications (HDS 5, HEC 26, and others). There are several videos by James Schall that illustrate culvert performance in a mobile flume, such as: Culvert Hydraulics Culverts shall be analyzed for various types of inlet and outlet flow control and evaluated for Aquatic Organism Passage (AOP) consistent with the NHDES Stream Crossing Rules and other guidance such as HEC 26. Consider the AASHTO Model Drainage Manual Chapter 11 for end treatments and other design guidance. See HDS 5 for information on grate design at culvert inlets. The basic principles of Culvert Liners was covered in another video by NHI.

Principle: To the extent practicable, culverts shall be constructed and aligned to fit natural channel profiles and bank forming cross sections.

Practice:

- All computations for culverts will be documented and design assumptions shall be explained. Pre and post calculations will be documented for all proposed culverts;
- Erosion prevention and sediment control will be evaluated for all culverts.
- Minimum pipe culvert cover for roadways types are:
  - Under pavement – 4 feet (some exceptions for existing profiles requiring substantial cost for compliance)
  - Under drives – 1 foot
  - Under grassed area – 2 feet
- Minimum pipe culvert sizes under main roadways are as follows:
  - State-Aid Highways – 18 inches (see guidance for bridge decks **Section 4.0.16**)
  - Primary and Secondary Highways – 18 inches
  - Interstate Highways – 24 inches
- Driveways – 12 inches (if adequate for ditch flow)

- Where the minimum cover requirements specified are not met; the strength of the pipe should be increased by sleeve or other material enhancement such as specifying a manufacturer designed plastic pipe, using a higher class of concrete, or ductile iron pipe.
- In mountainous terrain prone to flash runoff or areas containing a high percentage of impervious area or HSG C & *D* soils, culvert pipe sizes and underdrain requirements will be determined through a multi-disciplined review including Hydraulics and Geotechnical aspects.
- End sections shall not be used at pipe inlets that operate under high headwater conditions (vortex could develop); Headwalls will be used at these locations.
- Refer to **Table 4.0.21.a** for guidance on headwater for routine culverts in inlet control.
- In general culvert slopes should match the profile of the upstream and downstream channels, minimum slopes should not be less than 0.3% where practical.
- Equalizer culverts between wetlands, or ponds can be set level at 0%.
- Maximum culvert slopes should not be greater than 6% for RCP and 8% for CMP, consider energy dissipaters where maximum slopes are exceeded.
- Omit headers on all culverts ≤18”, except where there is an active stream.
- End sections should be used instead of headers at pipe outlets ≤ 48” where field conditions permit.
- See Chapter 6 (6-8 to 6-13) of the Highway Design Manual for guidance on pipe materials.

*Refer to Section 4.3.3 for guidance on energy dissipation.

4.0.20 Tailwater (TW)

Principle: The depth of water above the invert at the downstream end of a culvert for any flow rate is called tailwater. Tailwater can be estimated with a rating curve, a fixed water surface, or with known water surface elevations. It may be caused by an obstruction in the downstream channel causing impoundment or by the hydraulic resistance of the channel (typically determined using a normal depth calculation derived from the Manning’s equation).

Practice: Backwater calculations from the downstream hydraulic control feature are required to define tailwater. When appropriate, normal depth approximations may be used instead of backwater calculations.

4.0.21 Headwater (HW)

Principle: Energy takes the form of increased water depth on the upstream side of a culvert. Headwater depth is measured from the invert of the inlet to the water surface.

Principle: Analysis of steady varied flow or conveyance from cross section to cross section of the stream, such as determined with HEC-RAS, is less important when ponding occurs (because the approach velocity can be considered to be zero).

Principle: When pooling occurs at the inlet of culverts for design flows, attenuation happens, meaning that water flowing downstream is metered by the culvert unless excessive headwater results in flows that overtop the roadway.

Practice: The maximum allowable headwater depth of flow immediately upstream from a pipe culvert shall be controlled by the following:

- Hazard to human life
- Damage to stream & floodplain environment
- Damage to adjacent property
- Damage to the culvert and the roadway
- Encroachment on roadway structural material
- Traffic interruption

As a guide, **Table 4.0.21.a** may be useful under "normal conditions", however, the stream crossing rules must be considered.

Table 4.0.21.a – Headwater Criteria

Pipe Diameter	Maximum Allowable Headwater
18 – 30 inches	2 times the pipe diameter
36 – 48 inches	1.5 times pipe diameter
Greater than or Equal to 54 inches	1 times the pipe diameter

Notes:
- *Adequate freeboard must be provided for culverts prone to debris*
- *Vertical rise rather than diameter controls HW for shapes other than circular*

4.0.22 Energy Dissipation & Outlet Channel Stability

Principle: Since a culvert usually constricts the available channel area and roughness is less than the natural channel, flow velocities in the culvert are higher than in the channel. If not properly designed for, the increased velocity may cause streambed scour and bank erosion in the vicinity of the culvert outlet. The culvert design must not have an adverse impact on the stability of the downstream channel.

Principle: End Treatment for scouring associated with predictable higher velocities is necessary for many closed or "4 sided culverts". An open bottom culvert requires design for scour within the culvert and below the culvert in addition to that which is provided for at the outlet.

Hydraulic jumps are good energy dissipaters. Tailwater is required. A range of Froude numbers is used to classify and help select suitable dissipaters. Normal tailwater pools provide good plunge flow that may be adequate for energy dissipation for many existing culverts. For in-depth procedures and design criteria for additional outlet protection refer to HEC 14, Hydraulic Design of Energy, Dissipaters for Culverts and Channel. See also NHI video by James Schall: Energy Dissipaters.

Practice:

- Material and construction requirements for outlet stone protection is covered under *Standard Specifications.*
- Use HEC 14 to design internal or external energy dissipaters, Chapter 12 of the AASHTO Model Drainage Manual provides guidance to narrow down the design procedures in HEC 14.

Table 4.0.22.a – Energy Dissipaters and Limitations

HEC-14 Chapter	E. Dissipater Type	Froude Number[7] (Fr)	Allowable Debris[1]			Tailwater (TW)
			Silt/ Sand	Boulders	Floating	
4	Flow transitions	N/A	H	H	H	Desirable
5	Scour hole	N/A	H	H	H	Desirable
6	Hydraulic jump	> 1	H	H	H	Required
7	Tumbling flow[2]	> 1	M	L	L	Not Needed
7	Increased resistance[3]	N/A	M	L	L	Not Needed
7	USBR Type IX baffled apron	< 1	M	L	L	Not Needed
7	Broken-back culvert	> 1	M	L	L	Desirable
7	Outlet weir	2 – 7	M	L	M	Not Needed
7	Outlet drop/weir	3.5 – 6	M	L	M	Not Needed
8	USBR Type III stilling basin	4.5 – 17	M	L	M	Required
8	USBR Type IV stilling basin	2.5 – 4.5	M	L	M	Required
8	SAF stilling basin	1.7 – 17	M	L	M	Required
9	CSU rigid boundary basin	< 3	M	L	M	Not Needed
9	Contra Costa basin	< 3	H	M	M	< 0.5D
9	Hook basin	1.8 – 3	H	M	M	Not Needed
9	USBR Type VI impact basin[4]	N/A	M	L	L	Desirable
10	Rip-rap basin	< 3	H	H	H	Not Needed
10	Rip-rap apron[8]	N/A	H	H	H	Not Needed
11	Straight drop structure[5]	< 1	H	L	M	Required
11	Box inlet drop structure[6]	< 1	H	L	M	Required
12	USACE stilling well	N/A	M	L	N	Desirable

[1] Debris notes: N = none, L = low, M = moderate, H = heavy
[2] Bed slope (So) must be in the range of 4% < So < 25%
[3] Check headwater for outlet control
[4] Discharge, Q < 400 ft^3/s and Velocity, V < 50 ft/s
[5] Drop < 15 ft (4.6 m)
[6] Drop < 12 ft (3.7m)
[7] At release point from culvert or channel
[8] Culvert rise less than or equal to 60 in (1500 mm)

(Source: HEC-14)

Roadway Underdrain Guidelines

Upon request by the Bureau of Highway Design, the Geotechnical Section in the Bureau of Materials and Research will complete a geotechnical engineering evaluation of subsurface soil and groundwater conditions, and prepare a Geotechnical Engineering Design Report for the project providing recommended locations and type of underdrain with respect to the proposed project improvements. Standard underdrain configurations should be applicable in most instances where underdrain is needed, but there are situations where variation from the standard application may be required. Any unusual underdrain need or design should be referred to the Bureau of Materials and Research for review and concurrence. Non-standard designs should be included in the individual project's drainage documentation.

Practice: Material and construction requirements for underdrain is covered under *Section 605 of the Standard Specifications:* Standard Specifications

When considering existing underdrain removal or abandonment, Highway Design should evaluate project specific issues that consider:

- Depth of the underdrain from the proposed roadway surface;
- Underdrain location in regards to proposed roadway alignment, under the pavement or existing traveled way
- Potential issues with leaving abandoned underdrain in place, such as surface subsidence if the pipe corrodes and collapses in the future (observed with metal underdrain pipes). Underdrain that is greater than 6" diameter is typically filled if left in place;
- Presence of contaminated groundwater that could migrate through an abandoned system, and;
- Disturbance to historic properties or utilities with removal.

Highway Design should seek input from the respective DCE (District Construction Engineer) in the Construction Bureau if there are any underdrain construction concerns, especially if removal or abandonment of existing underdrain is anticipated.

Standard Underdrain

Principle: Standard underdrain is 6 inch diameter pipe, with perforations (weep holes) distributed uniformly or partially around the pipe circumference. When perforations only exist on one side of the pipe, the pipe should be installed so the perforations are positioned in the lower side of the pipe circumference. Longitudinal underdrain should be configured as a standard underdrain. It is typically located at the outside edge of the roadway and parallel to centerline. Refer to the standard typical sections in Volume 2 of the Highway Design Manual. Longitudinal underdrain is used in segments of roadway through cut areas, either soil or bedrock, or where groundwater and/or soil drainage characteristics indicate a saturated section may occur under normal conditions. There are circumstances where underdrain is appropriate in fill sections (shallow fills, fills over in situ soils which are not free draining, etc.). Longitudinal underdrain is generally not needed in roadway segments with deep groundwater levels, in segments with free draining subgrade soils, or in embankment fill segments.

If there is excessive roadway width (3 or more travel lanes or highway ramp merge segments) and longitudinal underdrain is recommended at both outside edges of the roadway, then additional longitudinal runs of underdrain may be needed under the travel way.

Practice:

- Run lengths consisting of 6-inch diameter pipe should be less than 600 feet long. Underdrain pipe over 600 feet long should be increased to 8-inch diameter for the entire length, but lengths should not exceed 1,200 feet without an outlet.
- Underdrain shall be designed with a 0.5% minimum drainage profile, with no sags (low points) trapping water in the individual runs of underdrain. If an adequate outlet for the underdrain cannot be provided, then the underdrain should not be installed.
- Underdrain is typically placed at either 6 feet below the pavement or 2 feet below the bottom of the sub-grade (whichever results in the greater depth). Refer to the standard typical sections Volume 2 - Highway Design Manual and Standard Plan sheet HW-3.
- Elbow joints (45 degrees, typical) are used to redirect an underdrain pipe from under the roadway to a nearby basin outside the edge of pavement. (See Standard Plan Sheet DR-4)
- A 32-ft maximum separation between parallel runs of longitudinal underdrain should be used as a guide when determining how many additional longitudinal runs are needed in wide roadway segments. Use interstate typical and maintain 4 ft. o/s of the travelled way in most cases. Underdrain below pavement with traffic could be configured as aggregate underdrain.

- A run of longitudinal underdrain should be considered along the boundary where a ramp and highway join if excessive roadway width is created. This would be in addition to any longitudinal underdrain recommended on the outside edge of the ramp, so there could be more than one run of longitudinal underdrain in ramp merging segments of roadway. (Underdrain below pavement with traffic could be configured as aggregate underdrain.)

Aggregate Underdrain

Principle: Aggregate underdrain is a more effective underdrain system than standard underdrain and is used for more difficult groundwater conditions. There are two configurations of aggregate underdrain, called Type 1 and Type 2. Type 1 aggregate underdrain consists of a trench filled with a uniform sized stone, which provides a medium full of voids for the collection and conveyance of groundwater. The stone-filled trench is surrounded by a non-woven geotextile fabric to minimize the amount of fines entering the voids in the stone fill, which could clog the underdrain or cause land subsidence (sink holes) over time. A Type 2 aggregate underdrain is similar in construction to a Type 1 aggregate underdrain, except that an underdrain pipe is installed within the stone filled trench (the invert of which is usually 6 inches off the bottom) to collect and direct the groundwater to an outlet.

Transverse underdrain (placed to cross the roadway centerline) is typically configured as an aggregate underdrain. Aggregate underdrain is more durable than standard longitudinal underdrain under traffic loads, providing better support to the roadway.

Transverse underdrain may be required where an extensive cut section transitions into a fill section on a roadway segment having a downward sloping profile grade that is greater than the roadway cross section slope. This is to intercept groundwater that can enter the free draining pavement structural section materials at the cut/fill interface. A transverse underdrain is also applicable to roadway areas experiencing groundwater breaking out from the pavement base, which is indicative of groundwater building up in the base course materials.

Practice:

- Transverse underdrain should be configured as a Type 2 aggregate underdrain with 6- inch pipe and 24-inch wide trench. The top of the aggregate underdrain trench should extend from the bottom of the overlying pavement structural section down to a depth that provides a minimum of 6 feet of cover from the pavement surface to the bottom of the trench. If possible, transverse underdrains should not be installed perpendicular to centerline, but have a skewed alignment across the roadway. This will minimize the potential of forming a roadway pavement dip over the pipe trench if fines are lost to the underdrain.

- Trench widths generally range between 18 and 36 inches, and the size should be based upon the pipe diameter, if included, or the volume of groundwater expected.
- Trench depths and configurations should be adjusted to meet the needs of the site. If an adequate outlet for the underdrain cannot be provided, then the underdrain should not be installed.
- Type 2 aggregate underdrain should be used if there is concern with survivability of the underdrain pipe under traffic loads, such as in additional runs of longitudinal underdrain in wide roadway segments, ramp crossings, or in roundabouts.
- Consult with Geotechnical Bureau for additional guidance

Underdrain Outlets & Flushing Basins

Principle: All underdrain should be provided an outlet at either end of the underdrain pipe in accordance with standard design procedures by installing a flushing basin, extending the pipe to a slope face (with or without underdrain headwall) or extending the underdrain with a carrying pipe to a nearby drainage structure. Flushing basins and cleanouts allow for periodic cleaning of the underdrain if they become clogged with sediment.

Practice:

- An underdrain outlet in a drainage ditch should be located on the side of the ditch, a minimum of 3 inches above the ditch line to prevent the outlet from being buried by sediment.
- Underdrain discharging into a drainage structure shall be a minimum of 6 inches above the flow line of the drainage pipe that discharges from the structure.
- Underdrain connected to a drainage structure solely to be used as a cleanout, should be placed as high in the structure as is feasible and the underdrain should transition to its typical depth (as described above) within 10 feet.
- When an underdrain cannot start at a CB or DI, a flushing basin will be installed at the beginning of the run.
- A flushing basin or cleanout, located outside the edge of pavement, should be provided at the beginning of all underdrain pipe runs (not needed for aggregate only underdrain) when it is not feasible to start from a CB or DI structure. Flushing basins and cleanouts should be placed with a 300 feet maximum distance from any other flushing basin or cleanout, sometimes necessitating a ‘Y’, or series of ‘Y’s in the underdrain run. (See Standard Plan Sheet DR-4)

Combined Stormwater & Underdrain Systems

Principle: Combined systems use the underdrain pipe as a carrying pipe for moving stormwater through the drainage system, as well as performing as an underdrain for the site. Typically, separate pipe systems for stormwater drainage and underdrain are used. There are occasional situations where a combined system may be appropriate; however, the use of a combined system should be considered as a last option. Combined systems may be an option in areas with inadequate space because of other utilities, a lack of Right-of-Way, or for other site constraints. Pipes with perforations distributed all around their circumference are not suitable as carrying pipes in a combined system.

Practice:

- Carrying pipes in the combined system are at least 12 inches in diameter.
- In a combined system the carrying pipe perforations are located only in the upper part of the pipe circumference, thus requiring an underdrain pipe with perforations only partially around the circumference being specified. This prevents water in the pipe from infiltrating into the ground from the pipe itself.

Underdrain Details

Principle: Underdrain should be included in the drainage summary and shown on the plans.

Practice: Refer to Standard Plans or Special Details in this document for examples of underdrain applications.

4.0.23 Other Underdrain Applications

Underdrains are used in a variety of other applications for highway related structures to provide subsurface drainage, to prevent seepage problems, or to prevent saturated soil conditions that could weaken the structure. These applications may require a site specific design, which should be generated and/or reviewed by Materials and Research. Several examples where underdrain is used in highway related structures are listed below. Outlets to a closed drainage system or to an area lower in elevation may be needed for these applications if present.

Wet Slopes – For an earth cut slope with a high groundwater table or subsurface water breakout occurring, an underdrain may be needed for the slope itself. Groundwater could destabilize the slope or saturate an adjacent roadway. The underdrain should be sited above, within or at the base of the slope where it will intercept groundwater to prevent these problems. Aggregate underdrain should be considered for these situations unless standard underdrain is considered adequate.

Water Treatment Ponds – Some treatment ponds utilize underdrain between pond segments to collect water infiltrating through the soil. Underdrain around the perimeter of a treatment pond may also be used to lower the groundwater table to limit the infusion of groundwater into the treatment pond. Depending upon conditions, standard underdrain or aggregate underdrain could be used.

Parking Lot – Large paved areas such as parking lots that have high groundwater tables could benefit from an underdrain system. Standard underdrain with lateral pipe runs 25 to 50 feet apart could be used.

Porous Pavement – Porous pavement consists of pervious asphalt or concrete surfacing with several feet of free draining granular soil installed below the pavement to provide treatment to the infiltrating surface water. In areas with a high groundwater table, or with poorly draining subgrade soils below the select materials, an underdrain system consisting of standard underdrain arranged in a lateral pattern described above for parking lots could be used.

MSE Walls – There are two applications for 6-inch underdrain pipe in a mechanically stabilized earth wall. One application at the back bottom edge of the reinforced soil zone is to keep groundwater out of the reinforced soil zone, which may weaken the shear strength of the soil. The other application is above the reinforcement zone to collect infiltrating surface water that may contain road salts, but this is only installed if the soil reinforcing elements are susceptible to chloride attack (i.e. metal strips). In this use, the underdrain pipe is usually placed above an impervious geomembrane layer.

Reinforced Soil Slopes – Underdrain is sometimes placed inside the fill at the back bottom edge of the reinforcement zone to keep groundwater out of the reinforced soil zone, which may weaken the shear strength of the soil. Either standard underdrain or Type 2 aggregate underdrain could be used, depending upon the volume of groundwater anticipated.

Slope Drainage

Principle: The purpose of a slope drain is to intercept and provide a permanent means for surface runoff to be conveyed down a slope in order to prevent erosion and to direct flow away from cut or fill slopes. Severe erosion may result when slope drains fail by overtopping, pipe separation, or plugging of outlet. Refer to most current Standard Specification Plans & Details (currently Standard No. DR-4 & Sect. 603 2.6).

Principle: The runoff could be from the road surface or from an area where runoff needs to be diverted away from the roadway.

Principle: It is desirable to construct slope drainage in areas where the lower discharge rate and volume can be treated effectively, rather than collect more water in a treatment pond or other facility with higher maintenance and cost requirements.

Practice:

- Permanent slope drain systems are designed for the 10 yr. storm event.
- Slope drainage will be shown on plans and cross sections.
- The designer must always determine an appropriate outlet location either on the slope or at the bottom of the slope. Clogging can be a problem for outlets at the bottom of the slope, and outlets on a slope may need erosion prevention. Slope drain should be constructed according to the Standard Detail Sheet and outlet above the toe of slope to prevent sediment from plugging the slope drain. Where space or right-of-way constraints exist and the drain outlets on the slope; appropriate slope protection must be installed to prevent erosion.
- Slope drainage will be collected in drainage structure(s) (typically a single DI-D) with outlet pipes. Refer to Standard Detail Sheet DR-4, or current.
- Slope drainage structures are normally used in conjunction with a curbed roadway typical where areas of surface runoff may potentially cause erosion problems to adjacent high fill slopes.
- Berm ditches are typically constructed at the top of a cut slope (earth or rock) to intercept drainage prior to it flowing over steep slopes and certain rock slopes before it reaches the roadway ditch. These ditches are often constructed to promote slope stability, minimize erosion of long and/or steep slopes, avoid dangerous icing on rock cut faces, minimize ice jacking of rock in rock cuts, and similar applications. Sometimes these are used in combination with an aggregate underdrain system to decrease subsurface water on the soil slope. The drainage is typically collected in either individual drainage structures or in a closed system in the berm ditch, and piped down the slope at the most appropriate location.
- All connections and joints must use watertight connecting bands.
- The need for anchoring pipe sections should be evaluated.

- A flared end section shall be attached to the outlet end of the pipe.
- Outlet protection must be provided where necessary to prevent erosion, a stone apron shall be used below the pipe outlet.

When slope drainage is constructed for temporary applications, the designer should refer to the Erosion & Sediment Control Practices in Chapter 6 of the Highway Design Manual and BMP guidance.

Material and construction requirements for slope drainage is covered under **Standard Specifications Section 603 2.6**.

Other areas to consider slope drains are on the low-side of a super-elevated roadway or a multilane normal crown section. Slope drainage should also be considered in non-curbed sag areas for snow curb.

Paved and metal sluices are not to be used because of issues with maintenance, durability, ineffective interception of flow, and negative effect when used in guardrail sections. In general, existing paved and metal sluices should be considered for replacement, depending on the project scope of work.

Where sediment-laden water is being conveyed it must pass through an appropriate BMP to provide removal of total suspended solids, see Chapter 6 of the Highway Design Manual, the NHDES Stormwater Manual, or other suitable reference if needed.

Hydraulic Modeling Technology

Improvements in hydraulic and hydrologic (H&H) modeling have been made in the past twenty years because of computer technology. Each type of model differs in the approach to the calculations. However, typically the software uses standard methods that have been used for generations. In many instances it is possible to adequately design drainage infrastructure without extensive modeling. The most common types of models used by NHDOT and consultants include the SRH2d engine developed by Dr. Li at the Bureau of Reclamation and the associated GUI called SMS developed with public funds and recommended by the FHWA. The Department holds licenses for proprietary software including HydroCAD, and Bentley MicroStation products. The public domain HEC RAS and HEC HMS are commonly used for floodplain and floodway studies, and stream restoration projects. HEC RAS models one or two dimensional steady or unsteady flow with boundary conditions defined for the upstream and downstream extents. Boundary condition options common to all models include normal depth, critical depth, known water surfaces, and rating curves. Steady flow utilizes profiles that run a specific flow rate for applying continuity and energy loss principles. Unsteady flow normally utilizes hydrographs and can be time consuming for 2d hydraulic models. Three dimensional hydraulic models (computational fluid dynamics, or CFD) are sometimes used for unique analysis, usually to improve accuracy in vertical acceleration (not modelled with 2d) or for vortexes and turbulence to improve understanding of scour. Another option for hydraulic modeling is a physical model, a scaled model is built and flows are studied for application in the real world using multi-dimensional analysis. The use of CFD is anticipated to be more commonly used than physical modelling in future years.

Principle: Typically there is more uncertainty in hydrologic assumptions than in hydraulics.

***Principle*:** Scaling errors exist when using a computer model to apply empirical guidance based on a limited sample of conditions.

***Principle*:** Sensitivity analysis for Manning's values or boundary conditions should not be confused w/ calibration.

Practice:

- Hydraulic models should be calibrated to a storm of record using gage records and recorded measurements. Anecdotal information is also used for calibration to a degree. Coastal models that utilize transducer data may not require calibration.
- HydroCAD models are commonly used by consultants and NHDOT for designs. The user should make every effort to evaluate error messages and hint messages provided by the application.
- SRH2d and SMS is used by NHDOT and state DOT users are sponsored by the FHWA. Many consultants have licenses and there are community versions for

reviewers. This software is one that has been accepted by FEMA for the NFIP (LOMR/CLOMR).

- Three dimensional roadway design models are used by NHDOT. With the release of SMS 13.1.9, 3d bridge files have been incorporated into hydraulic models.
- HEC RAS is routinely used for one dimensional analysis of culverts larger than 48” diameter and bridge design by NHDOT and consultants. The user should make every effort to evaluate warning messages and notes provided by the application.
- It is common practice to model or calculate the requirements for water treatment practices differently than one would for drainage capacity or AOP goals.
- The user should understand the limitations of models and make every effort to evaluate error messages.

4.0.24 Two Dimensional Hydraulics

Both 2d hydraulics and 2d hydrology will continue to advance rapidly in the near future. Three-dimensional design tasks and geometry will overlap with hydraulic modelling, including 2d simulations and computational fluid dynamics (CFD). The emphasis of this section is on 2d hydraulics, specifically that which is done with the US Bureau of Reclamation Sediment and River Hydraulics 2d modelling (SRH2d) and the US army corp. HEC RAS numerical models coupled with the associated SMS and RAS Mapper graphical user interfaces (GUI).

Principle: Publically available data must be merged with project files such as CAD and GIS components. The needed survey data including bathymetry and structure geometry should be identified at the scoping phase of projects. Doing so will improve work flow efficiencies. The SRH2d engine allows the user to incorporate the finite volume method for approximation of depth averaging of the Saint Venant equations as described in **Eq. 2** and **3** below. The program uses an unstructured mesh which is different than the mesh methods used in other 2d programs, such as HEC RAS. Analysis can be with steady state or unsteady hydraulic inlet boundary conditions. The modelling flow diagram is shown in **Figure 4.0.24.a**.

The Hydraulic Controls should be identified early in the scoping of the project.

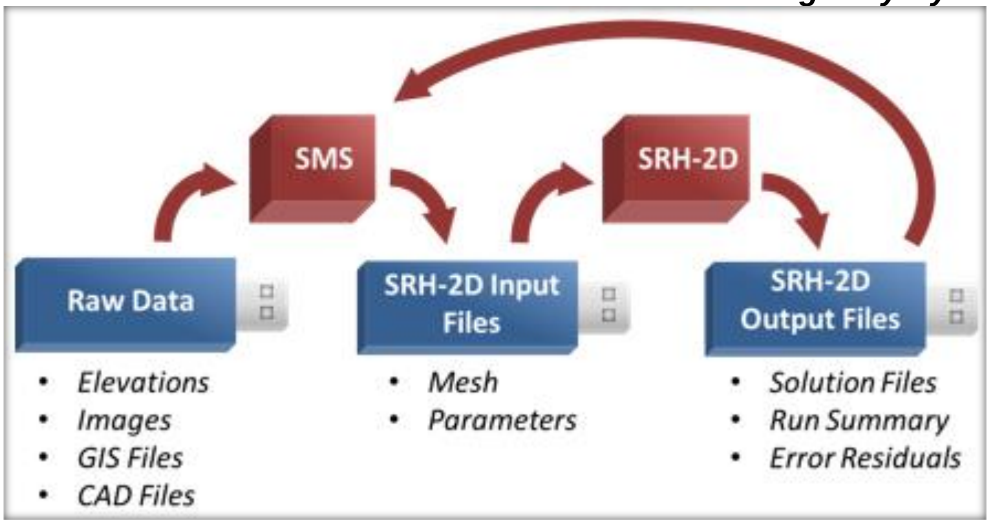


Figure 4.0.24.a FHWA suggested 2d Hydraulics workflow

The SRH2d program lets us determine water surfaces using hydraulics, physics, and numerical solution of partial differential equations, including the solving of differential equations simultaneously within the pre & post processing. Primarily there are 3 unknowns 1.) depth(h), 2.) depth-averaging velocity in the east-west direction, 3.) depth-averaging velocity in the north-south direction. Three unknowns require three equations, namely.

Conservation of mass in two dimensions:

$$\frac{\partial h}{\partial t}+\frac{\partial hU}{\partial x}+\frac{\partial hV}{\partial y}=e \qquad \textbf{Eq. 1}$$

Conservation of momentum in the x direction:

$$\frac{\partial hU}{\partial t}+\frac{\partial hUU}{\partial x}+\frac{\partial hUV}{\partial y}=\frac{\partial hTxx}{\partial x\partial}+\frac{\partial hTxy}{\partial y}-\frac{gh\partial z}{\partial x}-\frac{\tau bx}{\rho}+Dxx+Dxy \qquad \textbf{Eq. 2}$$

Conservation of momentum in the y direction:

$$\frac{\partial hV}{\partial t}+\frac{\partial hUV}{\partial x}+\frac{dyVV}{\partial y}=\frac{\partial hTxy}{\partial x}+\frac{\partial hTyy}{\partial y}-\frac{gh\partial z}{\partial y}-\frac{\tau by}{\rho}+Dyx+Dyy \qquad \textbf{Eq. 3}$$

All the equations are "depth averaged" which means depths are averaged at each element in the mesh. Equations **2** & **3** together are called the depth averaged Reynolds- Averaged Navier Stokes equations. Equations specific to derivation of scalar and vector quantities, such as acceleration are not covered in this manual. Inquire with the Specialty Section for more specific detail if needed. A model is a description of a system. For hydraulic models, the description is combination of parameters and geometry. The steps for modelling are:

- Determine purpose
- Import background data
- Develop components
- Set up simulation
- Specify parameters

- Run analysis
- Visualize results
- Application

To determine the purpose, one might ask what is the question? e.g.

- Floodplain Limits
- Velocity Distribution
- Flow Splits
- Min/Max Depths
- Hydraulic Structures
- Time Issues

Typically, the most important background information is elevation data. Components of the modelling geometry (grid/mesh), boundary conditions (inlet, outlet, structures), materials roughness – typically Mannings **n** which is likely somewhat less than the values used for 1d calculations. The materials file (GIS coverage) is defined using polygons for channel segments, floodplains, and other land cover. Studies are ongoing and guidance is anticipated within a few years (NCHRP 24-49). The mechanics are defined by how the program "engine" links the geometry, boundary conditions and materials. Data sets can be assigned to "scatter point" in the terrain or "mesh nodes". The mesh is essentially a series of elements draped over the terrain. Data sets that are read in from a solution file represent values for water surface, depth, velocity and other solutions. Each set of values are called functional data sets in SMS, and they are organized into folders within the modelling project. Data sets may be transient or steady state. Transient data includes one value for each node/point for each time step represented. Each data set can be shown with colored contours. Vector arrow can be plotted to show flow fields in the hydraulic solutions.

Principle: Precision is largely a product of the mesh geometry, and it can be structured or unstructured depending on the engine. SRH2d uses an unstructured mesh. Boundary conditions for the inflow include location, distribution across the mesh, direction and amount of flow. SRH2d provides a means to use steady flow for inlet boundary conditions whereas HEC RAS inlet boundary conditions for 2d models must be unsteady (hydrographs). Outlet boundary conditions are a control depth that often is a rating curve.

Principle: Running the analysis is when the numerical solutions for partial differentials occur. Computation times vary based mainly on the size /resolution of the mesh, length of time being simulated, and whether the analysis uses steady or unsteady inlet boundary conditions. SMS provides a Data Calculator that allows for computations to be carried out for all nodes in the network. Inquire with the Specialty Section Hydraulics Group for more information on what the Data Calculator can do.

Visualize results:

- Model output
    - Primary variables solved
        - Water surface elevations
        - Velocity (x,y)
        -
    - Derived
        - Depth
        - Speed
        - Froude
        - Bed shear
        - …

SRH-2D - ___XMDF.h5 file.

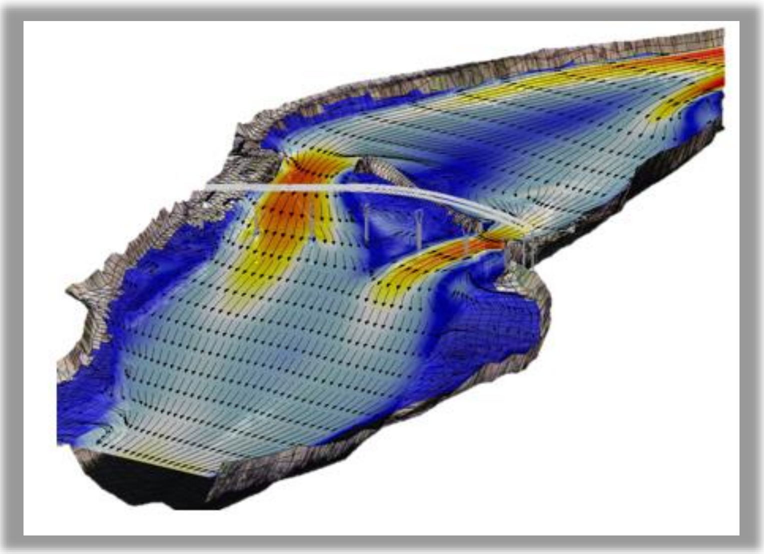


Figure 4.0.24.b Isometric view of a SRH-2d SMS model

Principle: Simplifying the visuals from 3d to 2d can be helpful for understanding or comparing velocity distributions and water surface profiles. It is often useful to view 2d cross sections or profiles from a mesh. A simulation contains much more information than can be shown in a single image. The user must filter and control the data that is displayed at any one time.

Modelling scenarios include validation by sensitivity analysis by varying boundary conditions, roughness, and in some cases mesh sizing. Preferably calibration is performed with data rather than merely anecdotal observations. Tidal domains typically involve a different means of calibration than riverine models. Refer to the FHWA primer on Coastal Modelling.

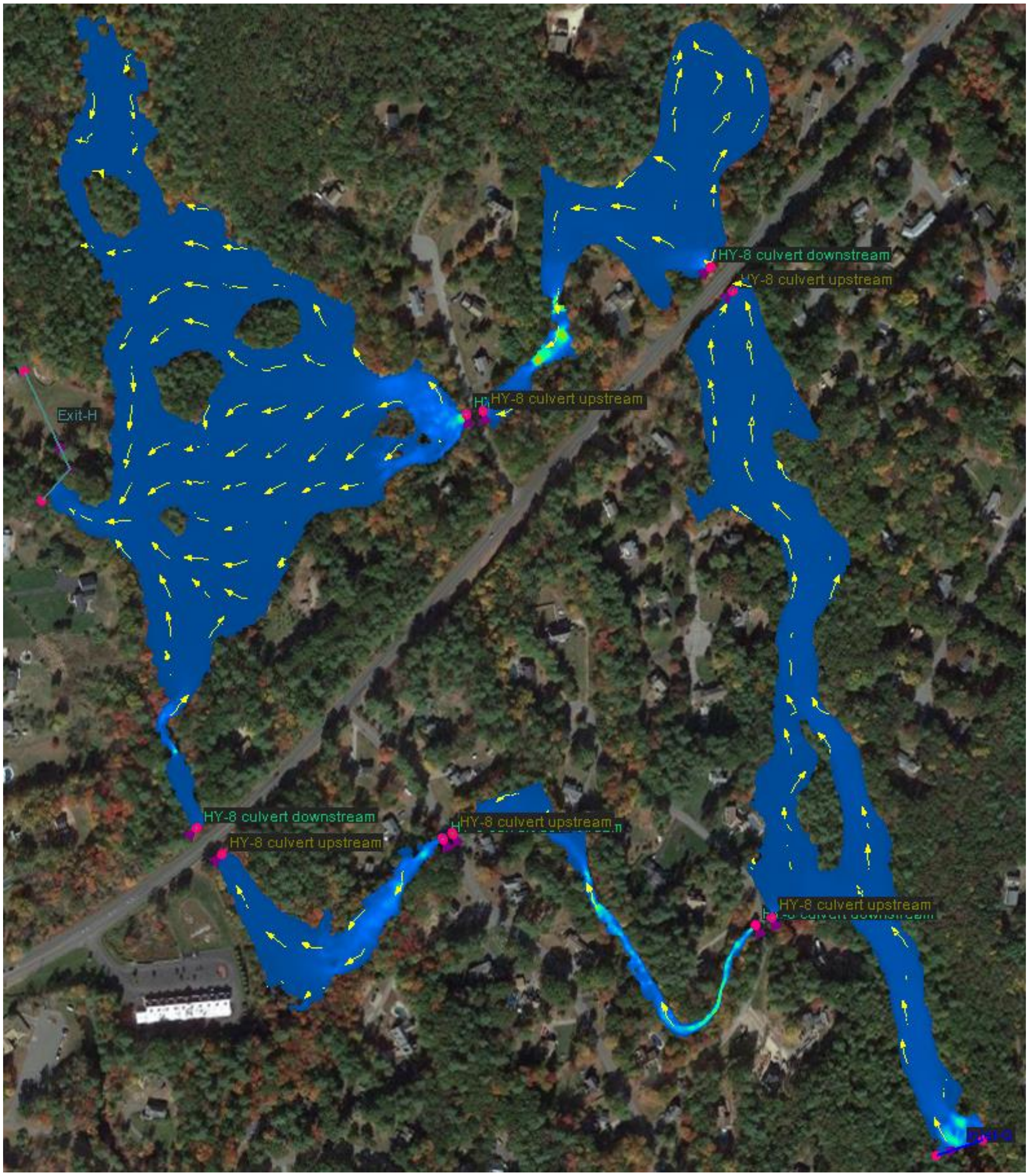


Figure 4.0.24.c The use of HY-8 culverts in a SRH-2d SMS model

Table 4.0.24.a – Software Applications

Application	HydroCAD	StormCAD	HY8	Hydraulics Toolbox	Spreadsheets
Rational Method	Not for routing	Yes		Yes	Yes
Gutter flow	Not preferred	Yes		Yes	Yes
Grate capacity	Not preferred	Yes		Yes	
Sizing pipes	Yes	Yes		Yes	
EGL / HGL		Yes			
Culvert design	Not preferred		Yes		
Channel linings				Yes	
Energy dissipation			Yes		
Stormwater pond design	Yes	No		Yes (HydroCAD preferred)	

Page intentionally left blank

GLOSSARY

Analysis – A term that means "to break apart" and that is applied to methods used to break down hydrologic data to develop a hydrologic model or design method (see synthesis).

Antecedent Moisture Condition – A classification of the state of soils from dry (I) to saturated (III). Dry soils produce less runoff than saturated soils. Antecedent condition II is typically used in hydrologic analysis.

Armoring – Placement of stone fill, rock riprap, concrete, asphalt pavement, erosion control mats, or other materials or manufactured products that resist the channel and bank erosive forces caused by stream flow.

Basin Development Factor (BDF) – an index of urbanization that accounts for man-made channel alterations, storm drains or sewers, and curb-and-gutter streets. Impervious area is not considered directly in the BDF.

Bridge – In NH a "bridge" means a structure, having a clear span of 10 feet or more measured along the center line of the roadway at the elevation of the bridge seats, spanning a watercourse or other opening or obstruction, on a public highway to carry the traffic across, including the substructure, superstructure and approaches to the bridge.

For purposes of qualification of bridge aid under this subdivision, but not applicable to RSA 234:4 or RSA 234:13, the term bridge shall include a combination of culverts constructed to provide drainage for a public highway with:

I. An overall combined span of 10 feet or more; and
II. A distance between culverts of 1/2 the diameter or less of the smallest culvert.

Catch Basin – A drainage structure, typically concrete, that is precast or built in place, having a grate as an inlet on the top of the structure, and frequently, but not always, serves as a "junction box" for runs of pipe. The catch basin has a sump below the invert of the lowest pipe, typically 3 feet deep, to allow for sediment collection. Catch basins also allow access to the adjoining runs of pipes for pipe cleaning.

Catchment – Smaller unit of a larger watershed drainage basin, the region of land upslope from a specific low point.

Closed System – A system composed of Catch Basins, Manholes, and Drop Inlet Basins with connecting drainage pipes that outlet via a single outlet.

Combined Systems – Closed drainage systems that also serve as sewage collection systems (sewers) in a municipality. In addition, the term is used for drainage systems with carrying pipes that have perforations and are used as underdrain pipes instead of, or in addition to, normal underdrain pipes.

Culvert – An open channel or conduit that could become submerged and which is used to convey flow under highways, railroad embankments, runways or other entities.

Debris – Floatable or other transportable material such as leaves, tree branches, seasonal plant vegetation, trash, toys, picnic tables, tires, lumber, fabric, etc…

Deterministic Modeling – Method of analysis that strives to break down components of hydrology into physically based parameters. Typically, these methods are used on ungaged watersheds and are most commonly contrasted with stochastic methods, which are based more on statistics. However, a unified approach to modeling will combine both stochastic and deterministic methods. Rainfall-runoff models, such as the U.S. Army Corp. HEC-HMS and the NRCS TR-20 models are examples of deterministic models that are quasi-empirical. Refer to HDS 2, Chapter 12 of the Maine DOT manual, or a hydrology text book for further discussion.

Drainage Manhole – A drainage structure similar to a catch basin but with a solid cover rather than a grate, and usually not having a sump, typically used as a “junction box” for multiple runs of pipes.

Drop Inlet (DI) – Drop Inlets are similar to Catch Basins except they have no sumps and they typically are the first drainage structure in a run of a closed drainage branch.

Drop Inlet D (DI-D) – These are reduced size drainage structures used for the interception of curb line flow via a grate on top of the structure and then each structure outlets typically via a slope pipe or occasionally into a closed drainage system. The pipe size is limited to 12 inches due to the interior diameter and this basin is not used to join multiple pipes.

Emergency Spillway – A secondary outlet that allows a percentage of higher flows and debris to pass, often times emergency spillways are designed to divert the estimated 100 year flow.

Energy Grade Line (EGL) – A line that represents the energy state of water flowing in a pipe, or channel. It is the sum of the pressure, velocity and elevation heads. The line is drawn above the hydraulic grade line (gradient) a distance equal to the velocity head ($V^2/2g$) of the water flowing at each section or point along the pipe or channel.

Dam Hazard Class – Env-Wr 101.09 “Class Structure” categories that define hazard potential sometimes based on potential loss of human life that would likely result from water levels and velocities causing the structural failure of a foundation of a habitable residential structure or a commercial or industrial structure which is occupied under normal conditions; or circumstance which would more likely than not cause one or more deaths. Refer to specific NHDES Dam Bureau hazard class definitions. Dams are classified from low hazard to high hazard, some highway stream crossings may meet dam.
criteria although efforts are made to reduce this potential. A Non Menace Dam is one that has less risk than a low hazard dam, this is the only Dam that NHDOT engineers are qualified to design and build.

Hydrology – The study of water cycles linking the hydrosphere. It encompasses the occurrence, distribution, movement, and properties of water on earth. Engineering hydrology is largely based on the theory of conservation of mass. Hydrologic methods have greater uncertainty than hydraulic fluid behavior.

Hydraulics – Applied science concerned with the flow of liquids, especially in pipes, channels, structures, and the ground. [HDS 7]

Hydraulic Grade Line (HGL) – The surface or profile of water flowing in an open channel or a pipe flowing partially full. If a pipe is under pressure, the hydraulic grade line is that level water would rise to in a small, vertical tube connected to the pipe. The HGL is the sum of the pressure and elevation heads.

Hydraulic Jump – A rise in water surface that is created in a rapidly varied flow situation where supercritical flow becomes subcritical flow.

Hyetograph – A time-dependent function of the rainfall intensity versus time.

Hydrograph – The graph of stage or discharge against time.

Impervious Area – Surfaces that water runs over not through, typically pavement and roofs.

Initial Abstractions – The portion of the rainfall that occurs prior to the start of direct runoff, not to be confused with "losses".

Inlet Control – If water can flow through and out of the culvert faster than it can enter, the culvert performs with inlet control. Flow capacity is controlled at the entrance by the headwater depth, cross-sectional area and type of inlet edge. Culverts with inlet control will always flow partially full and are in a state of shallow, high velocity known as supercritical flow.

Invert – The elevation where water flows within a drainage system and through ditch lines.

Lithology – The scientific study and description of rocks, especially at the macroscopic level, in terms of their color, texture, and composition. The gross physical character of a rock or rock formation.

Normal Depth – The depth of flow in a channel or culvert when the slope of the water surface and channel bottom is the same and the water depth remains constant. Normal depth occurs when gravitational force of the water is equal to the friction drag along the culvert and there is no acceleration of flow. In culverts, water flows at normal depth when outside the influence of the inlet and outlet tailwater. Normal depth is undefined for culverts placed at horizontal or adverse slopes. For each flow it can be calculated using the

Mannings equation based on the channel roughness, wetted area and hydraulic radius. Knowing normal depth aids in classifying the hydraulic slope of the culvert.

Open Channel Flow – A conduit with a free surface, depth of flow generally is used to represent the pressure term. Flow is classified as subcritical, critical, and supercritical and the concept of specific energy and conjugate (a.k.a. "alternative") depths connect the transition between supercritical and sub-critical flow regimes.

Ordinary High Water (OHW) Means the line on the shore, running parallel to the main stem of the river or stream, established by the fluctuations of water and indicated by physical characteristics such as a clear, natural line impressed on the immediate bank, shelving, changes in the character of soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means.

Outlet Control – If water can flow into the culvert faster than it can flow through and out, then the culvert performs with outlet control. Culverts with outlet control can flow either partially full or full. In this flow regime water is relatively deep and slow, known as subcritical flow.

Overland Flow – Both sheet flow and shallow concentrated flow if they both exist.

Practicable – capable of being accomplished within prudent natural, social, or economic constraints using readily available resources, reasonably reliable technology, and available practices that can be economically applied.

Resiliency – The ability of infrastructure to recover from overburden or disturbance.

Riprap – Riprap consists of a layer or rock, placed in the channel and structure boundaries in a manner which produces a well-graded mass that will limit the effects of erosion. Riprap is a very effective countermeasure when the riprapped area is of adequate size (length, width, and depth), it is of suitable gradation, and the correct installation procedures are followed. The designer may specify a minimum d_{50} (median stone diameter) for the rock comprising the riprap, which indicates the size for which 50% of the particles are smaller. The suitable shape of the individual stones shall be angular, meeting the gradation specified create interlocking riprap to provide stability of the slope or channel.

Runoff – The flow of water over land, from rain, snowmelt, or other sources. Scientists predict that less runoff from snow melt and more runoff from rain will likely occur in New Hampshire in the years ahead.

Runoff Prevention Method (RPM) – The means of reducing downstream impacts utilizing modern design techniques and technology, such as Low Impact Design (LID).

SCS Curve Numbers – The curve number is based on the area's hydrologic soil group, land use, and hydrologic condition, the two former being of greatest importance. Although the method is designed for a single storm event, it can be scaled to find average annual runoff values.

Sediment – Fine textured sand, silt, organic material or other particles that are transported by water flow or wind, eventually settling to the bottom of slow moving water bodies, standing water, or evaporated paths of flow. It is the matter that settles to the bottom of a fluid.

Spread – A measure of the accumulated flow in and transverse to the roadway gutter. This water often represents an interruption to traffic flow, splash-related problems, and a source of hydroplaning during rainstorms. Spread can be created due to snow curb or due to a winter sand berm that results from snow plowing.

Steady Flow – Fluid flow in which all the conditions at any one point are constant with respect to time, flow that is normally seen in relatively uniform channel sections. The flow modeled in HEC-RAS is either one-dimensional steady flow or one-dimensional unsteady flow.

Stochastic Hydrology – Statistics that deal with the probabilistic modeling of hydrologic processes that have *random components.* Stochastic models are used to describe the physical processes that are observed, and about which, data are recorded.[Univ. of Natal] The highway engineer is primarily interested in two variable processes precipitation and stream flow. The word “stochastic” is a statistical term describing time series when they are not purely random, but exhibit dependence in time. Other processes include: evaporation, infiltration, groundwater flow.

Stone Fill – Stone Fill is typically required for stability of embankment fill and soil cut slopes steeper than 2 horizontal to 1 vertical, although slopes at a flatter grade with water seepage or subject to submergence, such as in water quality treatment basins, could require stone fill. Stone fill is also used for erosion protection at pipe outlets, in drainage channels and for other drainage structures where expected water flow and shear stress may require it. Stone for stone fill shall be approved quarry stone, or broken rock of a hard, sound, and durable quality. The stones and spalls shall be so graded as to produce a dense fill with a minimum of voids.

Stop Logs – Devices used for temporary closure of an opening in a hydraulic structure. A generic term that is not intended to imply wood logs are used exclusive of other material, i.e., a log, wood plank, cut timber, steel, or concrete slab; or a beam of some synthetic material fitting into end guides between walls or piers to close or limit an opening to the passage of water. Long rectangular timbers or boards that are placed on top of each other inside a hydraulic structure and used to control weir flow or water elevation. Not the same as flash boards (see definition Env-Wr 101.17).

Subcatchments – The smaller units of catchments leading to specific culverts, streams, ponds, or structures. Watersheds and catchments have specific number codes created by

USGS, whereas subcatchments are more ad-hoc drainage areas that typically make up a design analysis.

Synthetic Unit Hydrograph – A unit hydrograph not directly based on measured rainfall and runoff data. The NRCS Curve Number Method used in HydroCAD uses synthetic unit hydrographs.

Synthesis – The term means "to put together" and is applied to the problem of hydrologic estimation using a known model.

Thalweg – Path of deepest flow in a stream.

Time of Concentration (t_c) – Generally defined as the time required for a drop of water to travel from the hydraulically most remote point to the outlet of a subcatchment. The most common method used to determine t_c is the velocity or hydraulic method developed by SCS. Typically, it is composed of sheet, shallow concentrated, and open channel flow.

Type II storm – A statistical method and procedure to model frequency and distribution of a rain event. Southern NH generally receives type III storm events while Northern NH receives type II storms.

Type III storm – A statistical method and procedure to model frequency and distribution of a rain event

Underdrain – A perforated pipe or trench filled with uniformly sized stone aggregate installed to collect groundwater and redirect it away from the structural box of the highway or other structure.

Uniform Flow – This occurs when the loss of potential energy equals the work done against the channel friction surface (the channel bottom slope = the friction slope), the water surface is parallel to the bed of the channel and the flow depth, channel discharge, and flow area do not change over a constant channel reach (shape and material).

Unsteady Flow – Turbulent flow in which properties of the flow change with respect to time.

Watershed – The total area of land, water and biota within the confines of a drainage divide. An estimate of surface flow to a single outlet is quantified for highway design, however, USGS has delineated areas using a standard Hydrologic Unit. This may include several catchments and subcatchments. The unique Hydrologic Unit Code (HUC) consists of two to twelve digits based on six levels of classification:

2-digit HUC first-level (region)
4-digit HUC second-level (subregion)
6-digit HUC third-level (accounting unit)
8-digit HUC fourth-level (cataloguing unit)
10-digit HUC fifth-level (watershed)
12-digit HUC sixth-level (subwatershed)

Hydrologic units are only synonymous with classic watersheds when their boundaries include all the source area contributing surface water to a single defined outlet point. The first four levels (2 to 8 digit) have been completed and certified for the entire USA. The fifth and sixth level are NRCS units for smaller scales. A comprehensive interrelated approach to area and natural resource management recognizes the needs of soil, water, air, plants, animals and people in relation to local social, cultural, and economic factors. The distinction between "watershed" and HUC could be significant in terms of water quality monitoring, total maximum daily load analysis, regional and national water quality patterns, and other research on the land/water relationship.

Additional Glossaries are provided in the HEC & HDS publication series, such as:

HEC 26, HDS 7 & HEC 18 to name a few.

APPENDICES

Appendix A

Mannings roughness values (n) are typically selected based on experience and comparisons to similar open channels or conduits. The tables in this appendix draw heavily from Ven Te Chow work in the 1950s and from USGS publications listed below:

- USGS 1584
- 85-4004
- Paper 2339
- Paper 1849

It is best to estimate roughness values in the field. However, it is not practical, or even necessary for most engineering studies due to project cost constraints and past experience. Mannings n is applicable to steady uniform flow, and it varies with channel material and flow depth. Typically, only channel material is considered by engineers.

There are several empirical equations to determine Mannings n analytically. Among them are Cowan's method. See addition discussion prior to the floodplain flow chart further on.

Table A.1 – Manning's Roughness Coefficients with Footnotes

I	Closed Conduits	Manning's n range [2]
A.	**Concrete pipe, smooth**	0.012 – 0.013
B.	**Corrugated metal or pipe arch**	
	1. 2 ⅔" by ¼" corrugation [3]	
	a. Plain or fully coated	0.024
	b. Paved invert (range values are 25% & 50% of circumference paved)	
	i. Flow full depth	0.021 – 0.018
	ii. Flow 0.8 depth	0.021 – 0.016
	iii. Flow 0.6 depth	0.019 – 0.013
	2. 6" by 2" corrugation (field bolted)	0.03
C.	**Vitrified clay pipe**	0.012 – 0.014
D.	**Cast-iron pipe, uncoated**	0.013
E.	**Steel pipe**	0.009 – 0.011
F.	**Brick**	0.014 – 0.017
G.	**Monolithic concrete**	
	1. Wood forms, rough	0.015 – 0.017
H.	**Cemented masonry**	
	1. Concrete floor & top	0.017 – 0.022
	2. Natural floor	0.019 – 0.025
I.	**Plastic pipe**	
	1. Corrugated polyethylene, smooth	0.009 – 0.015
	2. Corrugated polyethylene, corrugated	0.018 – 0.025
	3. PVC, smooth	0.009 – 0.011

II	Open channels, lined [4,5]	Manning's n range [2]
A.	**Concrete, with surface as indicated**	
	1. Formed, no finish	0.013 – 0.017
	2. Gunite, good section	0.016 – 0.019
B.	**Concrete, bottom float finished, sides as indicated**	

	1. Dressed stone in mortar	0.015 – 0.017
	2. Random stone in mortar	0.017 – 0.020
	3. Cement rubble masonry	0.020 – 0.030
	4. Dry rubble (riprap)	0.020 – 0.030
C.	**Gravel bottom, sides as indicated**	
	1. Formed concrete	0.017 – 0.020
	2. Random stone in mortar	0.020 – 0.023
	3. Dry rubble (riprap)	0.014 – 0.017
D.	**Asphalt**	
	1. Smooth	0.013
	2. Rough	0.016

III	**Open channels, excavated [4,5]**	**Manning's n range [2]**
A.	**Earth, uniform section**	
	1. Clean, new	0.016 – 0.018
	2. Clean, old (weathering)	0.018 – 0.020
	3. With short grass, few weeds	0.022 – 0.027
	4. In gravelly soil, clean	0.022 – 0.025
B.	**Fairly uniform section**	
	1. No vegetation	0.022 – 0.025
	2. Grass, some weeds	0.025 – 0.030
	3. Dense weeds, aquatic plants	0.030 – 0.035
	4. Sides clean, gravel bottom	0.025 – 0.030
	5. Sides clean, cobble bottom	0.030 – 0.040
C.	**Dragline excavated or dredged**	
	1. No vegetation	0.028 – 0.033
	2. Light brush on banks	0.035 – 0.050
D.	**Rock**	
	1. Based on mean section	0.013
	a. Smooth & uniform	0.035 – 0.040
	b. Jagged & irregular	0.040 – 0.045
E.	**Unmaintained channels**	
	1. Dense **weeds**, high as flow depth	0.08 – 0.12
	2. Clean bottom, brush on sides	0.05 – 0.08

3. Clean bottom, brush on sides at high stage of flow	0.07 – 0.11
4. Dense **brush,** high stage	0.10 – 0.14

IV Maintained Highway Channels [5,6]	Manning's n range [2]
A. Up to 0.7 ft. depth of flow	
1. Bermuda grass	
a. Mowed	0.045 – 0.07
b. 4" – 6" high	0.05 – 0.09
2. Good stand, any grass	
a. ~ 12" high	0.08 – 0.14
b. ~ 24" high	0.13 – 0.25
B. Depth of flow 0.7 – 1.5 ft.	
1. Bermuda grass	
a. Mowed	0.035 – 0.05
b. 4" – 6" high	0.04 – 0.06
2. Good stand, any grass	
a. ~ 12" high	0.07 – 0.12
b. ~ 24" high	0.10 – 0.20

IV Notes: *The values shown here are for velocities between 2 & 6 fps.*
See Clayton / Shuster chart for depths below 4" (see page 4-7).
Values are not for use in treatment swale design.
Many types of grass will bend over at higher depths of flow and these values are not for those types of grass.

V Street & expressway gutters	Manning's n range [2]
A. Asphalt or concrete pavement	
1. Smooth texture	0.013
2. Rough texture	0.016

Note*: For gutters with small slope, where sediment may accumulate, increase n values by 0.002*

VI	Natural stream channels	Manning's n range [2]
A.	**Minor streams (surface width at flood stage < 100 ft.)**	
	1. Fairly regular section	
	a. Some grass & weeds, little brush	0.035 – 0.05
	b. Dense weeds, depth of flow > weed height	0.035 – 0.05
	c. Weeds, light brush on the bank	0.05 – 0.07
	d. Weeds, heavy brush on the banks	0.05 – 0.06
	2. Mountain streams, no vegetation in channel, banks usually steep, trees & brush submerged at high stage	
	a. Bottom of gravel, cobbles, & few boulders	0.04 – 0.05
	b. Bottom of cobbles, with large boulders	0.04 – 0.07
	3. Heavy stand of timber, few downed trees, little undergrowth	
	a. Flood depth below branches	0.10 – 0.12
	b. Flood depth reaches branches	0.12 – 0.16
B.	**Major streams (surface width at flood stage > 100 ft.) [8]**	*

** Roughness coefficient is usually less than for minor streams of similar description because of less effective roughness from irregular banks and vegetation. Values of n may be somewhat reduced follow recommendations in footnote (8) if possible.*

Footnotes to Tables I – VI:

[1] Estimates are by the Bureau of Public Roads unless otherwise noted and are for straight alignment. A small increase in n may be made for channel alignment other than straight.

[2] Ranges for secs. I – III are for good to fair construction. For poor quality construction, use larger n values.

[3] Friction Losses in Corrugated Metal Pipe, by M.J. Webster & L.R. Metcalf, Corp. of Engineers, Dept. of the Army; published in the Journal of the Hydraulics Division, Proceedings of ASCE, Vol. 85, No. HY 9, Sept. 1959, Paper No. 2148, pp. 35-67.

[4] For important work & where accurate determination of water profiles are necessary, the designer is urged to consult the following references and to select n by comparison of the specific conditions with the channels tested:
Flow of Water in Irrigation & Similar Canals, by F.C. Scobey, US Dept. of Agriculture, Tech. Bulletin No. 625, Feb. 1939.
Flow of Water in Drainage Channels, by C.E. Ramser, US Dept. of Agriculture, Tech. Bulletin No. 129, Nov. 1929.

[5] Handbook of Channel Design for Soil & Water Conservation, prepared by the Stillwater Outdoor Hydraulic Laboratory in cooperation with the Oklahoma Agricultural Experiment Station. Published by the Soil Conservation Service, US Dept. of Agriculture, Publ. No. SCS-TP-61, March 1957, rev. June 1954.

[6] Flow of Water in Channels, Protected by Vegetative Linings, W.O. Ree & V. J. Palmer, Division of Drainage and Water Control, SCS, US Dept. of Agriculture, Tech. Bulletin No. 967, Feb. 1949.

[7] For calculations of stage or discharge in natural stream channels, it is recommended that the designer consult the local USGS office to obtain data regarding values of n applicable to specific streams. Where this procedure is not followed, the table may be used as a guide. The tabulated n values have been derived from data reported by C.E. Ramser (footnote 4) and from other incomplete data.

[8] The tentative values of n cited are principally derived from measurements made on fairly short but straight reaches of natural streams. Where slopes calculated from flood elevations along a considerable length of channel involving meanders and bends are to be used in velocity calculations using the Manning Formula, the value of n must be increased to provide for the additional energy loss caused by the bends. The increase may be in the range of 3 to 15 %.

[9] The presence of foliage on trees and brush under flood stage will increase the value on n, therefore, roughness coefficients for vegetation in leaf will be larger than for bare branches. Also, for trees in channels or on banks, and for brush on banks where submergence of branches increases with depth of flow, n will increase with rising stage.

(Source: Chow, 1959)

Table A.2 – Minimum, Normal, and Maximum Manning's n with Descriptions

Type of Channel and Description	Min.	Normal	Max.
EXCAVATED OR DREDGED			
Earth, straight & uniform			
1. Gravel, uniform section, clean	0.022	0.025	0.030
2. With short grass, few weeds	0.022	0.027	0.033
Earth, winding & sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
Rock cuts			
1. Smooth & uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
Channels not maintained, weeds & brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

Type of Channel and Description	Min.	Normal	Max.
NATURAL STREAMS			
Minor streams (top width at flood stage < 100 ft)			
Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same, but some weeds and stones	0.035	0.045	0.050
5. Same, lower stages, more ineffective sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080

8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
Mountain streams, no vegetation in channel, steep banks, trees & brush along banks submerged at high stages			
1. Bottom: gravels, cobbles & few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
Major streams (top width at flood stage > 100 ft)*			
Regular section with no boulders or brush	0.025	-	0.060
Irregular and rough section	0.035	-	0.100

** The n value for major streams is less than that for minor streams of similar description because banks offer less effective resistance.*

FLOOD PLAINS			
Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, few downed trees	0.080	0.100	0.120
5. Same, but with flood stage reaching branches	0.100	0.120	0.160

(Source: Chow, 1959)

Table A.3 – Values of n for Different Types of Vegetation on Overbank Areas

Overbank Cover	n Value		
	Min.	**Normal**	**Max.**
A. Pasture, no brush [1]			
1. Short grass	0.025	0.030	0.035
2. High grass	.030	.035	.050
B. Cultivated areas [1]			
1. No crop	.020	.030	.040
2. Mature row crops	.025	.035	.045
3. Mature field crops	.030	.040	.050
C. Brush [1]			
1. Scattered brush, dense weeds	.035	.050	.070
2. Sparse brush and trees, in winter	.035	.050	.060
3. Sparse brush and trees, in summer	.040	.060	.080
4. Medium to dense brush, in winter	.045	.070	.110
5. Medium to dense brush, in summer	.070	.100	.160
D. Trees			
1. Dense growth of willows, summer, straight	.110	.150	.200
2. Cleared land with tree stumps, no sprouts	.030	.040	.050
3. Same as above, but with dense growth of sprouts	.050	.060	.080
4. Dense stand of timber, a few down trees, little undergrowth, flood stage below branches	.080	.100	.120
5. Same as above, but with flood stage reaching branches	.100	.120	.160

[1] *Shallow depths accompanied by irregular ground surface in pastureland or brushland and by deep furrows perpendicular to the flow in cultivated fields can increase n vales by as much as 0.02.*

(Source: Chow, 1959)

Table A.4 – Sample Values of n for Agricultural Overbank Areas Under Various Stages for the Average Growing Season

Depth of Water (ft.)	Flood-plain Cover [1]				
	Corn	Pasture	Meadow	Small Grains	Brush & Waste
Less than 1	0.06	0.05	0.10	0.10	0.12
1 to 2	0.06	0.05	0.08	0.09	0.11
2 to 3	0.07	0.04	0.07	0.08	0.10
3 to 4	0.07	0.04	0.06	0.07	0.09
More than 4	0.06	0.04	0.05	0.06	0.08

[1] *From studies on the Nishnabotna River, Iowa.*

(Source: USGS)

From time to time designers need follow in the footsteps of prior studies by others, or carry out extensive floodplain modelling. The following procedure is one thorough method that could be used.

FLOODPLAIN ROUGHNESS

1. Determine extent of reach to which roughness factor will apply.
2. Determine if and where subdivision between channel and flood plain is necessary. Determine how base n will be assigned.

CHANNEL ROUGHNESS

3. Determine channel type, and estimate conditions at time of flow event. Compare the channel with photographs and descriptions of other channels.

For the entire channel

4. Determine the factors that cause roughness and how each will be accounted for.

6. Determine type and size of bed material.
7. Assign a base n from tables, formulas, and comparison with other channels and verification photographs.

By wetted perimeter

Assign a composite n for the entire channel, derived from individual segments of the channel

4. Determine the factors that cause roughness and how each will be accounted for.
5. Mentally divide channel into segments so that the roughness factor within a segment is fairly uniform.
6. Determine type and size of boundary material in each segment.
7. Assign a base n for each segment from tables, formulas, or comparison with other channels and verification photographs.
8. Apply adjustment factors for individual segments, if applicable.
9. Select the method for weighting n.

By area

10a. Estimate wetted perimeter for each segment of channel,
10b. Weight the *n* values by assigning weighting factors that are proportional to the wetted perimeter.

10a. Estimate area for each segment of channel.
10b. Weight the *n* values by assigning weighting factors that are proportional to the area.

11. Adjust for factors not considered in steps 7 and 8, including channel alignment, change in channel shape, vegetation, obstructions, and meander. Round off as desired for use in the Manning's equation.
12. Compare value determined with that for other channels and verification photographs to test for reasonableness.
13. For sand channels: Check flow regime by computing velocity and stream power for the above n ; determine regime from figure 2. The n from table 1 is valid only for the upper regime flow.

14. Determine type of flood plain, and estimate conditions at time of flow event; compare the flood plain with photographs and descriptions of other flood plains.

15. Determine method to be used in assigning n to flood plain, whether vegetation-density method will be used with boundary-roughness factors, or boundary-roughness factors only.

16. Determine if roughness is uniform throughout flood plain, or whether flood plain needs to be subdivided. (Following steps apply to each subdivision.)

17. Determine the factors that cause roughness and how each is to be accounted for.

18. Assign a base n_b from tables and comparison with other flood plains and verification photographs.

19. Determine the adjustment factors from tables.

Boundary-roughness method

Boundary-roughness method with vegetation-density method

20. Determine n_0 value from tables and formulas.

21. Determine vegetation density of representative sample area of flood plain.

22. Determine n for flood plain by using formulas.

23. Compare value determined with that for other flood plains and verification photographs to test for reasonableness.

Figure A.1 Flow chart of procedures for assigning *n* values (modified from Aldridge and Garrett, 1973, fig. 3)

Stream and location:

Reach:

Event for which n is assigned:

1. Is roughness uniform throughout the reach being considered?
If not, n should be assigned for the average condition of the reach.

2. Is roughness uniformly distributed along the cross section?
Is a division between channel and flood plain necessary?
(Channel roughness uses steps 3-13, flood plain roughness uses steps 14-23.)
Is roughness uniformly distributed across the channel?
If not, on what basis should n for the individual segments be weighted?

3. Describe the channel. Are present conditions representative of those during the flood?
If not, describe the probable conditions during the flood.

4. How will the roughness-producing effects of the following on the channel be accounted for?

Bank roughness:

Bedrock outcrops:

Isolated boulders:

Vegetation:

Obstructions:

Meander:

5 - 10. Computation of weighted *n* for the channel

Segment number and material	**Approximate dimensions, (ft)**		**Wetted perimeter, (ft)**	**Area, (ft^2)**	**Median grain size (mm)**	**Base *n* for segment**	**Adjust-ments**	**Adjusted *n***	**Weight factor**	**Adjusted *n* times weight factor**
	Width	**Depth**								
								Sum		
								Weighted *n* =		

11 - 13. Computation of *n* for the channel

Adjustment factors for the channel

Factor	**Describe conditions briefly**	**Adjustment**
Irregularity, n_1		
Alignment, n_2		

Obstructions, n_3		
Vegetation, n_4		
Meander, m		
	Weighted *n* plus adjustments	
	Computed *n* =	

14. Describe the flood plain.

Are present conditions representative of those during the flood?
If not, describe probable conditions during the flood.

15. Is the roughness coefficient to be determined by roughness factors or is it to include vegetation density method?

16. Is roughness uniformly distributed across the flood plain?
If not, how should the flood plain be subdivided?

17 - 23. Computation of *n* for flood plain

Adjustment factors not using vegetation-density method

Subsection	**Base n, n_b**	**Irregularity, n_1**	**Obstructions, n_3**	**Vegetation, n_4**	**Computed n**

Adjustment factors using vegetation-density method

Sub-section	**Base n, n_b**	**Irregularity, n_1**	**Obstructions, n_3**	**Vegetation, n_4**	**Boundary roughness, $n_0 = n_b + n_1 + n_3 + n_4$**	**Vegetation density, Veg_d**	**Effective drag, C^***	**Hydraulic radius, R**	**Computed n**

Figure A.2 Sample form for computing n values (modified from Aldridge and Garrett, 1973, fig. 4)

Appendix B

Table B.1 – Common Runoff Curve Numbers (CNs) for Pervious & Impervious Areas

	Curve Numbers for Hydrologic Soil Group [1]			
Cover Type and Hydrologic Condition	**A**	**B**	**C**	**D**
FULLY DEVELOPED URBAN AREAS (vegetation established)				
Open space [2] (*lawns parks, golf courses, cemeteries, etc.*)				
Poor condition (*grass cover < 50%*)	68	79	86	89
Fair condition (*grass cover 50% to 75%*)	49	69	79	84
Good condition (*grass cover > 75%*)	39	61	74	80
Impervious areas				
Paved parking lots, roofs, driveways, etc. (*excluding right-of-way*)	98	98	98	98
Streets and roads				
Paved; curbs and storm sewers (*excluding right-of-way*)	98	98	98	98
Paved; 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
Urban districts [3]				
Commercial and business 85% average impervious area	89	92	94	95
Industrial 72% average impervious area	81	88	91	93
Residential district by average lot size [3]				
1/8 acre or less (*town houses*) – 65% average impervious area	77	85	90	92
1/4 acre – 38% average impervious area	61	75	83	87
1/3 acre – 30% average impervious area	57	72	81	86
1/2 acre – 25% average impervious area	54	70	80	85

1 acre – 20% average impervious area	51	68	79	84
2 acre – 12% average impervious area	46	65	77	82
DEVELOPING URBAN AREAS				
Newly graded areas (*pervious areas only, no vegetation*)	77	86	91	94
CULTIVATED AGRICULTURAL LAND [4]				
Fallow				
Bare soil	77	86	91	94
Crop residue	74	83	88	90
Row crops				
Straight row	67	78	85	89
Straight row + Crop residue	64	75	82	85
Contoured	65	75	82	86
Contoured + Crop residue	64	74	81	85
Contoured & terraced	62	71	78	81
Contoured & terraced + Crop residue	61	70	77	80
Close-seeded legumes / rotated meadow				
Straight row	58	72	81	85
Contoured	55	69	78	83
Contoured & terraced	51	67	76	80
OTHER AGRICULTURAL LAND [5]				
Pasture, grassland, or range				
Poor condition	68	79	86	89
Fair condition	49	69	79	84
Good condition	39	61	74	80
Meadow, continuous grass, non-grazed	30	58	71	78
Brush, brush/weed/grass mix				
Poor condition	48	67	77	83
Fair condition	35	56	70	77
Good condition	30 [6]	48	65	73
Woods/grass combination [7]				
Poor condition	57	73	82	86
Fair condition	43	65	76	82
Good condition	32	58	72	79

Woods [8]				
Poor condition	45	66	77	83
Fair condition	36	60	73	79
Good condition	30	55	70	77
Farmsteads	59	74	82	86

[1] Average runoff condition, and Ia = 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 percent 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.

[4] Only CNs for hydrologically "good" conditions are listed for "Cultivated Agricultural Land" because NHDES Alteration of Terrain Rules require the use of "good" hydrologic condition when selecting the appropriate CN. Curve Numbers for hydrologically "poor" conditions, other cover types, and other hydrologic conditions are available from NRCS.

[5] Hydrologic condition is based on combinations of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface toughness.

Poor: Factors impair infiltration and tend to increase runoff.
Good: Factors encourage average and better infiltration and tend to decrease runoff.

- For conservation tillage poor hydrologic condition, 5% to 20% of the surface is covered with residue (less than 750 lbs./acre for row crops or 300 lbs./acre for small grain).
- For conservation tillage good hydrologic condition more than 20% of the surface is covered with residue (greater than 750 lbs./acre for row crops or 300 lbs./acre for small grain).

[6] If actual curve number is less than 30, use CN = 30 for runoff computation.

[7] CNs shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CNs for woods and pasture.

[8] Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed, but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and the litter and brush adequately cover the soil.

(Source: Adapted from Part 630 National Engineering Handbook)

Appendix C

STATE OF NEW HAMPSHIRE
INTRA-DEPARTMENT COMMUNICATION

DATE: dd, mm, year

FROM: Author, P.E.
Title

AT: Department of Transportation
Bureau of Highway Design

SUBJECT: Location
Stormwater
City or Town, NH

TO: Internal Client
Title

Urban or Rural stormwater ?

Vertical Datum: NAVD 88 ?

Overview

Review similar memos if needed.

Review of pertinent site data:

- Bullet list of data and documents pertinent to the review memo or report

Hydrology

Discusssion on the methods used and why. Description of watershed characteristics.

Hydraulics

Summary for analysis and modelling methods or calculations for culvert, closed stormwater system, treatment facilities, or bridge.

Conclusions

Appropriate for context of draft or final design. Any conditions that need to be addressed or concerns. Refer to prior documents for guidance if necessary.

ABC/xyz
Cc:

Design projects typically include "Supplemental Narratives" and Prosecution of Work (POW) documents that aid in permitting and design intent.

Appendix D

SAMPLE SCOPE OF WORK

Two-Dimensional Hydraulic Modeling Analysis for NHDOT Projects

(this document is revised from a similar sample scope of work suggested by the FHWA Resource Center)

This sample Scope of Work (SOW) provides the Department with a starting point template for a consultant contract SOW for a two-dimensional (2D) riverine hydraulic analysis. This document is not legally binding in its own right, and compliance with FHWA technical guidance described in this document is voluntary only. The types of projects that might justify two-dimensional hydraulic analysis include the partial list below. Further guidance on when 2D modeling is justified or otherwise required by Federal law or regulation can be found in FHWA HDS 7 "Hydraulic Design of Safe Bridges," Table 4.1.

- *Hydraulic design and scour analysis for a new bridge, replacement bridge, or culvert*
- *Scour evaluation of a new or existing bridge*
- *Design of scour countermeasures*
- *Major and complex culvert analysis and design*
- *Streambank stabilization*
- *Design of protection for road embankments in longitudinal floodplain encroachments*
- *Design of multiple-opening crossings*
- *Floodplain impact analysis of highway projects*
- *Habitat analysis*
- *Channel rehabilitation or realignment analysis*
- *Sediment transport analysis*
- *Tidal hydraulics – For some projects it may be sufficient to represent tidal boundary conditions when modeling bridge hydraulics, while for others, the addition of wind and wave action must be considered. For more information, refer to the FHWA Primer on Modeling in the Coastal Environment https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif18002.pdf*

This SOW is directed at the use of 2D hydraulic modeling for highways in the river environment. While some aspects of the content are applicable to coastal hydraulic modeling, the document is focused on, and intended for use in riverine settings. In coastal regions, hydraulic and hydrodynamic forces should be combined with coastal processes such as tidal generation and propagation, wave stresses created as waves break on the coast, and wind setup of the water surface. These topics are beyond the scope of this reference document.

The exact content of the SOW for a particular study will vary depending on the type of project and its purposes.

Throughout this sample SOW, instructional content for use by the Department is provided as italicized text within boxes (such as this text). These instructions should be deleted and not included in the actual SOW incorporated into a contract.

1. GENERAL

The Consultant shall perform the two-dimensional (2D) hydraulic modeling analysis in accordance with this Scope of Work (SOW).

1.1 Purpose

Two-dimensional hydraulic analysis is required for this project to accurately determine the values of certain hydraulic parameters under a range of flow conditions. The parameters to be calculated include flood elevation and lateral extent, flood depth, velocity (magnitude and direction), and shear stress, among others. The resulting values will be used for items listed below as well as other project elements not listed here (remove any that do not apply to a particular project):

- Verification of bridge waterway adequacy: determination of minimum bridge low chord profile, assessment of road overtopping location and magnitude, scour evaluation, etc. for a bridge design project.
- Scour evaluation of an existing bridge.
- Floodplain impact analysis or compliance documentation (Use of HEC RAS for LOMRs may be preferable until such time as 2d hydraulics becomes routine for FEMA review). The least costly modelling method for compliance w/ FEMA should be used unless a good reason exists for the application of 2d hydraulics.
- Design or evaluation of bridge scour for a proposed structure.
- Design or evaluation of stream instability countermeasures.
- Embankment protection analysis and design.
- Streambank stabilization design.
- Design and analysis of major and/or complex culverts.
- Accurate understanding of complex flow patterns for design of multiple bridge/culvert openings either in series, in combination, or both.
- Environmental permitting support and habitat analysis.
- Channel rehabilitation or realignment analysis.

The list above can be edited by omitting items not relevant to the particular project or adding other relevant items as needed.

1.2. Reference Documents

The following documents are incorporated by reference into this SOW. Conformance with the requirements and recommendations of these documents is expected in the performance of the work unless written justification is provided for deviation from any relevant recommendations.

- Most current version of the NHDOT Bridge Design & Drainage Design for Highways Manuals
- FHWA Reference Document “*Two-Dimensional Hydraulic Modeling for Highways in the River Environment*”

- FHWA HDS 2 *"Highway Hydrology"*
- FHWA HDS 7 *"Hydraulic Design of Safe Bridges"*
- FHWA HEC-18 *"Evaluating Scour at Bridges"*
- FHWA HEC-20 *"Stream Stability at Highway Structures"*
- FHWA HEC-23 *"Bridge Scour and Stream Instability Countermeasures"*

1.3 Software Requirements

The paragraph below calls for the use of SMS and SRH-2D, which is the modeling software currently recommended by the FHWA.

Note: The U.S. Government does not endorse products or manufacturers. Trademarks or manufacturer's names appear in this document only because the Government considers them essential to the objective of the document. The document includes them for informational purposes only and the Government does not intend them to reflect a preference, approval, or endorsement of any one product or entity.

The Consultant shall obtain and maintain the software and licenses required to develop, run, edit and apply the results of the two-dimensional analyses required under this SOW. Required software includes the graphical user interface Surface-water Modeling System (SMS) together with the Sedimentation and River Hydraulics-Two Dimensional (SRH-2D) model recommended by FHWA. In future years, it is possible that the Army Corp. HEC RAS modelling software will build-in features for the design of transportation infrastructure with 2d hydraulics. Until such time only one dimensional HEC RAS models should only be used for NHDOT projects (typically FEMA studies). Supporting geospatial software such as: ArcGIS, MicroStation, Carlson, OpenRoads, AutoCAD Civil 3D, or other comparable software may also be useful or required. Electronic work products submitted to the Agency shall be produced by the most current version of the relevant software at the time they are submitted.

2. DATA COLLECTION

2.1 Project Vertical Datum

All elevation values depicted in the hydraulic modeling, and in the presented results of the modeling, shall be reported in the North American Vertical Datum of 1988 (NAVD 88) and in US Survey feet, unless a different vertical datum and units are specified by the Department for the specific project or study. It is anticipated that the use of US Survey feet will cease after the implementation of GRAV D by NGS.

2.2 Horizontal Coordinate System

The horizontal coordinate system used in the model development and in the resulting model output will be NH 2011State Plane, FIPS 2800 Zone_US_ft. in the North American Datum of 1983 (NAD83).

2.3 Typical Data Provided by the NHDOT

The Department is responsible to provide the following:

- Record or as-built drawings of all highway and bridge facilities that fall within the model extents.
- Any available hydraulic analysis or design reports associated with highway and bridge facilities that fall within the model extents.
- Any hydraulic models (1D or 2D) that have already been developed for the reach of interest or adjacent to the reach of interest.
- Recent inspection reports for bridges owned by the NHDOT that fall within the model extents.
- Maintenance or asset management records of major structures and drainage systems within the model extent.
- Any available design documents and/or drawings for the current project.
- Previous scour analyses.

2.4 Additional Data Collected by the Consultant

The Consultant's services under this SOW will include collecting, obtaining, or developing the following data in support of two-dimensional hydraulic modeling:

- Information on flood hydrology from all available official sources (streamflow gauge data; Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) reports; previous hydrology studies for the same watershed; high-water marks; anecdotal information from adjacent landowners).
- Aerial imagery (a time history of aerial photographs may be desirable to inform assessments of channel migration rates).
- Any available, topographic and/or bathymetric data for the study reach that is in the public domain (refer to Section 2.5 for accuracy requirements). The Consultant will examine available topographic data and make recommendations on suitability for modeling to the Agency.
- Site specific soil survey and geotechnical analyses (if scour analysis is performed).

If suitable topographic data are not available, the Consultant shall acquire and process topographic digital data meeting the accuracy requirements described in Section 2.5 of this SOW, with coverage throughout the model domain. If suitable bathymetric data are not available the Consultant shall acquire and process bathymetric (below water) digital data meeting the accuracy requirements described in Section 2.5, extending along the entire stream channel reach within the model domain.

2.5 Requirements for Underlying Topographic and Bathymetric Data

The Consultant is responsible for verifying the suitability of the topographic and bathymetric data (hereafter referred to as "terrain") that are used for assigning elevations to the two-dimensional mesh. Suitable data, at a minimum, will have the following properties:

The traditional practice of collecting cross sections a fixed distance upstream and downstream of a bridge for every project is not acceptable, or adequate for 2d modelling. As a general rule of thumb, topographic data should be aquired for a min. distance of two floodplain widths upstream and downstream of the road.

- The horizontal coordinate system and projection, along with the vertical datum, are known and can be re-projected, if necessary, to the model's system and datum.
- The terrain data are in a digital format, or can readily be converted into a digital format, that can be read and used by the selected modeling software (for example, usable as SMS scatter data, or a RAS Mapper Terrain).
- Within the areas of key interest (e.g. in the vicinity of the project site and at significant hydraulic controls throughout the model domain), the terrain data meets the accuracy standard of NDDDA RMS 0.3 feet in Chapter 4 of the FHWA reference document *"Two-Dimensional Hydraulic Modeling for Highways in the River Environment"*
- Throughout the remainder of the model domain (outside the areas of key interest), the terrain data meets the accuracy standard of NDDDA RMS 0.6 feet in Chapter 4 of the FHWA reference document *"Two-Dimensional Hydraulic Modeling for Highways in the River Environment."*

2.6 Site Assessment

A thorough site assessment is essential to developing a successful hydraulic model study. The requirements below focus on the observations needed for modeling. Projects typically require a geomorphic and stream stability assessment.

The Consultant shall conduct a site reconnaissance visit for the purpose of documenting conditions on the ground. The Consultant shall prepare and submit, within one month of completing the field reconnaissance, a Site Assessment Memorandum. The memorandum will document the observations from the reconnaissance and will include:

- Copies of field notes.
- An organized documentation of potential roughness value assignments for various ground cover type (for example worksheets facilitating the use of Cowan's method).
- Descriptions and photographs of debris blockages.
- Descriptions and photographs of major hydraulic control features (crossings, diversions, critical depth sections, levees or other longitudinal floodplain encroachments, major obstructions, scour-resistant rock or other vertical grade controls, downstream water bodies, etc.).
- Document apparent high-water marks (e.g. marks on structures, seed lines, debris in trees, etc.) through GPS, descriptions, and photographs.
- Description and photographs of any indicators of lateral channel instability (e.g. recent bank retreat, cut banks, fresh point bars, channel braids, etc.).
- Description and photographs of any indicators of vertical channel instability (e.g. depositional areas, head cutting, incision terraces, exposed foundations of structures or bridge piers, etc.).
- Dimensional measurements of bridge and culvert openings within the study reach.

- Description of the likelihood of pressure flow and/or roadway and bridge deck overtopping at existing bridge (For bridge replacements or bridge scour evaluations).
- Description and photographs of any visible scour holes (For bridge replacements or bridge scour evaluations).
- Documented and photographed inlet and outlet end inspections (For culvert replacement or rehabilitation projects).
- Description and photographs of any scour holes at either end but especially the downstream end (For culvert replacement or rehabilitation projects).
- Photographs at locations of established FEMA model cross sections (For projects requiring FEMA documentation).

2.7 Hydrology

Unless the Department has specified a particular approach to obtaining or developing the discharge rates to be used in the hydraulic analysis, the Consultant shall submit a proposal to the NHDOT for the approach to hydrologic analysis. The proposed approach and resulting discharge values must be approved by the Department before proceeding with the hydraulic analysis. Flood frequency studies obtained from the U.S. Geological Survey (USGS), FEMA, US Army Corps of Engineers (USACE) or analyses conducted by state or local agencies are good resources. The following list reflects a general preference, in descending order.

- Adopt and use the flood-frequency relationship presented in previous studies by the NHDOT for the same reach (unless there is a good reason to update or revise).
- Adopt and use the flood-frequency relationship (peak discharge rate for various flood recurrence intervals) from the FEMA FIS for the reach of interest.
- Adopt and use the flood-frequency relationship provided for the reach by the USACE or other federal or state entities.
- Perform Flood-Frequency Analysis using annual peak flood values from streamflow gauges on the same stream reasonably near the study site.
- Perform detailed rainfall-runoff analysis to develop flood hydrographs for a range of recurrence intervals.
- Use USGS regional regression equations (possibly implemented in the USGS Streamstats website).

3. HYDRAULIC MODEL REQUIREMENTS

This section of the SOW sets out the expectations for an acceptable model under the contract. These requirements are important because there is a broad spectrum of quality and detail in hydraulic modeling. Failure to understand and agree on the modeling expectations will likely result in an outcome that doesn't meet the needs of the project or that requires excessive financial and schedule resources. Brief descriptions of the various elements of a suitable model are provided in this section. The FHWA Reference Document "Two-Dimensional Hydraulic Modeling for Highways in the River Environment" provides detailed guidance relevant to this section of the SOW.

3.1 General

The requirements in this section establish the expectations for the quality of the hydraulic model. The precise specifications for quality depend upon the purposes of the modeling study, and how the model results will be used. Certain main components are common to all two-dimensional modeling studies:

- An efficient, accurate geometric mesh of sufficient extent and appropriate detail.
- Correct boundary condition assignments (inlets, & outlets of domain, & at bridges and culverts).
- Appropriate Manning's n assignments (Results of NCHRP study 24-49 are anticipated within a couple of years).
- Correct handling of hydraulic structures and controls within the model domain.
- Appropriate model control parameters.

3.2 Extents of Model Domain

In establishing the model domain extents, the Consultant shall, at a minimum:

- Encompass at least the full inundation width of the largest flood recurrence interval to be simulated, typically the 500-year flood.
- Extend the domain far enough downstream of the locations of interest (e.g. the project site) that the uncertainty associated with any user-imposed water surface boundary conditions does not affect the model results at the location of interest.
- Extend the domain far enough upstream to fully depict any impacts caused by the project on the water surface and velocity magnitude (both increases and decreases).
- Where feasible, extend the domain upstream to a location where the flow is consolidated, without any need to distribute the flow among multiple inflow boundary condition assignments.
- If the inflow must be distributed among multiple inflow boundary conditions, extend the domain upstream far enough that the user-estimated flow distributions do not affect the accuracy of model results at the locations of interest.
- Demonstrate through sensitivity analysis that the model results at the locations of interest are not sensitive within a reasonable variation of the downstream water surface assignment or upstream flow boundary condition distribution.

Chapter 5 of the FHWA reference document *"Two-Dimensional Hydraulic Modeling for Highways in the River Environment"* provides detailed guidance for establishing the appropriate model domain limits, and the Consultant shall follow those recommendations to the extent practicable and must provide justification where certain recommendations cannot be followed.

3.3 Mesh Configuration, Density, Level of Detail

The appropriate configuration of the model mesh is project specific and dependent on the objective of the model study, on the topographic and geometric complexity of the study reach, and on the size and scale of hydraulically important features within the model domain. Note that with modern mesh development software, it is tempting and easy to develop an extremely detailed mesh that provides a high level of accuracy throughout the domain. The Department should guard against this tendency, however, because even though the development of such a model is easy for the Consultant, the end product can be extremely difficult to use later because of excessive model run times. A model delivered to the Department that requires many hours to run a single simulation is not acceptable because of its impact on project design costs and schedule.

The SOW should require adherence to the recommendations of the FHWA reference document "Two-Dimensional Hydraulic Modeling for Highways in the River Environment." To incorporate the level of detail and judgment required directly into a SOW would require an unreasonable amount of contractual volume. Therefore, the SOW text provided below is very general and mostly calls attention to the reference document.

The Consultant shall:

- Develop a sufficiently detailed mesh with small enough element sizes in the vicinity of the project and at significant hydraulic controls to achieve sufficient accuracy for the study purposes.
- Demonstrate that the mesh represents the terrain to a sufficient level of accuracy for the study purposes.
- For design projects, develop separate meshes for proposed conditions versus existing conditions.
- Check the mesh quality criteria (e.g. element area changes, aspect ratio, interior angle, etc.) and mitigate serious and/or widespread mesh quality issues.
- Provide appropriate variation in model density based on proximity to the project area and to significant hydraulic controls.
- Exercise due diligence in the tradeoff between small element sizes for more detail and accuracy versus larger elements sizes for greater speed of computation.
- Keep the total number of elements within a single model to less than 100,000 if feasible
- Consult with the Agency prior to submittal if the final model is to have considerably more than 100,000 elements.
- Obtain prior approval from the Department if the model run time for a single simulation is to be more than 2 hours when run on a standard grade business personal computer.

Chapter 5 of the FHWA reference document *"Two-Dimensional Hydraulic Modeling for Highways in the River Environment"* provides detailed guidance on appropriate configuration and density of the mesh. It also defines the appropriate level of accuracy for studies of various purposes. The Consultant shall follow those recommendations to the extent practicable and must provide justification where certain recommendations cannot be followed.

3.4 Manning's n Assignments

The Consultant should be expected to exercise diligence in assigning Manning's n values to the various regions of the model. This includes considering the authoritative references on the topic (identified in the FHWA reference document "Two-Dimensional Hydraulic Modeling for Highways in the River Environment"), and discerning the appropriate references for the stream setting (e.g. high-gradient, wide and broad flat floodplains, etc.). The model domain must be completely covered by Manning's n assignments.

The reference document explains depth variable Manning's n values available in some modeling programs and these may be appropriate where the effective roughness for a certain ground cover type is sensitive to the flow depth. Additionally, the Consultant should be expected to make a good faith effort to identify calibration data and to perform calibration or verification to the extent feasible. Ideally, the calibration event corresponds to the highest flow event with available data.

In assigning Manning's n values throughout the model domain, the Consultant shall:

- Assign Manning's n values that reflect the ground cover and resistance characteristics in each location.
- Define regions (e.g. polygons), not overlapping and together covering the entire model domain, each having a Manning n value that will be assigned to the mesh elements within that polygon.
- Assign Manning's n values that are defendable considering standard authoritative references on Manning's n assignment.
- Make use of depth-variable Manning's n values as appropriate.
- Make a good faith effort to obtain data for use in calibration or verification of the model's Manning's n assignments.
- Perform calibration or verification as allowed by the data obtained.
- Account for any differences in Manning's n values that are appropriate for the proposed versus existing condition.
-

Chapter 5 of the FHWA reference document *"Two-Dimensional Hydraulic Modeling for Highways in the River Environment"* provides detailed guidance and cites many authoritative references on assigning Manning's n values. Chapter 6 describes the processes of calibration and verification. The Consultant shall follow the recommendations of Chapter 5 to the extent practicable and must provide justification where certain recommendations are not followed. Additionally, the Consultant is expected to seek out and make use of calibration/verification data for use in developing and validating Manning's n values.

3.5 Structures

The Consultant should be expected to follow sound practices in representing structures of different types in a 2D model. The FHWA 2D reference document provides extensive descriptions of best practices for modeling highway bridge and culvert crossings, along with many other types of structures. The SOW text below is a partial summary of the recommendations of the reference document.

The Consultant is expected to develop model simulations that accurately represent the hydraulic effects of various manmade structures that exert a significant hydraulic control. The Consultant shall represent the effects of various types of structures as described below.

Weirs or Overtopped Embankments

- Accurately represent the horizontal alignment of the side slopes and the top with element edges forming break lines.
- Accurately represent the embankment cross section with at least two rows of elements on each side slope and two rows of elements on top and with accurate elevation assignments on all nodes.
-

Bridges and/or Culverts

- Accurately represent the bridge waterway opening shape and elevations.
- Accurately represent the locations, alignment, orientations and shapes of the abutments.
- Represent piers eithers as voids in the mesh or using drag forces applied at the pier locations.
- If the bridge low chord is expected to be partly or fully submerged, incorporate the effects of pressure flow (for instance, using the pressure flow feature in SRH-2D) and align element edges along the upstream and downstream faces of the bridge deck.
- Represent the waterways of large box culverts using the same approach as required for bridge waterways.
- Represent smaller or non-rectangular culverts, or large box culverts that are located well outside the area of interest, using 1D culvert modeling routines.
-

Spurs, Guide Banks, or Bendway Weirs

- Represent spurs, guide banks or bendway weirs by aligning elements for an accurate depiction of their horizontal layout, crest profile and cross section geometry.
- If spurs or guide banks are not to be overtopped in the simulation, they may be modeled as voids in the mesh.

Grade Control and Drop Structures

- Align element edges at the crest and at the toe of the grade control for accurate depiction the channel bed profile.
- If the actual step in the grade control is vertical, depict it with a steep, non-vertical slope.

Buildings in the Floodplain and Enclosures with Solid Fences

- Individual buildings or enclosed areas with solid fences expected to have a significant effect on the hydraulics in the area of interest should be represented by creating a void in the mesh within the building or enclosure footprint.
- Represent widely scattered smaller buildings (such as a residential subdivision without solid fences, outside the area of interest) by using a higher Manning n value to represent the aggregate effects of the buildings, rather than creating many voids in the mesh.

Note that small elements are typically needed to meet the requirements described above for the various structure types, while maintaining mesh quality. The FHWA reference document *"Two-Dimensional Hydraulic Modeling for Highways in the River Environment"* provides detailed guidance on properly modeling the structures of various types.

3.6 Boundary Conditions

The Consultant is responsible for assigning correct boundary conditions to the model. The essential boundary conditions are external: inflow and exit boundary conditions. Inflow boundary conditions put flow into the model, either a constant, steady discharge value or a time-varying discharge hydrograph. The inflow boundary conditions must reflect the hydrology that has previously been established or performed for the study.

Exit boundary conditions usually set the water surface elevations at the locations where flow exits the model domain. The water surface elevation at an exit location can be specified as a certain known value, a value that will be calculated based on normal depth, or as a time-varying stage hydrograph (such as in a tidal zone). The exit conditions must be based on the best available information for the specific reach being modeled. Such information may include previous studies or known values of the water surface elevation for certain flowrates.

The Consultant shall assign appropriate boundary conditions as needed. Such boundary conditions will include, but are not necessarily limited to:

- One or more inflow boundary conditions specifying a discharge value associated with the event being modeled (as developed or obtained in the earlier hydrology work).
- One or more outflow (exit) boundary conditions specifying starting water surface at the model's downstream external boundary(ies) based on the best available information, which could be:

- Assumed normal depth (if the flow conditions just downstream of the exit location are generally uniform), or
- A known value, taking information from another resource such as a previous hydraulic study, or
- A time-dependent water surface hydrograph (such as in a tidal waterway).

- Flow monitoring or continuity lines to track the discharge across the model at different locations.

The FHWA reference document *"Two-Dimensional Hydraulic Modeling for Highways in the River Environment"* provides detailed guidance on properly developing and assigning boundary conditions.

4. MODEL OUTPUT AND RESULTS

The output and results from a 2D model are primarily in the form of graphical plots and tables of values. The Consultant should be expected to prepare a variety of plots and tables to communicate the results to reviewers and the design team.

FHWA regulation 23 Code of Federal Regulations (CFR) Content of Design Studies [§650.117] requires project plans to show the water surface elevations of the base flood (i.e., 100-year flood) and overtopping flood [§650.117(c)].

4.1 Model Development Information

The Consultant shall prepare and submit the following, at a minimum, to clarify details of the model development:

- Plan view elevation contour plot of the terrain surface.
- Summary table of mesh statistics: number of elements, minimum and maximum elements size, etc.
- Plan view plots of the mesh elements for existing and proposed conditions.
- Plan view plots of mesh quality for existing and proposed conditions.
- Plan view plots of the Manning's n (material type) zones for existing and proposed conditions.
- Tables of the Manning's n values used for each zone.
- Plan view plots of mesh elevation contours for existing and proposed conditions, for comparison to the terrain surface.
- Cross section plots showing mesh elevations versus terrain.
- Plan view plot showing the locations of monitoring points and monitoring lines.

4.2 Sensitivity Analysis and Calibration/Verification

Sensitivity analysis reveals the degree to which the results of a specific model can be influenced by inaccuracies of certain input variables. The assigned water surface elevations at outflow boundaries usually incorporate an unknown amount of error, and if the project site is too close to the location of the outflow boundary this error might affect the model results where they matter the most. The Consultant shall, after running the model with the most likely outflow water surface elevation using the best available information (as determined under Section 3.6 above), perform sensitivity simulations with the assigned water surface values set higher and lower than the most likely value, covering a reasonable range of uncertainty. The Consultant shall prepare and submit water surface profile plots showing the longitudinal water surface profile along the main channel, left overbank and right overbank for different settings of the outflow boundary water surface elevations.

Assuming that adequate data were available to perform calibration or verification of Manning's n values (see Section 3.4 above), the Consultant shall prepare plan view plots showing the locations of observed high water marks or velocity measurements along with the model's computed water surface elevation contours or velocity magnitude contours. If calibration or verification was not feasible, the Consultant shall perform simulations with uniformly higher Manning's n values and uniformly lower Manning's n values, covering a reasonable range of uncertainty. The Consultant shall examine the results of the models and document the differences between the models in and around the project site location.

4.3 Results from Study Simulations

The Consultant shall prepare and submit the following, at a minimum, to explain the results of the simulations for each recurrence interval of interest:

- Summary tables of discharge across lines in various locations for existing and proposed conditions.
- Summary tables of water surface elevation, depth and velocity magnitude at various points throughout the model for existing and proposed conditions. In SMS, summary tables can be generated to document the minimum, maximum and average values for depth, water surface elevation, velocity and other variables along a line.
- Plan view contour plots of resulting water surface elevations for existing and proposed conditions.
- Plan view contour plots of resulting velocity magnitudes (including velocity vectors) for existing and proposed conditions.
- Plan view contour plots of the differences (differential plots) in results for proposed versus existing (possibly including multiple alternatives for proposed conditions).
- Water surface profile plots (along river channel, left overbank and right overbank) showing the terrain, existing conditions water surface, and proposed conditions water surface.

4.4 Additional Analysis for Bridge Waterway Capacity Design

For bridge design projects, the Consultant shall provide the following, in addition to the output and results described above:

- Determination of freeboard above the design-flood water surface for the low chord of a proposed bridge.

- Determination of freeboard above the design-flood water surface for the approach road profile.
- Determination of the distribution of flow through multiple bridge/culvert openings.
- Determination of overtopping event, if applicable.

4.5 Additional Analysis for Bridge Scour Evaluation

For bridge scour evaluation, the Consultant shall provide the following, in addition to the output and results described above:

- Contour plots of shear stress in the vicinity of the bridge waterway and in the main channel upstream and downstream of the bridge.
- Contour plots of velocity magnitude and vectors showing flow direction in the vicinity of the bridge opening.
- Calculation of clear-water or live-bed contraction scour at proposed or existing bridge.
- Calculation of local scour at bridge piers and abutments.
- Calculation of required riprap size for abutment, channel bank, or road embankment protection.
- Tables summarizing hydraulic variables used for scour analyses.

4.6 Additional Analysis for Culvert Design

For culvert design, the Consultant shall provide the following, in addition to the output and results described above:

- Determination of culvert headwater elevation.
- Determination of culvert outlet velocity.
- Calculation of expected outlet scour hole for the design event.

4.7 Additional Analysis for River Rehabilitation Design

For river rehabilitation design, the Consultant shall provide the following, in addition to the output and results described above:

- Contour plots of shear stress for lower discharge rates (bank-full flow and lower).
- Contour plots of stream power for lower discharge rates (bank-full flow and lower).

4.8 Additional Analysis for FHWA and FEMA Floodplain Regulatory Compliance

For bridge evaluations within a designated floodplain, the Consultant shall provide the following, in addition to the output and results described above:

- Tables of average water surface elevation along established FEMA model cross section lines for the 100-year flood, comparing proposed to existing conditions. In SMS, summary tables can be generated to document the minimum, maximum and average values for depth, water surface elevation, velocity and other variables along a line.
- Table of average water surface elevation along established FEMA model cross section lines for the 100- year flood with the regulatory floodway encroachments in place, comparing proposed and existing conditions.

- Table of the floodway widths at established FEMA model cross section lines for the 100-year flood, comparing proposed and existing conditions.
- Water surface elevations of the base flood (i.e., 100-year flood) and overtopping flood.

5. MODEL REVIEW

A two-dimensional hydraulic model can be developed through a rather mechanical process without much attention to accuracy or to hydraulically important features and controls. The model results can be misleading or unusable unless a thorough process is in place to ensure the quality of the modeling. Quality starts with assigning conscientious engineers, well-trained in hydraulics and two-dimensional modeling, to the model development effort. Quality is controlled and assured, however, by consistently applying best practices in model review.

The Consultant is responsible for delivering a model with reliable, accurate results that are useable for the model study's defined purposes. The Consultant shall perform and document the review processes described below. The documentation shall be provided in the form of a Review Memorandum.

5.1 Top-Down Review

The Agency's technical staff may choose to perform some elements of a top-down review as described below. The Consultant, however, must be responsible for conducting such a review before submitting deliverables to the Agency or the design team.

The Consultant's engineer in responsible charge, or a designated senior hydraulic engineer, shall perform a top-down review of the model prior to submitting it as final. The review shall be documented in the Review Memorandum, stamped by a registered professional engineer, which provides a narrative of the review, findings and conclusions. The Consultant shall perform and document the review steps below, at a minimum:

- Verify that current versions of the modeling software are being used.
- For steady-state model studies, verify that the simulation has run through enough time steps to achieve steady state (results no longer changing with time) at multiple monitoring line locations throughout the model domain.
- For steady-state model studies, verify that flow continuity is maintained at multiple monitoring line locations throughout the model domain for the final time step.
- For unsteady-flow models, plot flow hydrographs at multiple monitoring line locations throughout the model domain and verify that the hydrograph progression (shift in time, attenuation of peak discharge) appears reasonable.
- Verify that the water surface contours look reasonable.
- Explain differences between existing and proposed water surface contours.
- Verify that the resulting velocity magnitude contours and velocity vectors look reasonable.

- Explain differences between existing and proposed velocity magnitude contours and velocity vectors.
- Document whether the model results reveal a previously unknown problem with the design (for example, road overtopping where it was not expected).
- Verify that the ground surface as represented by the mesh elevations match the terrain closely, especially within the area of the project site.
- Verify that the computer time required to complete a simulation is reasonable, and not unduly burdensome.
- Follow up on adverse findings from the above review steps with further investigation of the modeling at specific locations where issues are found.
- Change the model to address issues or provide an explanation in the Review Memorandum as to why the issue is not a significant concern.

5.2 Bottom-Up Detailed Review of Model Input

During the model development process, a peer-level review must be conducted by the Consultant team that examines each part of the model input in detail. The peer reviewer should be someone trained in two-dimensional modeling but not otherwise involved in the current study. The peer review is considered a bottom-up review. Exhibit A, attached to this SOW, is a sample review checklist. The Consultant shall use this checklist or an equivalent. The review memorandum described in the previous subsection should make reference to this bottom-up review and include the completed checklist as an attachment.

The Consultant shall assign an engineer trained in 2D modeling but not otherwise involved in the current study to perform the following:

- Conduct a peer-level review examining each part of the model input in detail.
- Document the review by completing the review checklist in Exhibit A of this SOW or an alternative, equivalent checklist.
- Submit the completed checklist as an attachment to the Review Memorandum.

6. HYDRAULIC MODELING REPORT REQUIREMENTS

6.2 6.1 General Outline

FHWA regulation 23 CFR §650.115 "Hydraulic Design Standards" applies to all Federal-aid projects. The design standard requires development of a "Design Study" for each action in an encroachment (§650.115(a)). Regulation 23 CFR §650.117 "Content of Design Studies" requires such studies to contain the hydrologic and hydraulic data and design computations [§650.117(b)]. As both hydrologic and hydraulic factors and characteristics lead to scour formation, such data and computations apply to scour as well. As described earlier, project plans must show the water surface elevations of the base flood (i.e., 100-year flood) and overtopping flood [§650.117(c)].

Your Agency may have a standard outline already in use for hydraulic modeling studies. If so, it would be appropriate to use the same outline with minor modifications to accommodate two-dimensional modeling instead of one-dimensional modeling. Exhibit B is provided as a standard report outline for inclusion in a Scope of Work if your Agency doesn't already have a standard.

The Consultant shall thoroughly document the two-dimensional hydraulic modeling in a report following, in general, the outline presented in Exhibit B of this SOW. If the Consultant finds that this outline is not ideal or relevant for the study, minor deviations are acceptable, but significant deviations must be approved by Agency staff.

6.2 Professional Certification

The Consultant must affix to the report the signed stamp of a registered professional engineer in the state in which the project is located, with a statement that the signing engineer was in responsible charge of the work.

7. DELIVERABLES

The Consultant shall submit the following deliverables, at a minimum:

- The Hydraulic Modeling Report, as described in the previous section.
- The Site Assessment Memorandum.
- The Review Memorandum, complete with model review checklist as an attachment.
- If SRH-2D and SMS are used: SMS project files and SRH-2D model output files, with an index explaining what each file is (include only the files from simulations actually used and cited in the report) (Delete all case_RST*.dat files that are not used to support a simulation, as these unnecessarily increase the size of project archives.)
  - The 'File: Save As Package' option may be used in SMS to ensure that all project files are included in a zip file for transfer or archive.
  - The SMS 'Edit: Project Metadata' feature should be used to note important information, including:
    - Model developer
    - Location information

- Project purpose
- Source of terrain data
- Source of material roughness information
- Source of inflow data
- Structure notes
- Simulation summary (why specific simulations were performed)
- Calibration data information

- Results plots (if not already included in the report) that depict significant findings of the modeling.
- Supporting data, in electronic form, including:
  - The digital terrain model files (LiDAR, TIN files, etc.).
  - Projection files that established the horizontal coordinate system.
  - As-built drawings of existing conditions and design drawings of proposed conditions (only the sheets that depict the hydraulically significant features).
- All floodplain related information relevant to the project including the governing regulations, correspondence or contact with floodplain administrators, and documentation of analysis that demonstrates compliance.

Page intentionally left blank

REFERENCES

1. NHDOT, Utility Accommodation Manual, February 2010, or current rev.
2. NHDOT, Highway Design Manual, Chapter 6, current rev.
3. NHDOT, Policy for the Permitting of Driveways and Other Accesses to the State Highway System, March 10, 2000 or current rev.
4. NHDOT, "Construction Manual", June 1, 2006, or current rev.
5. NHDOT, Best Management Practices for Routine Roadway Maintenance in New Hampshire, dated August 2001, Rev. March 21, 2003, or current rev.
6. NHDOT, "Bridge Design Manual", Chapter 2, Section 2.7 Bridge Hydraulic Study, current rev.
7. NHDOT "Guidelines for Temporary Erosion Control and Stormwater Management", 2002, or current rev.
8. NHDOT Survey Manual, current rev.
9. NHDES, "New Hampshire Stormwater Management Manuals" Volumes 1, 2 & 3, December 2008
10. UNH, Stream Crossing Guidelines, May 2009.
11. The UNH Stormwater Center http://www.unh.edu/unhsc/presentations
12. AASHTO, "Highway Drainage Guidelines", 4th Ed. 2007, or current rev.
13. ASCE, "Curve Number Hydrology-State of the Practice"
14. FHWA, Highway Hydrology; (HDS 2), 2002, or current rev.
15. FHWA, "Urban Drainage Design Manual", HEC 22, August 2001, or current rev.
16. FHWA, "Hydraulic Design of Highway Culverts", Jerome M. Norman, Robert J. Houghtalen, and William J. Johnston, Hydraulic Design Series No. 5 (HDS 5), Office of Bridge Technology, FHWA, April 2012, 3rd ed. or current rev.
17. FHWA, Hydraulic Engineering Circular (HEC) 26. Culvert Design for Aquatic Organism Passage, October 2010 or current rev.
18. Open Channel Hydraulics", Ven Te Chow, PhD, University of Illinois, The Blackburn Press, 1959.
19. Elements of Applied Hydrology", Don Johnstone and William P. Cross, The Ronald Press Company, New York, 1949.
20. "Urban Storm Drainage Management", John R. Sheaffer, Kenneth R. Wright, Marcel Dekker Inc., 1982.
21. Maine DOT Drainage Manual
22. Penn DOT Drainage Manual

23. Conn DOT Drainage Manual
24. Washington State DOT Drainage Manual
25. CAL Trans Appendix F.
26. USGS, "Runoff Estimates for Small Rural watersheds and Development of a Sound Design Method", Joel E. Fletcher, A. Leon Huber, Frank W. Haws, and Calvin G. Clyde, Final Report 1977, FHWA, Office of Research and Development.
27. USGS, "Estimates of Flood Discharges at Selected Recurrence Intervals of Streams in New Hampshire", USGS SIR 2008-5206, by Scott A. Olson in cooperation with NHDOT.
28. "StormCad for Windows", User's Guide, StormCad v5,Haestad Methods Inc., 2003.
29. HydroCAD help files
30. NRCS, Musgrave & other
31. Katopodis, Chris, P. Eng. – Introduction to Fishway Design, Freshwater Institute, Department of Fisheries & Oceans, Manitoba, Canada.
32. Handbook of Culvert and Drainage Practice, Armco Drainage & Metal Products, Inc., copyright 1950, Printed in U.S.A. by R.R. Donnelley & Sons Company, Chicago.
33. Stuctural Analysis and Design of Pipe Culverts, National Cooperative Highway Research Program Report 116, Highway Research Board, 1971, by Raymond J. Krizek, Richard A. Parmelee, J. Neil Kay, and Hameed A. Elnaggar, Northwestern University, Evanston, Illinois.
34. Publication No. FHWA H1-98-032 Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures, NHI Course No. 132068, May 2001 or current rev.
35. Section 12 of the AASHTO LRFD Bridge Design Specifications *Buried Structures and Tunnel Liners*, , or current rev.
36. AASHTO LRFD *Bridge Construction Specifications, current rev.*
37. Technical Paper 40