



**Department of
Transportation**

HIGHWAY DESIGN MANUAL

Chapter 8

Highway Drainage

Revision 97

(Limited Revision with Errata issued by EB 22-022)

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Section	Changes
Throughout	Reformatted chapter for legibility, updated document references, removed obsolete references, and incorporated hyperlinks. Exhibits renumbered to accommodate new exhibits in revised Section 8.6.1.1.
§8.6.1.1 Allowable Headwater	Section has been updated to meet guidance from the <i>New York State Flood Risk Management Guidance for Implementation of the Community Risk and Resiliency Act</i> .
Exhibit 8-3 (formerly Table 8-2)	“Design Flood Frequencies (in years) for Drainage Structures and Channels” table has been revised to reflect community risk and resiliency standards for local roads.
Appendix 8B	Appendix renamed to remove SPDES Permit number. Updated document references, removed obsolete references, and updated hyperlinks.
Appendix 8B, Exhibit 1	Removed “Streambank restoration projects” as an example since they are no longer included in the SPDES Permit list of routine maintenance activities.
Appendix 8B, Exhibit 3	Values revised to reflect 2015 Stormwater Design Manual updates.
Appendix 8B §2.3.2.2.1	Kinderhook Lake watershed added to list of Phosphorus Restricted Watersheds
Appendix 8B §2.3.2.5	Stormwater Management Design Manual page references corrected.
Appendix 8B §2.3.2.7	Removed reference to inspections after rain events to reflect the current SPDES Permit and NYSDOT MURK 6 forms.
Appendix 8B §2.4 (22)	Removed reference to Draft NYSDOT Stormwater Management Practices Operation & Maintenance Manual (no longer on NYSDOT website) and replaced with reference to NYSDEC Maintenance Guidance for Stormwater Management Practices.
Appendix 8B §2.5	Section updated to include NYSDEC’s electronic NOI submission process.
Section 8.6.2.3 Errata issued with EB 22-022	Corrected the standard sheets referenced for cut-off wall details. Corrected minor typographical errors.

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8.1 INTRODUCTION

Highway drainage is an important consideration in the design of many projects. The term drainage is defined in several different ways, including the process of removing surplus groundwater or surface waters by artificial means, the manner in which the waters of an area are removed, and the area from which waters are drained. A project may alter the existing drainage. When this occurs, drainage features should be provided which protect the highway, adjacent landowners, and the traveling public from water, while maintaining water quality and protecting other environmental resources.

The American Association of State Highway and Transportation Officials (AASHTO), the Federal Highway Administration (FHWA), the U.S. Army Corps of Engineers (USACE), the National Resource Conservation Service (NRCS), and the U.S. Geological Survey (USGS) are the predominant source of guides, manuals and other documents to aid in the design of highway drainage features. In addition, "NYSDOT Guidelines for the Adirondack Park" provides information for consideration when designing projects within the Adirondack Park. (Refer to [Chapter 2](#), Section 2.3.4 of the *Highway Design Manual* for further information regarding the use of this guideline.) AASHTO's "Highway Drainage Guidelines" presents an overview of highway drainage design. Procedures, formulas, and methodologies are not presented in detail. AASHTO's "Drainage Manual" provides procedures, formulas, methodologies, and example problems. FHWA's "Hydraulic Design Series" and "Hydraulic Engineering Circulars" provide guidance, formulas, and example problems on various subjects. The USACE, NRCS, and the USGS provide guidance regarding specific hydrologic, and hydraulic methodologies.

Rather than repeat the detailed information in the publications mentioned above, this chapter provides an overview of highway drainage which is consistent with these publications and refers to them as necessary throughout the text.

Departmental drainage design criteria, Regional experience, and other guidance is given where it may differ from the information presented in the referenced publications. Refer to [Chapter 5](#), Section 5.1.2 for guidance regarding deviation from other design elements (drainage design criteria).

8.2 LEGAL ASPECTS OF HIGHWAY DRAINAGE

The Department is obligated by State and Federal laws and regulations to protect:

- The highway from rainfall and runoff.
- Adjacent land beyond the highway from the discharge of artificially collected and concentrated flow from highway channels.
- Floodplains.
- Water quality and natural resources.

Questions regarding our legal obligation to protect adjacent landowners from the Department's alteration of existing drainage should be addressed to the Office of Legal Affairs for opinion. Questions regarding water quality and protecting natural resources should be directed to the Regional Environmental Contact.

The legal aspects of highway drainage are discussed at greater length in "The Legal Aspects of Highway Drainage" (Chapter 5 of AASHTO's "Highway Drainage Guidelines") and "Legal Aspects" (Chapter 2 of AASHTO's "Drainage Manual").

8.2.1 State Drainage Law

State drainage law is derived from common law based on two historical lines of thought: the old English common law rule ("common-enemy rule") and the "civil law rule". These rules developed into the "reasonable use rule". The law in New York seems to be based on the common law rule, modified by the law of reasonable use. Common law is that body of principles which developed from immemorial usage and custom and which receives judicial recognition and sanction through repeated application. These principles were developed without legislative action and are embodied in the decisions of the court. State drainage law is not located in "McKinney's Consolidated Laws of New York Annotated".

State drainage law defines surface waters (runoff) and natural watercourses (natural channels) and establishes the legal consequences of their alteration. Each situation is unique, and the circumstances involved play a prominent role in determining legal liability, as well as rights and duties. When in doubt, legal opinion should be sought from the Office of Legal Services.

Projects which alter existing drainage patterns should be progressed in accordance with the following guidance which is based on "Drainage of Surface Waters":

1. Every effort shall be made to perpetuate the natural drainage pattern that existed prior to the construction of the highway. Collection and diversion of flows should be avoided whenever possible to limit the Department's liability from these actions.
2. When existing drainage patterns are disturbed by collection, diversion, elimination of ponding areas, or increasing stream velocities, provisions shall be included in the contract documents to return the drainage pattern downstream of the project to approximately the conditions existing before the project, as quickly as is feasible.

3. Whenever possible, the natural drainage pattern shall be re-established within the highway right of way.
4. Downstream drainage easements (usually permanent easements as described in [Chapter 5](#), Section 5.5.6) shall be taken for all drainage from the highway boundary (right of way) to a point downstream where the pre-project drainage pattern has been re-established. This point will usually be the location at which all collected waters would have entered the stream had the project not been built. However, the point may be that place at which the velocity returns to its natural state. At times, this may involve a considerable length which should require special studies and investigations. Economics may dictate taking an easement to a major water course without determining the point of re-established conditions. (Note: It may be argued that in rough terrain there would be little chance of downstream improvements being made and, therefore, there is no need to take downstream easements. If land in these areas is inexpensive, it would cost little to protect the Department from some future court action. There is no guarantee that a piece of property will never have capital improvements.)
5. Upstream drainage easements (permanent easements) shall be taken where necessary to provide adequate storage for headwater resultant from a drainage facility. These easements should be large enough to accommodate access to adequately maintain the drainage facility. Contact the Regional Maintenance Group to verify the size and location of the easement before the appropriation map is scheduled to be produced.
6. Consider improving existing downstream structures, to protect downstream landowners from increased flooding potential, when the flow reaching the structure is increased significantly because of the proposed highway improvement. An equally acceptable solution would be the creation of upstream storage areas.
7. Existing structures which become inadequate by the loss of their upstream storage areas due to highway construction shall be improved.

8.2.2 State and Federal Environmental Laws and Regulations

The Environmental Manual (TEM) contains guidelines – prepared consistent with the various state and federal laws and regulations – which should be followed during project development. This guidance generally reflects State and Federal interagency concurrence on the most expeditious methods for the progress of Department activities. Copies of the laws and regulations are maintained by the Regional Environmental Contacts.

Refer to the [Project Development Manual](#) (PDM), Appendix 1, for a list of federal and state laws, rules, and regulations related to the environment, and guidelines for their implementation.

Exhibit 8-1 lists areas of environmental concern associated with highway drainage and the corresponding reference in the TEM.

Exhibit 8-1 Areas of Environmental Concern

Area of Environmental Concern	Location of Guidance in TEM
Wetlands	4.4.1
Wild, scenic and recreational rivers	4.4.3
Coastal zone	4.4.6
Floodplains	See note 1
Water quality	See note 2
Endangered species	4.4.9.3

Note 1. IPDG 24 Flood Plain Management Criteria for State Projects is still in force, although it is not in the EPM. A copy of IPDG 24 may be obtained from the Regional Hydraulics Engineer or the Office of Structures

Note 2. Contact the Environmental Science Bureau for guidance on water quality and State Pollutant Discharge Elimination System (SPDES) regulatory requirements.

Sections 8.2.2.1 to 8.2.2.7 discuss these areas of environmental concern in greater detail.

8.2.2.1 Wetlands

In addition to the discussion presented in Sections A through C, wetlands are also regulated under water quality. Refer to Section 8.2.2.5.C.

A. Executive Order (EO) 11990 Protection of Wetlands (May 24, 1977)

EO 11990 was enacted to minimize the destruction, loss or degradation of wetlands and to preserve and enhance the natural and beneficial values of wetlands through proper planning. FHWA NY Division has developed a programmatic wetland finding to streamline the wetland findings process for simple/minor projects.

TEM 4.4.1 Appendix D includes information on when projects may use the programmatic wetland finding and the additional information which must be included in Design Approval Documents in order to qualify for this use. Additionally, TEM 4.4.1 includes information on processing an individual wetland finding under EO 11990, public notification requirements, approval by FHWA and requirements for the Design Approval Document.

B. Article 24 of the Environmental Conservation Law (Title 6 of the State of New York Official Compilation of Codes, Rules and Regulations, 6NYCRR, Part 663-665)

This article establishes regulations to preserve, protect and conserve freshwater wetlands. A New York State Department of Environment Conservation (NYSDEC) Freshwater Wetlands Permit is required for any project activities, including excavation, erecting structures, grading, grubbing, filling, draining, clear-cutting, or work on drainage structures or channels within the established boundary of a state regulated wetland or within its adjacent 100 ft. wide buffer area. NYSDEC jurisdictional wetlands are generally 12.4 acres or larger and are mapped by NYSDEC.

The Adirondack Park Agency regulates activities in and adjacent to freshwater wetlands on all lands within the Adirondack Park pursuant to Article 24 and Adirondack Park Agency Rules and Regulations (9 NYCRR 578). Within the Adirondack Park, all wetlands 1.0 acre or larger are regulated (100 ft. adjacent area) and all wetlands smaller than 1.0 acre are regulated if there is a free exchange with open water (e.g., streams, ponds, lakes). Regulated activities include any form of excavation, filling, draining, polluting, clearcutting, and erecting structures.

C. Article 25 of the Environmental Conservation Law (6 NYCRR Part 661)

This article establishes regulations to preserve, protect and enhance tidal wetlands. A NYSDEC Tidal Wetlands Permit is required for all regulated project activities, including dredging, grading, excavating or constructing bridges or drainage structures within tidal wetlands areas and adjacent areas that extend up to 300 ft. inland from the wetland boundary (up to 150 ft.) within New York City).

8.2.2.2 Wild, Scenic, and Recreational Rivers

A. Wild and Scenic Rivers Act (16 USC 1271, 36 CFR 251, 297, and 43 CFR 8350)

The federal Wild and Scenic Rivers Act requires consultation with the National Park Service for any proposed federal activity affecting a "Listed, Study or Inventory River". Department activities shall not affect the free-flowing character or scenic value of designated rivers or affect the future designation of inventory or study rivers. Regulated activities include expanding or establishing new river crossings or adjacent roads, clearing, and filling.

B. Article 15 of the Environmental Conservation Law (6 NYCRR Part 666)

The Wild, Scenic and Recreational Rivers Act was developed to protect and preserve, in a free-flowing condition, those rivers of the state that possess outstanding natural, scenic, historical, ecological and recreational values. Project activities within a designated river or its immediate environs (generally 0.5 mi. each side, outside the Adirondack Park and 0.25 mi. each side, within the Adirondack Park) must be designed to prevent significant erosion or direct runoff into the river. NYSDEC has jurisdiction over rivers on public lands within the Adirondack Park and on lands outside the Park. The Adirondack Park Agency has jurisdiction over rivers on private lands within the Adirondack Park.

8.2.2.3 Coastal Zone

A. Article 34 of the Environmental Conservation Law (6 NYCRR Part 505), the Coastal Erosion Hazard Act

This article requires a permit from NYSDEC for any project proposed within a coastal erosion hazard area.

B. Article 42 of the Executive Law (19 NYCRR 600 and 601), The Waterfront Revitalization

and Coastal Resources Act

This article requires project activities in coastal areas to be consistent with NYS Department of State's (NYSDOS) 44 coastal policies and, to the maximum extent practicable, with approved municipal Local Waterfront Revitalization Plans (LWRP's). NYSDOS coastal policies and LWRP's include provisions to protect wetlands and surface water resources from erosion and sedimentation and other non-point source pollution.

8.2.2.4 Floodplains

A floodplain or flood prone area is any land or area susceptible to being inundated by water from any source. A flood or flooding means a general and temporary condition of partial or complete inundation of normally dry land areas from the overflow of inland or tidal waters, or the unusual and rapid accumulation or runoff of surface waters from any source.

A. National Flood Insurance Program (NFIP)

NFIP regulations are contained in 44 CFR Parts 59-77. The following acts describe the program:

1. The National Flood Insurance Act of 1968 (PL 90-448), as amended, was enacted to provide previously unavailable flood insurance protection to property owners in flood-prone areas.
2. The Housing and Urban Development Act of 1969 (PL 91-152) added mudslide protection to the program.
3. The Flood Disaster Protection Act of 1973 (PL 93-234) added flood-related erosion to the program and requires the purchase of flood insurance as a condition of receiving any form of federal or federally-related financial assistance for acquisition or construction purposes with respect to insurable buildings and mobile homes within an identified special flood, mudslide, or flood-related erosion hazard area that is located within any community participating in the program. (A community includes any State or area or political subdivision thereof which has authority to adopt and enforce flood plain management regulations for the areas within its jurisdiction.) In addition, the act requires that on and after 7/1/75, or one year after a community has been notified by the Federal Insurance Administrator (FIA) of its identification as a community containing one or more special flood, mudslide, or flood-related erosion hazard areas, no federal financial assistance shall be provided within such an area unless the community participates in the program.

To qualify for the sale of federally subsidized flood insurance, a community must adopt and submit to the FIA as part of its application, flood plain management regulations. New York's regulations are contained in 6 NYCRR Part 502.

It is possible to comply with the federal requirements regarding the encroachment of a highway on a floodplain and still be faced with future legal liabilities because of the impact of the highway on the floodplain and the stream. The Regional Hydraulics Engineer should

review these potential liabilities and ensure that their evaluation is considered when the final highway location is selected.

B. Executive Order (EO) 11988 Floodplain Management (May 24, 1977)

EO 11988 requires each Federal agency to take the following actions:

1. To reduce the risk of flood loss, to minimize the impact of floods on human safety, health and welfare, and to restore and preserve the natural and beneficial values served by floodplains, and
2. To evaluate the potential effect of any actions it may take in a floodplain, to ensure its planning programs reflect consideration of flood hazards and floodplain management.

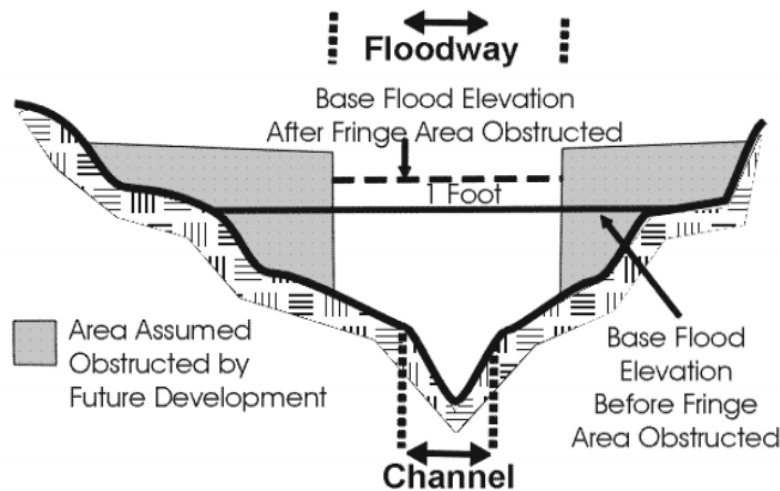
These requirements are contained in the Federal Aid Policy Guide (FAPG) under 23 CFR 650 Subpart A, Location and Hydraulic Design of Encroachments on Flood Plains.

C. Article 36 of the Environmental Conservation Law (ECL) - Participation in Flood Insurance Programs, and Part 502 (6 NYCRR) - Flood Plain Management Criteria For State Projects

This article establishes regulations (6 NYCRR 502) for Departmental and other State agency implementation to insure that the use of State lands and the siting, construction, administration and disposition of State-owned and State financed facilities are conducted in ways that will minimize flood hazards and losses. As previously discussed, the regulations are required for the State to continue its qualification as a participating community in the NFIP administered by the Federal Insurance Administration of the Department of Housing and Urban Development.

Projects which involve flood hazard areas shall be progressed in accordance with the criteria in Section 502.4. A flood hazard area consists of the land in a floodplain within a city, town or village subject to a one-percent or greater chance of flooding in any given year. A Flood Hazard Boundary Map (FHBM), or Flood Insurance Rate Map (FIRM) is available from the municipality, NYSDEC, or the Regional Hydraulics Engineer, for those flood hazard areas which have been delineated by the FIA.

Exhibit 8-2 illustrates the flood hazard area and the 100 year flood plain.

Exhibit 8-2 Flood Hazard Area

From *FEMA 480 – National Floodplain Insurance Program (NFIP) Floodplain Management Requirements: A Study Guide and Desk reference for Local Officials*, February 2005

8.2.2.5 Water Quality

A. Section 401 of the Federal Water Pollution Control Act

This section (33 USC §1341) authorizes state environmental agencies to certify that proposed dredge and fill disposal projects will not violate state water quality standards before a federal permit is granted. NYSDEC administers Section 401 through its 401 Water Quality Certification (WQC) Program. The issuance of any federal permit (e.g., U.S. Coast Guard Section 9, U.S. Army Corps of Engineers (USACE) Section 10 and Section 404) is contingent on receipt of a NYSDEC 401 WQC.

B. Section 402(p) of the Federal Water Pollution Control Act

This section (added in 1987; 33 USC §1342(p)) requires the Environmental Protection Agency (EPA) to establish final regulations governing stormwater discharge permit application requirements under the National Pollutant Discharge Elimination System (NPDES) program. NYSDEC implements the federal program through its State Pollutant Discharge Elimination System (SPDES) program.

Under the NYSDEC SPDES Stormwater Construction General Permit, a Notice of Intent must be filed and a Stormwater Pollution Prevention Plan (SWPPP) must be prepared as discussed in Appendix B, Section 2.4 of this chapter, and Chapter 4.4.8 of *The Environmental Manual*. Refer to Section 8.8, Erosion and Sediment Control and Stormwater Management, and Appendix B, *NYS DOT Design Requirements and Guidance for State Pollutant Discharge Elimination System (SPDES) General Permit for Construction Activity* for additional guidance regarding the need for coverage under the SPDES General Permit.

NYSDEC does not have jurisdiction on federal land within New York State. For projects on Indian Lands, refer to the NPDES Construction General Permit in [The Environmental Manual](#) (which references the *Environmental Procedures Manual* (EPM) Chapter 4.3, Attachment 4.3.C.).

C. Section 404 of the Federal Water Pollution Control Act

This section (33 USC §1344) prohibits the unauthorized discharge of dredged or fill material, including incidental discharge from excavation activities, into waters of the United States, including federal-jurisdictional wetlands. USACE Section 404 Permits are required for the placement of fill materials into waters of the United States associated with the construction, repair or replacement of highways, bridges and drainage structures and facilities.

D. Section 10 of the Rivers and Harbors Act of 1879

This section (33 USC 403 and 33 CFR 320-325) prohibits the unauthorized obstruction or alteration, including the excavation or deposition of materials in and the construction of any structure in or over designated navigable waters of the United States without a Corps of Engineers Permit.

E. Section 1424 (e) of the Safe Drinking Water Act - Sole Source Aquifers

This section, EO 12372, and a 1974 FHWA/EPA MOU requires certain Federally funded projects over designated sole source aquifers to be coordinated with EPA and modified, if necessary, to protect the water quality and quantity of the aquifer.

Projects over state primary water supply and principal aquifer areas should also consider potential contamination of aquifers and include best management practices as appropriate. Although no permitting program exists, other regulations such as SPDES, Protection of Waters, Wetlands Act, etc. can also be used to require project modification.

F. Article 15 of the NYS Environmental Conservation Law

This section (ECL Article 15-0501 and 15-0505, and 6 NYCRR Part 608) regulates the disturbance of the bed and banks of any protected stream (class CT or higher) and the excavation and fill in any navigable water. The Department is exempt from permit requirements. However, the Department is obligated through a 1996 MOU to coordinate with DEC and modify projects if necessary.

G. Water Quality Classifications and Standards (6 NYCRR Part 700-705)

These regulations classify surface and groundwaters according to their potential best usage (Part 701) and establish water quality standards to protect water quality (Part 703). Department projects, whether or not a permit is required, cannot result in a violation of established water quality standards (e.g., no increase in water turbidity that causes a

substantial visible contrast to natural conditions).

8.2.2.6 Endangered Species

The Federal Endangered Species Act of 1973, as amended, Section 7 (16 USC 1536) prohibits federal agencies (e.g., FHWA, USACE, Coast Guard) from taking actions (e.g., approving funding or issuing permits) that would be likely to jeopardize the continued existence of endangered or threatened species. Projects with activities involving drainage, wetlands and surface waterbodies must be evaluated for potential impacts affecting federally listed or proposed endangered or threatened species.

8.2.2.7 Fish and Wildlife

The Fish and Wildlife Act of 1956 (16 USC 742a et seq.), the Migratory Game-Fish Act (16 USC 760c-760g) and the Fish and Wildlife Coordination Act (16 USC 661-666c) express the concern of Congress with the quality of the aquatic environment. The United States Fish and Wildlife Service is authorized to review and comment on the effects of a proposal on fish and wildlife resources under federal permit processes. It is the function of the regulatory agency (e.g., USACE, Coast Guard) to consider and balance all factors, including anticipated benefits and costs in accordance with the National Environmental Procedures Act, in deciding whether to issue the permit (40 FR 55810, December 1, 1975).

Federal Aviation Administration (FAA) guidance regarding attracting wildlife (prohibits stormwater facilities that may attract waterfowl within certain distances of runways) can be found in:

[FAA Advisory Circular 150/5200-33B: HAZARDOUS WILDLIFE ATTRACTANTS ON OR NEAR AIRPORTS](#)

8.2.3 Connections to State Drainage Facilities

Sections 8.2.3.1 and 8.2.3.2 discuss the different types of connections to state drainage facilities which shall be considered.

8.2.3.1 Sanitary Sewer Connections

Private or municipal sanitary sewer connections may exist within our right of way.

A. Private Sanitary Sewer Connections

The discharge of private sanitary sewer systems into a state highway drainage facility is illegal. Department representatives who suspect a private sanitary sewer connection to a storm sewer, culvert, or open channel within the right of way should contact the local representative of the Department of Health for their recommendation in removing the subject trespass. If the sanitary sewer pipe terminates outside the right of way and the effluent flows into a highway drainage facility, the Department's representative should

contact the Department of Health and request the necessary corrective action be taken. (This arrangement has been agreed to by Memorandum of Understanding between the Department of Transportation and the Department of Health dated 4/11/66.) The cost of corrective action shall be paid for by the private owner.

B. Municipal Sanitary Sewer Connections

Existing sanitary sewer systems may be separate or combined with our existing storm drainage system. The Department will participate in the relocation of separate, and combined systems if our project affects them. If the municipality desires any improvements to their system, this is a betterment and should be treated as such.

8.2.3.2 Storm Drainage Connections

A. Private Storm Drainage Connections

1. Existing Connections. It is the Department's responsibility to maintain existing surface water drainage across and along the highway right of way. Due to the nature of typical section design for cuts, fill and curbed highways, the collection and redirection of the flow of surface water will continue. In many corridors, the highway drainage system has become the only way that the stormwater or groundwater discharge from adjacent development can be conveyed to a natural watercourse. The Department's highway work permit process, and a municipality's site plan approval process, generally ensure that discharges from developments are designed in a manner that does not increase flows that have the potential to damage the highway or downstream properties. If adding discharge to a State system is unavoidable, the applicant shall assume the cost of altering the downstream section of the State system to accommodate the increased flow.

There will be locations where connections to the state's storm drainage system were constructed without our knowledge or approval and where damage has occurred or may occur in the future. In situations where designers suspect this to be the case, they should first consult with the Resident Engineer for whatever historical information may be available. If there is potential for damage, or if damage has occurred in the past and it is clear that we cannot correct the situation at nominal cost, a coordinated approach by the Department and the municipality is advisable.

2. Proposed Connections. Private development adjacent to State Highways will not be allowed to significantly increase either the runoff velocity or rate of runoff as it enters the State highway drainage system. This policy is administered through the highway work permit process. (Refer to Procedure 7.12-2 of the "Manual of Administrative Procedures", and the "Policy and Standards for Entrances to State Highways" for additional guidance.) Generally, a retention or detention system is necessary to restrict the flow rate of a development's drainage discharge to the pre-development rate. In situations where the development is discharging near the downstream end of a relatively large drainage area, an undetained connection may be appropriate. This allows the increased flow from the development to pass downstream prior to the peak flow from the watershed. Detention in these situations can actually increase the watershed peak flow. Proper analysis of the watershed's characteristics is necessary.

Consideration shall be given to redevelopment projects in corridors with older storm drainage facilities. There may be an existing development that was constructed with stormwater discharge to the state's highway drainage facilities without proper consideration of the effect on our system. If this site is being proposed for redevelopment, any deficiencies in the design of the original development's storm drainage system shall be corrected at the expense of the owner.

B. Municipal Storm Drainage Connections

Proposed storm drainage systems may be progressed in cooperation with a municipality. The municipality shall participate in the cost of cooperative projects as discussed in Section 8.7.8.

8.3 HYDROLOGY

Hydrology is a science concerned with the occurrence, distribution, and movement of water. The necessity and extent of the hydrologic analysis to be performed is based on the type of project.

Typically, a design discharge (usually the peak discharge) must be determined for a unique design flood frequency and storm duration. On occasion, the peak discharge must be determined from plotting a hydrograph. An overview of the process of performing a hydrologic analysis, including criteria (design flood frequency) and methodologies for determining the peak discharge (with and without the use of a hydrograph), are discussed in this section. The publications referred to in the text regarding specific methodologies (i.e. "Urban Hydrology for Small Watersheds (TR-55)",) are necessary to guide the user through the process. Hydrology is discussed at greater length in "Hydrology" (Chapter 2 of the "Highway Drainage Guidelines"), "Hydrology" (Chapter 9 of the "AASHTO Drainage Manual"), and "Hydraulic Design Series" (HDS) No. 2, "Highway Hydrology".

8.3.1 Type of Project vs Extent of Hydrologic Analysis

The extent of the hydrologic analysis to be performed should be based on project type as follows (Not every project will require a hydrologic analysis for all locations within the project limits.):

1. Construction on new alignment - A hydrologic analysis is required to determine the need for, and necessary capacity of drainage features (culverts, open channels, storm drainage systems, etc.).
2. Reconstruction on existing horizontal and vertical alignment - A hydrologic analysis shall be performed for all drainage features located within the project limits having a history of flooding, and for open and closed drainage systems with a remaining service life less than the design life of the highway improvement.
3. Resurfacing, Restoration and Rehabilitation (3R) - A hydrologic analysis is required for all drainage features having a history of flooding, or in need of replacement or major repair. Extension of a culvert to flatten side slopes usually does not require a hydrologic analysis, but shall require a hydraulic analysis to establish the flow of water in the new drainage pattern.
4. Pavement Preventive Maintenance Projects - No hydrologic analysis is required.
5. Culvert replacement or relining project - A hydrologic analysis is required.

8.3.2 Hydrologic Analysis

The overall process which should be used to conduct the hydrologic analysis for a given project is listed below and discussed in Sections 8.3.2.1 through 8.3.2.5.

1. Conduct preliminary research.
2. Take an initial field trip to the project site.
3. Determine if the design discharge should be calculated with or without a hydrograph.
4. Select a methodology and design flood frequency, and calculate the design discharge.
5. Take a final field trip to verify the analysis/design and to recheck flood damage potential.

8.3.2.1 Preliminary Research

Preliminary research should take place before conducting the first field trip in order to make the trip more productive:

1. Obtain available topographic information, including USGS quadrangles, county or municipal topographic maps or other recent surveys. Outline the drainage patterns and areas on a contour map.
2. Determine soil characteristics of each drainage area for the project. This information may be obtained from the NRCS County Soil Survey or from the Regional Geotechnical Engineer.
3. The Regional Hydraulics Engineer, or appropriate Regional Design group, should be contacted to determine if there is a Flood Insurance Study with an associated Flood Insurance Rate Map, and Flood Boundary and Floodway Map for the area.
4. Determine if the site is at or near a location listed in the USGS publication "[Maximum Known Stages and Discharges of New York Streams 1865 - 1989, with Descriptions of Five Selected Floods, 1913-85](#)" or "[Maximum Known Stages and Discharges of New York Streams and their Annual Exceedance Probabilities through September 2011](#)". If the site is listed, valuable hydrologic data may be provided which should be used to check the reasonableness of the flow rate computations performed in accordance with Section 8.3.2.4. In addition, the USGS publishes "Water Resources Data New York Water Year" which contains flow rates and other data for streams. Three separate volumes are published to cover the state (eastern excluding Long Island, western and Long Island).
5. Obtain available aerial photographs. These photographs can help determine lateral migration of stream channels, land use, type of terrain, tree cover, etc. Recent aerial ortho photographs are available as a part of the GIS datasets. GIS ortho images are available dating back to approximately 1995 through the GIS group. The Office of Technical Services is also a good source for earlier photos. NYSDEC has aerial ortho images dating back to around 1940. Photogrammetric images are also now being provided with highway projects. Google Earth may also be used as a source for aerial imagery.

6. Obtain any previous field reconnaissance notes. They may be of significant value in the determination of drainage patterns and areas.
7. Check with the Resident Engineer's Office to see if they have historical records of high water flow for the subject drainage area.
8. Check Highway Record Plans for the area. They can be used as a guide to size proposed structures.
9. Perform preliminary flow rate calculations for cross drainage locations.

8.3.2.2 Initial Field Trip - Data Collection

After the preliminary research has been completed, and preliminary flow rates calculated, the initial field trip should be taken. (Be sure to bring a camera, take lots of pictures, and always try to include a person or other item of known size in the picture to act as a visual scale.)

The main items to be investigated are:

1. Drainage patterns and drainage areas.
2. Land use.
3. Soil types.
4. Existing and previous flood conditions.
5. Location of natural and man-made detention features.

A. Drainage Patterns and Drainage Areas

Topographic information that covers the drainage areas should be taken into the field to verify the drainage patterns and areas. In addition, aerial photographs, record plans, and field survey or notes may be useful in the field. Cross culverts which carry water into or out of the area being investigated should be noted.

In the initial phases of a design, especially for projects on a new alignment, the roadside drainage patterns have normally not been determined. The proposed location of cross-drainage can be determined by laying out the centerline of the proposed improvement on a contour map. Drainage patterns and areas may have to be changed before the project is finalized, however, and these changes may require redesign later.

B. Land Use

The type of land and land use should be considered. Note whether the land is wooded, fallow (crop land kept free of vegetation during the growing season), plowed and planted with crops, or developed (containing a high percentage of roof areas, paved parking lots, grass lawns, etc.). Land use and cover have a large effect on peak flow rate.

C. Soil Types

Confirm preliminary soils information. Attempt to verify the runoff characteristics of the soils within the drainage area and their capabilities of resisting erosion. Look for locations where soil erosion may become, or is a problem. These locations, which are frequently channel banks, may need stone filling. The Regional Geotechnical Engineer should assist, if deemed necessary, with this determination.

D. Existing and Previous Flood Conditions

The magnitude of previous flooding conditions at the location being investigated should be determined. Evidence of past flooding may not be easily discernible since time will disguise flood damage. Evidence may be gathered by checking the following sources:

1. Determine High Water Elevations in Channels and at Structures – Existing stream channels (when there is a definite channel) show flooding effects in various ways. These effects are visible as different degrees of erosion in the stream bank or the stream bottom. Other indications which may be evident are high water marks shown as mud lines on concrete surfaces or rock faces along stream embankments, and debris which has been deposited along the channel slopes. High water marks are only to be used as indicators. High water elevation determination for an existing structure will enable the designer to more accurately calculate a maximum flow rate at the structure. The information needed is the slope, type and size of the existing structure and the estimated maximum headwater and tailwater depths.
2. Conduct Personal Interviews – Personal interview of area residents may be helpful in determining previous flooding conditions; however, the information obtained by this method may be biased. The interviewer should ask if local residents have photographs or a video taken at the time of the flooding. Information obtained from interviews of local residents and local maintenance personnel is usually helpful and can be used as an indication of high water elevations in conjunction with field observations.
3. Deposition and Scour at Existing Structures – Signs of flooding conditions at existing structures are deposition of stream bed material and scour holes near structure inlets. Scour holes near structure outlets are not as indicative of flood conditions since normal flows through a structure can cause scour and erosion at the outlet. Deposition of stream bed material, which will consist of sand, gravel and/or debris, can occur within the upper portions of a drainage structure which does not have any significant outlet control. The deposition and scour holes are indications of high headwater at the inlet. This indicates that the culvert may be undersized.
4. Debris – The size and weight of deposited material is also a general indication of the velocity of the stream during maximum flow conditions. As the stream channel grade steepens and the flow rate increases, the velocity will be faster and the heavier debris is more easily moved by the flowing water. By observing these general characteristics of a stream and culvert, a better determination of past velocities and flow rates can be made to confirm the design flow rate.

5. Regional Department of Environmental Conservation Office – Some of these offices have flood control staff who may provide information regarding flood prone areas.

E. Natural and Man-made Detention Features

The existence of any detention features should be verified. Natural detention features can take the form of wetlands, ponding areas, reservoirs, and lakes. Man-made features can include flood control dams, highway embankments, and culvert locations. These all have the effect of increasing the time of concentration, which may reduce the flow rate at the point under consideration downstream. If the feature is near the headwater of the drainage area, the effects will not be as great as the effects if the feature is closer to the point under consideration. If the storage feature is close to the point under consideration, an analysis may need to be performed using HEC-HMS "Hydrologic Modeling System" (Hydraulic Engineering Center), or another similar method to determine the proper flow rate. In some cases, detention features can reduce the required size of a downstream facility appreciably.

8.3.2.3 Peak Discharge (With or Without a Hydrograph)

Discharge is the rate of the volume of flow per unit of time. The peak runoff rate, or peak discharge, should generally be determined without calculating a hydrograph when the effects of water storage are not considered. A hydrograph should be used to establish the peak discharge when an analysis of the effects of water storage on the drainage area under evaluation needs to be considered. Examples are ponds, reservoirs, or other water storage areas within the drainage area, and when stormwater management needs to be considered.

A hydrograph is a graphical plot of discharge (flow) versus time for a specific location and recurrence interval (design flood frequency). A unit hydrograph is a special hydrograph that describes the discharge resulting from a one inch rainfall event. The height of a hydrograph is the peak flow rate, and the time to the peak discharge is the time of concentration. The shape of a hydrograph is dependent upon the watershed characteristics. A narrow, high hydrograph describes a watershed that produces runoff quickly after a rainfall event, and a broad, short hydrograph describes a watershed that produces runoff slowly after a rainfall event. The most commonly used hydrograph is the NRCS dimensionless unit hydrograph.

8.3.2.4 Flow Rate Determination

Several methods for determining flow rates are listed in items 1 through 4 below. Methods 1 and 2 are based upon rainfall effects and do not consider snowmelt. In some areas, runoff from snowmelt will regularly exceed peak events due to rainfall. Each flow rate determination method is unique. Users should be familiar with each method, and be able to select the most appropriate method for each location and situation. Additional methods are presented in the publications referenced in Section 8.3.

Other agencies, such as the USACE, have studied various larger streams and rivers in New York State. Their flow rate calculations tend to be conservative because their purpose is flood control on those rivers and streams. The FEMA has studied a large percentage of the rivers

and streams in the State through the NFIP. Flood study results should be closely reviewed because they were developed by different consultants over several decades, using a variety of methods, and they are of variable quality.

Recommended methods, which are briefly explained in sections A through D, include:

1. Rational Method. Computes a peak discharge.
2. Modified Soil Cover Complex Method ("Urban Hydrology for Small Watersheds", NRCS TR-55, is the basis for this method). Computes a peak discharge directly using a formula and by plotting a hydrograph.
3. Regression Equations. Computes a peak discharge.
4. Historical Data.

The design flood frequencies provided in Exhibit 8-3 are recommended for use with the Rational Method, the Modified Soil Cover Complex Method, and the regression equations. The design flood frequency is the recurrence interval that is expected to be accommodated without exceeding the design criteria for the open channel, culvert, or storm drainage system.

Exhibit 8-3 Design Flood Frequencies (in years) For Drainage Structures and Channels¹

Road type or Functional Class	Culvert ²	Storm Drainage Systems	Driveway Culverts	Ditches ⁴
Interstates and Other Freeways	50	10 ⁵	n/a	25
Principal Arterials	50	10 ⁵	25	25
Minor Arterials	50 ⁶	5 ⁷	10	10
Major Collectors	50 ⁶	5 ⁷	10	10
Minor Collectors	50 ⁶	5 ⁷	10	10
Local Roads & Streets w/ AADT>400	50 ⁶	5 ⁷	10	10
A or B type highways (AADT < 400) ^{8, 10}	50 ⁶	5 ⁷	10	10
C ^{8,9,10}				

NOTES

1. The values in this table are typical. The selected value for a project should be based upon an assessment of the likely damage to the highway and adjacent landowners from a given flow and the costs of the drainage facility. Note: 100-year requirements must be checked if the proposed highway is in an established regulatory floodway or floodplain.
2. The check flow, used to assess the performance of the facility, should be the 100 year storm event.
3. Relocated natural channels should have the same flow characteristics (geometrics and slope) as the existing channel and should be provided with a lining having roughness characteristics similar to the existing channel.
4. Including lining material (All ditches should have a lining material and not be left untreated).
5. As per 23CFR650A, and Table 1-1 of HDS 2, a 50-year frequency shall be used for stormwater design at the following locations where no overflow relief is available:
 - a. sag vertical curves connecting negative and positive grades.
 - b. other locations such as underpasses, depressed roadways, etc.
6. A design flood frequency of 10 or 25 years is acceptable if documented in the Design Approval Document, and when identified after design approval, in the drainage report. A design flood frequency of 10 or 25 years should be used in the design of driveway culverts and similar structures.
7. Use a 25-year frequency at the following locations where no overflow relief is available:
 - a. sag vertical curves connecting negative and positive grades.
 - b. other locations such as underpasses, depressed roadways, etc.
8. Dead end highways should use the Local Road Standard as a minimum for sizing, but the roadway and structure should be armored to handle a larger event without washing out.
9. Existing structures are considered acceptable unless there are known flooding issues which require a more detailed design. In such a case, the A/B standard should be followed.
10. See HDM Chapter 4, Table 4-1 for the definition of Type A, B and C Low Volume Highways.

A. Rational Method

This method is recommended to determine the peak discharge, or runoff rate, from drainage areas up to 200 acres. If a hydrograph is required to consider the effects of storage, use the Modified Soil Cover Complex method, or a similar method.

The Rational Method assumes the following:

1. Peak discharge occurs when all of the drainage area is contributing,
2. A storm that has a duration equal to the time of concentration (T_c) produces the highest peak discharge for the selected frequency,
3. Intensity is uniform over a duration of time equal to or greater than the T_c , and
4. The frequency of the peak flow is equal to the frequency of the intensity.

The rational method formula is:

$Q = CiA$, where:

Q = peak discharge or rate of runoff (cfs)
 C = runoff coefficient
 i = intensity (in/hr)
 A = drainage area (acres)

1. Runoff coefficient. The runoff coefficient selected shall represent the characteristics of the drainage area being analyzed. A weighted runoff coefficient (C_w) should be used in the Rational formula for drainage areas having different runoff characteristics. C_w should be calculated as follows:

$C_w = \sum C_i A_i / A$, where

C_i = runoff coefficient for subarea "i"
 A_i = subarea

Refer to Exhibit 8-4 for recommended runoff coefficients.

Exhibit 8-4 Values of Runoff Coefficient (C) for Use in the Rational Method

Type of Surface	Runoff Coefficient (C) ¹
Rural Areas	
Concrete, or Hot Mix Asphalt pavement	0.95 - 0.98
Gravel roadways or shoulders	0.4 - 0.6
Steep grassed areas (1:2, vert.:horiz.)	0.6 - 0.7
Turf meadows	0.1 - 0.4
Forested areas	0.1 - 0.3
Cultivated fields	0.2 - 0.4
Urban/Suburban Areas	
Flat residential, @ 30% of area impervious	0.40
Flat residential, @ 60% of area impervious	0.55
Moderately steep residential, @ 50% of area impervious	0.65
Moderately steep built up area, @ 70% of area impervious	0.80
Flat commercial, @ 90% of area impervious	0.80

NOTE

1. For flat slopes and/or permeable soil, use lower values. For steep slopes and/or impermeable soil, use the higher values.

- Intensity. Determine intensity i.e., the rate of rainfall upon the drainage area, using intensity-duration-frequency (IDF) curves developed for the area being analyzed, a duration equal to the time of concentration (T_c), and a frequency equal to the design flood frequency.

IDF relationships are based upon statistical analysis of rainfall data. They describe, for a given flood frequency, the average intensity of rainfall for a storm of a given duration (equal to the time of concentration). The statistical data for New York State is based upon "Technical Paper No. 40" (TP-40) and the "NOAA Technical Memorandum NWS HYDRO-35". The methodology for developing IDF curves is presented in "Drainage of Highway Pavements", Highway Engineering Circular (HEC) No. 12. To construct a set of IDF curves for a given location, HEC-12 uses six data points from HYDRO-35: the 2-year 5, 15 and 60 minute rainfalls and the 100-year 5, 15 and 60 minute rainfalls. the 60 minute rainfall for each intermediate return period is calculated from these points, and then the rainfall intensities for other durations are calculated. IDF curves for some locations are available from the Regional Design Group or should be constructed from known rainfall data.

To obtain the intensity, the T_c must first be estimated. The T_c is defined as the time required for water to travel from the most remote point in the watershed to the point of interest. The time of concentration path is the longest in time, and is not necessarily the longest in distance. Various methods can be used to determine the T_c of a drainage area. The method used to determine the T_c should be appropriate for the flow path (sheet flow, concentrated flow, or channelized flow). The minimum T_c used shall be 5 minutes.

The velocity method can be used to estimate travel times (T_t) for sheet flow, shallow concentrated flow, pipe flow, or channel flow. It is based on the concept that the travel time for a flow segment is a function of the length of flow (L) and the velocity (V):

$$T_t = L/60V$$

, where T_t , L, and V have the unit of minutes, ft, and ft/s.

The T_t is computed for the principal flow path. When the principal flow path consists of segments that have different slopes or land covers, the principal flow path should be divided into segments and the travel time associated with each flow segment should be calculated separately and summed to determine the T_c .

Velocity is a function of the type of flow, the roughness of the flow path, and the slope of the flow path. A number of methods are available for estimating velocity, and are based on the type of flow. After short distances, sheet flow tends to concentrate in rills and then gullies of increasing proportions, and is usually referred to as shallow concentrated flow.

The velocity of shallow concentrated flow can be estimated by using the following equation:

$$V = 3.28k S^{0.5}$$

where

- V = velocity in (ft/s)
- k = Value from Table 8-4
- S = slope (percent)

The value of k is a function of the land cover as shown in Exhibit 8-5.

Exhibit 8-5 k Values for Various Land Covers and Flow Regimes

k	Land cover/flow regime
0.076	Forest with heavy ground litter; hay meadow/ overland flow
0.152	Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland/overland flow
0.213	Short grass pasture/overland flow
0.274	Cultivated straight row/overland flow
0.305	Nearly bare and untilled/overland flow
0.457	Grassed waterway/shallow concentrated flow
0.491	Unpaved/shallow concentrated flow
0.619	Paved area/shallow concentrated flow; small upland gullies

Manning's equation, provided in Section 8.4.1.1, can be used to determine flow velocities in pipes and open channels.

3. Area. This is the drainage area contributing flow to the drainage feature under evaluation.

B. Modified Soil Cover Complex Method

This method is recommended to determine the peak discharge, and a hydrograph, from drainage areas up to 1 mi² (640 acres).

"Urban Hydrology for Small Watersheds", NRCS TR-55, applies the modified soil cover complex method to determine a peak discharge and a hydrograph. Only a peak discharge may be determined using the Graphical Peak Discharge method. A peak discharge and a hydrograph may be determined using the Tabular Hydrograph method. (NRCS TR-55 methodology is taken from TR-20, "Computer Program for Project Formulation Hydrology"). TR-20 is capable of more complex hydrologic and hydraulic (routing) analyses. Users should be aware of the limitations presented throughout NRCS TR-55.

Input requirements for both the Graphical Peak Discharge method and the Tabular Hydrograph method include:

1. Q – runoff (in.). Estimating runoff is covered in NRCS TR-55 Chapter 2. Variables used to calculate Q include P – rainfall (in.), I_a – initial abstraction (in.), and S – potential maximum retention after runoff begins (in.). Values of P should be calculated for the project site based on the 24 hour rainfall events as shown in Technical Paper NO. 40. The other variables should be calculated as discussed in Chapter 2.
2. T_c – time of concentration (hr.). Follow the procedure outlined in NRCS TR-55 Chapter 3. The minimum T_c for design is 6 min.
3. A_m – drainage area (mi.²). This is the drainage area under consideration.
4. Appropriate rainfall distribution (Type I, IA, II, and III). This information is necessary to compute the unit peak discharge, q_u, for the Graphical method, and the tabular hydrograph unit discharge, q_t, for the Tabular method. All New York State counties have a Type II distribution, except the following, which have a Type III distribution:

Bronx	Kings	New York	Queens	Richmond
Nassau	Suffolk	Dutchess	Orange	Putnam
Rockland	Sullivan	Ulster	Westchester	

The Tabular Hydrograph method requires T_t, travel time (hr.), in addition to 1. through 4. above. Follow the procedure outlined in [NRCS TR-55](#), Chapter 3 to determine T_t.

C. Regression Equations

This method is recommended for calculating a peak flow rate for use in the design of a natural channel relocation, and for culvert design (when the culvert is installed to pass a natural channel (e.g., a perennial stream).

[Magnitude and Frequency of Floods in New York](#), USGS Scientific Investigations Report 2006-5112, presents a series of regression equations for use to calculate the peak discharge, for recurrence intervals of 1.25, 1.5, 2, 5, 10, 25, 50, 100, 200 and 500 years,

for six hydrologic regions in New York State. For urban streams, this method shall be modified by the method recommended in [Evaluation of Six Methods for Estimating Magnitude and Frequency of Peak Discharges of Urban Streams in New York](#), USGS Water-Resources Investigations Report 84-4350. USGS has also developed the [StreamStats](#) tool for estimating stream flows utilizing regression equations.

Refer to [Magnitude and Frequency of Floods in New York](#) for details on limitations (drainage area, etc.) for each hydrologic region. These regression equations are not valid and should not be used for regulated streams, diversions, or other manmade changes, unless supplemented with a routing procedure such as HEC-HMS "Hydrologic Modeling System" or TR-20, "Computer Program for Project Formulation Hydrology".

D. Historical Data

[Maximum Known Stages and Discharges of New York Streams 1865-1989, with Descriptions of Five Selected Floods, 1913 - 1985](#), USGS Water-Resources Investigations Report 92-4042, lists maximum known stages and discharges for 1280 sites on 863 New York streams. The report lists the sources and categories of data, maximum stage and discharge, and location data for each site. Data from this report should be used, as appropriate, to check/verify peak flow estimates calculated by any of the other methods discussed in Sections A through C above. "[Maximum Known Stages and Discharges of New York Streams and their Annual Exceedance Probabilities through September 2011](#)" list maximum known stages and discharges at 1,400 sites on 796 streams within New York. Other historical data may be found in newspaper or other media accounts as well as a number of other local sources.

8.3.2.5 Final Field Trip to Verify Design

Sometimes not all of the design problems are immediately evident during the initial field trip. The designer may have to request a more accurate field survey and/or prepare cross sections to help identify other design parameters. Once the final hydrologic analysis has been completed and the designer has selected some alternate solutions which meet the design criteria, a final field trip should be taken to confirm the hydrologic and hydraulic design for the project.

During the final field trip, special consideration should be given to the effects of future potential flooding upstream or downstream. The design high water elevations are now known, so verification of the effects of these elevations can be accomplished more accurately than previously. Changes to existing drainage patterns which may result in potential damage to adjacent landowners should also be given close scrutiny at this time to avoid the potential for legal problems or claims.

8.4 HYDRAULIC PRINCIPLES

Flow in any open or closed conduit (e.g., open channel or pipe) where the water surface is at atmospheric pressure, or free, is termed open channel flow. The principles of flow in open channels are discussed in many of the publications referenced throughout the text. One such publication is [Introduction to Highway Hydraulics](#), Hydraulic Design Series No. 4 (HDS 4).

8.4.1 Types of Open Channel Flow

Open channel flow can be characterized as:

1. Steady or unsteady. Steady flow occurs when the quantity of water passing any section of an open channel or pipe is constant. Unsteady flow is characterized by variations in flow.
2. Uniform or non-uniform. Flow is considered uniform if velocity and depth are constant, and non-uniform or varied if velocity and depth of flow changes from section to section.
3. Subcritical, critical, or supercritical. These types of flow are related to depth of flow. Subcritical flow is considered tranquil and occurs at depths greater than critical depth, d_c (velocity less than critical). Critical flow occurs at critical depth. Critical depth is defined as the depth of minimum energy content. Energy of flow is discussed in Section 8.4.2. Supercritical flow is considered rapid or shooting and occurs at depths less than critical depth. The Froude number is used to define these types of flow. Refer to Section 8.4.1.2 for the equation used to calculate the Froude number.

The capacity of open channels and storm drains is determined assuming steady uniform flow. Steady uniform flow is discussed further in Section 8.4.1.1.

If steady flow is assumed to exist, but a change occurs in either the depth of flow, velocity, or cross section, the conduit should be designed based on the principles and equations for steady nonuniform flow. For nonuniform or varied flow, the energy equation should be used as the basis for the analysis.

8.4.1.1 Steady Uniform Flow

Steady uniform flow occurs when the conduit slope, cross section (size and shape), and roughness are reasonably constant along the conduit. Manning's equation is used to calculate the velocity and/or depth in an open channel or pipe under open channel conditions.

Manning's equation is:

$$V = \left(\frac{1.486}{n} \right) R^{0.67} S^{0.5} \quad \text{where:}$$

V = Velocity (ft/s)

n = Manning's roughness coefficient

R = Hydraulic radius = A/P (ft)

A = Cross sectional area of flow (ft²)

P = Wetted perimeter (ft)

S = Slope of the energy grade line, and the flow line, S_o (ft/ft). For steady uniform flow, S = conduit slope.

Combining the Manning equation with the continuity equation, Q=VA, results in a commonly used form of the Manning equation which is used as the basis to determine the capacity or discharge (Q) of an open channel, or pipe.

$$Q = A \left(\frac{1.486}{n} \right) R^{0.67} S^{0.5}$$

For a given geometric cross section, slope, roughness, and a specified value of discharge, a unique depth occurs in steady uniform flow called the normal depth (d_n). This depth is primarily a function of roughness. When d_n is known, calculating the discharge is a matter of inserting the appropriate values into the formula. When d_n is not known, and this is generally the case with open channels and pipes, the solution requires a trial and error procedure unless charts, tables, nomographs, or computer software are used.

8.4.1.2 Subcritical, Critical, and Supercritical Flow

The flow regime (subcritical, critical, or supercritical) is determined by solving for the Froude number (F_R).

$$F_R = \frac{V}{\sqrt{gD}}, \text{ where}$$

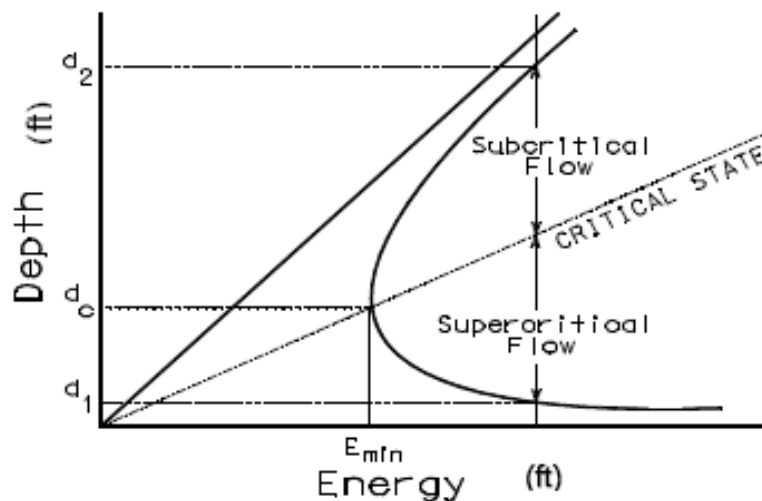
- V = Mean velocity (ft/s)
- g = Acceleration due to gravity (32.2 ft/s²)
- D = Hydraulic depth (ft) = A/T
- A = Cross sectional area of flow (ft²)
- T = Top width of flow (ft)

and

- $F_R < 1$ Subcritical Flow
- $F_R = 1$ Critical Flow
- $F_R > 1$ Supercritical Flow

Open channel flows which result in flow depths within 10% of critical depth, or which result in Froude numbers between 0.9 and 1.1, should be avoided because flow is unstable and may result in either subcritical flow or supercritical flow. As the flow approaches critical depth from either limb of the specific head curve, (as shown in Exhibit 8-6) a very small change in energy is required for the depth to abruptly change to the alternate depth on the opposite limb of curve. If the unstable flow region cannot be avoided, the least favorable type of flow should be assumed for design.

Exhibit 8-6 Specific Energy Diagram



8.4.2 Energy of Flow

Open channel flow contains energy in two forms, potential and kinetic. The potential energy is due to the position of water above some datum, and is equal the water depth (d), plus the elevation of the channel bottom above the datum plane (z). Kinetic energy is due to the velocity of water and is represented by the velocity head ($V^2/2g$). Specific energy or specific head (E) is equal to the depth of water plus the velocity head ($d + V^2/2g$). Energy can also be expressed in terms of total energy or total head ($E + z$). Total head may be used in applying the energy equation, which states that the total head (energy) at any section is equal to the total head (energy) at any section downstream plus the energy (head) losses between the two sections. The energy equation is used as the basis for determining minor losses in storm drainage systems, and in evaluating the hydraulic performance of culverts operating under outlet control.

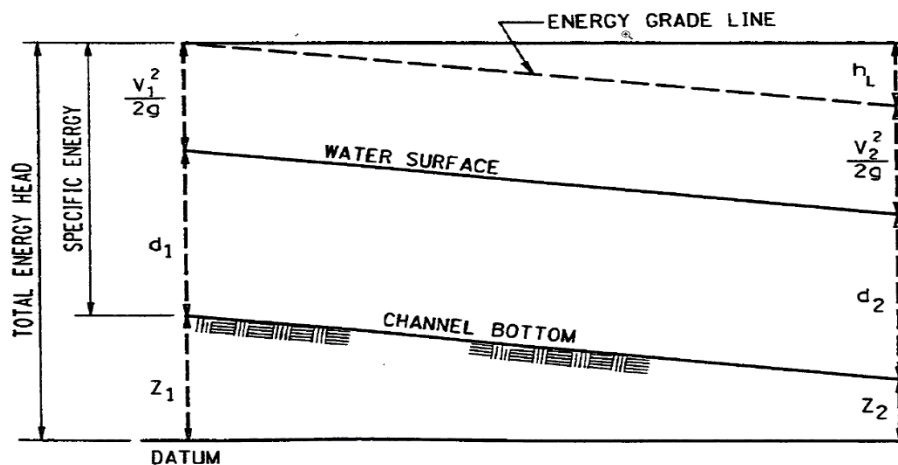
The energy (Bernoulli) equation is usually written:

$$z_1 + d_1 + \frac{V_1^2}{2g} = z_2 + d_2 + \frac{V_2^2}{2g} + h_L$$

- V = Average velocity (ft/s)
- g = Acceleration due to gravity (32.2 ft/s²)
- d = Depth of flow (ft)
- z = Channel elevation (ft)
- h_L = Head losses due to friction, transition, and bends between sections 1 and 2.

In the equation, the subscripts 1 and 2 refer to the upstream and downstream channel locations respectively. Exhibit 8-7 illustrates these concepts.

Exhibit 8-7 Total Energy in Open Channels



8.5 OPEN CHANNELS

Open channels may exist or be proposed within the roadside. Examples of existing channels which may be encountered are natural channels or watercourses (streams, creeks, etc.), and man-made (artificial, roadside) channels constructed as part of the original highway or a subsequent rehabilitation. The need for roadside channels and natural channel relocation or protection are influenced by the proposed location, terrain, and type of project.

Open channel design is discussed at greater length in "Hydraulic Analysis and Design of Open Channels" (Chapter 6 of the "Highway Drainage Guidelines"), "Channels" (Chapter 10 of the "AASHTO Drainage Manual"), and "Hydraulic Design Series" (HDS) No. 4.

8.5.1 Types of Open Channels

8.5.1.1 Roadside Channels

Roadside channels are a broad category of artificial channels located within the highway right of way which often parallel the roadway. Various types of roadside channels, some of which are referred to as ditches, are discussed in Sections A through F.

A. Gutters

Gutters are one component of a storm drainage system, and are also used to minimize embankment erosion. Gutters are:

1. Created when curb is placed at the edge of pavement. Refer to Chapter 3, Section 3.2.9, for guidance regarding types of curb, limitations on curb use, warrants for curb use, curb alignment and typical section details, and curb material selection and installation details.
2. Formed without curb as depicted on Standard Sheet 624-01, and illustrated in Figure 3-2 of Chapter 3. Refer to Chapter 3, Section 3.2.10 for guidance regarding gutter placement at driveways.

Gutter flow should be intercepted by a grate inlet or a combination inlet, and conveyed through storm drains, or a chute to a point of discharge.

B. Chutes

Open (lined with stone filling, or paved) or closed (pipe) chutes should be provided to convey water collected in gutters down steep embankments when curbing is used to minimize embankment erosion. An appropriate pay item, selected from Section 620 - Bank and Channel Protection of the Standard Specifications, should be provided at the chute outlet to minimize or eliminate erosion.

Recommended pay items for flexible lined and rigid lined open chutes are located in Section 620 – Bank and Channel Protection and Section 624 - Paved Gutters, respectively. Closed chutes and their appurtenant structures should be provided by including the

appropriate pay items found in Section 603 - Culverts and Storm Drains, Section 604 - Drainage Structures, and Section 655 – Frames, Grates and Covers. Chutes should be detailed in the plans.

C. Roadway Ditches

Roadway ditches should be provided in cut sections to remove surface runoff from the highway cross section. Refer to Chapter 3, Section 3.2.14.1 and Chapter 10 for safety considerations regarding roadway ditches.

D. Toe-of-Slope Ditches

Toe-of-slope ditches should be provided where it is necessary to convey water collected in a roadway ditch, or from adjacent slopes, to a natural channel or another point of outlet.

Refer to Chapter 3, Section 3.2.14.2, for recommendations regarding the ditch location within the roadside. Channels placed behind guide rail (installed for other reasons) should have side slopes of 1:2 (vertical:horizontal).

E. Intercepting Ditches

An intercepting ditch should be provided where it is necessary to prevent runoff from reaching the road section. These are normally locations with long, steep, backslopes.

Use a semi-circular cross section and relatively steep grades to dispose of the water intercepted by these ditches as fast as possible. A trapezoidal or flat bottom shape will tend to hold water for a longer time, resulting in potential saturation of the top of the slope which could result in slope failure. A trapezoidal channel may be required if there is a large volume of water. Side slopes of 1:2 are recommended and acceptable because these ditches are not accessible to errant vehicles.

The Regional Geotechnical Engineer should be consulted to verify the necessity of an intercepting channel and to determine if the channel may cause slope failure.

F. Median Swales (Ditches)

These ditches should be provided to drain portions of the roadway, and the median area of divided highways. When median swales do not drain directly to a natural channel, a grate inlet, drainage structure, and pipe, or a transverse ditch and culvert should be provided to intercept the collected water and pass it underneath the embankment for further conveyance or disposal. The grate inlet and drainage structure should be flush with the approach ditch line. Existing drainage structures located in the median should be reviewed for their impact on the roadside environment. Existing drainage structures may have surface pipes connected to them which are intended to increase the hydraulic efficiency of the inlet. If these surface pipes are existing and are not located behind guiderail installed for other reasons, they should be modified with a commercially available safety slope end section or a fabricated safety grate system to reduce the potential hazard to errant vehicles.

Refer to Chapter 3, Section 3.2.8, for guidance regarding geometric criteria (median width, etc.).

8.5.1.2 Natural Channels

A natural channel is a surface watercourse created by natural agents, which is characterized by the bed and banks that confine the flow of surface water. The flow in a natural channel may be periodic (e.g., ephemeral stream, intermittent stream), or continuous (e.g., a perennial stream).

Work in the vicinity of natural channels should be limited, because the best way to protect a stream is to avoid disturbing it. When work in the vicinity of natural channels is unavoidable, the natural channel shall be protected from the consequences of building and maintaining the highway, and the highway shall be protected from the channel flow. It is not desirable for the embankment to encroach upon, or to cause the relocation or alteration of a natural channel. When these situations are unavoidable, temporary provisions to protect the channel from construction operations, and permanent provisions to protect the embankment from the channel, and vice versa, shall be provided. The Department's Regional Environmental unit should be contacted to determine the stream classification (A, B, C, or D), the nature of the aquatic community, and the need for permits. These issues will influence the conditions for working in the channel.

Many of the legal aspects of highway drainage discussed in Section 8.2.2 will influence work in the vicinity of channels.

8.5.2 Channel Design Criteria

Adherence to the following criteria is recommended:

1. Roadside channels.

- a. Roadside channels should be lined to minimize or prevent erosion. Flexible linings are preferred – turf, and stone filling, in order of preference. Rigid linings are acceptable when conditions warrant their use (i.e., when increased channel capacity is needed and channel geometry is fixed).

The flow in roadside channels should be subcritical whenever possible to avoid the adverse characteristics of supercritical flow (erosion, etc.). An exception to this is chutes. The flow in chutes is commonly supercritical because of the normally steep channel slope.

To prevent deposition of sediment, the minimum slope for turf lined roadside channels should be 0.5%.

- b. The spread of water in a gutter and onto the traveled way should not exceed the spread and puddle depth (if curb is used to create the gutter) criteria stated in Section 8.7.4.4.C
- c. On long steep embankments where curbing is warranted to minimize embankment

erosion, a grate inlet, drainage structure, and closed chute is preferred to intercept and convey gutter flow down the embankment and ensure that the water will be contained within the channel. The minimum size round pipe for a closed chute should be 15 inches, except in unusual circumstances where a 12 inch round pipe may be used with the approval of the Regional Design Engineer or NYSDOT Resident Engineer.

- d. The design criteria for roadway ditches are presented in Chapter 3, Section 3.2.14.1. This includes the typical trapezoidal ditch cross section recommendations (fore slope, back slope, depth, and width), and safety considerations.
2. Natural channels. The bed and banks of relocated natural channels shall be protected from erosion by providing a lining material which is consistent with the roughness characteristics of the remaining (not relocated) channel. Permit requirements may necessitate coordinating lining specification with other agencies (Adirondack Park Agency, etc.). The geometrics and alignment of the proposed channel should be consistent with the existing channel. Provisions for fish passage shall be provided if necessary.

Where permanent stream relocation is required to meet project goals, channel work should be kept to a minimum and stream conditions should be inspected early in the design process, to determine the location and frequency of existing pools, percentage of overhead cover, amount of in-stream structure (boulders, undercut banks, buried logs, etc.) and the pool:riffle ratio. The new stream channel should be designed to replicate or improve these characteristics.

To facilitate colonization of aquatic macro invertebrates, plants and algae, the original stream bed material should be collected and relocated into the new stream channel. This should not be done until immediately before flooding the new channel. The cross section of the new stream channel should slope towards the center. For a stream up to 50 feet wide, the center line elevation should be 8 inches below the edges of the bed. On curved alignments, the slope should be towards the outside of the curve and the outside bank should be appropriately stabilized to minimize erosion.

New channel banks should be sloped 1:3, vert.:horiz., (unless adequate woody vegetation is already established) and stream banks should be planted with native trees and shrubs as close to the stream channel as flow and flood considerations allow. When stabilizing new stream banks, utilize, as appropriate, geotextiles and soil bioengineering measures such as willow wattles, live willow stakes and native woody plantings or other mutually reinforcing vegetative control measures in lieu of, or to supplement, stone fill or riprap.

In planning any new section of stream channel over 150 feet long, in-stream fisheries habitat should be enhanced by proper placement of large boulders, singly or in clusters (where no danger of scour or erosion is anticipated) to create pools and increase in-stream habitat diversity. The groups of boulders should be placed at the intervals necessary to establish a pool:riffle ratio matching the existing stream channel as closely as possible.

8.5.3 Hydraulics - Design and Analysis

As discussed in Section 8.4.1.1, the rate of flow or discharge (Q), the depth of flow (d), and the velocity of flow (V) depend on the channel's geometry (shape and size), roughness (n), and slope (S). Roughness and velocity are of particular concern because the velocity in the channel will influence the type of lining necessary, and the type of lining will define the roughness.

The design process involves establishing and analyzing a channel section – geometry (shape and size), lining material and roughness, and slope – which will convey the peak discharge (Q), determined in Section 8.3, at a depth which allows the water to flow within the selected geometry at a velocity which will not cause the lining to be displaced by the flow of water.

Current peak flows shall be increased to account for future projected peak flows for culvert design. Based on the USGS developed “Future StreamStats” tool, calculated flows in Regions 1, 2, 7, 8, 9, 10, and 11, and Cortland and Oswego Counties in Region 3 shall be increased by 20%. Current peak flows in Regions 4, 5, and 6, and Cayuga, Onondaga, Seneca and Tompkins Counties in Region 3 shall be increased by 10%. These flow increases shall be applied for all methodologies used to determine current flow rates.

8.5.3.1 Geometry (Shape and Size)

Following are considerations regarding geometry:

1. Gutters are triangular in cross section. The gutter's cross sectional dimensions will depend on the presence of parking lanes, shoulders, curb type and height, and whether or not a formed gutter (from Standard Sheet 624-01) is used.
2. Chutes should be trapezoidal (open) or circular (closed).
3. Roadway ditches should be trapezoidal or V shaped. The geometrics of the roadway channel should be determined based on the guidance in Chapter 3, Section 3.2.14.1 and Chapter 10.
4. Toe-of-Slope ditches should be trapezoidal.
5. Intercepting ditches should be semi-circular or trapezoidal.
6. Median swales and natural channels should be trapezoidal. The geometrics of a median swale will depend on the width and side slopes (embankment slope or cut slope) adopted for the median.

8.5.3.2 Linings, Lining Preferences, and Roughness

If the underlying channel material is susceptible to erosion, either a flexible or rigid lining should be provided to minimize erosion. Refer to Sections A and B for a list of the most commonly used flexible and rigid linings, and the methodologies recommended for design. Refer to Section C for a guidance regarding lining preferences, and Section D for suggested references regarding roughness.

A. Flexible Linings

Flexible linings include turf, stone filling (fine, medium, and heavy), and dry rip-rap. Specifications covering this work are located in Section 610 and 620, respectively, of the Standard Specifications.

Two methods are recommended for flexible lining design and analysis:

1. The permissible velocity approach – presented in GDP-10 "Bank and Channel Protective Lining Design Procedures", and
2. The permissible tractive force (shear stress) approach – presented in "Design of Roadside Channels With Flexible Linings" (HEC 15), HDS 4, and Chapter 10 of the AASHTO Drainage Manual, "Channels".

These methods are not recommended when the discharge under consideration exceeds 50 ft³/s. "Design of Riprap Revetment", HEC-11, is recommended for discharges greater than 50 ft³/s.

B. Rigid Linings

Rigid linings include grouted rip-rap, gabions, asphalt concrete, and portland cement concrete. Specifications covering this work are located in Section 620 and Section 624, respectively, of the Standard Specifications.

Rigid lining design and analysis are presented in HDS 4. The following provisions should be incorporated into the design when paved (asphalt concrete or portland cement concrete) linings are necessary:

1. Include cutoff walls at the terminal ends to minimize undermining.
2. Specify an apron of stone filling at the end of the paved channel to minimize erosion.
3. Depress the channel so that the lining is below the surrounding ground surface.
4. Specify underdrains where ground water flow is a problem. Underdrains are described in Chapter 9, Section 9.3.8 and the Comprehensive Pavement Design Manual.
5. Detail superelevation or additional outer wall height at bends and junctions.
6. Specify a 1.5 ft wide strip of sod on each side adjacent to the paved lining.

C. Lining Preferences

Flexible linings – turf and stone filling, are preferred over paved linings for the following reasons:

1. Maintenance – historically, paved channels have presented a maintenance problem. Paved linings:

- a. Accelerate the velocity and flow rate of water which may cause unacceptable erosion and undermining of the down stream channel section,
 - b. Are susceptible to frost damage as well as slope and bank settlement because of their inherent rigid nature.
2. Cost – paved linings are more expensive.
 3. Aesthetics – turf and stone filling are more aesthetically pleasing because they blend into the roadside environment more readily than paved linings.

The following lining recommendations, in order of preference, are made based on the statements above.

1. Chutes. Open chutes should be lined with stone filling, asphalt concrete, or portland cement concrete. Turf is not acceptable because it will erode away. The outlet area of a closed chute should be provided with an apron of stone filling to prevent erosion.
2. Roadway and Toe-of-Slope ditches. These ditches should be lined with turf, stone filling, asphalt concrete, or portland cement concrete.
3. Intercepting ditches. These ditches should be lined with asphalt concrete because it is desirable to remove water from the top of slope without infiltration to minimize the potential for slope failure. Lining selection should be verified by the Regional Geotechnical Engineer.
4. Median Swales. Median swales proposed for divided highways should be lined with turf or stone filling.

D. Roughness

Each lining has a roughness associated with it which is represented by the Manning roughness coefficient "n". Refer to HDS 4, or "Channels" (Chapter 10 of the "AASHTO Drainage Manual") for recommended values of "n" for natural and roadside channels. "Estimates of Roughness Coefficients for Selected Natural Stream Channels With Vegetated Banks in New York", Appendix 1, USGS Open File Report 93-161 is also available as a source of guidance to estimate Manning's roughness coefficient.

8.5.3.3 Slope

The channel's slope will be dictated by adjacent terrain and right of way constraints. As a result, the slope of roadside and natural channels will normally be as discussed in items 1 through 7.

1. Gutters. Slope will be the same as the vertical alignment of the traveled way.
2. Chutes. Slope should parallel the embankment.
3. Roadway ditches should parallel the traveled way. The slope may be greater than the grade of the traveled way to outlet the roadway ditch to a natural channel or another channel (culvert, etc.).
4. Toe-of-Slope ditches should parallel the toe-of-slope. The slope may increase in the vicinity of the outlet to a natural channel or another channel (culvert, etc.).
5. Intercepting ditches should parallel the top of the cut slope.
6. Median swales should parallel the traveled way. The slope may be greater than the grade of the traveled way to increase channel capacity or to outlet the swale to a natural channel or another channel (culvert, etc.).
7. Relocated natural channels should maintain the existing channel slope.

Abrupt changes in slope will result in the deposition of transported material (when the proposed slope is flattened from a steep approach slope) or scouring (when the proposed slope is greater than the approach slope).

8.5.4 Maintenance

The following two maintenance activities should be considered during a field trip for inclusion as part of the project.

1. Cleaning and reshaping turf lined channels. (Refer to Section 203 of the Standard Specifications for a pay item covering this work.)
2. Repairing or replacing failed or inadequate linings.

8.6 CULVERTS

A culvert is usually a closed conduit (e.g., a pipe), but may be an open conduit (e.g., an arch), installed to convey runoff collected in roadside channels, and natural channels such as streams, underneath an embankment. Generally, a culvert is open at both ends and is not connected to a drainage structure. Sometimes, a culvert is connected to a drainage structure located in the median, or in a ditch. Work in natural channels should be performed consistent with water quality standards. Best management practices to control erosion, sedimentation, and water disturbances shall be provided consistent with Section 8.8.2, and recommendations from the Regional Environmental Contact.

Any single structure with a span greater than 20 ft. is a bridge. These structures require a different procedure for hydraulic analysis and should be coordinated through the Regional Structures Group.

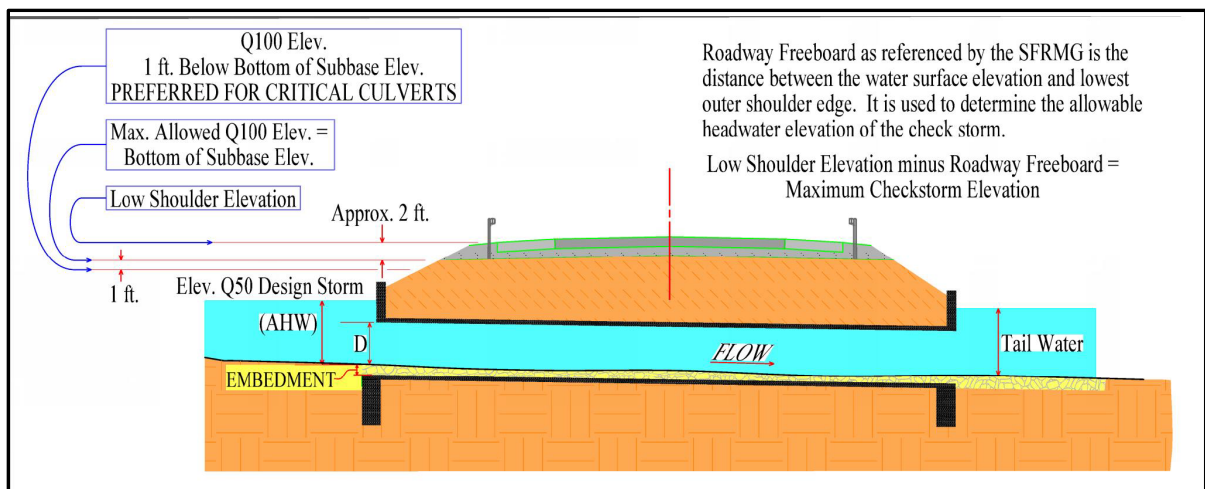
Culverts are discussed at greater length in "Hydraulic Design of Highway Culverts" (Chapter 4 of the "Highway Drainage Guidelines"), "Culverts" (Chapter 11 of the "AASHTO Drainage Manual"), and "Hydraulic Design Series" (HDS) No. 5, "Hydraulic Design of Highway Culverts". Refer to Chapter 19 of this manual for guidance regarding reinforced concrete box culverts and similar structures.

8.6.1 Hydraulic Design Criteria

8.6.1.1 Allowable Headwater

Allowable Headwater (AHW) is defined as the maximum depth of water which can be tolerated at the inlet of the culvert. It is measured from the elevation at the inlet of the culvert to the water surface elevation necessary to achieve the hydraulic design requirements at the location, as illustrated in Exhibit 8-8.

Exhibit 8-8 Culvert Allowable Headwater



The hydraulic design of culverts essentially consists of an analysis that evaluates the performance of a culvert to convey flow from one side of the roadway to the other. The following criteria should be established, when investigating the hydraulic performance of a culvert:

1. Design Flood Frequency
2. An estimate of the Design Discharge associated with that frequency
3. Allowable Headwater (AHW) elevation based on the selected Flood Frequency and headwater considerations.

A. Design Flood Frequency:

Design flood frequency is established based on the functional classification of the Highway and its criticality to New York’s economy, and the health, safety and welfare of its residents and visitors. The determining factors for criticality may be found in the “[NYS Flood Risk Management Guidance for Implementation of the Community Risk and Resiliency Act](#)” (FRMG), and are also listed in Section 8.6.1.1.C. The design flood frequency and check storm criteria for culverts crossing state highways is summarised in Exhibit 8-12.

B. Design Discharges:

An estimation of the design discharges is dependent upon the watershed draining to the culvert. Exhibit 8-9 below summarizes acceptable methods to be used based on watershed area. The proper use of each method can be found in Section 8.3.2.4 *Flow Determination*. It is the Engineer’s responsibility to verify that the appropriate method for the conditions is used. See Section 8.3 for more information on hydrology.

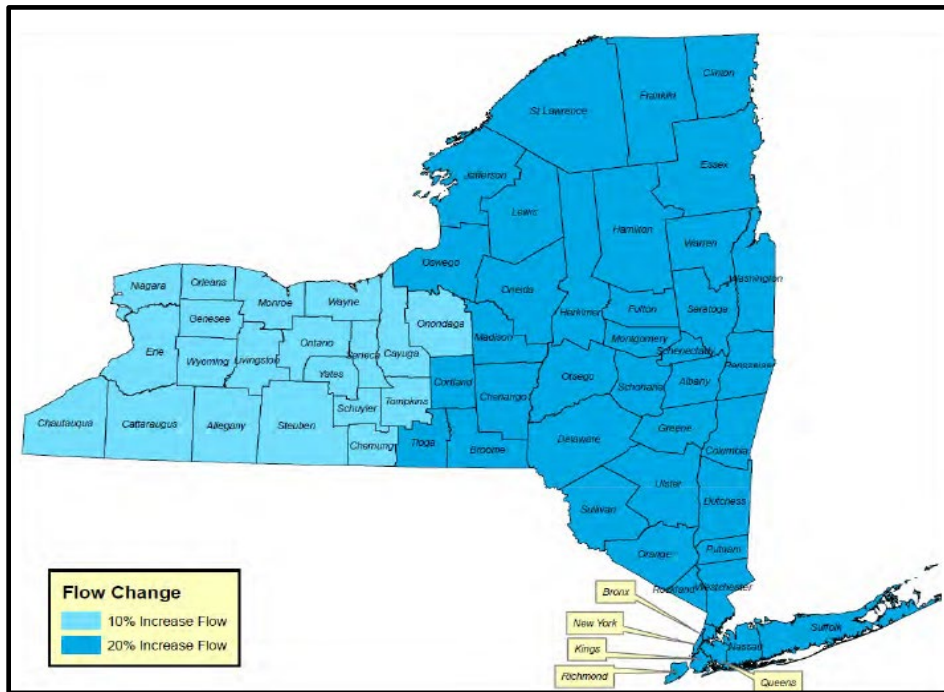
Exhibit 8-9 Methodology for Estimating Design Discharge Based on Drainage Area

Drainage Area	Methodology
0-200 Acres	Rational Method
Up to 640 ¹ Acres	USDA NRCS TR-55 Method
>500 Acres	USGS Regression Equations (StreamStats)

1. For drainage areas less than 640 acres (1 Sq. Mile), where the limits of the USGS Regression Variables are valid, the USGS Regression Equations should be used to compute design discharges.

Computed design discharges shall be increased as discussed in Section 8.5.3 *Hydraulics-Design and Analysis*. The figure below gives a visual representation of which increase in flow should be applied to which counties.

Exhibit 8-10 Design Discharges Multipliers by County

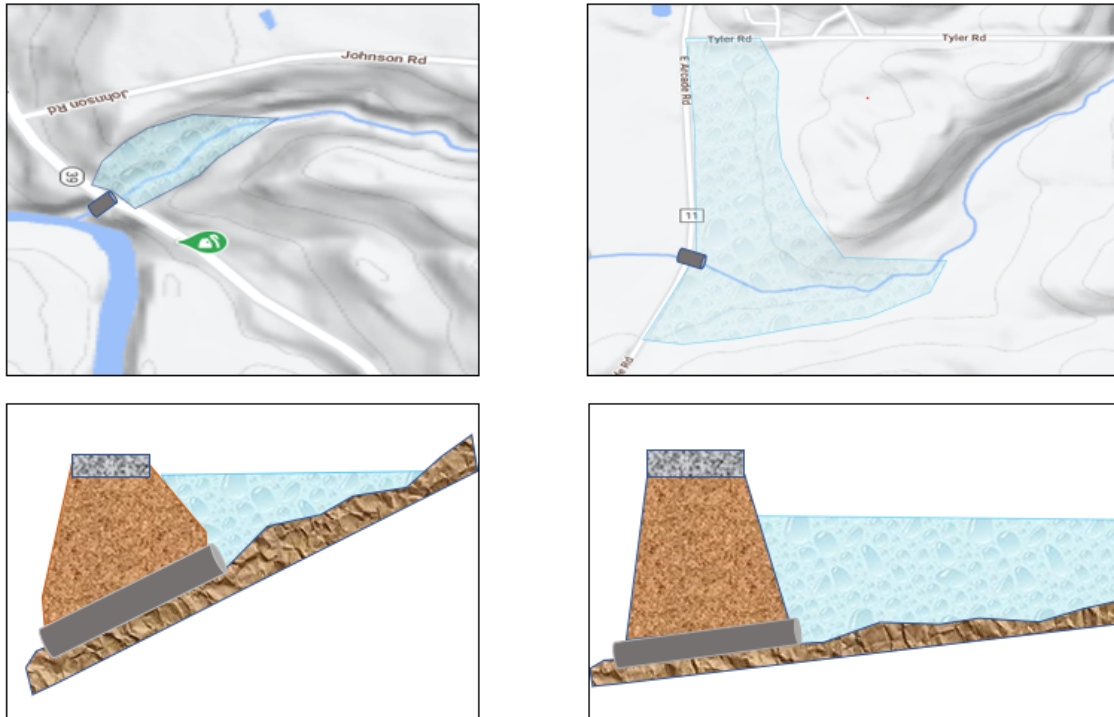


C. Allowable Headwater (AHW) elevation:

Existing field conditions, channel geometry, and culvert hydraulics will control the allowable headwater depth.

The stream terrain upstream of the culvert should be considered when determining AHW. A very steep terrain upstream of the culvert can lead to substantial headwater ponding with minimal transverse spread that can impact nearby property. However, too much ponding along the roadway embankment can lead to piping, which can compromise the structural integrity of the highway.

If the upstream terrain is very flat, a minimal raise in backwater elevation can lead to substantial transverse spread of the flood waters. This can have an impact on more distant properties. Exhibit 8-11 Illustrates the effect of backwater based on terrain upstream of the culvert.

Exhibit 8-11 The Effect of Backwater Based on Terrain Upstream of the CulvertSteep Terrain Upstream of CulvertFlat Terrain Upstream of Culvert

In selecting an Allowable Headwater Elevation (AHW), the following should be considered:

- Anticipated upstream and downstream flood risks, for a range of flood frequencies.
- Damage to culvert and roadway
- Traffic interruption due to culvert or roadway damage
- Hazard to human life and safety
- Allowable Headwater (AHW) / Culvert Depth (D) Ratio
- Low point in the roadway grade line
- Roadway elevation above structure
- Elevation at which flood waters will flow to adjacent water courses
- Stability of the roadway embankment above the culvert
- Current and future land use that may be impacted

Allowable headwater also depends on whether the culvert is on a non-critical or critical roadway. Critical roadways are part of a designated evacuation route or provide sole access to any of the following facilities where practical detour routes are not available in case of loss or closure:

- Facilities designed for bulk storage of chemicals, petrochemicals, hazardous or toxic substances or floatable materials
- Hospitals, rest homes, correctional facilities, dormitories, patient care facilities
- Major power generation, transmission, or substation facilities
- Major communications centers, such as civil defense centers
- Major emergency service facilities, such as central fire and police stations

Additional information on criticality may be found in the Transportation Infrastructure section of the [FRMG](#) and should be determined during the project scoping phase. The designer should demonstrate through design documentation or verification that consideration of the higher of these flood risk management guidelines to determine design flow elevation were evaluated. Note that the following criteria only apply to allowable headwater design. All other elements of culverts including stone aprons shall be designed to meet the design flood frequency unless otherwise noted.

As noted in Exhibit 8-3, minor arterials, collectors/local roads, and streets as well as driveways may have a lower design flood frequency if found to be appropriate and documented in the Design Approval Document if they are considered to be non-critical. Culverts on state highways as well as critical roadways and critical driveways shall consider the guidance shown in Exhibit 8-12 to the greatest extent practical. This consideration should be given to the highest applicable design flow elevations practicable, considering feasibility, project costs, costs of flooding, funding eligibility, risk tolerance, environmental effects, and historic preservation per design documentation or verification.

Roadway Freeboard as defined in the SFRMG is the distance from the water surface elevation of the check storm flow to the lowest shoulder elevation of the roadway (see Exhibit 8-8). This is intended to be a rough approximation of the bottom of the roadway subbase layer. The table below references the bottom of subbase as the control point. If this is known from record plans or a new culvert is being placed, the actual elevation of the bottom of subbase should be used. If it is not known, the designer should use 2 feet below the low shoulder for non-critical culverts or three feet below the low shoulder for critical culverts as the check storm elevation.

Exhibit 8-12 Flood Risk Management Guidelines for Culverts

Highway Criticality	Design/Checked Flood Frequency (Years)	Adjusted for Climate Change per Figure 8.6.1.2 (Yes/No)	Guidance Criteria			
			Rise D ≥ 5 ft.		Rise D < 5 ft.	
			Allowable Headwater/Depth Ratio ³ (AHW/D)	Check Elevation (ft)	Allowable Headwater/Depth Ratio ³ (AHW/D)	Check Elevation (ft)
Critical Highways ⁴	50 ² Design	Yes	≤ 1 for D > 5 ft. ≤ 1.2 for D = 5 ft.	--	≤ 1.5 for D ≤ 4 ft. ≤ 1.2 for D > 4 ft. < 5 ft.	--
	100 ⁵ Check	Yes	--	Below bottom of Subbase ¹ 1' below subbase preferred	--	Below bottom of Subbase ¹ 1' below subbase preferred
	500 Check	No	--	No overtopping of the roadway	--	No overtopping of the roadway
Non-critical Highways ⁴	50 Design	Yes	≤ 1 for D > 5 ft. ≤ 1.2 for D = 5 ft.	--	≤ 1.5 for D ≤ 4 ft. ≤ 1.2 for D > 4 ft. < 5 ft.	--
	100 ⁵ Check	Yes	--	Below bottom of subbase ¹	--	Below bottom of subbase ¹
For Culverts located in Tidal Waterways, see note 4 below						

NOTES:

- Where the elevation of the bottom of subbase is not known, 2 feet below the low shoulder for non-critical culverts or three feet below the low shoulder for critical culverts may be used as the check storm elevation.
- Critical Culverts should pass the projected Q₅₀ storm through the culvert while maintaining the appropriate AHW/D ratio with or without pressure flow.
- All culverts that increase headwater should be discussed in the drainage report with regard to flooding conditions, velocities, scouring, economy, future maintenance (e.g., installing a smaller pipe/structure within the original culvert), etc. If damage to the culvert is possible or if adverse upstream flooding conditions exist (from the accumulation of ice and debris at the culvert's inlet), then culvert should be resized to reduce the AHW/D ratio, in order to minimize flood damages within acceptable limits.
- Culverts located in tidal areas shall meet all the criteria in Exhibit 8-12 above and apply applicable coastal design (tidal) criteria set out in the [FRMG](#) that incorporate a range of sea-level rise projections, up to and including the high sea-level rise projection for critical culverts and up to and including the medium sea-level rise projection for non-critical culverts. These projections are applicable for the structures full, expected service life and can be found in [6 NYCRR Part 490](#) as well as Exhibit 8-14.
- Culverts within a regulated floodway shall have no rise in headwater elevation when compared to existing conditions for the 100-year storm. Designers should refer [6 NYCRR Part 502](#) for additional information and exceptions. All other culverts should be designed to maintain or decrease the existing depth of headwater. If it is not practicable to do so, then an assessment of the increased flood risk shall be performed as discussed below.

If any of the allowable headwater design criteria or check storm criteria can't be met; it should be documented as a non-conforming feature. The reason(s) for not meeting these criteria shall be documented in the Project Drainage Report and the Design Approval Document. At locations where it is not feasible to design a culvert that maintains or decreases the existing depth of headwater, an assessment of increased flood risk to adjacent property should be performed. This assessment should be based in the findings of the project drainage report and should be documented in the DAD by including the following when applicable:

1. Likely damage to the highway and adjacent landowners, including but not limited to the potential for damage to existing and future structures or crop lands.
2. Resultant damage to waters of the State such as scour, erosion, barriers to aquatic organisms, adverse deposition of stream bed material due to reduced velocities, etc. caused by increased velocities, constrictions and/or raised headwater.
3. A list of any required flood risk-management measures, including the need to take additional property, if necessary.
4. A summary that identifies the optimum design alternative with respect to minimizing flood risk while protecting properties and natural resources.
5. Sufficient justification for the standard(s) that cannot be met.

Exhibit 8-13 Culverts on Regulated Floodways vs. Other Streams

Stream Condition	Adverse Effect ¹	Allowable Rise in BFE	Notes
FEMA Mapped Floodway Stream (Regulated Floodway)	None Allowed	No rise in BFE	A detailed hydraulic analysis can be carried out to illustrate that the new, replaced or rehabbed alternative will have 0.00 feet rise in the BFE when compared to existing conditions.
FEMA Mapped Floodplain (No Regulated Floodway Established)	None Allowed	Up to 1-foot rise in the BFE ²	Rises in BFE are allowed under 6 NYCRR 502 for new, replaced or rehabbed alternative if the rise is ≤1 foot. Previous records should be checked for existing adverse effects ¹ .
All other streams (No Regulated Floodway Established)	None Allowed	Up to 1-foot rise in the BFE ²	Previous records should be checked for existing adverse effects ¹ .

NOTES:

1. Adverse Effects can be categorized as one or more of the following:
 - i. Impact on private properties (houses, businesses, upstream infrastructure, etc.).
 - ii. Records of existing structure overtopping during storm events.
 - iii. Records of existing structure affected by debris that leads to flooding.
 - iv. Roadway freeboard will be less than 2 feet because of a culvert rehabilitation.
 - v. Likely damage to waters or other natural resources of the State.
2. New and replacement culverts should result in no rise or reduced headwater at the culvert.

D. Rehabilitated Culverts:

1. Riverine Locations:

Existing culverts that are in riverine locations, which are scoped to be rehabilitated by methods described in Section 8.6.7, should be evaluated for criticality, risk and potential damage to waters of the State as described in Section 8.6.1.1.C. If it is determined that culvert rehabilitation is the best course of action, materials and sizing should be selected to keep the rise in headwater to a minimum while extending the service life of the culvert to the desired lifespan. However, if the culvert is in a FEMA Mapped Floodplain Area, Exhibit 8-13 should be used as guidance to evaluate flood risks in such areas.

2. Culverts in Tidal Areas:

The rehabilitation of culverts in Tidal Areas should be evaluated for criticality and risk as described in Section 8.6.1.1.C, incorporating the projected sea-level rise projections in Exhibit 8-14.

If the allowable headwater design or check criteria cannot be met for rehabilitated culverts in conditions A or B above, the reasons shall be documented in the Project Drainage Report and the Design Approval Document along with the assessment of increased flood risk to adjacent property.

Exhibit 8-14 Projected Sea-level Rise

(a) Mid-Hudson Region

Time Interval	Low Projection	Low-Medium Projection	Medium Projection	High-Medium Projection	High Projection
2020s	1 inch	3 inches	5 inches	7 inches	9 inches
2050s	5 inches	9 inches	14 inches	19 inches	27 inches
2080s	10 inches	14 inches	25 inches	36 inches	54 inches
2100	11 inches	18 inches	32 inches	46 inches	71 inches

(b) New York City/Lower Hudson Region

Time Interval	Low Projection	Low-Medium Projection	Medium Projection	High-Medium Projection	High Projection
2020s	2 inches	4 inches	6 inches	8 inches	10 inches
2050s	8 inches	11 inches	16 inches	21 inches	30 inches
2080s	13 inches	18 inches	29 inches	39 inches	58 inches
2100	15 inches	22 inches	36 inches	50 inches	75 inches

(c) Long Island Region

Time Interval	Low Projection	Low-Medium Projection	Medium Projection	High-Medium Projection	High Projection
2020s	2 inches	4 inches	6 inches	8 inches	10 inches
2050s	8 inches	11 inches	16 inches	21 inches	30 inches
2080s	13 inches	18 inches	29 inches	39 inches	58 inches
2100	15 inches	21 inches	34 inches	47 inches	72 inches

6 NYCRR Part 490 Projected Sea-Level Rise

In no case should the flood-risk management guidelines outlined in this section be used if doing so would increase the risk of flood damage upstream or downstream of the culvert. If the application of this this guidance would result in increased flooding, or the need to take additional property or floodproof private property, the designer should analyze an appropriate range of options to determine the optimum approach to minimize flood risk while protecting

properties and natural resources. These options shall be documented in the Project Drainage Report and Design Approval Document.

8.6.1.2 Minimum Pipe Size

The minimum size round pipe recommended for all highway functional classifications, including ramps is 24 in. Smaller diameter pipes (18 in. and 15 in.) are acceptable when shallow depths and utility conflicts prohibit using a 24 in. pipe.

The minimum size round pipe recommended for driveways and field entrances is 15 in., except in unusual circumstances where the Regional Design Engineer or NYSDOT Resident Engineer may approve a 12 in. round pipe.

8.6.1.3 Multiple Barrels

Multiple barrels (more than one pipe) should be considered when the allowable headwater criteria cannot be met with a single barrel installation and an improved inlet design. These locations will normally occur at sites with low fills or wide channels. Multiple barrels are not a desirable alternative to convey perennial streams because of the potential for clogging. Cleaning operations are labor intensive and disrupt the stream's ecosystem.

8.6.2 Pipe Design Criteria

Acceptable culvert materials are steel (see note below), reinforced concrete, aluminum, polyethylene and polypropylene. The design criteria for these materials consist of design life, anticipated service life, structural criteria, and economics.

Note: Steel shall not be specified for culverts installed to act as equalizer channels or at other locations such as canals where the water level is expected to remain relatively constant. The near constant water levels result in localized metal loss rates in excess of those anticipated (refer to Section 8.6.2.2), and these water levels usually are at a location – on the sidewalls – where the stress levels are higher than at the invert.

The material specified shall be the most economical which satisfies all the pipe criteria (design life, anticipated service life, and structural criteria) in addition to meeting the hydraulic criteria (allowable headwater, etc.). Refer to section 8.6.2.4 for further discussion regarding selection of the final culvert material.

8.6.2.1 Design Life

Design life is defined as the number of years of in-service performance which the pipe is desired to provide.

Design life is assigned by location, taking into consideration the following factors:

1. Initial cost of the pipe material, installation, backfill, etc.

2. Cost to rehabilitate or replace.
3. Disruption to traffic during rehabilitation or replacement once an installation reaches the end of its design life.

The design lives, by location, shall be as shown in Exhibit 8-15:

Exhibit 8-15 Design Lives

Location	Design Life (Years)
Driveways	20
Significant Locations ¹	70
Other Locations	50

NOTES

1. Significant locations are defined as follows:
 - a. Highways functionally classified as Interstates and Other Freeways. (Refer to Section 2.4.1 of Chapter 2 for these definitions.)
 - b. Natural watercourses, or channels, such as perennial streams.
 - c. Under high fills – 15 ft. or greater.
 - d. Locations with high traffic volumes.
 - e. Locations where long off-site detours would be required if the culvert failed. Consider taking user costs into account.

8.6.2.2 Anticipated Service Life

Anticipated service life is defined as the number of years it is anticipated that the culvert pipe material will perform as originally designed or intended. For steel, it is defined as the time when the culvert invert or flow line would be completely removed if the design metal loss (2 or 4 mils/yr) occurred uniformly throughout the length of the culvert.

The anticipated service life for concrete, polyethylene, and polypropylene is 70 years. Aluminum is anticipated to last for 70 years under normal conditions. When high velocities, potentially abrasive bed loads from adjacent channel sections, or high concentrations of industrial waste are present or suspected, 70 years should not be anticipated for aluminum.

The anticipated service life for metallic coated (galvanized) steel pipe is based on metal loss rates established through research conducted by the Department. For the purpose of establishing the anticipated service life, the galvanized coating is not considered an additional coating and is termed "without additional coating". (Refer to "Metal-Loss Rates of Uncoated Steel and Aluminum Culverts In New York", Research Report 115 for a complete explanation of the methodology used.) Metal loss rates have been defined for two geographical locations, Zone I and Zone II, as shown in Exhibit 8-16.

Exhibit 8-16 Metal Loss Rates for Steel by Geographic Location

Zone I (2 mils/yr)		Zone II (4 mils/yr)
Region 1	except	Albany, Greene, and Schenectady Co.
Region 2	except	Montgomery Co.
Region 3	except	Cortland, Tompkins Co.
Region 4	--	--
Region 5	except	Cattaraugus Co.
--	--	Region 6
Region 7	--	--
--	--	Region 8
--	--	Region 9
--	--	Region 10
--	--	Region 11

Exhibit 8-17 provides the anticipated service life for various gauge steel in Zone I and Zone II (with and without an additional coating). Additional anticipated service life is assigned to steel pipe by providing a metallic coating other than galvanizing (i.e., by specifying aluminum coated (Type 2) pipe), or by providing a coating in addition to the metallic (galvanized) coating. (This additional life is also based on research performed by the Department.) Not all coating options are available for each type of steel pipe. (Refer to Exhibit 8-18 for a list of additional coatings available for use with the various types of pipe.) When a metallic coating, or an additional coating option is not available, use increased gauge, or select a material other than steel, to achieve the anticipated service life required to satisfy the design life criteria.

Additional coating alone shall not be used to extend the anticipated service life of galvanized steel pipe or structural steel plate pipe installed to pass natural watercourses (streams, creeks, rivers, etc.) when the water surface is above the area protected by the additional coating. The appropriate gauge galvanized steel pipe shall be specified in these locations to meet the design life criteria.

Exhibit 8-17 Anticipated Service Life, in years, for Steel (with and without additional coating)^{1,2}

Gauge	Metallic Coated (Galvanized)		Metallic Coated (Aluminum Coated - Type 2) ³ & Metallic Coated (Galvanized) w/ Paved Invert or Fully Paved ³		Metallic Coated (Galvanized) w/ Polymer Coating ⁴		Metallic Coated (Galvanized) w/ Polymer Coating and Paved Invert ⁵		Metallic Coated (Galvanized) w/Paved Invert ⁶ (Structural Steel Plate)	
	§707-02/§707-09		§707-02		§707-02		§707-02		§707-09	
	Zone I	Zone II	Zone I	Zone II	Zone I	Zone II	Zone I	Zone II	Zone I	Zone II
18	26	13	51	38	51	38	66	53	Gauge not manufactured	
16	32	16	57	41	57	41	72	56		
14	40	20	65	45	65	45	80	60		
12	54	27	79	52	79	52	94	67	89	62
10	69	34	94	59	Gauge not manufactured with this coating				104	69
8	84	42	109	67					119	77
7	94	47	Coating option not specified for these gauges						129	82
5	109	54							144	89
3	124	62							159	97
1	140	70							175	105

NOTES

1. For culverts whose diameter, or equivalent diameter, is 10 ft or greater:
 - a. in Zone I - specify a paved invert for 12 gauge culverts or specify a 10 gauge culvert.
 - b. in Zone II - specify a paved invert for all culvert regardless of gauge.
2. Use caution in designing culverts on grades steeper than 6±% carrying potentially abrasive bed loads. Do not rely on polymer coating alone to increase the service life in abrasive conditions. Use fully paved pipe or paved invert. In very severe conditions, consider use of concrete or polyethylene. Aluminum is not recommended due to the potentially abrasive bed load.
3. The Aluminum Coated - Type 2 metallic coating is expected to have the same anticipated service life as metallic coated (galvanized) pipe with a paved invert or fully paved. Additional coating (i.e., paved invert or fully paved) adds 25 years to the anticipated service life of metallic coated (galvanized) steel pipe.
4. Additional coating (i.e., polymer coating) adds 25 years to the anticipated service life of galvanized steel pipe.
5. Additional coating adds 40 years to the anticipated service life of galvanized steel pipe.
6. Additional coating adds 35 years to the anticipated service life of galvanized steel pipe.

Exhibit 8-18 Additional Coating Options

Additional Coating	Corrugated Steel (§ 707-02)	Corrugated Structural Steel Plate (§ 707-09)
Paved Invert (Bituminous)	Type I and II only	Not available
Fully Paved (Bituminous)	Type I and II only	Not available
Polymer	Type I and II only	Not available
Polymer & Paved Invert (Bituminous)	Type I and II only	Not available
Paved Invert (Portland Cement Concrete)	Not available	Available

8.6.2.3 Structural Criteria

Factors which affect the structural performance of a pipe include:

1. Maximum and minimum height of cover (Refer to Section 8.6.2.3.A) - all pipe materials.
2. Bedding conditions and installation methods (Refer to Section 8.6.2.3.B) - concrete and metal.
3. The end treatment for the pipe (square, miter, skew) as this will affect the structural strength of the pipe's end - all pipe materials. (Not to be confused with an end section, which attaches to the pipe's end.) (Refer to Section 8.6.2.3.C)

A. Height of Fill Tables

The height of fill tables provided in Appendix A shall be used to obtain the thinnest gauge steel or aluminum, or lowest class concrete pipe which meets the height of fill requirements of the site. Tables for the following pipe materials are provided:

<u>Material</u>	<u>Table</u>	<u>Page</u>
Corrugated Steel Pipe (§707-02 Type I)	Table 8-22	8A-1
Corrugated Steel Pipe Arch (§707-02 Type II)	Table 8-23	8A-2
Corrugated Steel Pipe (§707-02 Type IR)	Table 8-24	8A-3
Corrugated Steel Pipe (§707-02 Type IIR)	Table 8-25	8A-4
Corrugated Structural Steel Plate Pipe (§707-09)	Table 8-26	8A-5
Corrugated Structural Steel Plate Pipe Arch (§707-09)	Table 8-27	8A-7
Corrugated Aluminum Pipe (§707-13 Type I)	Table 8-28	8A-9
Corrugated Aluminum Pipe Arch (§707-13 Type II)	Table 8-29	8A-10
Corrugated Aluminum Pipe (§707-13 Type IR)	Table 8-30	8A-11
Corrugated Aluminum Pipe (§707-13 Type IIR)	Table 8-31	8A-12
Corrugated Aluminum Structural Plate Pipe (§707-14)	Table 8-32	8A-13
Corrugated Aluminum Structural Plate Pipe Arch (§707-14)	Table 8-33	8A-14

Reinforced Concrete Pipe (§706-02)	Table 8-34	8A-16
Reinforced Concrete Elliptical Pipe (§706-03)	Table 8-35	8A-18
Smooth Interior Corrugated Polyethylene Pipe (§706-12)	Table 8-36	8A-19
Polypropylene Pipe (§706-08)	Table 8-37	8A-20

The same type (corrugation, gauge, or class) of material shall be used throughout the length of the culvert for ease of installation and consistency in design.

The maximum height of fill over the top of circular and elliptical reinforced concrete pipe is dependent on bedding details and installation methods shown on Standard Sheet 203-04. As a result, designers have to anticipate, or specify on the plans as appropriate, the bedding detail and installation method for each pipe selected.

B. Bedding Conditions and Installation Methods

Bedding conditions and installations methods – except pipe jacking – for concrete and metal (steel and aluminum) pipe are detailed on Standard Sheets 203-04 and 203-05. Standard Sheet 203-05 is also applicable for smooth interior corrugated polyethylene pipe.

The standard sheet(s) need to be considered to:

1. Specify and estimate pay items not included for payment under the pipe item.
2. Determine the relationship to the height of fill. (This is only necessary for concrete pipe.)
3. Determine the need to provide additional details or notes in the plans which are not covered in the standard sheet.

Manufacturers of reinforced concrete pipe suitable for jacking do not produce this type of pipe in a size less than 42 inches in diameter. Steel pipe is available for jacking applications where a diameter less than 42 inches is needed.

Contact the Regional Geotechnical Engineer as soon as possible in the design process to request soil boring in the area of the proposed pipe jacking. These borings will be used to determine a suitable method of jacking, pipe wall thickness, and other technical issues.

C. End Treatment

Ring compression, buoyancy, and scour should be considered when specifying an end treatment. (Not to be confused with an end section, which attaches to the pipe's end.)

1. Ring compression. The embankment transfers live and dead loads which result in ring compression on the pipe. Square ends are strongest and should generally be used to resist these forces.

- a. Metal pipes and mitered ends - A miter is a flush-entrance culvert where the barrel is mitered to the slope of the embankment. Miter normal to culvert axis only, when this type of treatment is desired. Do not specify combination skew and miter, and do not extend miter below $75\pm\%$ of nominal pipe diameter or rise dimension for pipe-arches. The flattest miter shall be one vertical on three horizontal. Warp flatter embankment slopes to meet miter or use other end treatment.
 - b. All materials - The skew angle between the perpendicular to the highway centerline and the structure axis should generally not exceed 20° . An anchor bolt system embedded in concrete cutoff walls, collars, slope paving, etc. should be provided when the skew exceeds 20° . Skewed culverts should be checked for unbalanced loading from the embankment.
2. Buoyancy and scour. The effects of buoyancy at the inlet and outlet should be considered. The hazard is greatest for flexible pipes (steel, aluminum, polyethylene), increasing rapidly with diameter; however, concrete structures are not totally immune. Concrete pipe has been known to lose downstream sections, either from scouring and/or buoyancy effects. Concrete box structures have also moved because of buoyant effects. The hazard of buoyancy is increased when flat slopes for highway embankments exist. To cope with these uplift forces, concrete cutoff walls, full headwalls or headwalls with flared wingwalls, may be required.

Flexible pipes should be anchored to these concrete end walls to insure their stability, as well as protect against scour. Refer to Exhibit 8-19 for typical anchor bolt details. The straight anchor bolt is considered the industry standard and does not have to be detailed in the plans. When needed, hook bolts shall be detailed on the plans. (Hook bolts were developed for use in masonry walls.) The materials requirements for both types of bolts are covered Section 707-20 of the Standard Specifications.

It is recommended that all sites where the proposed culvert is 30 inches in diameter or larger (35-inch span x 24-inch rise pipe arches) and incorporates end sections be evaluated by the Regional Geotechnical Engineer for the need to include a concrete cut-off wall at each end. Cut-off walls can be used on smaller culverts if channel velocities, soil conditions, steep grades or other such conditions warrant. If a cut-off wall is not used, the criterion below for using a toe-plate extension shall be used. Usage of cut-off walls shall not preclude the inclusion of proper channel protection and/or dissipation devices as discussed in Section 8.6.4.3.

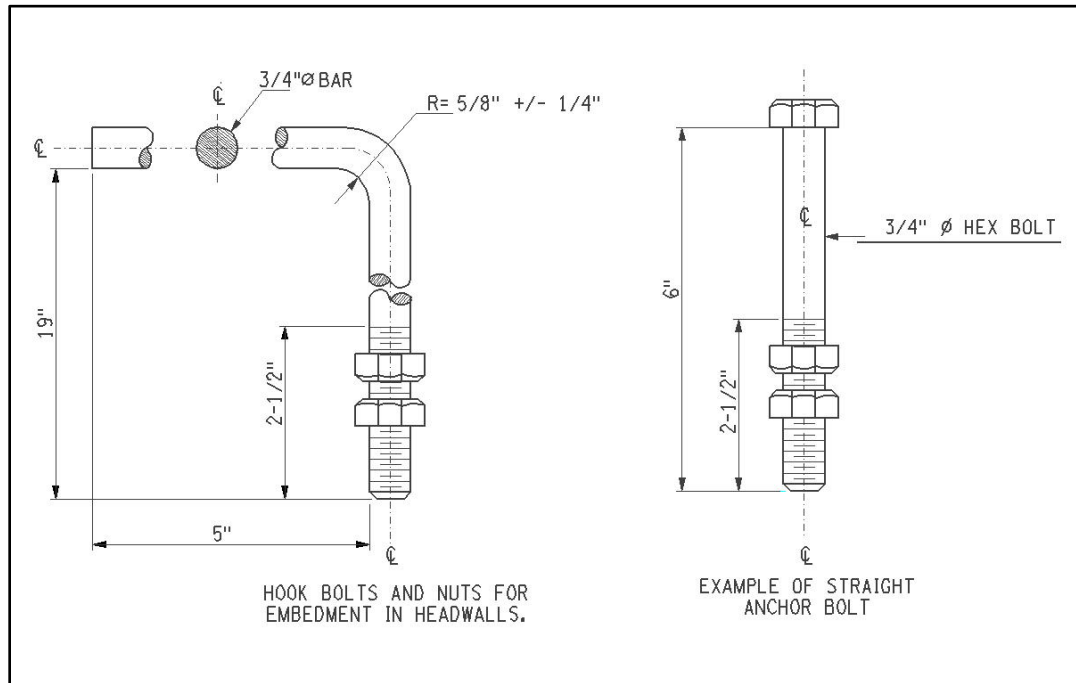
The designer should indicate in the drainage table (or similar table, e.g. table of culvert installations) where cut-off walls shall be included. The designer shall include in the estimate the appropriate item numbers and quantities of Class A, Concrete for Structures, and Trench and Culvert Excavation required to construct the prescribed cut-off walls. As stated in Standard Specification Section 603-5.01 General, the cost of anchor bolts for cut-off walls is included in the cost bid for End Sections and need not be estimated separately.

Refer to Standard Sheets 603-01, 603-02 and 603-04 for end section and cut-off wall details.

Toe plate extensions (for the outlet) shall be specified in the drainage structure table when the downstream channel is at a grade of 5% or greater.

Regardless of the design, adequate precautions must be observed to ensure the safety of the traveling public by providing for a barrier system or placing the end treatment at an appropriate location within the roadside as discussed in Section 8.6.6.

Exhibit 8-19 Typical Anchor Bolt Details



8.6.2.4 Engineering and Economic Analysis

Materials shall be specified which satisfy engineering requirements at the lowest overall cost. This cost should include all expenses necessary to install and maintain the pipe throughout the life of the highway. The installation cost includes not only the pipe itself but the pipe excavation, backfill, labor, and any other special features. Generally, the design with the least installation cost is the first choice. Occasionally, however, there may be other quantifiable lifetime factors which negate the initial savings. These can include maintenance costs which occur on a regular predictable basis or specific material features which offer additional resistance to corrosion, silting, sliding or rupture.

A discussion on cost analysis and justification of the final design should be included in the Drainage Report.

Note: In the absence of firm cost data and without an engineering reason dictating the choice of one pipe material, an optional specification should be used. Special specifications are available to specify alternate culvert materials for side drains (driveways and ditch crossings) and cross drains (culverts under gravel or dirt roads) not installed under high type pavement (603.85xxxx15, 603.86xxxx15, 603.87xxxx15). Under the optional items, the designer selects the size and shape of the pipe and the contractor picks the material based on the specification. Therefore, pipes should be sized based on the hydraulically least efficient material.

8.6.3 Culvert Design - Overview

Culvert design should proceed in accordance with the following sequence of steps:

1. An initial pipe material, shape, and size should be selected which are consistent with the criteria stated in Sections 8.6.1 and 8.6.2.
2. The headwater depth should be calculated based on inlet, and outlet control – the greater of the two values is the controlling headwater. (Section 8.6.3.1 discusses the factors which influence inlet and outlet control.) The controlling headwater elevation should be compared with the allowable headwater criteria stated in Section 8.6.1.1. If the controlling headwater is greater than the allowable headwater, a larger size pipe or a different culvert configuration shall be designed and analyzed.
3. The outlet velocity should be calculated to determine the need for scour protection, channel protection, or an energy dissipator.

8.6.3.1 Inlet Control and Outlet Control

Calculating headwater depth based on inlet control and outlet control may be done manually or using computer software. The manual design method is presented in Chapter III of HDS 5. It is based on the use of design charts and nomographs located in Appendix D of HDS 5, should be used for the shape under consideration. The HY-8 computer software is also acceptable for design.

Exhibit 8-20 lists the factors which influence a culvert's hydraulic performance under inlet and outlet control. I

Exhibit 8-20 Factors Influencing Culvert Performance in Inlet and Outlet Control

Factor	Inlet Control	Outlet Control
Headwater Elevation	X	X
Inlet Area	X	X
Inlet Edge Configuration	X	X
Inlet Shape	X	X
Barrel Roughness		X
Barrel Area		X
Barrel Shape		X
Barrel Length		X
Barrel Slope		X
Tailwater Elevation		X

A. Inlet Control

A culvert operates in inlet control when the barrel (pipe) will convey more flow than the inlet will accept. The flow is supercritical (shallow and high velocity), and the flow control is at the upstream end of the culvert. As a result, the headwater and the inlet configuration (inlet area, edge configuration, and shape) influence the culvert's performance.

1. **Headwater Elevation.** The headwater elevation is the water surface elevation of the headwater. Headwater, or headwater depth, is the depth of the water surface measured from the inlet invert. The headwater depth shall not exceed the allowable headwater criteria stated in Section 8.6.1.1.
2. **Inlet Area.** The inlet area is the cross-sectional area of the face of the culvert and is the same as the barrel (pipe) area, unless a tapered inlet is specified. A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section.
3. **Inlet Edge Configuration (End Treatment).** Improvements to the inlet edge configuration should be considered (headwalls with bevels, end sections, and mitered ends). Improved inlets reduce the energy losses at the inlet, thereby decreasing the headwater depth required to convey the flow and reducing the pipe size. Entrance loss coefficients (k_e) vary from 0.2 to 0.9, depending upon the type of structure and the configuration of the entrance; e.g., 0.2 applies to a concrete pipe with the socket end projecting from the fill, and beveled headwalls. 0.5 applies to most end sections, and 0.9 applies to a corrugated metal pipe or pipe arch projecting from the fill with no headwall or end section and pipes mitered to fills. Table 12 of HDS 5 lists k_e for the various types of entrances. For culverts in inlet control, a more efficient inlet edge configuration may allow for a more economical pipe to be chosen. The type of inlet has some influence on capacity in outlet control, but generally, the edge geometry can be ignored when the culvert is operating in outlet control.

The following recommendations are made regarding the design of entrances:

- a. **Bevels.** Bevels are a simple and economical method to improve the efficiency of culvert inlet flows. Beveled edges reduce the contraction of the flow by effectively enlarging the face of the culvert. Beveled edges should be used when concrete headwalls or concrete box culverts are required. Refer to HDS-5 Figure III-3 for bevel edge details. These details shall be included in the contract plans whenever headwalls or concrete box culverts are specified.
- b. **Headwalls.** Headwalls may be used to improve the hydraulic efficiency of culverts and should also be used to prevent erosion of the embankment or to protect the end of the pipe from scour, if necessary.
- c. **End sections.** End sections should be provided with culverts that do not utilize headwalls.

- Concrete end sections may be used when needed to retain the embankment.
 - A galvanized steel or aluminum end section, one standard pipe diameter larger than the pipe, should be specified for polyethylene pipe.
 - The cost of specifying an end section should be considered for metal pipes over 54 in. in diameter, and for concrete pipes over 48 in. in diameter. Generally, a mitered inlet is preferred to improve the hydraulic efficiency of corrugated metal pipes in excess of 54 in.
- d. Mitered. Mitered edges are only recommended for metal pipe over 54 in. in diameter. The structural integrity of the pipe's end should be considered when detailing a mitered edge. Refer to Section 8.6.2.3.C.

Tapered inlets (side and slope tapers) are a less common type of culvert improvement and are discussed in Chapter IV of HDS 5.

4. Inlet Shape. Recommended shapes include circular (all materials), pipe arch (steel and aluminum), elliptical – horizontal and vertical (reinforced concrete), and rectangular (reinforced precast and cast-in-place concrete). The preferred shape is circular due to the numerous materials options available to meet the various height of cover requirements. Other shapes should be considered when site conditions limit height of cover or allowable headwater criteria cannot be achieved. Refer to Chapter 19 "Concrete Culverts and Similar Structures " for guidance regarding the design of these structures.

Exhibit 8-21 illustrates several different examples of inlet control. The type of flow depends on the submergence of the inlet and outlet ends of the culvert. These flow types have been standardized by the USGS and are included in the HY8 output. (Refer to HDS-5 for an explanation of the various flow types.) Depending on the tailwater, a hydraulic jump may occur downstream of the inlet.

Exhibit 8-21a illustrates a condition where neither the inlet nor the outlet end of the culvert are submerged. The flow passes through critical depth just downstream of the culvert entrance. The barrel flows partly full over its length, and the flow approaches normal depth at the outlet.

Exhibit 8-21b illustrates that submergence of the outlet end of the culvert does not assure outlet control. The flow just downstream of the inlet is supercritical and a hydraulic jump forms in the culvert barrel.

Exhibit 8-21c is a more typical situation. The inlet end is submerged and the outlet end flows freely. Critical depth is located just downstream of the entrance and the flow is approaching normal depth at the downstream end of the culvert.

Exhibit 8-21d is an unusual condition which shows that submergence of the inlet and outlet does not assure full flow. In this case, a hydraulic jump will form in the barrel. The median inlet provides ventilation of the culvert barrel. If the culvert were not ventilated, sub-atmospheric pressures could develop which might create an unstable condition during

which the barrel would alternate between full flow and partly full flow.

Exhibit 8-21 Flow Profiles for Culverts in Inlet Control

Exhibit 8-21a HY-8 flow type 1-S2n.

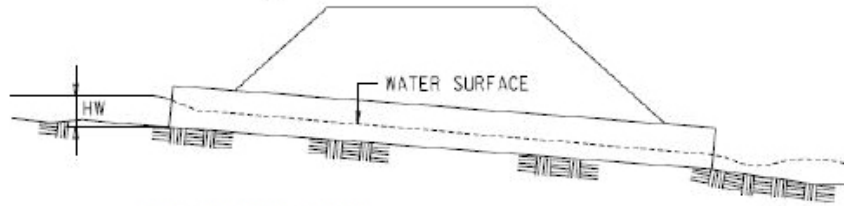


Exhibit 8-21b HY-8 flow type 1-S1f.

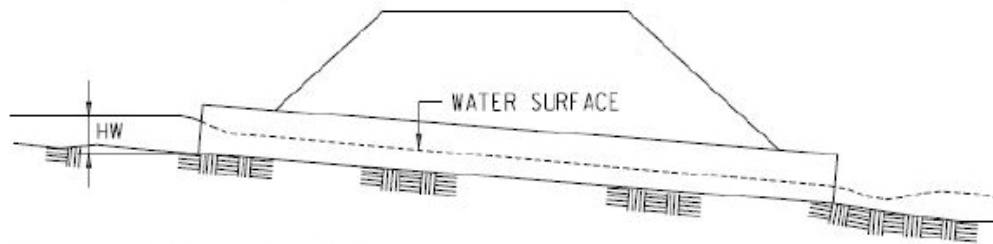


Exhibit 8-21c HY-8 flow type 5-S2n.

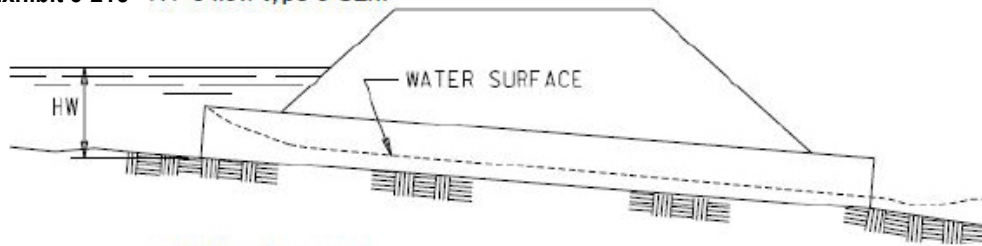
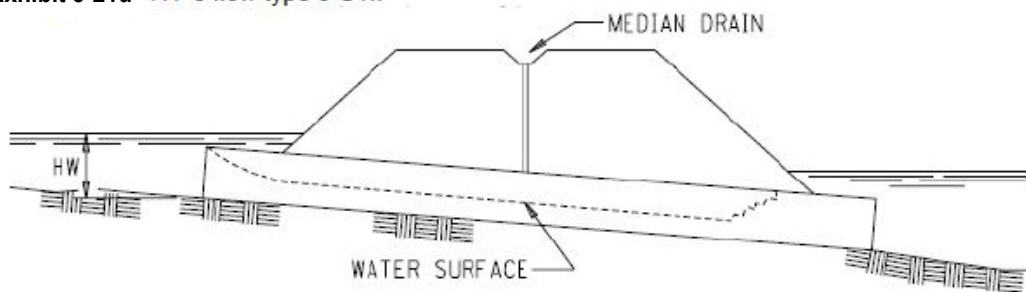


Exhibit 8-21d HY-8 flow type 5-S1f.



Note: HW = headwater depth

B. Outlet Control

Flow in a culvert operating in outlet control is subcritical (relatively deep and of lower velocity) and is controlled by the downstream end of the culvert. The inlet will accept more flow than the barrel will convey. In addition to all of the factors which influence inlet control, outlet control is affected by the hydraulic roughness, area, shape, length, and slope of the barrel and the water surface elevation at the outlet (tailwater).

1. Hydraulic roughness. Pipes have an associated roughness which is represented by the Mannings n value. The Mannings n values provided below should be used.

Pipe Material	Mannings n Value
Reinforced Concrete Pipe (all shapes)	0.013
Smooth Interior Corrugated Polyethylene Pipe	0.013
Corrugated Steel, and Aluminum Pipe (Type IR, IIR)	0.013
Corrugated Steel, and Aluminum Pipe (Type I, II) 2 2/3 x 1/2 - See Note	0.024
Corrugated Steel, and Aluminum Pipe (Type I, II) 3 x 1 or 5 x 1	0.026
Corrugated Structural Steel, and Aluminum Plate Pipe and Pipe Arch	0.032

NOTE: The following n values should be used for 2 2/3 x 1/2 corrugated metal culverts operating under outlet control, and in the design of storm drainage systems.

12 in	0.012
15 in	0.014
18 in	0.015
21 in	0.017
24 in	0.018

Additional coatings for steel pipe (paved inverts, and fully paved pipe) shall not be specified to lower the hydraulic roughness. Department research has determined that these coatings do not last for the entire design life of the pipe.

When pipes with a paved invert are specified, the design of the protective lining for the downstream channel should be based on the higher initial velocity that will exist while the paving is still intact. Use a Manning’s n of 0.019 for all pipe specified with a paved invert regardless of corrugation configuration. For pipe specified as fully paved, design the channel protective lining using an n of 0.013.

2. Area and shape. The charts and nomographs in HDS 5, and the HY-8 software, incorporate area and shape into the hydraulic analysis.
3. Length. Sometimes a pipe will build up a headwater depth higher than the allowable headwater due to the hydraulic roughness and the length of pipe. To reduce the headwater in this situation, use a pipe with a smoother interior, a larger pipe or multiple pipes. A smoother pipe is preferred, unless outlet velocities are a concern. The use of twin structures (multiple barrels) is discouraged unless site conditions are such that debris is not a problem.
4. Slope. The slope (grade) of a culvert should be made compatible with the grade of the upstream and downstream channel to aid in reducing the amount of deposition

at the structure's inlet and scour at the outlet. When a culvert is placed on an extremely flat grade, below a channel with a steep grade, an increased potential for deposition at the culvert's inlet or inside the culvert will occur. The exact location of the deposition of material will depend on the location of the vertical grade break in relation to the culvert's inlet. If a steep approach grade is carried continuously through the structure with a break to a flat grade at the outlet channel, then deposition will occur at the culvert's outlet. If the tailwater depth is deep enough, the deposition will encroach back into the culvert, thereby reducing the culvert's efficiency.

If there is a grade break from a flat grade through a culvert, to a steep grade at the outlet, a scour problem will occur. If the grade break is severe, scour will occur to the extent that the end of the culvert will be undermined with a possible loss of the culvert and a portion of the highway embankment.

If the culvert's vertical alignment cannot be established to conform to the inlet and outlet channels, then the culvert's end, and the channel, shall be protected from scour and erosion (e.g., by providing a headwall and stone filling).

5. Tailwater. Tailwater is the depth of the downstream water surface measured from the outlet invert. The tailwater depth may affect the headwater depth at the inlet of the structure.

Tailwater controls may exist due to drainage structures downstream of a proposed structure. Information should be obtained so an analysis can be performed on the existing downstream structure to determine what effect it has on the proposed culvert. If an effect does exist on the tailwater of the proposed structure, it is sometimes desirable and more economical to replace the existing downstream structure at the same time the proposed structure is built. The maximum and minimum elevations of the tailwater should be recorded during a field review.

Site conditions such as tidal effects from oceans, water surface variations in lakes controlled by power or recreation authorities, barge canals, flood stages in large streams and rivers, etc., may give a variable effect on the tailwater, other than that computed from Manning's equation. These restrictions on the tailwater may force the culvert to act in inlet control when the tailwater surface is at a low elevation or to act in outlet control when the tailwater is at its maximum elevation. The culvert should be designed for the condition which yields the largest culvert size. The effect of the variable tailwater may be eliminated by placing the outlet of the culvert above the maximum high-water elevation of the control feature.

Exhibit 8-22 illustrates several different examples of outlet control. Exhibit 8-15a represents a full flow condition with both the inlet and outlet submerged. The barrel is in pressure flow throughout its length. This condition is often assumed in calculations, but seldom actually exists.

Exhibit 8-22b depicts the outlet submerged with the inlet unsubmerged. For this case, the headwater is shallow so that the inlet crown is exposed as the flow contracts into the culvert.

Exhibit 8-22c shows the entrance submerged to such a degree that the culvert flows full

throughout its entire length while the exit is unsubmerged. This is a rare condition. It requires an extremely high headwater to maintain full barrel flow with no tailwater.

Exhibit 8-22d is more typical. The culvert entrance is submerged by the headwater and the outlet end flows freely with a low tailwater. The barrel flows partly full over at least part of its length (subcritical flow) and the flow passes through critical depth just upstream of the outlet.

Exhibit 8-22e is also typical, with neither the inlet nor the outlet end of the culvert submerged. The barrel flows partly full over its entire length.

8.6.3.2 Outlet Velocity

The outlet velocity (V_o) is usually higher than the velocity in the channel leading up to the culvert. V_o shall be calculated to determine the need for additional channel protection in the vicinity of the culvert's outlet or for culvert redesign. Refer to Section 8.6.4.3 for a discussion of the use of energy dissipators.

The outlet velocity should be calculated as follows using the formula $V_o = Q/A$:

1. If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The velocity at normal depth is assumed to be the outlet velocity.
2. If the controlling headwater is based on outlet control, determine the area of flow at the outlet based on the barrel geometry and one of the following:
 - a. critical depth, if the tailwater is below critical depth,
 - b. the tailwater depth, if the tailwater is between critical depth and the top of the barrel,
or
 - c. the height of the barrel if the tailwater is above the top of the barrel

Exhibit 8-22 Flow Profiles for Culverts in Outlet Control

Exhibit 8-122a HY-8 flow type 4-FFt, 6-FFn.

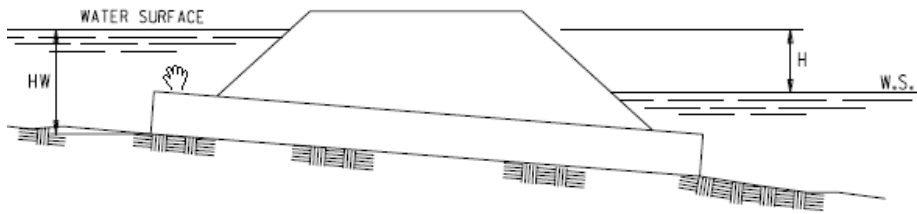


Exhibit 8-22b HY-8 flow type 3-M1f.

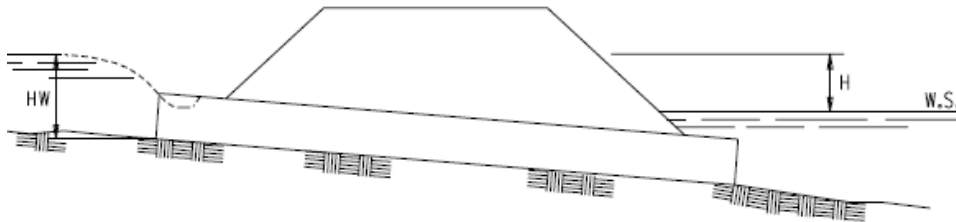


Exhibit 8-22c HY-8 flow type 6-FFt, 6-FFc.

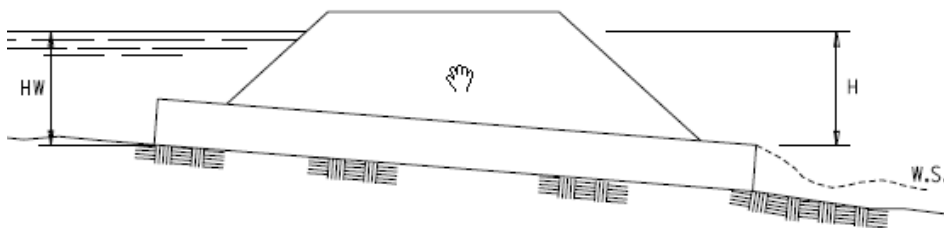


Exhibit 8-22d HY-8 flow type 7-M1t, 7-M2t, 7-M2c.

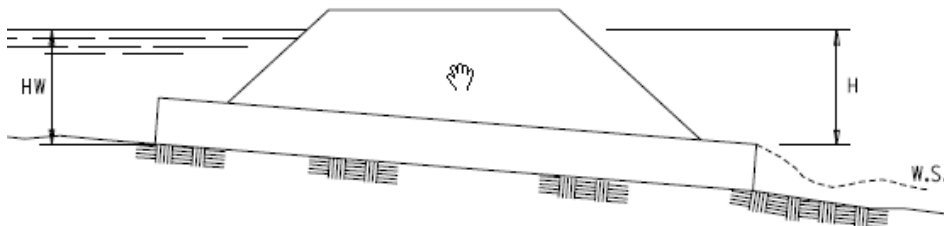
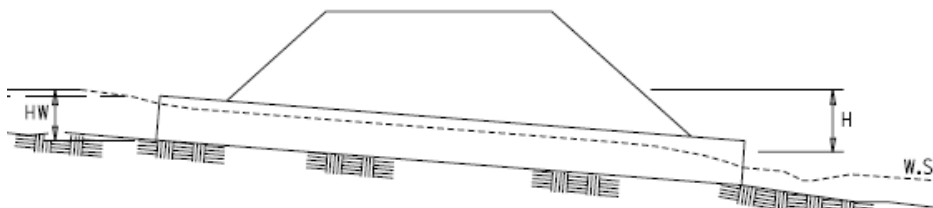


Exhibit 8-22e HYDRAIN (HY8) flow type 2-M2c, 3-M1t, 3-M2t.



Note: HW = headwater depth, H = headloss

8.6.4 Site Considerations

Conditions may exist which are unique to a site and warrant the consideration of measures designed to address the situation. Such conditions include not only general environmental considerations, but special conditions as fish passage, the need for an energy dissipator, and debris control.

8.6.4.1 General Environmental Considerations

The construction of highway drainage facilities can have both short- and long-term negative impacts on aquatic life. Where fish resources are concerned, the necessity to protect the aquatic ecosystem will influence many decisions regarding whether to install a bridge, open or closed culvert, as well as low flow channel design, velocity control, pool:riffle ratios, shading, stabilization techniques, methods and timing of construction and temporary and permanent fish and wildlife passage.

Removal of stream side vegetation can reduce or eliminate shading, causing water temperatures to increase by up to 10° F and thus negatively affecting habitat for cold-water fish species, such as trout. Removal of vegetation can also destabilize stream banks, resulting in erosion, habitat loss, increased aquatic pollution and impaired ecosystem function.

The protection of fish and wildlife resources should be considered from the earliest stages of project development and coordinated with the erosion and sediment control plan. The physical, hydrological and ecological characteristics of the stream or water body should be investigated. Coordinate with the Regional Environmental Unit and appropriate environmental permitting agencies to determine how to avoid or minimize disturbance to the maximum extent practicable and incorporate best management practices to improve habitat, as appropriate.

Whenever practicable, locate highway alternatives in areas causing the fewest unavoidable impacts to fish and wildlife resources. Select locations that cross streams at right angles and avoid crossing at bends and through pools. Minimize removal of riparian trees and maintain vegetated filter strips along water bodies to provide natural bank stabilization, erosion control, shading, water quality and wildlife travel corridors. Use temporary and permanent erosion control measures to minimize transport of sediment into water bodies and use turbidity curtains, cofferdams and dewatering basins when work is required in a water body. Establish the construction schedule considering the spawning times of resident and migratory fish and wildlife and seasonal low water flow periods.

Where highway projects segment or isolate critical reptile or amphibian habitat or breeding areas, special consideration should be given to allow adequate passage (several of these species are listed as protected, threatened or endangered). Culverts constructed "in the dry" or with an elevated dry shelf, may be used to allow herptiles to pass from one side of the road to the other without crossing the pavement. Low fencing (drift fencing) should be used with the culverts to direct wildlife to the culvert openings. Case-by-case requirements should be discussed with the Regional Environmental Unit and appropriate environmental permitting agencies.

8.6.4.2 Fish Passage

Culverts installed to convey natural channels should be aligned to minimize the effect on fish-supporting streams with resident or migratory fish populations. Sufficient water depth and suitable water velocity for fish passage should be provided during periods of low flow. The exact requirements should be discussed with the Regional Environmental Unit.

When designing for fish passage, consider the flow regimes at the time of year when peak resident or migratory fish movement usually occurs (usually spring and/or fall). Simulating natural stream bottom conditions in a culvert is the most desirable design option to accommodate fish and wildlife passage. To achieve this goal and accommodate fish passage during low flow periods, various options are available as follows:

1. In locations where adequate foundation support exists, an arch shaped culvert may be used. The natural stream bed will remain in place.
2. A precast or cast in place reinforced concrete box culvert with a broad shaped "V" or dish shaped floor may be used. The center line floor elevation should be 8 in below the outer floor elevation.
3. In locations with poor foundation material, oversized depressed culverts can be installed, and the natural stream bed material placed inside.

To accommodate fish passage during high flow periods, avoid concentrating flows such that the velocity of water inside the culvert is increased above normal levels upstream and downstream of the culvert. Culverts should not result in water velocities throughout a distance that exceeds the sustained swimming speeds of trout during the fall and spring spawning seasons (Refer to Report No. FHWA-FL-90-006 "Fish Passage Through Culverts, page 12, Figure 9.). As an example, trout require a resting pool every 100 ft with a water velocity of 2 ft/s. Consider the placement of energy dissipating baffles or boulders inside culverts or using oversized depressed culverts and placing natural stream bed material inside to provide habitat and resting pools for migrating fish species. The culvert's outlet should not be suspended above the stream channel or cascade onto rocks; rather the outlet flow should gently merge with the tailwater pool, thereby facilitating the upstream movement of fish.

Culverts should be installed during low flow conditions (August-September) and "in the dry". Temporary dams, water diversions, cofferdams, sediment basins, and water bypass should be considered. The specific type or combination of measures necessary will depend on the depth of water, velocity, discharge, duration of installation, site conditions, and the type of culvert specified. Any restrictions regarding the timing of construction activities or other requirements for work in the channel should be stated in a Special Note. (Refer to Chapter 21, Section 21.4, for the desired format of Special Notes.)

8.6.4.3 Energy Dissipators

Energy dissipators are any device designed to protect downstream areas from erosion or scour by reducing the velocity of flow to acceptable limits. There are two types of scour which should be considered – local scour and channel degradation. Local scour extends a limited distance from the outlet and may occur in both a roadside channel and a natural channel when the outlet velocity is significantly higher than that which the outlet channel lining or natural bed material

can withstand. Channel degradation is typical to a natural channel, is independent of culvert performance, and is a result of natural causes.

There are two approaches which may be taken to protect the channel from scour – mitigation and monitoring. Both approaches should be coordinated with the Regional Maintenance Engineer because they involve follow up reviews to monitor the site. Mitigation entails providing a minimum level of protection such as stone filling, followed up by periodic monitoring. Monitoring the site entails providing no protection for the channel in the vicinity of the culvert's outlet, other than the channel lining material; however, the structural integrity of the culvert should be protected by providing a headwall, cutoff wall, or end section to prevent undermining of the pipe's end.

There are a number of methods by which water's energy may be dissipated. Refer to Hydraulic Engineering Circular (HEC) No. 14 "Hydraulic Design of Energy Dissipators for Culverts and Channels" or "Energy Dissipators", (Chapter 12 of the "AASHTO Drainage Manual") for guidance regarding design considerations and procedures.

8.6.4.4 Debris Control

The need to control debris at a culvert's inlet should be considered. When debris accumulates, the culvert will normally cease to convey water effectively. This can endanger the traveling public and result in damage to the culvert, the highway, and adjacent property. A debris control structure should not be specified without consulting with the Regional Maintenance Engineer because planned maintenance, rather than emergency maintenance during flooding periods, is an essential design consideration.

A discussion of the field survey data required to determine the need for debris control structures, the types of debris control structures, and the classification of debris is provided in Highway Engineering Circular (HEC) 9, "Debris Control Structures Evaluation and Countermeasures". HEC 9 is recommended as a source of guidance if debris control structures are determined to be necessary based on a field review of the drainage area and its debris problems.

8.6.5 Maintenance

All existing culverts within the project limits should be examined to determine the culvert's structural condition and hydraulic adequacy. The need to clean, rehabilitate, or replace the installation should be considered.

8.6.5.1 Cleaning

The pay items located in Section 203 – Excavation and Embankment should be used for culverts in need of cleaning. These culverts should be tabulated in the Contract Documents. Specification provisions contained in Section 619 – Maintenance and Protection of Traffic should not be used or relied upon for culverts in need of cleaning prior to construction. Section 619 provisions are for conditions which arise during construction. Culvert cleaning may be included in federal participating shares which contain other items of work where the cleaning work is incidental to the other federal-aided work.

8.6.5.2 Rehabilitation and Replacement

An inventory of existing culverts should be organized by material type. The age of each culvert should be determined and compared with the anticipated service life provided in Section 8.6.2.2. This information should form the basis to determine whether or not the culvert is reaching the end of its design and anticipated service life, and therefore is a candidate for rehabilitation or replacement.

The Regional Maintenance Group routinely inspects culverts over 5 ft in diameter. Their inspection reports should be reviewed to provide additional information regarding the condition and the need to repair or replace the culvert.

Replacement will seldom be a viable alternative in significant locations where long detours or extensive disruption to traffic will occur. When replacement is not considered desirable, existing pipes may be relined. Specification provisions for this work are provided in Section 602. Consideration should be given to any impact additional headwater will have on floodplain criteria, and adjacent landowners.

Replacement culverts should be designed for future traffic, future maintenance activities, a horizontal offset to the traffic barrier, and where permitted, bicyclists and pedestrians. Therefore, the culvert length should accommodate the minimum roadway width for bridge replacements in Section 2 of the [NYSDOT Bridge Manual](#). Although the roadway widths in the Bridge Manual were developed for bridges, they are appropriate for less costly culvert structures. When exceptions are appropriate, they should be approved as nonstandard features in accordance with the [Project Development Manual](#).

8.6.6 Safety - Roadside Design

Decisions regarding roadside design should be made consistent with Chapter 10. In general:

1. Any unnecessary drainage structures that would be hazardous to an errant vehicle should be eliminated. Examples might include: headwalls protruding above embankments; non-traversable ditches, particularly those perpendicular to traffic; check dams in ditches; etc.
2. Potentially hazardous drainage structures that are necessary should be relocated farther from the roadway, if practical, and if doing so would result in an adequate clear zone. Relocating drainage structures should not result in a discontinuous embankment.
3. Drainage structures which cannot be eliminated or relocated, should be modified to reduce the hazard, if such modification can be made without adversely affecting hydraulic performance.
4. Designers should refer to [HDM Chapter 10](#), Sections 10.2.1.2 and 10.3.2.2 for design guidance for culvert-end safety grates.
5. If the above four measures, considered in combination, will not be able to produce an adequate clear zone, then that roadside area should be shielded from traffic with an appropriate barrier system.

8.6.7 Rehabilitation of Culverts and Storm Drains

8.6.7.1 General

Replacement of a buried pipe system will seldom be a viable alternative in locations where long detours or extensive disruption to traffic will occur. When replacement is not viable or economically desirable, existing pipes may be rehabilitated by one of the following options:

A. Structural paving of the invert with Portland Concrete Cement (PCC).

This is an excellent rehabilitation methodology when the culvert has maintained its original shape, even if it exhibits considerable invert deterioration. Structural invert paving should be the predominant choice for rehabilitating large diameter arches and culverts, where using a new pipe lining method is cost prohibitive. Invert and barrel deterioration are the prime considerations when deciding on the extent and boundaries of the structural invert paving operation. These boundaries should be clearly shown on the contract documents. For worker access safety reasons, **structural paving of the invert should not be specified in culverts with diameters smaller than 48"**, except when no other option is feasible.

Design details on structural invert paving along with extended guidance can be found in Section 8.6.7.6. Structural invert paving may also be used in conjunction with shotcreting of the entire remaining culvert barrel (i.e. beyond the structural paving limits) further improving the structural capacity of the rehabilitated culvert. This hybrid rehabilitation scheme is recommended when the circumferential integrity of the culvert is questionable.

B. Lining with Shotcrete.

This is a cost-effective solution for large size culverts and arches that are experiencing distress in sidewalls, roof, and invert. The culvert should not display any significant structural deficiencies, buckling or other forms of deformation. It can also be used in combination with paving inverts. The plans need to indicate the reinforcement details and its limits along the culvert periphery. Utilizing welding or stainless steel mechanical anchors in reinforcement anchoring are the only options allowed. Lining with shotcrete could encompass the entire barrel of the pipe. A minimum cover of 2 inches over the corrugation crests is recommended. If for other reasons a designer would like to specify a thicker cover this must be clearly indicated on the plans. To accommodate the proper installation and application equipment, the item can only be used in culverts with a minimum diameter of 48 inches.

C. Lining with Cured in Place Pipe (CIPP).

This consists of a CIPP lining inserted into an existing culvert. When lining with CIPP, the designer must provide on the plans all necessary information so that the bidding Approved List installers are able to calculate the design (dead and live) loads on each liner based on the same critical design input parameters. The provided information must **at minimum** include, but it is not limited to, the culvert maximum depth of cover, estimated modulus of soil reaction E' and estimated water table elevation above culvert invert for the site. This information should be clearly noted on the plans, as: "The maximum depth of cover is

estimated from the top of the pipe to...”, or “The estimated water table elevation is measured from the culvert invert”. By providing this information on the plans, we improve the uniformity of bidding, since all submitted bids will be relying on the same design input parameters. Subcontractors mobilization cost should also be considered when selecting relining methodologies. For example, if CIPP lining has been selected for twelve culverts with bends while HDPE lining has been proposed for three straight run culverts, unless the three straight run culverts were of very large diameter (approaching 42” in diameter), it may be cost prohibitive to mobilize another subcontractor (HDPE lining) when the CIPP one will already be on site. Designers should be selecting the most cost-effective repair scheme.

D. Lining with New Pipe.

This rehabilitation technique can restore both structural and hydraulic capacity of a pipe. It is appropriate for culverts ranging in diameters from 12 through 120 inches. The existing (host) pipe should be relatively free of large bulges and in relatively good alignment. If bulges, alignment issues or other obstructions exist, they should be (if possible) eliminated in order to accommodate the unobstructed insertion of the new liner pipe. Lining pipe generally comes in 20-foot lengths and requires adequate end access space to accommodate insertion. When end access is limited, shorter pipe lengths may be utilized but have to be special ordered. Designers should contact lining pipe manufacturers or suppliers in advance to determine availability of short lengths. All available pipe rehabilitation materials are presented in Section 8.6.7.6.

When end access of an existing large pipe (72 inches or larger) is limited, Galvanized Steel Plate, Steel Structural Plate and Galvanized Steel Tunnel Liner Plate may be cost effective pipe lining options. This is true only if structural paving is not an option. These methodologies will also restore a culvert’s structural integrity.

Designers should never specify the grout mix design used to fill the annular space between the existing pipe and the lining pipe. It is the manufacturer's representative's responsibility to recommend a suitable grout compatible with the lining material used. The grout material recommendation must be included in the submitted written proposal to the EIC, in accordance to the specifications. The compressive strength of the grout is completely ignored (it is assumed that it carries no load and it does not contribute to the structural capacity of the composite “old pipe-grout-liner pipe” structure) in the liner thickness design calculations. Completely deteriorated conditions (i.e. it possesses zero remaining structural capacity) are assumed for the existing pipe during these calculations. Consequently, the proposed liner is required to possess adequate structural capacity to carry the entire calculated dead and live load. The maximum depth of cover for the culvert to be relined, estimated modulus of soil reaction E' , and the estimated water table elevation should be clearly noted on the plans.

Due to the potential for adverse environmental impacts associated with culvert rehabilitation activities (e.g. in-stream habitat, sediment dynamics, water/air quality), it is essential that designers coordinate the project development with the Regional Environmental Unit at an early stage in order to identify potential impacts and opportunities for mitigation.

The reader is strongly encouraged to review Section 8.6.7.6 for a detailed presentation of the above-mentioned rehabilitation techniques. Additional guidance regarding the rehabilitation of culverts is provided in AASHTO’s Highway Drainage Guidelines, Volume XIV.

8.6.7.2 Evaluating the existing culvert and site conditions

During hydraulic calculations to determine the minimum required diameter which satisfies the facility's hydraulic demand, the proposed lining pipe's wall thickness and/or the corrugation pattern may dictate the choice of lining materials.

It is imperative that the existing (host) culvert is thoroughly inspected in order to determine the most appropriate type of rehabilitation. The inspection should determine culvert's dimensions, material type, overall condition and structural integrity as well as site and/or existing pipe end accessibility for inserting lining pipe. Inspectors should clearly map the location and extent of distressed areas as well as all existing obstructions/buckles which may impact the size and insertion of the proposed lining pipe. Buckled pipes can be jacked near to their original shape provided that working room and proper access are available. However, excessive buckling of the existing pipe may severely obstruct and consequently preclude the use of any lining pipe. If visual inspection of the existing pipe is not feasible, a robotic or other remote inspection method is warranted.

Special order lining pipe lengths may be available for some types of lining pipes. If needed and as it was recommended in the "Lining with a new pipe" section, Designers should contact the pipe manufacturer or supplier to determine availability of shorter lining pipe lengths and approximate material costs.

If significant pipe perforations and/or backfill subsidence has been observed, consult with the Regional Geotechnical Engineer to determine the extent of any voids that may exist in the backfill material in the area above and immediately adjacent to the culvert.

8.6.7.3 Hydraulics & Service Life

All material used in culvert and storm drain rehabilitation should meet structural, hydraulic and service life requirements as identified in this chapter. For design calculation purposes, the existing culvert and any annular fill material (e.g., grout) used in the lining application are assumed to provide no additional service life nor contribute to the structural capacity of the lining pipe.

8.6.7.4 Geotechnical Issues

Consult with the Regional Geotechnical Engineer to determine and map the extent of any voids that may exist in the backfill material adjacent to the culvert. Voids not immediately adjacent to the culvert, which may have developed via infiltration of backfill fines into the culvert, are typically filled from above using a series of drill holes. For the sake of uniform bidding purposes, when voids are present, the Plans should include the following details: general site conditions, access, proposed end treatments, profiles and grade staging, voids mapping, any special situations, relevant restrictions, etc. The Regional Geotechnical Engineer should be contacted for selecting the voids filling material in the backfill. Very rarely culvert bearing embankments require grouting immediately prior to culvert rehabilitation. Extensive embankment grouting (filling of voids beyond culvert vicinity) is pursued only when the Designer and/or a Geotechnical Engineer decide(s) that the observed settlement poses a serious and immediate threat to the embankment integrity. Established practice for most cases

dictates completing the lining work prior to addressing any settlement related issues.

8.6.7.5 Cost

Each rehabilitation methodology has its own pay item which includes all labor and materials cost necessary to complete the installation. The depth of cover (or fill height above the culvert) and ground water table elevation impact the bidding price of a liner. The fill material in the backfill area above the pipe is paid for under a separate item provided by the Geotechnical Engineer.

8.6.7.6 Available lining methodologies and recommended conditions for use:

A. Adequate Structural Capacity.

If the culvert possesses reduced but adequate structural capacity and the culvert has maintained its original shape some structural capacity can be restored by rehabilitating the barrel. Under these conditions, the culvert could be effectively rehabilitated by selecting one of the following two methodologies:

1. Structural Paving of Inverts with Portland Cement Concrete (PCC)

This is a relatively low-cost solution for rehabilitating circular culverts and arches experiencing invert distress, primarily caused by abrasion or a combination of abrasion and corrosion. Generally, if the pipe or arch has maintained its original shape and does not experience other major structural deficiencies except the invert loss, the culvert can be rehabilitated by structurally paving the invert, regardless if the invert shows considerable deterioration. Due to the higher cost of lining pipes using other approved rehabilitation methodologies, the effectiveness of invert paving for invert distressed drainage facilities should be explored first. Structural invert paving is appropriate when there are no other major structural deficiencies in the culvert besides the invert deterioration, and the pipe is also of sufficient size (a minimum culvert diameter of 48" is recommended) to accommodate safe execution of this work. The designer should clearly indicate in the plans the concrete cover over the corrugation crests, the type and layout of the concrete reinforcement, and the paving area limits along the periphery of the culvert. Note that the periphery paving limits should always extend beyond the area of significant corrosion loss, allowing reinforcement to be attached onto sound metal locations on both sides of the invert. The 602 standard specification requires that all reinforcement details shall be shown on the plans.

Design details for structural invert paving of culverts spanning up to 10 feet and bearing up to 20 feet of fill over the crown are available in .dgn format under "Resource Links" at:

<https://www.nysdot.gov/divisions/engineering/design/dgab/hdm/chapter-8>. Repairs for culverts falling outside these parameters need to be designed on an individual basis and the Office of Structures can be consulted for assistance. Welded wire fabric reinforcement embedded in a 4" deep/thick concrete slab over the corrugation crest is recommended for

structural invert paving of round pipes and arches spanning up to 6 feet. The welded wire fabric reinforcement can be attached directly onto the corrugations by welding or utilizing stainless steel mechanical anchors. These are the only two anchoring options allowed for welded wire fabric reinforcement embedded in 4" thick concrete slab for culverts spanning less than 6 feet.

Reinforcement bars are recommended for all arches (regardless of span) and for all round pipes spanning between 6 and 10 feet. The reinforcement is embedded in a 6" or 8" thick concrete slab, depending on the culvert's span, with a minimum of 2" reinforcement cover. Shear transfer is achieved by anchoring (welding) shear studs to sound metal on the corrugation crests (see relevant drawing detail Figure 2). Reinforcement bars shall only be attached to shear studs (never directly to the culvert walls) by wire tying (see relevant drawing detail Figure 1). Reinforcement bar sizes will be selected based on the recommendations established by the Office of Structures (see Figure 3). All reinforcement and shear studs should be covered with concrete and the concrete should be sloped in such a way as to prevent water ponding on the side walls. Small areas of suspected metal loss around culverts circumference do not necessarily preclude the use of this item as an effective rehabilitation technique, since the remaining circumference above the structurally paved invert area could also be lined with shotcrete. Since these treatments modify culverts hydraulics characteristics, designers must consult with the Hydraulics Design section on this topic. Designers are directed to use a Manning's "n" value of 0.013 for hydraulic calculations in the paved area of the culvert. Structural paving of corrugated pipe inverts with PCC results in reduced roughness, leading to increased flow velocity and sediment transport competence. This may in turn negatively affect aquatic habitat connectivity as the barrel will retain less streambed sediment than it did in its prior unpaved condition. It is recommended that surface roughening techniques (e.g. raking the fresh concrete perpendicular to the flow) be considered for use where appropriate and practicable. Designers are also reminded that a culvert with a previously paved invert could be re-rehabilitated with concrete lining.

2. Lining with Shotcrete

If the entire culvert circumference (or even when an area beyond the extent of structural invert paving but less than 20% of the total corroded area) exhibits signs of minor corrosion, lining with shotcrete is a viable option. Designers may utilize the Design Standard details for structural paving developed by the Office of Structures Design which are available in .dgn format under "Resource Links" at: <https://www.nysdot.gov/divisions/engineering/design/dqab/hdm/chapter-8>, after simply substituting the concrete with the appropriate shotcrete item. As mentioned in the previous section, shotcrete can be used in conjunction with structural invert paving to address corrosion beyond the extent of paving. Shotcrete can also be used for localized rehabilitation of a distressed culvert section. If properly specified, this method can also restore culvert structural capacity. Designers are directed to use a Manning's n value of 0.013 for hydraulic calculations in the shotcreted area of the culvert. For worker access safety reasons, this lining methodology should not be utilized in culverts with diameters smaller than 48".

B. Inadequate Structural Capacity.

If the structural integrity of the culvert is questionable, the following six lining options may be considered:

1. Lining with High Density Polyethylene Pipe (HDPE)

High Density Polyethylene Pipe meeting ASTM F894 (Profile Wall) or ASTM F714 (Smooth Wall) may be used in association with the height of cover table provided below. There are two types of HDPE liners: a profile wall often without exterior corrugations, and a smooth solid wall pipe. The exterior and interior diameters of these liners vary among manufacturers. Considerable confusion has been generated when specifying the required size of these two lining pipes. Profile wall pipe is specified by the inside diameter, and as the required load increases, the wall profile thickens. Smooth wall pipe is specified by the outside diameter and the actual inside diameter diminishes with greater loads, as the wall thickens. Smooth wall HDPE lining pipe has a much higher material cost because of the larger resin volume required for its fabrication compared to the profile wall. For good grouting practice purposes Standard Specification 602 requires a minimum annular space of 1 inch between the host pipe and the liner. Therefore, the outside diameter of the liner pipe must be a minimum of 2 inches smaller than the diameter of the existing (host) pipe. This is an important size constraint when calculating the hydraulic capacity of the relined installation. As a result of these dimensional constraints, designers are at times forced to specify the more expensive smooth wall liner pipe instead of the less expensive profile wall. This is the case, because for the same load carrying capacity, the smooth wall liner pipe possesses a thinner wall section, allowing for more interstitial space between the proposed liner and the host pipe. To prevent problems during construction between the two wall types, both types have their own specification and the Designers' wall selection should be clearly indicated in the plans. The original specification has been retained as an optional item. It can be used when abundant interstitial/annular space exists and the consequences of a potentially arbitrary swap in the field are not a major concern. Currently available diameters for these pipes range from 18 to 60 inches for solid wall pipe, and 18 to 96 inches for profile wall pipe.

Various joint types can be specified to hold the liner segments aligned and firmly together through the liner insertion process and until the grout sets in place. The issue of joint type selection for HDPE liners is addressed in the revised 602 specification as: "*Perform all butt fusion and extrusion welding of HDPE pipe in accordance with the Manufacturer's recommendation*" and "*Alternate joining methods will be subject to approval by the Director, Materials Bureau*". Two mechanical joints are currently approved, a threaded joint and a smooth exterior bell and spigot joint. A mechanical joint wrap process has been evaluated and approved with some dimensional restrictions associated with its use at this time. However, butt fusion and extrusion welding are still the most durable joining methods, effectively creating a continuous liner pipe but they are also more labor intensive and hence more expensive than other joining methods. When alignment breaks or pinch points are encountered in a project, designers should only consider butt fusion and extrusion welding as the joining methods of choice, as they provide the most reliable joints for these challenging site conditions. Designer choices

should be clearly stated in the notes and placed in a very prominent spot in the contract documents. The notes should also explicitly state that these joint selection restrictions solely apply to the host pipes possessing these challenging morphological conditions. Again, the maximum depth of cover for the culvert to be relined, estimated modulus of soil reaction E', and the estimated water table elevation should be clearly noted on the plans.

HDPE Liner Diameter (inches)	18	21	24	27	30	33	36	42	48	54	60	66	72	84	96
Maximum Allowable Cover (feet) ¹	30	30	33	26	26	26	26	23	23	26	26	26	26	33	36

¹Maximum vertical distance between the top of the pipe and the top of pavement.

2. Lining with Cured in Place Pipe (CIPP)

Lining with Cured in Place Pipe (CIPP) is an option for round pipes when end access is limited, pipe alignment includes bends, or when other lining methods would seriously reduce culvert hydraulic capacity. CIPP can restore both the structural and the hydraulic capacity of a pipe. Although this item is relatively expensive compared to other relining items, it could be the best rehabilitation alternative if all or some of the above site conditions exist. Designers are responsible for making this selection decision based on site conditions and associated regional installation cost. This item should very rarely be considered as a viable rehabilitation method for arches or rectangular culverts. CIPP is best suited for circular drainage facilities with limited access to and ranging in diameter from 12 to 42 inches. CIPP technology can be applied to lining larger diameter culverts, but in these cases, the installation cost is usually prohibitive and CIPP lining should only be considered for very short lengths. The ability to prepare (“wet”) the liner off site as opposed to “wet” it on site (near the culvert’s access point) greatly impacts operational cost. High depths of cover and elevated water tables demand thicker CIPP liners. These adverse site conditions coupled with large diameter and/or long host culverts, result in very heavy liners, which forces the installer to perform the “wetting” operation over the culvert’s access point. This additional cost may render the overall cost of this operation prohibitive. Consult with the Materials Bureau when dealing with large diameter and/or excessively long liners.

Lining Concrete Pipe with Cured in Place Pipe (CIPP) is an option created to utilize a partially deteriorated design methodology and can ONLY be used for lining concrete pipes. This item should ONLY be used when the concrete pipe still possesses some measurable structural capacity, but inspections have raised some concerns, which can be alleviated by lining. These concerns may include, but are not limited to, damaged or separated joints, infiltration of ground water from faulty joints or incipient cracks (cracks not threatening the structural capacity of the concrete pipe) or as a precautionary measure for abrasion protection. The designer may follow the same guidance as with CIPP (see above), but this concrete pipe pay item should be selected. It is anticipated that invoking the partially deteriorated design methodology, ONLY where warranted, will result in substantial material cost savings and reduced overall bid prices.

A Manning's coefficient "n" value of 0.013 should be used for hydraulic calculations in all CIPP sections. The CIPP relining item is often selected by designers as a result of two very important features. The usually improved Manning's coefficient of the rehabilitated culvert coupled with the smallest cross section reduction possible as a result of selecting CIPP vs. any other relining methodology. Once again, only round culverts should be lined with this method. The anticipated service life of this treatment is 70 years.

It is imperative that all obstructions in the culvert are removed prior to initiating the CIPP lining procedure. The bid price for the CIPP item includes the cost of removing any obstructions, along with re-establishing culvert to secondary pipe connections and the removal of any protruding pipes as required. It should be noted that Department policy for storm drainage allows addition of pipe connections onto a pipe only within a structure (e.g. manhole, drop chamber, etc.) and never in between such structures, so finding all pipe to pipe connections should be a straightforward process. A thorough culvert inspection is required to determine the number of existing "pipe to pipe" connections and the extent, if any, of obstruction removal. If the culvert opening (36" in diameter or less) prohibits a human-led visual inspection, a closed-circuit television inspection should be performed by experienced personnel trained in locating breaks, obstacles and service connections during the design phase of the project. This information should be made available to all bidders to assist them in preparing an accurate bid.

Designers should be aware that in the field, the liner is inverted and filled with water, which pushes the liner through the culvert. The water also holds the resin-impregnated liner in position and in contact with the host pipe. The water is circulated through a mobile boiler which raises its temperature in excess of 160°F. The heat of the circulating water cures the resin and transforms the formerly flexible liner into a solid continuous conduit. The liner curing process typically lasts several hours. Once the liner is completely cured, the curing water is removed, then, all (if any) service connections are restored, using robotic cutting devices, especially for culverts less than 36" in diameter. The completed liner should be inspected, often using a closed-circuit television, especially for culverts less than 36" in diameter.

Environmental impact concerns and information gathered from the extensive use of CIPP, have led to a series of new product developments as well as amendments to installation procedures. Curing water from CIPP installations utilizing a styrene-based resin contains some styrene residual. NYS Surface Water and Groundwater Quality Standards and Groundwater Effluent Limitations (6 New York State Department of Codes, Rules and Regulations (NYCRR) Part 703) set quantitative and qualitative standards for effluents to NYS water bodies. These standards may vary for different classes of receiving waters or for discharging it to groundwater. NYS Water Quality Standards include specific limit guidelines for discharging certain pollutants (including styrene) to various surface water classes and groundwater. The limit of 0.05 ppm applies to surface water bodies with classifications A, A-S, AA and AA-S while the limit of 0.005 ppm applies to groundwater with GA (fresh groundwater) classification. The general conditions listed in 6 NYCRR Part 701 apply to all water classifications. They dictate that discharges shall not cause impairment of the best usages of the receiving water (as specified by the water classifications) both at the location of discharge or at any

other location which may be affected by such discharge. Note that thermal loading considerations (6 NYCRR Part 704) are directly related to the release temperature and discharge rate of the curing water. Provisions for handling and/or disposal alternatives as well as other control procedures are included in the specifications to address the presence of styrene in the curing water and other potential releases to water and air from the byproducts of the CIPP installation. These provisions include:

- Some procedural changes to enhance control of the CIPP process and leakage of resin, including utilizing a preliner bag and excavating a temporary resin control pit at the outlet.
- Allowing the use of non-styrene based resins containing less than five percent volatile organic compounds (VOCs) with less than 0.1 percent hazardous air pollutants (HAPs). These resins are now included in the 602 Standard Specifications and can be used by the Contractor in any CIPP site. The cured liner should also contain less than 0.1 percent of water quality pollutants (as listed in 6 NYCRR Parts 700-705)". NYSDOT may determine and dictate that at certain areas of greater environmental concern (i.e. in the vicinity of class A, A-S, AA and AA-S water sources or at other locations presenting other needs) non-styrene based resins MUST be used. If the Department requires the use of only a designated resin type, the resin type requirements shall be clearly noted in the contract documents; otherwise, the Contractor may select the resin type from the resin approved list included in the CIPP Approved Installer's Materials Procedure document, which is kept on file at the Main Office, Materials Bureau.
- In all CIPP installations utilizing a styrene based resin, it is required to collect the curing water for:
 - Reuse in another curing operation
 - Treatment or disposal at an off-site facility; or
 - Release on site after treatment to standards dictated by NYSDEC and with approvals from NYSDEC.

Summarizing, the resin type alternatives for CIPP work are to either use a non-styrene based resin or follow additional controls when using a styrene-based resin.

Areas of environmental concern may include, but are not limited to, wetlands, fishing streams and ponds, upstream of public water sources, densely populated urban areas where residents are concerned about the short-lasting odor of the styrene during curing, etc. Waters classified as Class A are: a source of water supply for drinking, culinary or food processing purposes; primary and secondary contact recreation; and fishing (i.e. waters suitable for fish, shellfish, and wildlife propagation and survival). Although numerical standards are not listed for other classes of streams in 6 NYCRR Part 703, the general requirements of Part 701 apply (i.e., discharge must not cause impairment of the best usages of the receiving water).

The designer shall determine the location of each installation in respect to environmental resources including the classes of streams/watercourses in the vicinity of the project. The regional environmental unit and/or "Environmental

Viewer” (A GIS application currently available at the Office of the Environment Intradot site and under the manual / applications tab, subsection GIS Application) shall be consulted for stream classification and other relevant environmental information. For installations close to and presenting the potential for curing water releases to class A, A-S, AA and AA-S streams or releases of other concern, the requirement for using a non-styrene based resin should be considered.

If designers determine that a non-styrene based resin shall be used in a specific CIPP installation, then the relining project’s contract documents have to clearly and unambiguously specify that “a resin containing less than five percent volatile organic compounds (VOCs) with less than 0.1 percent hazardous air pollutants (HAPs) must be used”. It should also clearly state that “the cured liner should contain less than 0.1 percent of water quality pollutants (as listed in 6 NYCRR Parts 700-705)”. Use of these resins will result in a significant increase to the rehabilitation cost, approximately double to the materials cost of using regular styrene based resins in 2009 market prices. Therefore, the decision to specify non-styrene based resins should not be made lightly and never prior to evaluating whether the use of styrene based resins combined with other alternative provisions to control the release of the curing water provide both enough site environmental protection and project cost savings. These provisions (listed now in the 602 Standard Specification) include, but they should not be limited to: removal (pumping out) of the curing water at the end of the CIPP installation, transporting it to an appropriate disposal facility instead of free draining it at the outlet, providing for a resin catchment pit excavated right at the culvert’s outlet to create ideal conditions for the collection of the trace amounts of styrene, temperature control of the released curing water, as well as combinations of the above. Again, designer’s stipulations on environmental protection provisions should be clearly indicated on the plans as they may substantially impact the operational cost. These stipulations should be limited to the class of the neighboring stream or water course class which may be impacted by the release of the curing water, the need for utilizing a styrene or a non-styrene based resin, the maximum allowed release temperature of the curing water and the size of the resin catchment pit. It is recommended that the resin catchment pit dimensions are: 12 to 15 feet long, twice the culvert diameter wide and one foot deep. If a rip-rap layer has been installed at the culvert outlet (potentially even in the stream bed), the proposed depth of the catchment pit will be equal to the thickness of the rip rap layer (it could even be over a foot deep) and as indicated on the record plans. If the natural bedrock is encountered near the culvert invert elevation, the catchment pit bottom elevation must be just (at least) below the culvert invert, so that the CIPP installer is able to create the proposed temporary catchment pit. CIPP installers will decide as to the catchment pit lining, as they are responsible for preventing the liner curing process byproducts and waste from reaching the surrounding ground and percolate / leach into the groundwater. The excavation, temporary storage of the fill and restoration of the downstream channel will be paid for under pay item 206.0201 *Trench and Culvert Excavation*.

The majority of the environmental controls in any CIPP project are dictated by the written agreement (Materials Procedure) between the Approved List manufacturer/ installer and the Materials Bureau. If designers have any concerns about using the CIPP relining item at a specific location, especially when styrene based resins are employed, they should first consult with the Materials Bureau.

Current designs address fill heights up to a maximum of 50 feet. If the fill height over the CIPP installation exceeds 50 feet at any point along the culvert alignment, contact the Materials Bureau to coordinate an evaluation of the proposed CIPP liner design.

3. Lining with Polyvinyl Chloride Pipe

The 602 relining specification also allows the contractor to use PVC pipe meeting ASTM F1803, ASTM F949, and two new standards, ASTM F679 or ASTM D3034 (small diameters 12" or 15") as relining material. PVC pipe is currently available in diameters ranging from 12 inches through 36 inches on the current Approved List. Designers must consult the Approved List for the addition of new manufacturers and/or products as well as for the currently approved sizes for each PVC material. PVC lining pipes are corrosion and abrasive resistant. A Manning's coefficient value of 0.013 should be used for all hydraulic capacity calculations related to PVC pipes.

Some PVC pipes are more brittle than other flexible relining materials. Therefore, designers should determine prior to specifying it, that the condition and shape of the host culvert will allow for an unobstructed insertion of the PVC lining pipe.

Since the 602 relining specification requires a minimum annular space of 1" for effective grouting, the outside diameter of the liner pipe needs to be a minimum of 2" smaller than the inside diameter of the existing (host) pipe when calculating the hydraulic capacity of the new installation. Profile wall pipe (ASTM F949) is specified by the pipe's **nominal** inside diameter while the respective outside diameters can be found in ASTM F949. ASTM F1803 lists only the **nominal** inside diameter of these respective materials, therefore designers must consult with the pipe manufacturer to obtain the outside diameter information. Both ASTM F949 and ASTM F1803 pipes possess joints that are flush with the outside diameter. However, ASTM F679 and ASTM D3034 pipes do not have a joint flush with the outside diameter of the pipe but rather a bell and spigot joint protruding from the outer pipe shell. Therefore, for ASTM F679 & ASTM D3034 pipes, it is **the outside diameter of the joint and not the outside diameter of the pipe** which dictates the maximum allowable liner diameter for a particular application. Because of this confusion, the lack of readily available info for some PVC pipes and in order to properly evaluate selected PVC pipe size compliance with the minimum required annular space for effective grouting, **designers are strongly encouraged to consult with the Materials Bureau about the maximum outside dimension of any PVC pipe, if this information is not available to them.**

The table below provides the allowable fill heights for each size of PVC liner pipe. If the fill height exceeds 50 feet at any point along the pipe alignment, contact the Materials Bureau for approval of the proposed liner design. The anticipated service life of a PVC liner is 70 years.

PVC Liner Diameter (inches)	12	15	18	21	24	30	36
Maximum Allowable Cover (feet ¹)	50	46	33	50	50	50	26

¹ Maximum vertical distance between the top of the pipe and the top of pavement.

4. Lining with Tunnel Liner Plate

In some closed drainage systems, space constraints may limit lining options which involve installing full lengths of pipe sections. In these cases, galvanized tunnel liner plate may be a viable option. Tunnel liner plate can be inserted and assembled within the host culvert as the laying length of individual sections is 18 inches. Galvanized Tunnel Liner Plate requires a PCC invert to provide the required service life and hydraulic performance.

Structural paving of the invert improves the hydraulic efficiency of the culvert and protects the tunnel liner bolt flanges from abrasion caused by bedload. When site conditions dictate that only tunnel liner plates can accommodate the structural demand of the culvert, but hydraulics analysis dictates that a low Manning material must be used, shotcreting the invert or HDPE lining is a valid solution. Consult the Materials Bureau for further guidance when such cases arise.

5. Lining with Polymer Coated Corrugated Steel Pipe (CSP)

If the rehabilitated pipe is anticipated to be subjected to potentially abrasive bed loads or to be exposed to high concentrations of industrial waste, the Polymer Coated CSP item is a suitable relining material for these conditions. The suggested Manning’s coefficient “n” value will range from 0.013 to 0.026, depending on the corrugation pattern selected. The recommended Manning’s coefficient n” values for all drainage materials are provided in Section 8.6.3.1.

The height of fill tables provided in Appendix A shall be used to determine if the 12 gauge steel pipe meets the site’s height of fill requirements.

6. Lining with Steel Structural Plate Pipe or Pipe Arches with PCC Paved Invert

Steel structural plate can be partially or fully assembled and pushed or pulled into place within the pipe. This lining option should be considered for large diameters where other items are unavailable. If the plate sections are too large to be inserted into the host culvert through the exposed ends, then an excavation and a field cut through the culvert roof would be required. Every effort should be made to ensure that the excavation and field cut are located where it will be the least disruptive to traffic.

Designers must clearly indicate in the plans how to pave the invert of the plate pipe or arch. The minimum extent of the invert paving will be the bottom 30 percent of the inside circumference for round pipes, the bottom 35 percent of the inside periphery for arch spans up to 10 ft and the bottom 40 percent of the inside periphery for arch spans greater than 10 ft. If this paving coverage encompasses (with some margin of safety) at least 6 in. circumferentially beyond the limits of the culvert's wetted perimeter for mean flow conditions, then this paving coverage will be sufficient. If it does not, then a greater percentage of coverage should be used. A minimum cover of 2 in. over the crests of the corrugation and the steel fabric reinforcement should also be noted on the plans.

8.7 STORM DRAINAGE SYSTEMS

Storm drainage systems should be considered to protect the roadway and the traveling public from stormwater when right of way limitations, or the presence of curbs and sidewalks do not allow stormwater to run off of the roadway. Depending on the adjacent terrain, storm drainage systems consist of some or all of the following items: curb, gutter, inlets, drainage structures, storm drains, and an outfall or outlet, which discharges collected stormwater to a point of release in the receiving waters (i.e., a stream, river, lake, storage facility, etc.). Pumping stations and flap gates are other items which are not as common. These other items are not discussed in this section, and should be designed consistent with the guidance provided in the references.

There are two principal concerns when designing a storm drainage system:

1. Preventing excessive spread of storm water onto the traveled way, and
2. The protection of adjacent natural resources, and property.

The first concern is addressed through the design of curbs, gutters, and inlets. The second concern is addressed through the design of the storm drainage system outlet, and the type of receiving waters. Regardless of the design or size, there will be occurrences when the capacity of a storm drainage system is exceeded.

Storm drainage design is an iterative process. Due to the time-consuming nature of storm drainage design, the use of drainage software is recommended. A computer assisted design will generally be more cost effective than manual methods because the design iterations are automated.

This section presents storm drainage design criteria, and provides an introduction to the components and considerations regarding storm drainage system design. Detailed explanations, formulas, and procedures are provided only to the extent necessary to convey the subject. The following publications should be referred to for a complete discussion of the formulas and design procedures used during design:

1. "Storm Drain Systems" (Chapter 9 of the "AASHTO Highway Drainage Guidelines") provides an overview of storm drainage design. This reference does not provide a discussion of procedures, e.g., detailed discussion and formulas regarding the calculation of the hydraulic gradeline.
3. "Storm Drainage Systems" (Chapter 13 of the "AASHTO Drainage Manual"), and "Hydraulic Engineering Circular" (HEC) No. 22, "Urban Drainage Design Manual" provides procedures, formulas, and example problems.

8.7.1 Planning and Coordination

The preliminary layout and design of storm drainage systems and the planning for stormwater management should begin early in the design process when right of way taking lines are being developed. Potential locations of receiving waters, either existing (natural watercourses, or surface waterbodies) or proposed (recharge basins, leaching basins), should be identified at this time. Additionally, slopes, adjacent terrain and development, environmental resources, and locations of existing drainage facilities and utilities should be identified. All these factors should be considered in establishing the highway alignment and the right of way lines.

When a highway typical section is changed to include sidewalks, curbs, or any element which prohibits stormwater from flowing off the roadway, a storm drainage system will generally need to be added also. Adequate funds must be programmed during the scoping stage to provide for this addition.

8.7.2 Hydrologic Analysis

A hydrologic analysis shall be progressed consistent with the guidance in Section 8.3. The Rational Method shall be used to calculate the peak discharge to be conveyed by the gutter, and storm drains. Storm drainage systems which involve detention storage, pumping stations, and large or complex storm drainage systems require the development of a runoff hydrograph to determine the peak discharge. Exhibit 8-3, in Section 8.3, provides the design flood frequencies to be used for storm drainage system design.

As previously defined in Section 8.3, the T_c is defined as the time required for water to travel from the most remote point in the watershed to the point of interest. For storm drainage system design, the T_c generally consists of two components, one for inlet spacing (discussed in Section 8.7.4.4.C), and the other for pipe sizing (discussed in Section 8.7.5.3.A).

The flow path having the longest T_c to the point of interest in the storm drainage system will usually define the duration used in selecting the intensity value in the Rational Method. Exceptions to the general application of the Rational Equation exist. For example, a small relatively impervious area within a larger drainage area may have an independent discharge higher than that of the total area. This may occur because of the high runoff coefficient and high intensity resulting from a short T_c . If an exception does exist, it can generally be classified as one of two exception scenarios.

The first exception occurs when a highly impervious section exists at the most downstream area of a watershed and the total upstream area flows through the lower impervious area. When this situation occurs, two separate calculations should be made as follows:

First, calculate the runoff from the total drainage area with its weighted runoff coefficient and the intensity associated with the longest T_c .

Secondly, calculate the runoff using only the smaller, less pervious area. The typical procedure would be followed using the runoff coefficient for the small, less pervious area and the intensity associated with the shorter T_c .

The results of these two calculations should be compared and the largest value of discharge should be used for design.

The second exception exists when a smaller, less pervious area is tributary to the larger primary watershed. When this scenario occurs, two sets of calculations should also be made.

First, calculate the runoff from the total drainage area with its weighted runoff coefficient and the intensity associated with the longest T_c .

Secondly, calculate the runoff to consider how much discharge from the larger primary area is contributing at the same time the peak from the smaller, less pervious tributary area is occurring. When the small area is discharging, some discharge from the larger primary area is also contributing to the total discharge. In this calculation, the intensity associated with the

T_c from the small, less pervious area is used. The portion of the larger primary area to be considered is determined by the following equation:

$$A_c = A \frac{T_{c1}}{T_{c2}} \quad \text{where:}$$

- A_c = the most downstream part of the larger primary area that will contribute to the discharge during the T_c associated with the smaller, less pervious area.
- A = the area of the larger primary area.
- T_{c1} = the time of concentration of the smaller, less pervious, tributary area.
- T_{c2} = the time of concentration associated with the larger primary area as is used in the first calculation.

The runoff coefficient (C value) to be used in this computation should be the weighted C value that results from combining C values of the smaller, less pervious tributary area and the area A_c . The area to be used in the Rational Method would be the area of the less pervious area plus A_c . This second calculation should only be considered when the less pervious area is tributary to the area with the longer time of concentration and is at or near the downstream end of the total drainage area.

Finally, the results of these calculations should be compared, and the largest value of discharge should be used for design.

8.7.3 Gutters

This section considers gutters as that part of the typical section which conveys stormwater along the edge of the traveled way, shoulder, or parking lane.

8.7.3.1 Types of Gutter Section

The gutter section may or may not incorporate curb. When curb is used, the gutter section is termed a conventional curb and gutter section. The cross slope of a conventional curb and gutter section may be continuous or composite. If the spread of water is limited to the shoulder, parking lane, or traveled way, the cross slope is continuous. If the spread is allowed to encroach onto the traveled way from the shoulder or parking lane, a composite section will exist due to the differing cross slopes of these typical section elements. Refer to Chapter 3, Section 3.2.9, for guidance regarding types of curb, limitations on curb use, warrants for curb use, curb alignment and typical section details, and curb material selection and installation details.

A gutter section without curb may be considered a shallow swale section, is formed as detailed on Standard Sheet 624-01, and is illustrated in Figure 3-2 of [HDM Chapter 3](#). Refer to Chapter 3, Section 3.2.10 for guidance regarding the placement of shallow swale sections at driveways. Formed gutter shall not be placed to interfere with a crosswalk.

8.7.3.2 Gutter Capacity

Gutter flow calculations are necessary to establish the capacity of the gutter within the spread and puddle depth criteria in Section 8.7.4.4.C. Gutter capacity is determined based on a modified form of Manning's equation. This equation is presented below to illustrate the basis of the computation:

$$Q = (0.56/n)S_x^{1.67}S^{0.5}T^{2.67}, \text{ where:}$$

- Q = Flow rate (ft³/s)
- n = Manning's roughness coefficient
- T = Width of flow or spread in the gutter (ft)
- S_x = Cross slope (ft/ft)
- S = Longitudinal slope (ft/ft)

Each variable is discussed below.

Q is determined for the drainage area contributing flow to the gutter. Desirably, overland flow should be intercepted before it reaches the roadway. This will allow the gutter to primarily convey water running off of the roadway. Manning's n values are provided in the references and are based on the gutter surface. T is based on Q, n, S_x, and S. S_x is based on the design criteria (i.e., Chapter 2 or 7 of this manual) for the project. S is based on the vertical alignment of the roadway and drainage considerations. (Guidance regarding vertical alignment is provided in Chapter 5, Section 5.7.4, of this manual.)

Using the modified form of Manning's equation, and the relationship between spread and depth of flow or puddle depth at the curb (d), $d = TS_x$, spread and puddle depth are determined and compared to the criteria in Section 8.7.4.4.C. Nomographs are provided in the references, and computer software will aid in the determination of spread and puddle depth. The formulas, nomographs, and software are specific regarding what type of gutter section exists (e.g., conventional (uniform or composite cross slope) or shallow swale section).

Inlets are placed when the spread or puddle depth in the gutter section exceed the criteria, and at other locations as discussed in Section 8.7.4.4.

8.7.4 Inlets

Inlets are placed to intercept the concentrated flow of stormwater. Typical locations are in gutter sections, paved medians, and in roadway and median ditches. When curb is used to form the gutter section, the inlet shall be placed in front of the curb. Inlets shall not be recessed back from the curbline due to the potential hazard they present by creating a discontinuous curb line.

8.7.4.1 Types of Inlets

This section discusses three major types of inlets.

1. Curb-opening inlet. This inlet provides a vertical opening in the curb. No grate is specified. The curb-opening inlet is not depicted on the standard sheets because it not hydraulically efficient and as a result, rarely used.
2. Gutter inlets. Gutter inlets include grate inlets and slotted inlets.
 - a. Grate inlets provide a horizontal opening in the channel section (gutter or ditch) and are recommended for gutter locations where curb is not proposed, and locations off the roadway. Three grate types – rectangular, parallel bar (with and without cross bars), and reticuline – are illustrated on Standard Sheets 655-01, 655-02, and 655-04. The frames associated with these grates are also illustrated on the standard sheets.

In addition to the grates used by the Department, the references mentioned in Section 8.7 discuss two other types of grates, the curved vane and the tilt-bar. These grates are not included on the standard sheets.

- b. Slotted inlet (pipe interceptor drain). This inlet consists of a pipe cut along the longitudinal axis with a grate of spacer bars to form slot openings. This type of inlet is acceptable for use, although not depicted on the standard sheets.
3. Combination inlet. This inlet consists of both a curb-opening (depicted as the curb box on Standard Sheet 655-05) and a grate placed in a side-by-side configuration, and is recommended for gutter locations where non-mountable or mountable curb is proposed because of the added capacity offered by the curb opening should the grate become clogged. Three sizes of cast frames (F1, F2, and F3) are available. Each size is available with either a mountable or unmountable cast curb box, both of which are also illustrated on the standard sheet.

8.7.4.2 Inlet Selection

The predominant types of inlets used by the Department are the grate inlet and the combination inlet. The curb-opening inlet and the slotted inlet are used less often.

1. Grate Inlets. Parallel bar grate inlets without the cross bars have superior hydraulic capacities and should be used in locations outside the roadway, or on highways with full or partial control of access where bicycles and pedestrians are not allowed (such as interstates, other freeways, and parkways). Bicycles and pedestrians should be accommodated on all highways with uncontrolled access by providing one of the following types of grate:
 - a. Reticuline. This grate is superior for pedestrian and bicycle safety.
 - b. Rectangular. Similar to parallel with cross bars.
 - c. Parallel bar with cross bars. The cross bars shall be specified in the drainage structure table.

A maximum grate opening to safely accommodate pedestrians has not been specified by the Department. Where pedestrian traffic is a concern within the roadway, sidewalks should be considered in accordance with Section 18.6 of Chapter 18.

4. Combination inlets are valuable when debris is a problem and at low points where water velocities are low. Whenever inlets are placed along curbs, the combination inlet should be used. When an inlet is placed along a short radius (less than 100 foot) curb return, it is difficult to construct a combination inlet and a grate inlet should be used. The type of grate specified should be consistent with the guidance in item 1.

8.7.4.3 Interception Capacity of Grate Inlets and Combination Inlets

Inlet interception capacity is the flow intercepted by an inlet under a given set of conditions. The capacity of an inlet depends on its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Gutter flow that is not intercepted by an inlet is termed carryover, bypass, or run-by. Inlet interception capacity of the grates discussed in Section 8.7.4.1 have been investigated by the FHWA. The results of this investigation are reported in Volumes 1 through 4 of the "Bicycle-Safe Grate Inlets Study". The references in Section 8.7 contain formulas and charts, and software is available for use to determine inlet capacity.

A. Interception Capacity of Grate Inlets and Combination Inlets on Continuous Grade

The interception capacity of grate inlets and combination inlets is considered to be the same when they are placed on a continuous grade operating under roadway depth and spread criteria. The added capacity offered by the curb opening is ignored during design. The interception capacity (Q_i) of a grate inlet on grade is based on the efficiency of the grate (E) and the total gutter flow (Q). The efficiency of an inlet is the percent of total flow that the inlet will intercept. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and to a lesser extent, pavement roughness.

B. Interception Capacity of Grate Inlets and Combination Inlets In Sag Locations

A grate inlet operates as a weir up to about a 0.4 ft depth of water over the grate and as an orifice for depths greater than 1.4 ft. The interception capacity of the combination inlets shown on the standard sheets is essentially equal to that of a grate inlet in weir flow. Between 0.4 ft and 1.4 ft, a transition from weir to orifice flow occurs. Inlets located in a roadway sag will operate as a weir (a defined discharge vs depth relationship exists). When inlets are placed in a ditch sag outside the roadway, the inlet may operate as an orifice if sufficient depth is allowed to develop over the inlet.

8.7.4.4 Inlet Locations

The location of inlets is determined by geometric controls which require inlets at specific locations, the use and location of flanking inlets in sag vertical curves, and the criterion of spread on the roadway. In addition, providing access for storm drain maintenance may be considered at this time even though the storm drains have not been sized. Generally, an inlet should be provided every 300 ft for pipes less than or equal to 24 inches in diameter, 400 ft for pipes 27 to 36 inches in diameter, and 500 ft for pipes 42 to 54 inches in diameter. Inlets may be spaced farther apart when flow velocity in the storm drain will provide for the transport of suspended solids and thus be self-cleaning. As this is primarily a maintenance consideration, a recommendation from the Regional Maintenance Group should be considered.

The following should be obtained or prepared to aid in inlet location design:

1. A plan to delineate the drainage areas. The contributory areas and the corresponding runoff coefficients for each proposed inlet should be shown on a small scale plan (preferably at 1"=200' or 1"=500' scale). If the contributing areas are within the roadway section, then large scale plans (1"=20' or 1"=40') should be sufficient. If the contributing drainage areas extend beyond the plan limits, other maps should be used where available or estimated extensions should be drawn on the plans to show these extended contributing drainage areas.
2. Profile.
3. Typical sections.
4. Superelevation diagrams.

A. Geometric Controls

There are a number of locations where inlets should be placed without regard to the contributing drainage area. These inlets should be marked on the plans prior to any computations regarding discharge, spread, inlet capacity, or bypass. Examples of such locations are listed as items 1 through 4 below.

1. Roadway Low Points or Sag Vertical Curves. In curbed areas where cross-sectional transition zones occur, it may be necessary to prepare curblines profiles in order to determine actual low point locations. See Section B below for information on flanking inlets.
2. Up-grade of intersections, and major driveways to intercept gutter flow before it reaches these areas.

The depth of gutter flow shall not overflow a driveway opening when the driveway's grade descends from the gutter. The typical driveway section can usually be designed so as to allow reasonable gutter depth, preventing the necessity of installing an excessive number of inlets to maintain shallow flow past the driveway opening.

3. Upgrade of pedestrian crosswalks, to intercept gutter flow before it reaches them. Zero by-pass need not be attained as it is unlikely that significant pedestrian

movement will occur during peak rainfall.

To comply with Americans with Disabilities Act (ADA) requirements for pedestrian surfaces, inlets shall not be placed within a Pedestrian Access Route (see note, below), including locations within curb ramp openings or between the transverse markings of a crosswalk, unless the inlet openings are ADA-compliant. Grates with openings that meet ADA surface requirements are available, but due to their smaller openings, may not be adequate for highway drainage flows. Grates that are not ADA-compliant may be located under the transverse markings of crosswalks, provided they do not intrude into the area between the transverse markings. If the presence of a non-ADA compliant grate within the pedestrian access route is unavoidable, and the grate cannot meet the ADA surface requirements, it must be justified as a nonstandard feature, as discussed in Chapter 2, Section 2.8.

NOTE: A Pedestrian Access Route (PAR) is defined in the Proposed Public Right-of-Way Accessibility Guidelines as “a continuous and unobstructed path of travel provided for pedestrians with disabilities within or coinciding with a pedestrian circulation path.” Refer to Chapter 18 of this manual for surface requirements for a PAR.

4. Up-grade of cross slope reversals. When cross slope transitions occur on very flat profiles or when they occur at crest and sag vertical curves, intermediate low points may exist. These low points are best handled through adjustment in transition length, but occasionally will require inlets. Inlets should be located to intercept the gutter flow before the transition, and the cross slope becomes too flat for efficient pick up. This is done to reduce the water flowing across the roadway to prevent icing in the winter and hydroplaning during warmer temperatures. The curb line profiles of all transitions should be carefully plotted when these conditions are anticipated.

B. Inlets in Sag Locations

When a sag vertical curve in a curbed section connects a negative grade with a positive grade, inlets should be placed with one at the low point and one at each side of this point. The inlets at each side are known as flankers. The purpose of the flanking inlets is to act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded. Flanking inlets should be located so that they will function before water spread exceeds the spread criteria provided in Section C below for inlets on grade.

C. Inlet Spacing on Continuous Grades

Spread is the criterion used for locating inlets between those required by geometric or other controls. Spread and puddle depth criteria are established to protect the traveling public. **Spread should not encroach beyond one half the width of the right most travel lane for all functional classifications of highways except for Interstates and other freeways with shoulders. Spread should be limited to the shoulder of Interstates and other freeways. Puddle depth should be 0.5 inches less than the curb height regardless of functional classification.**

The following should be considered regarding the spread criteria. Spread should be increased or decreased as deemed necessary:

1. Gutter flow can utilize the full parking lane or shoulder width because these typical section elements are outside the travel lane.
2. The half lane flooding criteria can be very restrictive when the typical section does not include parking lanes or shoulders. Highways without parking lanes or shoulders have inherent traffic constraints. Consideration should be given to the effects of half lane flooding relative to inlet spacing and drainage system cost.
3. Highways with a design speed greater than 40 mph will have a higher potential for hydroplaning if travel lanes are covered with water.

When the spread criteria is exceeded, it should be noted and explained in the Drainage Report.

Inlet spacing necessary to meet the spread and puddle depth criteria should not be misused. These are maximum flooding criteria and when used in an average parking lane or wide shoulder could result in considerable by-pass. A design which allows considerable by-pass upstream should not result in the need to place numerous inlets near the low point to intercept the remaining stormwater.

The interception capacity of the upstream inlet will define the initial spread. As flow is contributed to the gutter section in the downstream direction, spread increases. The next downstream inlet is located at the point where spread in the gutter reaches the design spread, or a value less than the design spread to control the amount of water flowing to the sag. Therefore, the spacing of inlets on a continuous grade is a function of the amount of upstream bypass flow, the tributary drainage area, and the gutter geometry.

A summary of the steps provided in the references for manually determining inlet spacing is presented below to illustrate the design process for spacing inlets on a continuous grade.

- Step 1. As discussed in Section 8.7.4.4.A, mark on a plan the location of inlets which are necessary without regard to the contributing drainage area.
- Step 2. Start at a high point on the roadway grade, at one end of the job if possible, and work towards the low point. Then begin at the next high point and work down-grade toward the same low point.
- Step 3. Select a trial drainage area approximately 300 ft to 500 ft long below the high point for the location of the first proposed inlet, outline the area on the plan, and describe the inlet's location (e.g., by number and station). Include any area that may drain over the curb and onto the roadway. As previously mentioned, the flow from these areas desirably should be intercepted before it reaches the roadway.

Compute the trial drainage area and calculate the flow in the gutter using the rational method and the rational equation. As previously discussed in Section 8.3.2.4.A, values for the following additional variables will have to be established in order to calculate the flow:

The runoff coefficient (C), or the weighted runoff coefficient (C_w) for the trial drainage area (A).

The T_c for inlet spacing. This is the time required for water to flow from the hydraulically most distant point of the unique drainage area contributing only to that inlet. Typically, this is the sum of the times required for water to travel overland to the pavement gutter and along the length of the gutter between inlets. When the T_c to the inlet is less than 5 min., a minimum T_c of 5 min. shall be used. In the case of a constant roadway grade and relatively uniform contributing drainage area, the T_c for each succeeding inlet may be assumed to be constant.

The rainfall intensity. This is determined from an IDF curve using the T_c and the design frequency.

- Step 4. Determine the spread (T), and depth of water at the curb (d), based on the gutter capacity relationship shown in Section 8.7.3.2.

As previously discussed, this relationship requires the following input variables:

Q , determined in step 3 above. For the first inlet in a series, the flow to the inlet is the same as the total gutter flow since there was no previous bypass flow (Q_b).

S , the gutter's longitudinal slope at the inlet, determined from the roadway profile.

S_x , the gutter's cross slope, determined from the cross section.

n , manning's roughness coefficient, determined from tables in the references based on the gutter's material.

Compare the calculated T and d with the criteria. If the calculated T and d is less than or close to the criteria, continue with step 5, otherwise, increase or decrease the length to the inlet and repeat steps 3 and 4 until appropriate values for T and d are obtained.

- Step 5. Select the inlet type (discussed in Section 8.7.4.2) and dimensions (e.g. width (W) and length (L), and calculate the flow intercepted by the grate (Q_i). Q_i was discussed in Section 8.7.4.3 and Section 8.7.4.3.A.
- Step 6. Calculate the bypass flow (Q_b). Q_b is the total gutter flow (step 3), within the constraints established by step 4, minus the flow intercepted by the grate (step 5).

- Step 7. Proceed to the next inlet down the grade. As in step 3, select a trial drainage area approximately 300 ft to 400 ft below the previous inlet, and calculate the flow. The flow between inlets is termed incremental flow.
- Step 8. Determine the total gutter flow by adding the incremental flow in the gutter from the area between the two inlets (step 7) to the bypass flow from the previous upstream inlet (step 6).
- Step 9. Determine T and d , and compare to the criteria (step 4). If T or d violate the criteria, adjust the drainage area selected in step 7 and recompute the incremental flow, and total flow (step 8) until T and d are within the design criteria.
- Step 10. Select the inlet type and dimensions and calculate Q_i (step 5). Calculate Q_b (step 6).
- Step 11. Repeat steps 7 through 10 for each subsequent inlet down to the low point. The low point should be designed as discussed in Section 8.7.4.4.B.

8.7.5 Storm Drains

Storm drains convey stormwater which is intercepted by the inlets, to an outfall where the stormwater is discharged to the receiving waters. The storm drainage system consists of differing lengths and sizes of storm drain connected by drainage structures. A section of storm drain which connects one drainage structure to another is often termed a segment or run. Sections of storm drain in series are often referred to as the trunk line.

8.7.5.1 Storm Drain Design Criteria

A. Hydraulic Criteria

Pipe size, the velocity of water, pipe slope, and the hydraulic gradeline are the hydraulic criteria which will influence storm drain design.

1. Size. Pipe shall be sized no larger than necessary to minimize material and trenching costs. The minimum round pipe size shall be 15 inches, except in unusual circumstances where the Regional Design Engineer or NYSDOT Resident Engineer may approve use of a 12 inch round pipe.
2. Velocity. 3 ft./s is the minimum velocity recommended to prevent sediment deposition. Significant drops in velocity in downstream storm drains should be avoided due to the likelihood of sediment deposition.
3. Slope. Where possible, pipe should be designed using grades which parallel the roadway grade to minimize excavation, sheeting, and backfill. The slope specified should satisfy the velocity criteria and will be controlled by the hydraulic analysis.
4. Hydraulic gradeline (HGL). The HGL is a line coinciding with the level of water

under open channel flow conditions. In a storm drain flowing under pressure, the HGL is the level to which water would rise in a vertical tube at any point along the pipe. The HGL should be calculated throughout the system and the water level should be kept a minimum of 8 in. below the top of grate elevation at each drainage structure.

B. Pipe Criteria

Storm drain pipe is available in a number of materials, shapes, and sizes. The materials recommended for use in a storm drainage system are reinforced concrete, steel, aluminum, polyethylene and polypropylene. Circular storm drains are preferred. The size and material specified should satisfy the hydraulic criteria in Section 8.7.5.1.A, in addition to the pipe criteria recommended in items 1 through 3.

1. Design Life. The design life shall be 70 years.
2. Anticipated Service Life. The anticipated service life for concrete, aluminum, polyethylene and polypropylene is 70 years. The anticipated service life for steel (with and without additional coating) in Zones I & II is as shown in Exhibit 8-23. For the purpose of determining anticipated service life, metallic coated (galvanized) pipe is not considered to have an additional coating. All other options are considered additional coatings.
3. Structural Criteria. Refer to Section 8.6.2.3.A for the maximum and minimum height of fill limitations for these materials.

Exhibit 8-23 Anticipated Service Life, in years, for Steel (with and without additional coating)¹

Gauge	Metallic Coated (Galvanized)	Metallic Coated (Aluminum Coated - Type 2) ² & Metallic Coated (Galvanized) w/ Paved Invert ³	Metallic Coated (Galvanized) w/ Fully Paved ⁴	Metallic Coated (Galvanized) w/ Polymer Coated ⁵	Metallic Coated (Galvanized) w/ Polymer Coated and Paved Invert ⁶
	§ 707-02	§ 707-02	§ 707-02	§ 707-02	§ 707-02
16					72
14			70		80
12		79	84	74	94
10	69	94	99		
8	84				

NOTES:

1. The anticipated service life for steel pipe used in a storm drainage system is based on the Zone I, 2 mil per year, metal loss rate. This rate applies in Zone II only when all drainage is received from paved surfaces exclusively. When storm drainage systems are located in Zone II and these systems receive open drainage or bed loads other than ordinary road debris, use the anticipated service life values for open systems (culverts) in Zone II - see Section 8.6.2.2, Exhibit 8-10. Anticipated service lives are not shown for options which do not meet the criteria.
2. Aluminum Coated - Type 2 pipe is expected to have the same anticipated service life as metallic coated (galvanized) pipe with a paved invert.
3. Additional coating adds 25 years to the anticipated service life of galvanized steel pipe.
4. Additional coating adds 30 years to the anticipated service life of galvanized steel pipe.
5. Additional coating adds 25 years to the anticipated service life of galvanized steel pipe.
6. Additional coating adds 40 years to the anticipated service life of galvanized steel pipe.

8.7.5.2 Placement and Utilities

Generally, trunk lines should be placed under the parking lanes, shoulders, or just inside the curb line to leave the sidewalk area available for existing, relocated, or future utilities. When a highway is being widened, twin trunk lines may be used to avoid repeated roadway crossings and disruption to traffic, with the resultant increase in maintenance of traffic costs. Generally, the trunk line should be placed below the lowest point on the typical section with branch lines connecting other low points of the typical section.

Deep cuts and conflicts with utilities shall be avoided where possible. Some instances (see preceding paragraph) may dictate a trunk line on both sides of the roadway with few or no laterals, while other instances may call for a single trunk line. Such features are usually a function of economy, but may be controlled by traffic disruption, utilities or other physical features. Drainage structures placed in proximity to telephone, signal or other overhead utilities may cause conflicts. Construction sequencing should be considered if a utility pole is to be relocated. If the pole is not going to be relocated, sheeting or some other method to maintain the pole in place during excavation for a drainage structure may be required.

8.7.5.3 Storm Drain Hydraulics

This section discusses storm drain capacity design, headlosses, and the hydraulic gradeline (HGL).

A. Storm Drain Capacity Design

The hydraulic design of storm drains is based on the assumption that flow is steady and uniform, and the average velocity is constant within each run of pipe. Storm drains may be sized to operate under the principles of open channel, or pressure flow. Each flow condition has its advantages and disadvantages. Storm drains should be designed to flow nearly full (i.e., as an open channel). This approach will allow flow in addition to the design flow. A storm drain operating under pressure flow is sensitive to increases in flow, and grate displacement may occur. After the preliminary location of inlets, connecting pipes, and the outlet(s) have been determined, the discharge to be carried by each run, and the pipe size and slope required to convey this discharge should be computed.

The references provide a multi-step procedure and design form for storm drain capacity design. This section discusses using the rational equation, and the storm drain capacity design. Prior to performing the hydrologic and storm drain capacity calculations, a working plan layout and profile of the storm drainage system should be prepared which establishes the location of storm drains, direction of flow, location and numbering of drainage structures and manholes, and the location of utilities (e.g., water, gas, or underground cables). After all storm drains are sized, headlosses, and the hydraulic grade line should be calculated as discussed in Sections 8.7.5.3.B and C.

1. Storm Drain Hydrologic Calculations. Starting with the most upstream storm drain run, determine the discharge from the total contributing drainage area tributary to the inlet located at the upstream end of the storm drain run being sized. Based on this discharge, determine the minimum size storm drain as discussed in item 2 of this section. This process should be continued, run by run, to the inlet upstream of the storm drainage system outlet.

The rational equation is expressed as follows for storm drain design.

$$Q = (\sum CA) i / 360$$

The product of the runoff coefficient (C), or the weighted runoff coefficient (C_w), and the corresponding drainage area tributary to each inlet (referred to as an incremental area) upstream of the storm drain run under design (referred to as the incremental CA) is summed to get a "total CA".

Using an Intensity-Duration (IDF) curve or data, the rainfall intensity (i) is based on the larger of two times of concentration (the inlet T_c , or the system T_c), or the minimum T_c of 5 min. These times of concentration are discussed below.

The inlet T_c is the value associated with the inlet spacing computations (i.e., the T_c associated with the drainage area or incremental drainage area contributing flow to the inlet). The system T_c is the time for water to travel from the most remote point in the storm drainage system to the upstream end of the storm drain under consideration. The system T_c will be the same as the inlet T_c for

the most upstream storm drain run. For all other runs, the system T_c is computed by adding the system T_c and the section T_c from the previous run together, to get the system T_c at the upstream end of the run under consideration. The section T_c is the travel time for water to flow through the storm drain.

2. Storm Drain Capacity Calculations. Storm drains are sized based on Manning's equation, by varying the slope and size as necessary to convey the discharge within the criteria stated in Section 8.7.5.1. The references contain charts, and software is available to determine pipe size, slope, and flow velocity. For storm drains downstream of the first run, an approximate crown drop for the downstream pipe may be calculated at this time to off-set potential energy losses. This is discussed in Section 8.7.5.3.B, item 3a.

The pipe invert at the upper and lower ends of the section of pipe may also be established and recorded at this time, and are computed based on the pipe's length, slope, and minimum depth requirements (e.g., minimum height of fill, or utility conflicts).

B. Headloss or Energy Loss

Headloss (a loss of energy) results from frictional and other disturbances which water encounters as it flows throughout the storm drainage system. The flow regime (i.e., supercritical or subcritical) within each storm drain run is used to determine those headlosses which are included in the HGL calculation. Prior to computing the HGL, energy losses in each storm drain run, and the drainage structures must be estimated. (The terms drainage structure, manhole, and access hole should be considered synonymous in this section.)

Headloss due to pipe friction (often termed major losses) represents most of the losses in the system and is caused by the interior roughness of the pipe. If flow in the outflow storm drain is supercritical, the friction loss is not included in the headloss calculation.

Headloss due to the other disturbances are termed velocity head losses, or minor losses, and are caused by water's turbulence as it enters and leaves the storm drainage system and the various drainage structures. Access hole losses do not apply when the flow in two successive pipes is supercritical (i.e., flow is supercritical in the inflow pipe and the outflow pipe).

Other minor losses (bend losses, transition losses, and junction losses) are discussed in the references but are not considered in this chapter. Bend losses are not considered because each storm drain should be a straight run of pipe from one drainage structure to another. Changes in horizontal alignment should be made at a drainage structure. Transition losses occur when the storm drain changes size within a pipe run. No standard pay items exist for transition sections and they should be avoided. A drainage structure should be used when a change in pipe size is necessary. A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of a drainage structure. There are no standard pay items for junctions and pipe connections should be made at a drainage structure.

Items 1 through 3 discuss exit loss, pipe friction loss, and inlet and access hole losses and their calculation.

1. Exit loss, H_o . The exit loss is a function of the change in velocity at the outlet of the last downstream storm drain. For a sudden expansion, such as an endwall, the exit loss is:

$$H_o = 1.0 \left[\frac{V_o^2}{2g} - \frac{V_d^2}{2g} \right], \text{ where :}$$

V_o = average outlet velocity, ft./s

V_d = channel velocity downstream of outlet, ft./s

g = 32.2 ft./s²

Note that when $V_d = 0$, as in a reservoir, the exit loss is one velocity head. For part-full flow where the pipe outlets in a channel with moving water, the exit loss may be reduced to virtually zero.

2. Friction slope (S_f) and pipe friction loss (H_f). The friction slope is the hydraulic gradient for that run or length of pipe (L). The hydraulic gradient is the slope of the hydraulic gradeline.

H_f , in meters, is computed as follows:

$$H_f = S_f L$$

See section 7.1.6 of HEC-22, Urban Drainage Design Manual, for calculation of friction slope values.

3. Inlet and Access Hole Losses. HEC 22 discusses inlet and access hole losses.
 - a. Preliminary Estimate. HEC 22 presents an approximate method for computing minor losses at drainage structures (H_{ah}). The method involves multiplying the velocity head of the outflow pipe ($V_o^2/2g$) by a headloss coefficient (K_{ah}). Values of K_{ah} are provided in Exhibit 8-24.

$$H_{ah} = (K_{ah})(V_o^2/2g)$$

Exhibit 8-24 Headloss Coefficients

Structure Configuration	K_{ah}
Inlet - straight run	0.5
Inlet - angled through	
90°	1.50
60°	1.25
45°	1.10
22.5°	0.70
Manhole - straight run	0.15

Manhole - angled through	
90°	1.00
60°	0.85
45°	0.75
22.5°	0.45

This method can be used to estimate the initial pipe crown drop across a drainage structure to offset energy losses at the structure. The crown drop is then used to establish the appropriate pipe invert elevations. This method should only be used for preliminary estimation and not for the actual design process.

Similarly, a common practice is to match crowns or place the outflow pipe invert 1 in. lower than the inflow pipe invert.

- b. Energy Loss Methodology. The head loss encountered in going from one pipe to another through a drainage structure is commonly represented as being proportional to the velocity head at the outlet pipe. Using K to signify this constant of proportionality, the energy loss is approximated as $K (V_o^2/2g)$. Experimental studies have determined that when the inflow pipe invert is below the water level in the access hole, the K value can be approximated as:

$$K = K_o C_D C_d C_Q C_P C_B, \text{ where:}$$

K = adjusted loss coefficient

K_o = initial head loss coefficient based on relative manhole size

C_D = correction factor for pipe diameter (pressure flow only)

C_d = correction factor for flow depth (non-pressure flow only)

C_Q = correction factor for relative flow

C_p = correction factor for plunging flow

C_B = correction factor for benching

These coefficients are discussed in items 1 through 6 below.

For cases where the inflow pipe invert is above the access hole water level, the outflow pipe will function as a culvert, and the access hole loss and the access hole HGL can be computed using procedures found in HDS 5. If the outflow pipe is flowing full or nearly full under outlet control, the access hole loss (due to flow contraction into the outflow pipe) can be computed by setting K , in the equation provided in item 3a, above, equal to k_e , provided in Table 12 of HDS 5. If the outflow pipe is flowing under inlet control, the water depth in the access hole should be computed using the inlet control nomographs in HDS 5.

1. Relative Manhole Size. K_o is estimated as a function of the relative manhole size and the angle of deflection between the inflow and outflow pipes.

$$K_o = 0.1(b/D_o)(1-\sin\theta) + 1.4 (b/D_o)^{0.15} \sin \theta, \text{ where:}$$

θ = the angle between the inflow and outflow pipes, degrees

b = manhole diameter, ft.

D_o = outlet pipe diameter, ft.

2. Pipe Diameter. C_D is only significant in pressure flow situations when the ratio of the depth of water in the access hole above the outflow pipe invert (d) to the outflow pipe diameter, d/D_o , is greater than 3.2. d is approximated as the level of the hydraulic gradeline at the upstream end of the outflow pipe. C_D should be set equal to one for open channel flow design.

$$C_D = (D_o/D_i)^3, \text{ where:}$$

D_o = the outflow pipe diameter, ft.

D_i = the inflow pipe diameter, ft.

3. Flow Depth. C_d is significant only in cases of free surface flow or low pressures, when d/D_o ratio is less than 3.2. In cases where this ratio is greater than 3.2, C_d is set equal to one.

$$C_d = 0.5(d/D_o)^{0.6}, \text{ where:}$$

d = water depth in access hole above the outlet pipe invert, ft

D_o = the outflow pipe diameter, ft

4. Relative Flow. C_Q , is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes, and is only applied to situations where there are 3 or more pipes entering the structure at approximately the same elevation. Otherwise, $C_Q = 1.0$.

$$C_Q = (1 - 2 \sin \theta) \left(1 - \frac{Q_i}{Q_o} \right)^{0.75} + 1, \text{ where :}$$

θ = the angle between the inflow and outflow pipes

Q_i = flow in the inflow pipe, ft.³/s

Q_o = flow in the outflow pipe, ft.³/s

5. Plunging Flow. C_p corresponds to the effect of another inflow pipe, plunging into the drainage structure, on the inflow pipe for which the head loss is being calculated. The correction factor is only applied when $h > d$. Additionally, the correction factor is only applied when a higher elevation flow plunges into an access hole that has both an inflow pipe and an outflow pipe at the bottom of the access hole. Otherwise $C_p = 1.0$.

$$C_p = 1 + 0.2 \left[\frac{h}{D_o} \right] \left[\frac{h-d}{D_o} \right], \text{ where:}$$

h = vertical distance of plunging flow from flow line of the higher elevation inlet pipe to the center of outflow pipe, ft.

d = water depth in access hole relative to the outlet pipe invert, ft.
 D_o = the outflow pipe diameter, ft.

6. Benching. C_B is obtained from Exhibit 8-25. Benching tends to direct flows through the manhole, resulting in a reduction in headloss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Exhibit 8-25 Correction Factors for Bench Types

Bench Type	Submerged ¹	Unsubmerged ²
Flat or depressed floor	1.0	1.0
Benched one-half of pipe diameter	0.95	0.15

1. pressure flow, $d/D_o > 3.2$
2. free surface flow, $d/D_o < 1.0$

Note: The Section 604 standard sheets do not refer to benching types as do the drainage publications. A flat floor should be considered similar to a drainage structure without a sump, a depressed floor should be considered similar to a sump, and a benched one-half of pipe diameter condition should be considered similar to a formed invert.

C. Hydraulic Gradeline

The hydraulic gradeline (HGL) is a line coinciding with the level of flowing water at any point along an open channel, and is the last characteristic to be established relating to the hydraulic design of storm drains. When water is flowing through the pipe and there is a space of air between the top of the water and the inside of the pipe (the crown), the flow is considered as open channel flow and the HGL is at the water surface.

The HGL should be below the crown of the pipe in order to maintain open channel flow conditions. An exception to this will be when the tailwater submerges the outfall outlet. As previously mentioned in Section 8.7.5.3.A, a concern with storm drains designed to operate under pressure flow conditions is that inlet surcharging and possible manhole lid displacement can occur if the HGL rises above the ground surface. As the HGL calculations are performed, frequent verification of the existence of the desired flow condition should be made. Storm drain systems can often alternate between pressure and open channel flow conditions from one run to another.

Most storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and drainage structure losses are summed to determine the upstream HGL levels. As previously stated in Section 8.7.5.3.B, if supercritical flow occurs, pipe and drainage structure losses are not carried upstream. When a storm drain run is identified as being supercritical, the computations are advanced to the next upstream pipe run to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime.

The references provide a multi-step procedure and design form(s) to aid in the manual

calculation of the hydraulic grade line (HGL). Software will calculate the HGL for the user based on the hydraulic characteristics of the storm drainage system (e.g., tailwater elevation, flow regime in the storm drains, friction loss, access hole losses, etc.).

HGL calculations begin at the outfall with the tailwater elevation, and are worked upstream taking each storm drain run and drainage structure into consideration. If the proposed system connects to an existing downstream system, the HGL calculation must begin at the outlet end of the existing system, and proceed upstream through the existing system, then upstream through the proposed system to the upstream inlet. The flowline elevation of the proposed outlet should be equal to or higher than the flowline of the receiving waters. If this is not the case, there may be a need to provide flap gates, pump, or otherwise lift the water.

The tailwater elevation will be the greater of the water surface elevation in the receiving waters, or $(d_c + D)/2$ + the outlet invert elevation (where d_c is the critical depth, and D is the pipe diameter). These possibilities are discussed further in 1 through 3 below.

1. If the pipe outlet is submerged, the HGL at the pipe outlet is equal to the tailwater elevation. Data regarding the water surface elevation in natural channels (which serve as the receiving waters) may be determined from flood insurance studies performed for the area.
2. If the pipe outlet is not submerged, and the design is performed manually, the HGL at the pipe outlet is computed as follows:
 - a. If the tailwater at the pipe outlet is greater than $(d_c + D)/2$, use the tailwater elevation as the beginning HGL elevation.
 - b. If the tailwater at the pipe outlet is less than $(d_c + D)/2$, use $(d_c + D)/2$ plus the outlet invert elevation as the beginning HGL elevation.
3. If the pipe outlet is not submerged, and the design is performed using software, the tailwater elevation will be calculated based on the depth of flow in the pipe and the outlet invert elevation.

Once the tailwater elevation is established, the friction loss in the pipe is calculated (major loss), and added to the minor losses. These major and minor losses are added to the tailwater elevation to arrive at the HGL which is experienced just inside the inflow pipe, and is compared with the depth of flow and the crown of the inflow pipe. If the inflow pipe is submerged by the depth of water in the drainage structure, open channel flow conditions will not exist in the inflow pipe, and this depth will be used to continue calculation of the HGL upstream. If the inflow pipe is not submerged, the depth of flow in the inflow pipe is used to continue calculation of the HGL upstream. Note, at this point, a new "tailwater elevation" has been calculated for the inflow pipe (which is the outflow pipe for the upstream drainage structure). This process is repeated for each pipe in the system.

8.7.5.4 Outfall

The outfall is the point at which stormwater discharges from the storm drainage system. If used, a ditch transfers stormwater from the storm drain outlet, to the point of release, termed the receiving waters. Otherwise, the storm drain outlet discharges directly to the receiving

waters. The receiving waters should be either a natural channel or a man-made facility (e.g., a stream or a storage facility).

Energy dissipation (e.g., a stone apron or stone lined channel) should be considered to protect the storm drain outlet from scour, to lower the velocity of water before discharge into a natural channel, thereby minimizing the turbidity in the receiving waters, and to minimize erosion of the stream bed and banks.

8.7.6 Drainage Structures

Drainage structures provide support for grate inlets, and combination inlets, and serve as a point of connection for the storm drains. Sections 8.7.6.1 and 8.7.6.2 provide general guidance regarding the type and size of drainage structure to be specified.

8.7.6.1 Types of Drainage Structures

The pay items in Section 604 of the standard specifications provide a number of alternatives for drainage structures including:

1. Rectangular drainage structure. This is the preferred pay item. Rectangular drainage structures are detailed on the Section 604 Standard Sheets.
2. Rectangular drainage structure with round option. This pay item should only be selected if shape is unimportant in the design and construction of the drainage system. Placement of round structures in lieu of rectangular structures may shift the centerline of the drainage structure as well as the centerline of pipe entries. This may cause construction problems or adversely affect adjacent features.
3. Round precast manhole. This pay item should be used to provide a structure for connecting pipelines on differing alignments where a drainage structure with a grate inlet is not necessary.
4. Special drainage structure. Regions which have established special drainage structure details shall include the details in the contract documents. When a top slab is used, the top slab should be as small as possible and yet provide adequate structural stability. Concrete block and mortar structures are not acceptable due to the damage they incur from salt and highway loads.

The following shall be specified in the Drainage Structure Table as necessary:

1. Precast drainage structures. These are preferred to cast in place by the construction industry and are usually more cost effective. Precast structures may not be economical in some Regions due to shipping costs. Contractors have the option of providing precast or cast in place drainage structures unless specified otherwise. Any requirements regarding precast versus cast in place drainage structures should be noted in the drainage structure table.
2. Scoops, formed inverts, and sumps (refer to Standard Sheet 604-02). A scoop, formed invert, or sump may be specified for the drainage structure. Each option has a specific

function. A scoop should be specified for the upstream drainage structure beginning a trunk line or branch line. Formed inverts should be specified to reduce head losses and facilitate self-cleaning when sumps are not specified. Sumps (termed catch basins in the references) require periodic cleaning to be effective, and may become an odor and mosquito nuisance if not properly maintained. In areas where site constraints dictate that storm drains be placed on relatively flat slopes, and where maintenance is provided, sumps can be specified to collect sediment and debris. The Regional Maintenance Group should be consulted before specifying sumps.

8.7.6.2 Size of Drainage Structures

Drainage structures should be large enough to accommodate the inlet frame and grate, maintenance personnel, and connecting pipes. To avoid excessive head losses, structures should be no larger than necessary.

Exhibit 8-27 gives the inside dimensions of rectangular drainage structures (types A through U). Rectangular drainage structure types A through P utilize top slabs and accommodate a wide range of pipe sizes. Types Q through U do not utilize top slabs, are more economical, and are limited to the smaller pipe sizes.

Exhibits 8-28 through 8-31 give the minimum internal rectangular drainage structure wall dimensions necessary for the various pipe sizes and pipe entry skew angles (θ). Exhibits 8-32 and 8-33 give the maximum size round pipe that can be accommodated by drainage structure types Q through U, also based on pipe size and θ . Exhibits 8-28 through 8-33 are based on the following relationships between θ , wall thickness of drainage structure ($a = 8$ in used), outside pipe diameter (d), and the knockout clearance (3 in max.).

1. $b = a (\tan\theta)$
2. $c = d/\cos\theta$
3. Knockout opening = $b + c + 6$ in.
4. knockout opening \leq internal wall dimension

Exhibit 8-26 illustrates these terms

Exhibit 8-26 Drainage Structure Pipe Entrance

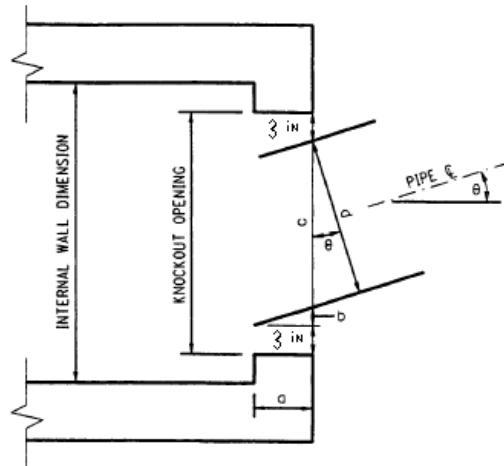


Exhibit 8-27 Inside Dimensions of Drainage Structures (Types A Through U) ¹⁻⁴

Structure Type	Inside Dimensions (In.)		Structure Type	Inside Dimensions (In.)	
	WIDTH	LENGTH		WIDTH	LENGTH
A	36	36	M	36	80
B	48	36	N	48	80
C	60	36	O	60	80
D	80	36	P	80	80
E	36	48	Q	34 (33)	36 (36.5)
F	48	48	R	34 (33)	46 (46.5)
G	60	48	S	24 (24)	32 (32.5)
H	80	48	T	26 (27)	42 (42.5)
I	36	60	U	34 (34)	38 (38)
J	48	60			
K	60	60			
L	80	60			

NOTES:

1. Dimensions are taken from the Section 604 Standard Sheets.
2. Surface Flow is always parallel to the length of a structure.
3. Structures larger than 60 in. x 60 in. shall be designed on a case by case basis.
4. Two sets of dimensions are shown for structure types Q through U. Dimensions outside the parenthesis are for precast structures, dimensions inside parenthesis are for cast in place structures.

Exhibit 8-28 Necessary Internal Wall Dimensions for Type A Thru P Drainage Structures Based on Skew Angle and Nominal Pipe Diameter (Concrete and Smooth Interior Corrugated Polyethylene ¹)

Skew Angle	Nominal Inside Pipe Diameter										
	12	15	18	21	24	30	36	42	48	54	60
0	36	36	36	36	36	48	60	60	80	80	80
7.5	36	36	36	36	48	48	60	60	80	80	80
15	36	36	36	36	48	48	60	80	80	80	Special Design
22.5	36	36	36	48	48	60	60	80	80	80	
30	36	36	48	48	48	60	80	80	80		
37.5	36	48	48	48	60	60	80	80			
45	48	48	60	60	60	80	80				

NOTE:

1. Dimensions are in inches.

Exhibit 8-29 Necessary Internal Wall Dimensions for Type A Thru P Drainage Structures Based on Skew Angle and Nominal Pipe Diameter (Metal) ¹

Skew Angle	Nominal Inside Pipe Diameter										
	12	15	18	21	24	30	36	42	48	54	60
0	36	36	36	36	36	48	48	60	60	80	80
7.5	36	36	36	36	36	48	48	60	60	80	80
15	36	36	36	36	36	48	48	60	80	80	80
22.5	36	36	36	36	48	48	60	60	80	80	80
30	36	36	36	48	48	48	60	80	80	80	Special Design
37.5	36	36	48	48	48	60	80	80	80		
45	36	48	48	60	60	80	80	80			

NOTE:

1. Dimensions are in inches.

Exhibit 8-30 Necessary Internal Wall Dimensions for Type A Thru P Drainage Structures Based on Skew Angle and Horizontal Elliptical Concrete Pipe Dimensions ¹

Skew Angle	Pipe Rise x Span						
	19x30	22x34	24x38	27x42	29x45	32x49	34x53
0	48	48	60	60	60	80	80
7.5	48	48	60	60	80	80	80
15	48	60	60	60	80	80	80
22.5	60	60	60	60	80	80	80
30	60	60	80	80	80	80	
37.5	60	80	80	80	Special Design		
45	80	80	Special Design				

NOTE:

1. Dimensions are in inches.

Exhibit 8-31 Necessary Internal Wall Dimensions for Type A Thru P Drainage Structures Based on Skew Angle and Metal Pipe Arch Dimensions ¹

Skew Angle	Pipe Span x Rise								
	17x13	21x15	24x18	28x20	35x24	42x29	49x33	57x38	60x46
0	36	36	36	36	48	60	60	80	80
7.5	36	36	36	48	48	60	60	80	80
15	36	36	36	48	48	60	80	80	80
22.5	36	36	48	48	60	60	80	80	80
30	36	48	48	48	60	80	80	80	
37.5	48	48	48	60	60	80	80	Special Design	
45	48	60	60	60	80	80	Special Design		

NOTE:

1. Dimensions are in inches.

Exhibit 8-32 Maximum Size Round Pipe (Concrete and Smooth Interior Corrugated Polyethylene) and Skew Angle for Type Q Thru U Drainage Structures^{1,2}

Skew Angle	Q		R		S		T		U	
	Width	Length	Width	Length	Width	Length	Width	Length	Width	Length
	34 (33)	36 (36½)	34 (33)	46 (46½)	24 (24)	32 (32½)	26 (26½)	42 (42½)	34 (34)	38 (37½)
0	21	24	21	30	12	21	15	30	21	24
7.5	18	24	18	30	12	18	12	24	21	24
15	18	21	18	30		18	12	24	18	21
22.5	15	18	15	24		15		24	15	18
30	12	15	12	24		12		21	15	15
37.5		12		21				21	15	15
45				15				12		

NOTES:

1. Dimensions are in inches.
2. Dimensions outside the parenthesis are for precast structures, dimensions inside parenthesis are for cast in place structures.

Exhibit 8-33 Maximum Size Round Pipe (Metal) and Skew Angle for Type Q Thru U Drainage Structures^{1,2}

Skew Angle	Q		R		S		T		U	
	Width	Length	Width	Length	Width	Length	Width	Length	Width	Length
	34 (33)	36 (36½)	34 (33)	46 (46½)	24 (24)	32 (32½)	26 (26½)	42 (42½)	34 (34)	38 (37½)
0	24	24	24	36	15	24	18	30	24	24
7.5	24	24	24	36	15	21	15	30	24	24
15	21	24	21	30	12	21	15	30	21	24
22.5	18	21	18	30		18	12	24	18	21
30	15	18	15	24		15		24	15	18
37.5	12	15	12	24		12		18	12	15
45		12		18				15		12

NOTES:

1. Dimensions are in inches.
2. Dimensions outside the parenthesis are for precast structures, dimensions inside parenthesis are for cast in place structures.

8.7.7 Storage Facilities

Stormwater storage facilities are often referred to as either detention or retention facilities.

For the purposes of this section, detention facilities are those that are designed to reduce the peak discharge and only detain runoff for a short period of time. These facilities are designed to completely drain after the design storm has passed. Recharge basins are a special type of detention basin designed to drain into the groundwater table. Two types of recharge may be proposed - surface or subsurface. Surface recharge basins should be designed in accordance with "Design Construction and Maintenance of Recharge Basins", Geotechnical Design Procedure GDP-8. Subsurface recharge can be accomplished through the use of leaching basins. "Design of Leaching Basins", Research Report 157, should be used as a source of information regarding leaching basin design.

Retention facilities provide for storage of stormwater runoff, and release via evaporation and infiltration only. Retention facilities which provide for slow release of stormwater over an extended period of several days or more are referred to as extended detention facilities. In addition to extended detention facilities, retention facilities include infiltration basins, and swales.

The need for and type of storage facility selected should be based on the following:

1. Site conditions. If a natural watercourse or body of water is not within reasonable proximity of the outfall, a recharge basin, leaching basin, or retention type storage facility should be provided to receive the collected stormwater. The specific type of storage facility selected should be based on right of way constraints, soil permeability, and Regional experience.
2. The legal aspects of highway drainage. The collection and diversion of surface waters may result in discharges which create a flooding problem which did not exist prior to the project. A storage facility should be provided to mitigate the increased discharge resultant from the storm drainage system.
3. Environmental considerations. SPDES/NPDES permitting requirements may warrant a retention type storage facility for peak flow attenuation, and first flush considerations. The decision to provide a storage facility and the type of retention facility proposed should be consistent with the guidance in Section 8.8.

8.7.8 Shared Costs

Municipally collected storm water may be carried in the State highway drainage system if the municipality shares in the added cost of the state system resultant from municipal participation.

The method used to calculate the Municipality's and State's share of the drainage work is based on funding (State versus Federal). The method outlined in Section 8.7.8.1 shall be used for 100% State funded drainage work. Federally funded work shall be calculated using one of the two methods outlined in Section 8.7.8.2. (Note: The methodologies presented in these sections applies to cost only. Refer to Chapter 21, Section 21.5, for guidance regarding establishment of quantities for entry into TrnsPort Estimator and the establishment of Engineering "shares".)

Refer to Chapter 14 of this manual for guidance on, and an example of, the necessary resolution to be obtained by the Region from the municipality.

8.7.8.1 Additional Cost Method - State Funding

The municipality's share of the estimated cost of the drainage system shall be based on the additional cost of pipe, excavation, drainage structures, and any other pertinent items which contribute to the added cost of the drainage system to accommodate the municipality.

8.7.8.2 Contributory Flow or Separate Flow - Federal Funding

The Municipality's share in the cost of the drainage system should be calculated using either of the two methods presented in Sections A and B.

A. Contributory Flow

The formulas contained in Exhibit 8-34 shall be used to calculate the municipality's share, and the State's share, of the drainage system when using this method:

Exhibit 8-34 Contributory Flow Formulas

Municipality's Share ¹	State's Share ¹
$\frac{Q_1}{Q_1 + Q_2} \cdot TotalCost$	$\frac{Q_2}{Q_1 + Q_2} \cdot TotalCost$

1. Q₁ = Municipality Flow, Q₂ = State Highway Flow

B. Separate Flow

The formulas contained Exhibit 8-35 shall be used to calculate the municipality's share, and the State's share, of the drainage system when using this method:

Exhibit 8-35 Separate Flow Formulas

Municipality's Share ¹	State's Share ¹
$\frac{X}{X + Y} \cdot Z$	$\frac{Y}{X + Y} \cdot Z$

1. X = Estimated Cost of Separate Municipal System
 Y = Estimated Cost of Separate State System,
 Z = Total Estimated Cost of Combined System

8.7.9 Maintenance

Existing storm drainage systems should be reviewed to determine the need to clean the drainage structures and storm drains. Section 203 of the Standard Specifications contains a pay item for cleaning storm drains, and a separate pay item for cleaning drainage structures and manholes.

8.8 EROSION AND SEDIMENT CONTROL AND STORMWATER MANAGEMENT

Various state and federal laws and regulations, including the State Environmental Quality Review Act (SEQRA), the National Environmental Protection Act (NEPA), and the Clean Water Act (CWA) require that erosion and sediment control be addressed as part of project development. The Federal Highway Administration (FHWA) requires that all federally-aided projects involving clearing and grubbing, grading, or excavation have formal erosion and sediment control plans (E&SC) included in the Plans, Specifications, and Estimate (PS&E). Additionally, erosion and sediment control is a concern to regulatory and resource agencies in the protection of threatened and endangered species; wild, scenic and recreational rivers; fish and wildlife resources; the control of non-point source pollution; the protection of coastal and inland waterways resources, and the protection of surface drinking water supply sources. It is a violation of State Water Quality Standards to cause turbidity that will result in a substantial visible contrast to natural conditions.

On December 8, 1999, the United States Environmental Protection Agency (EPA) issued a Final Rule dealing with stormwater discharges to surface waters of the United States, including federal jurisdictional wetlands. Coverage under a non-point source State Pollutant Discharge Elimination System (SPDES) Construction General Permit must be sought from the New York State Department of Environmental Conservation (NYSDEC) if any project involves a cumulative disturbance of 1.0 acre or more of non-Tribal Indian land during the life of the contract by clearing and grubbing, grading, filling or excavation. If a project will disturb 1.0 acre or more on Tribal Indian lands, coverage under a National Pollutant Discharge Elimination System (NPDES) Construction General Permit must be sought from EPA.

8.8.1 Determining the Need for an Erosion and Sediment Control Plan and SPDES/NPDES Stormwater Permits

The need for an erosion and sediment control (E&SC) plan, and coverage under SPDES/NPDES general permits, should be determined by following the guidance provided in steps 1 and 2 below. These steps are consistent with EPM Chapter 4.3.

- Step 1. Determine if the project involves clearing and grubbing, grading, excavation or fill. If the project does not involve clearing and grubbing, grading, excavation or fill, an E&SC plan is not necessary. If the project does involve clearing and grubbing, grading, excavation or fill, an E&SC plan shall be prepared consistent with the guidance in Section 8.8.2. Go to Step 2 to determine the need for coverage under SPDES/NPDES general permits.

- Step 2. Determine if the project is subject to and requires coverage under the SPDES General Permit for Stormwater Discharges from Construction Activity, from the NYSDEC, or a NPDES Construction General Permit from the EPA, or both.
 - a. A project is subject to and requires coverage under a NYSDEC SPDES General Permit if:
 1. The project involves a cumulative disturbance of 1.0 acre or more of non-Tribal Indian Land or 5,000 square feet or more in the New York City East of Hudson Watershed during the life of the contract by clearing and grubbing, grading, excavation or fill (excluding the area involved in routine maintenance activities,

as per Appendix B, Section 2.3.1), and

2. The project could involve a stormwater discharge (for example, runoff from the 100-year, 24-hour storm event) to surface waters of the United States (see definition on next page), either indirectly through stormwater sewers or directly to waterways, and
3. The stormwater discharge is from sources other than a storm sewer covered by an existing SPDES (point source) permit.

If the project requires coverage under the NYSDEC SPDES General Permit, refer to Section 8.8.3 and Appendix B.

A project is not eligible for coverage under the NYSDEC SPDES General Permit if there is a disturbance of two or more acres of land with no existing impervious cover and the Soil Slope Phase is identified as an E or F in the USDA County Soil Survey, and if these disturbances are within watersheds of AA or AA-s waterbodies. A construction project is ineligible for General Permit coverage if it is in an archaeologically sensitive area or there is a potential to affect an historic property, unless there is documentation that such impacts have been resolved.

- b. A project is subject to and requires coverage under an EPA NPDES Construction General Permit if the project:
 1. Will result in a disturbance of 1.0 acre or more total land area on Tribal Indian lands, and
 2. Involves a stormwater discharge to surface waters of the United States, either indirectly through stormwater sewers or directly to waterways.

If the project requires coverage under a NPDES Construction General Permit, refer to section 8.8.4.

Definition - "Waters of the United States" means:

1. All waters which are currently used, were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide;
2. All interstate waters, including interstate "wetlands";
3. All other waters such as interstate lakes, rivers, streams (including intermittent streams), mudflats, sandflats, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds, the use, degradation, or destruction of which would affect or could affect interstate or foreign commerce including such waters:
 - a. which are or could be used by interstate or foreign travelers for recreational or other purposes;
 - b. from which fish or shellfish are or could be taken and sold in interstate or foreign

commerce; or

- c. which are used or could be used for industrial purposes by industries in interstate commerce;
4. All impoundments of waters otherwise defined as waters of the United States under this definition;
5. Tributaries of waters identified in items 1 through 4 of this definition;
6. The territorial sea;
7. Wetlands adjacent to waters (other than waters that are themselves wetlands) identified in items 1 through 6 of this definition.

Waste treatment systems, including treatment ponds or lagoons designed to meet the requirements of the Clean Water Act (CWA) are not waters of the United States. This exclusion applies only to man-made bodies of water which neither were originally created in waters of the United States (such as disposal areas in wetlands) nor resulted from the impoundment of waters of the United States.

8.8.2 Erosion and Sediment Control

Erosion and sediment control is any temporary or permanent measure taken to reduce erosion, control siltation and sedimentation, and ensure that sediment-laden water or wind does not leave the site. Guidelines for providing erosion and sediment control are discussed in Section 8.8.2.1. The E&SC plan resulting from these guidelines is discussed in Section 8.8.2.2. Note: The guidance in Sections 8.8.2.1 and 8.8.2.2 is based on Attachment 4.3.A of EPM Chapter 4.3

8.8.2.1 Erosion and Sediment Control Guidelines

There are a number of publications and sources of guidance regarding erosion and sediment control and the preparation of an E&SC plan. This section discusses erosion and sediment control guidelines within the context of the *NYS Standards and Specifications for Erosion and Sediment Control*. Section 8.8.2.2 discusses the preparation of the E&SC plan.

The following is recommended guidance:

1. Preserve the existing vegetative groundcover on the project site as much as possible to protect the soil surface and limit erosion.
2. Sediment control practices/measures, where necessary, should be designed to protect the natural character of rivers, streams, lakes, coastal waters, wetlands, or other waterbodies on-site and minimize erosion and sedimentation off-site from the start of land disturbance activities to establishment of permanent stabilization.
 - a. The off-site impacts of erosion and sedimentation related to land clearing, grading, and construction activities should not be any greater during and following land disturbance activities than under pre-development conditions.

- b. Pursuant to Part 700 et seq. of Title 6, Chapter X of NYCRR:
3. Toxic and other deleterious substances shall not be discharged in amounts that will adversely affect the taste, color or odor thereof, or impair the waters of the state for their best (classified) usages,
 4. Suspended, colloidal, and settleable solids shall not be discharged in amounts that causes substantial visible contrast to natural conditions, or causes deposition or impairs the waters for their best (classified) usages.

Stream reaches on-site and downstream of construction areas should not have substantial visible contrast relative to color, taste, odor, turbidity, or sediment deposition from the reaches upstream of the construction area. Impacts such as these which result from construction or developmental activities are a violation of Part 700 Water Quality Standards and may be subject to enforcement actions.

Erosion and sediment control measures shall be constructed in accordance with an E&SC plan, established during design phases I through VI, which is included in the contract documents. **The plan shall not be left for construction personnel to develop.** Refer to Section 8.8.2.2 for specific elements of the E&SC plan.

8.8.2.2 Erosion and Sediment Control Plan

The complexity of the E&SC plan and the types, locations, and quantities of various erosion and sediment control measures will be dependent upon the:

1. Type of project (new construction; reconstruction; resurfacing, restoration, and rehabilitation (3R), etc.),
2. Scope of the project (for example, 3R projects with resurfacing and minimal disturbance, versus a 3R project which disturbs a significant amount of soil during the construction of new or lengthened turning lanes), and
3. Natural and man-made resources requiring protection.

The E&SC plan developed by the design team is intended to supplement, not replace, Section 209 of the Standard Specifications. An appropriately developed and detailed plan will help the Contractor understand the Department's expectations concerning the work required under Section 209 and should assist the Engineer-In-Charge (EIC) in assuring that erosion and sediment control is adequately provided.

Consistent with Sections 209 and 107-12 of the Standard Specifications, EICs will still need to obtain the Contractor's schedule of operations and plans for temporary and permanent erosion and sediment control measures not otherwise addressed in the Department's E&SC plan.

Note: The Contractor has to identify erosion and sediment control measures for their staging areas, borrow areas and work operations that the designer could not have foreseen in design. Contract provisions for these erosion and sediment control measures are included in Section 100 of the Standard Specifications. Payment should not be provided under Section 209.

The design team should include an E&SC plan in the contract documents which identifies and

shows the following, as appropriate:

1. The location of sensitive on-site and adjacent off-site natural and man-made resources requiring protection, including but not limited to, surface waterbodies, wetlands, special wildlife habitat areas, existing trees and other vegetation to remain, adjacent residential and commercial properties and public recreation facilities.
2. When necessary, protection should include delineation with a fencing type barrier and signing to keep construction equipment from entering these zones, and/or measures such as silt fence, to protect the resources from sedimentation. Note: No standard specification exists for fencing type barrier. Item No. 607.41010010 is an acceptable general approval, special specification which covers this work. Provisions for silt fence are contained in Section 209 of the Standard Specifications.
3. The location of temporary and permanent structural and vegetative measures to be used for soil stabilization, runoff control, and sediment control for each stage of the project from land clearing and grubbing to project close-out (on large scale plans, drainage plans, or special plans). Provide dimensional details, size, and length for these measures, as necessary, on the miscellaneous details plan sheets or the special details plan sheets.
4. Engineering and environmental judgement should be used to assure that only those erosion and sediment control practices that are necessary and effective are included in the contract documents. To that end, feedback is necessary and important between designers, EICs, Regional Landscape Architects, Regional Environmental Contacts/Regional Landscape Environmental Managers.
5. Conversion of temporary controls to permanent controls. Identify temporary practices that will be converted to permanent facilities.
6. Construction staging. Provide an implementation schedule for staging temporary erosion and sediment control practices, including the timing of initial placement and the duration that each practice is to remain in place. In addition, provide for the removal of temporary measures, when temporary measures are not to be converted to permanent measures. Whenever practical, the area of disturbance on a project site should not exceed 5 acres at any one time. If the SWPPP preparer knows that a project will disturb more than 5 acres at once, it should be noted in the e-NOI submitted to DEC. If it becomes necessary during construction to exceed 5 acres of disturbance at any time, the EIC or CEC shall notify DEC using form HC 209.
7. For contract work not covered under Section 209, a maintenance schedule should be provided in the body of specifications, or in a notes section on the E&SC plan sheet, to ensure continuous and effective operation of erosion and sediment control practices throughout construction. Note: Section 209 of the Standard Specifications contains maintenance requirements for the work covered by that section.

A description of the temporary and permanent structural and vegetative measures incorporated into the contract documents that will be used to control erosion and sedimentation for each stage of the project, from land clearing to project close-out, should be included in the Drainage Report, or the Design Approval Document

8.8.2.3 Erosion and Sediment Control Products

In addition to traditional erosion and sediment control practices, the Department has standard specifications for other practices that prevent or reduce erosion by adhering to the ground surface by stapling or spraying. They are not intended, however, for controlling slope instability. §713-07 *Rolled Erosion Control Products and Soil Stabilizers* identifies the slopes for which different Classes and Types may be used, and what shear stresses they will withstand.

A. Rolled Erosion Control Products

Rolled Erosion Control Products (RECPs) are commonly referred to as erosion control mats or blankets. An RECP is a temporary or long-term non-degradable material manufactured or fabricated into rolls. They are designed to reduce soil erosion on critical areas such as steep slopes, channels or shorelines by assisting in the growth, establishment and protection of vegetation. They are available in three Classes: short-term, intermediate, and permanent.

The advantages of RECPs are that they effectively control erosion by dissipating the energy of raindrops, they can withstand concentrated flows, they aid in the establishment of vegetation, and no heavy equipment is required. The disadvantage is that the slope preparation can be very labor intensive, as the slope should be at or near final grade to ensure that the RECP has intimate contact with the soil. If the contact is not adequate, water can flow under the RECP, and rutting can occur.

Permanent RECPs are Turf Reinforcement Mats (TRMs). TRMs are rolled erosion control products composed of nondegradable synthetic fibers, filaments, nettings, and/or wire mesh, processed into a permanent, three-dimensional matrix which may be supplemented with natural or fiber components (composite). In the past, RECPs and TRMs were combined in the same category. The industry has defined a clear difference and AASHTO National Transportation Product Evaluation Program is considering different testing for TRMs.

TRMs are designed to enhance vegetation reinforcement during and after maturation. TRMs are typically used in critical hydraulic applications, such as high-flow channels, steep slopes, stream banks, and shorelines, where erosive forces exceed the limits of natural, unreinforced vegetation. TRMs need to be protected from concentrated flows of water until establishment of vegetation. To establish vegetation, the prepared seedbed and TRM should be protected using practices to divert the runoff away from the TRM, such as a top of slope berm, a temporary pipe (in the case of swales) or a pipe slope drain. Class III Type A and B TRMs are meant to be filled with topsoil immediately after installation and to be used in conjunction with a Class IV soil stabilizer, Class I or II RECP, or mulch.

Class III Type C and D TRMs, which contain a composite, do not need to be filled with topsoil unless recommended by the manufacturer. Seeding is recommended on the topsoil for most TRMs. TRMs with a composite will require seed placement in contact with the soil. Payment for soil stabilizers, Class I or II RECPs, or mulch, are to be paid for separately when used with a TRM.

B. Soil Stabilizers

Soil Stabilizers (Class IV products) are short-term duration, hydraulically applied products that bind soil particles together to prevent erosion, and should be applied in accordance with the manufacturer's recommendations. They form a protective covering that conforms to the ground surface and dry to form a breathable, high strength blanket. They are for use on slopes 1:2 or flatter, and are not to be used in channels.

There are three types of Soil Stabilizers in §713-07 *Rolled Erosion Control Products and Soil Stabilizers*.

1. Type A is a soil binding system consisting of either a Bonded Fiber Matrix (BFM). or a Polymer Stabilized Fiber Matrix (PSFM).
 - A. A Bonded Fiber Matrix is a continuous layer of elongated fiber strands held together by a water resistant bonding agent. It eliminates direct rain drop impact on soil, it allows no gaps between the product and the soil and it has a high water-holding capacity. It will not form an impermeable crust that can inhibit growth, and will biodegrade completely. BFMs last up to six months and require a cure time up to 48 hours without rain to develop intimate soil contact. Seed would be spread prior to BFM application.
 - B. A Polymer Stabilized Fiber Matrix is a wood cellulose fiber mulch and stabilized polyacrylamide emulsion that works as a flocculent to control erosion. PSFMs last up to six months and require a cure time up to 24 hours.
2. Type B is a polyacrylamide (PAM) and calcium solution intended to reduce the erodibility of bare soils during construction activities or to enhance the performance of mulching on permanent slopes. Polyacrylamides hold soils in place by ionically bonding them together to increase the particle size. This results in greater infiltration through the soil pores and increased resistance to erosive forces.
3. Type C has been added under Class IV. It is a soil binder made up of wood fibers, cotton fibers, interlocking fibers, polymer gels, and hydro-colloid tackifiers, a Flexible Growth Medium (FGM) or Cotton Fiber Reinforcement Matrix (C-FRM). It works similar to a Bonded Fiber Matrix. BFMs require up to 48 hrs. without rain to cure, whereas FGMs require no curing time to develop intimate soil contact. There is a slight cost difference with FGMs and C-FRMs being a little more expensive. BFMs provide protection up to six months. FGMs and C-FRMs are rated for up to a year, and require no cure time to develop intimate soil contact, so they provide protection as soon as applied. FGMs are the highest-performing, hydraulically applied technology and, according to Utah Water Research Laboratory tests, provide greater than 99 percent erosion control effectiveness when subjected to a 5-inches-per-hour rain event for durations of 60 minutes; to a 6-inches-per-hour rain event at TRI (Texas Research International) Environmental for 30 minutes; and to three consecutive, 50-year storm events at San Diego State University.

Type A, B, and C soil stabilizers may be used alone and are approved for use with Class III, Types A and B TRMs where those products are used on slope applications.

8.8.3 SPDES General Permit for Stormwater Discharges from Construction Activity

The SPDES General Permit provides for stormwater qualitative and quantitative control through requirements contained in the body of the general permit. Designs shall be progressed consistent with the permit wherever practicable. However, strict conformance may not always be possible because of the unique limitations placed on the Department by lineal, narrow rights of way, soil type, and existing adjacent development. See Appendix B, Section 2.1.4, for a discussion of additional constraints that may make conformance to the SPDES General Permit impractical.

Where conformance with the General Permit cannot be achieved, the Regional design staff shall provide an explanation in the SWPPP, the project files (Drainage Report, or Design Approval Document, as appropriate) and shall also inform the NYSDEC Regional Permit Administrator or NYSDEC Regional Water Engineer at their regularly scheduled coordination meetings or at project level meetings prior to project lettings.

The SPDES General Permit can be found at:

https://www.dec.ny.gov/docs/water_pdf/constgp020001.pdf

8.8.3.1 Contents of SPDES General Permit

The SPDES General Permit is divided into seven parts as follows:

- Part I Permit Coverage and Limitations
- Part II Permit Coverage
- Part III Stormwater Pollution Prevention Plan (SWPPP)
- Part IV Inspection and Maintenance Requirements
- Part V Termination of Permit Coverage
- Part VI Reporting and Retention of Records
- Part VII Standard Permit Conditions

The entire SPDES permit should be read to establish familiarity with these documents and recognize the necessary coordination between the Construction, Maintenance, and Design groups to implement the permit requirements.

8.8.3.2 Stormwater Management Plans, Details, and Certifications

Each SWPPP shall consist of erosion and sediment control plans and stormwater management plans. Refer to Appendix B, Section 2.4, for information that should be included in a SWPPP. In addition to plans, details, and notes that describe the proposed work, there are other components of a SWPPP to be addressed during the design phase:

Special Note. The “Special Note for Stormwater Pollution Prevention Plans Made Pursuant to the SPDES Stormwater General Permit for Stormwater Discharges from Construction Activity” is to be included in the Proposal by the designer. It is intended to indicate to the Contractor what portions of the contract documents are in the SWPPP, and to convey the Contractor’s obligations under the SPDES General Permit. This note is in EPM Chapter 4.3, Attachment 4.3.B.3.

Contractor/Subcontractor SPDES Permit Certification Form (CONR 5). This form is to be included in the Proposal by the designer. All Contractors and Subcontractors will be expected to sign a copy

of this form, agreeing to comply with the terms and conditions of the SWPPP.

The Regional Design Group shall transmit to the Regional Construction Group, at or about the time of PS&E submittal to DQAB, the components of the SWPPP the Design Group produces.

8.8.4 NPDES Construction General Permit

Stormwater discharges from construction sites on Tribal Indian lands in New York State are permitted under the NPDES Construction General Permit issued by The United States Environmental Protection Agency (EPA).

The NPDES Construction General Permit is located at <https://www.epa.gov/npdes/stormwater-discharges-construction-activities>.

The NYSDEC does not have jurisdiction over Tribal Indian lands in New York State under the SPDES Program.

8.8.4.1 Eligibility

Coverage under the NPDES permit is required for all projects that will result in a disturbance of 1.0 acre or more total land area on Indian lands in New York and involve a stormwater discharge to surface waters of the United States, either indirectly through stormwater sewers or directly to waterways.

If the project involves a disturbance of less than 1.0 acre on Indian lands and 1.0 acre or greater disturbance on non-Indian lands then coverage under the SPDES General Permit is required. If a 1.0 acre disturbance occurs on both Indian and non-Indian lands, then coverage must be sought under both the NPDES Construction General Permit and the SPDES General Permit.

8.8.4.2 Authorization

The NOI should be sent either electronically or via mail to the appropriate address in the NPDES Construction General Permit at the time the final PS&E package is submitted to DQAB. Permit number "NYR10I000" should be included on the NOI. Construction is authorized to commence seven calendar days after acknowledgement of receipt of the NOI is posted on the EPA NPDES website (<https://e-enterprise.gov>)

The NOI shall be signed by the Regional Director or official designee.

8.8.4.3 Storm Water Pollution Prevention Plans (SWPPP)

A SWPPP shall be prepared, prior to submission of an NOI, for each construction project requiring coverage under the NPDES Construction General Permit. Contents of the stormwater pollution prevention plan are described in the NPDES Construction General Permit.

8.8.5 MS4 Stormwater Outfall Inventory Mapping

The NYSDEC SPDES General Permit for Stormwater Discharges from Municipal Separate Storm Sewer Systems (MS4) states that, “An MS4 must, at a minimum must develop and maintain a map showing the location of all outfalls and the names and location of all Waters of the United States that receive the discharges from those outfalls”. An outfall is “any point where a separate storm sewer system discharges to either the Waters of the United States or to another MS4”. Outfalls include discharges from pipes, ditches, swales, and other points of concentrated flow”. This will not include pipe or box culverts which carry streams under a highway, cross culverts that convey drainage from ditch to ditch or culverts that discharge to groundwater.

This outfall mapping is required only in designated urbanized areas and the Department has developed a methodology for identifying and documenting outfalls.

Designers are required to complete the “Stormwater Outfall Inventory” (HC-107) form for any project which is located within the designated urbanized areas. Refer to EI 07-033 for guidance on completing and implementing this form. A fillable PDF is located at this link: https://www.dot.ny.gov/main/business-center/contractors/construction-division/construction-repository/HC_107.pdf

8.9 DRAINAGE REPORT

A Drainage Report should be provided when earthwork, modification of existing drainage features, or new drainage features are included in the project. The report should document the analysis performed to produce the final drainage design, and build upon the information included in Chapter 3, Section 3.3.3.4. and 3.3.3.7. of the Design Approval Document, when provided. Refer to Appendix 7 of the Project Development Manual for the complete format and content of design approval documents.

The Drainage Report should be available for review when design approval is sought or when advance detail plans are complete, depending on the extent of the drainage work. Include a table of contents, an introduction, and sections which discuss the hydrologic analysis and hydraulic design for each type of drainage feature (open channels, culverts, storm drainage systems) included in the project. Other sections should be provided as needed to document the drainage design.

8.9.1 Introduction

The introduction should provide an overview of existing drainage conditions, including known drainage problems, and proposed drainage conditions.

8.9.2 Hydrology

Discuss the information used to perform the hydrologic analysis. State:

1. The methodology, design flood frequency, and basis therefore, used to calculate the discharge to be conveyed by each drainage feature.
2. Any adjustments to the runoff coefficient (C) made to account for future changes in land use or watershed urbanization.
3. The reference, method, and source of values used to compute the peak discharge, time of concentration, and rainfall intensity.
4. All assumptions, site observations, and other hydrologic information used in the analysis.
5. The drainage area and location map for each drainage feature.
6. Output from computer programs, or manual calculations used to compute the peak discharge.

8.9.3 Open Channels

Provide the design criteria and output from computer programs, or manual calculations, used to design the channel (cross section, dimensions, linings) and determine the capacity. Also include the velocity and depth of flow.

All open channel systems meeting the requirements of the NYSDEC SPDES permits shall be noted as such in the contract documents should be provided to the Regional Maintenance Group.

8.9.4 Culverts

Provide the design criteria and:

1. The output from computer programs, or manual calculations used to determine the size and shape. The output should include all of the factors which influence culvert performance as discussed in Section 8.6.3.
2. A table which compares the hydraulic and pipe design criteria with the culvert design. (i.e., for each culvert - state the allowable headwater and the design headwater, state the design life of the pipe material and the anticipated service life of pipe material specified.)
3. The selection of pipe material based on the economics of the various materials available shall be included as discussed in Section 8.6.2.4. The factors which have been considered in the pipe material selection decision as discussed in Section 8.6.2 shall also be included in the report.
4. A Culvert Inventory Coding Sheet for all new culverts. A copy should be provided to the Regional Maintenance Group to update the NYSDOT's statewide Culvert Inventory and Inspection System (CIIS) database.
5. All culverts design in accordance with the USACE Nationwide Permits for habitat connectivity shall be noted as such in the contract documents and should be provided to the Regional Maintenance Culvert Management Group.

8.9.5 Storm Drainage Systems

Provide the design criteria and the output from computer programs, or manual calculations used to design and analyze gutter capacity, inlet capacity, storm drain capacity, the hydraulic gradeline, the outfall, and the receiving waters.

The selection of pipe material based on the economics of the various materials available shall be included as discussed in Section 8.6.2.4. The factors which have been considered in the pipe material selection decision as discussed in Section 8.6.2 shall also be included in the report

All storm drainage systems meeting the requirements of the NYSDEC SPDES permits shall be noted as such in the contract documents should be provided to the Regional Maintenance Group.

8.9.6 Erosion and Sediment Control and Stormwater Management

Describe the erosion and sediment control and stormwater management practices that are part of the project.

All Permanent erosion and sediment control measures which meet the requirements of the NYSDEC SPDES permits shall be noted as such in the contract documents should be provided to

the Regional Maintenance Group.

8.9.7 Special Considerations

Discuss any special considerations such as wetlands, stream temperature considerations, permits, etc., which influenced the drainage design.

8.9.8 References

Include a list of all references which were used to design the proposed drainage features, and the erosion and sediment control and stormwater management measures. (Example, Highway Design Series Number 4, "Introduction to Highway Hydraulics" was used to design all roadside channels.)

8.10 PLANS AND SPECIFICATIONS

This section supplements the guidance presented in Chapter 21 of this manual.

8.10.1 Plans

In general, plans should be prepared consistent with the guidance provided in Chapter 21, Section 21.2.2 A through Z. The intent of this section is twofold: to illustrate that drainage information is presented to the Contractor in different sections of the plans, and to reiterate and build upon guidance contained in Chapter 21 regarding drainage.

8.10.1.1 Typical Sections

Typical sections are discussed in Chapter 3 of this manual and the Comprehensive Pavement Design Manual. Roadside channels are the only drainage feature shown on the typical sections. The following information regarding roadside channels should be provided on the typical section:

1. Gutters (when used within the roadway section). Indicate gutter pay item number, and reference drainage details if applicable. When curbs are proposed, indicate pay item number.
2. Roadway ditches. Indicate lining material pay item number, cross section (width, depth, fore and back slopes), and distance from shoulder (i.e., "varies"). Refer to the drainage details, and ditch profile if applicable.
3. Toe-of-slope ditches. Indicate lining material pay item number, cross section (width, depth, fore and back slopes), and distance from shoulder (i.e., "varies").
4. Intercepting ditches. Indicate lining material pay item number, cross section (width, depth, fore and back slopes), and distance from roadway ditch (i.e., "varies"). Refer to the drainage details if applicable.
5. Median swales (divided highways). Indicate lining material pay item number, and cross section (width, depth, fore and back slopes).

8.10.1.2 Miscellaneous Tables

1. Drainage Structures. Provide a Table of Drainage Structures. The table should include a summary of the work to be performed and should include relevant information which is not included elsewhere within the contract plans. The quantity breakout for each item should not be included in the table. This information should be made available to the contractors as supplemental information to bidders.

2. Gutters. Provide a table for each type of gutter proposed, by pay item number. Include the station limits, offset (i.e. left or right, in the direction of increased stationing), and quantity.
3. Curb. Provide a table for each type of curb proposed, by pay item number. Include the station limits, offset (i.e. left or right, in the direction of increased stationing), and quantity.
4. Stone Filling. Provide a table for each type of stone filling proposed, by pay item number. Provide a separate entry for each location with the corresponding quantity. This table should refer to or be placed with the drainage detail(s) for this work which shows the pay limits.

8.10.1.3 Miscellaneous Details

Provide any details which differ from the Standard Sheets (e.g., drainage structures) and designs not covered by the typical sections. Examples include:

1. Ditch designs which differ from the typical section,
2. Gutter sections which differ from the standard sheets,
3. Stone lined aprons, energy dissipators, or closed chutes (downspouts) which are necessary to provide permanent soil erosion and sediment control, and
4. Details of work before abandoning existing drainage structures.
5. Temporary soil erosion and sediment control details which differ from the standard sheets

8.10.1.4 Special Plans

The following items should be provided in this section of the plans as necessary:

1. Storm Water Management Plan. Provide a legend designating the type of temporary soil erosion and sediment controls to be used and note the location of these controls on the plan. Wetland mitigation areas may also be shown on this plan or on a separate plan as noted in item 2 below.
2. Wetland Mitigation Plan. This plan should include a tabulation of the species to be planted. Include item numbers, quantities, symbology, and other pertinent information deemed necessary by the Regional Landscape/Environmental Group.
3. Wetland Mitigation Details.

8.10.1.5 Plan - 1'=40' Scale or Larger

1. Existing topographic features. Show culverts (size, material or type, and direction of flow), natural watercourses (streams), lakes, swamps, wetlands.

2. Drainage. Show proposed drainage features - culverts, drainage structures, storm drains. Show control elevations for each drainage feature - culvert inlet and outlet invert elevation, invert elevations for storm drains connecting to drainage structures, top of grate (TG) or top of frame (TF) elevation. (Consider providing a note which states that the Contractor shall verify the TG or TF elevations.) Indicate locations to receive a stone apron. Reference drainage structure table or culvert table as appropriate.

Depending on the type and scope of project, Regional preference, and adjacent development, the drainage information may be shown on Utility and Drainage Plans, a separate Drainage Plan, and/or on a 1"=20' scale Intersection Plan.

For every culvert conveying a live stream (a stream that shows as a blue line on a USGS map), the following information is to be shown in a box near the location of the culvert on the plan:

1. 100 year flood discharge, and high water elevation.
2. Design flood frequency, peak discharge, and high water elevation.(If the flood of record exceeds the 100 year flood, the plans should also indicate the peak discharge, high water elevation, and date of occurrence of the record flood.)

There should be a design high water elevation shown for highway embankments encroaching into flood plains and rivers.

8.10.1.6 Profile

1. Special ditches. Show the grade, starting elevation and station, ending elevation and station, and which side of the horizontal control line the ditch is to be located.
2. Culverts and storm drainage systems. Show drainage structures and which side of the horizontal control line (HCL) the structure is located, and storm drains - indicate pipe diameter, material type, and which side of the HCL the pipe is to be located (if applicable). Show control elevations as discussed in Section 8.10.1.5, and reference the drainage structure table.

8.10.2 Specifications

The following sections of the Standard Specifications are used to specify drainage work:

- 201 Clearing and Grubbing
- 203 Excavation and Embankment
- 206 Trench, Culvert and Structure Excavation
- 209 Soil Erosion and Sediment Control
- 602 Rehabilitation of Culvert and Storm Drain Pipe
- 603 Culverts and Storm Drains
- 604 Drainage Structures
- 620 Bank and Channel Protection
- 624 Paved Gutters
- 655 Frames, Grates and Covers.

8.10.3 Special Notes

Special notes should be provided to convey special directions, provisions, or requirements peculiar to the project. Refer to Chapter 21, Section 21.4 for additional guidance regarding the use and format of special notes.

For all projects subject to the SPDES Stormwater General Permit, designers shall include the special note, contained in EPM Chapter 4, Section 4.3.B.3, in the PS&E package submitted to DQAB.

8.11 DRAINAGE SOFTWARE

This section provides guidance regarding drainage software to be used for highway drainage design and analysis. The drainage software shown in Exhibit 8-36 should be used for highway drainage design and analysis.

Exhibit 8-36 Software for Highway Drainage and Analysis

Drainage Design and Analysis	Highway Drainage Software Application ¹	
	Hydrologic Analysis	Hydraulic Design & Analysis
Culvert Design & Analysis	WMS ² , TR-55 ³ , StreamStats ⁵	HY-8 ⁴
Storm Drainage Systems	Storm & Sanitary, TR-55	Storm & Sanitary
Ditch Design & Analysis	Storm & Sanitary	Storm & Sanitary
Stormwater Management Design & Analysis (Ponds, etc.)	WMS, HydroCAD	HydroCAD

NOTES:

1. The listed software applications are available to Department personnel.
2. WMS – Watershed Modeling System (WMS)
3. TR 55 – Technical Release No. 55 – developed by the National Resources Conservation Service (NRCS).
4. Federal Highway Administration Culvert Analysis
5. Online regression analysis tool from USGS. [USGS StreamStats Application](#)

8.12 REFERENCES

This section contains two lists of references. Section 8.12.1 lists all current references for Chapter 8. Section 8.12.2 lists all volumes and chapters in the "Highway Drainage Guidelines" and the "AASHTO Drainage Manual" as a source for additional information.

8.12.1 References for Chapter 8

1. "Bank and Channel Protective Lining Design Procedures", Geotechnical Design Procedure-10, 2015, Geotechnical Engineering Bureau, New York State Department of Transportation, 50 Wolf Road, Albany, N.Y., 12232.
2. Bicycle-Safe Grate Inlets Study Volume 1, "Hydraulic and Safety Characteristics of Selected Grate Inlets on Continuous Grades", June 1977, P. Burgi and D. Gober, National Technical Information Service, Springfield, VA., 22161.
3. Bicycle-Safe Grate Inlets Study Volume 2, "Hydraulic Characteristics of Three Selected Grate Inlets on Continuous Grades", May 1978, P. Burgi, National Technical Information Service, Springfield, VA., 22161.
4. Bicycle-Safe Grate Inlets Study Volume 3, "Hydraulic Characteristics of Three Selected Grate Inlets in a Sump Condition", September 1978, P. Burgi, National Technical Information Service, Springfield, VA., 22161.
5. Bicycle-Safe Grate Inlets Study Volume 4, "Hydraulic Characteristics of Slotted Drain Inlets", February 1980, C. Pugh, National Technical Information Service, Springfield, VA., 22161.
6. Bicycle-Safe Grate Inlets Study Volume 5, "Hydraulic Design of General Slotted Drain Inlets", October 1980, C. Pugh, National Technical Information Service, Springfield, VA., 22161.
7. "Comprehensive Pavement Design Manual", 2008, Office of Technical Services, New York State Department of Transportation, 50 Wolf Road, Albany, N.Y., 12232.
8. "Computer Program for Project Formulation Hydrology", Technical Release 20, January 2005, United States Department of Agriculture, Natural Resources Conservation Service, Washington, D.C.
9. "Design, Construction and Maintenance of Recharge Basins", Geotechnical Design Procedure-8, August 2015, Geotechnical Engineering Bureau, New York State Department of Transportation, 50 Wolf Road, Albany, N.Y., 12232
10. "Design of Leaching Basins", Research Report 157, November 1992, Office of Technical Services, New York State Department of Transportation, 50 Wolf Road, Albany, N.Y., 12232
11. Vacant
12. "Project Development Manual", Plan Sales Unit, New York State Department of Transportation, 50 Wolf Road, Albany, N.Y., 12232.
13. "Environmental Procedures Manual", Office of Environment, New York State Department of Transportation, 50 Wolf Road, Albany, N.Y., 12232.

14. "Estimates of Roughness Coefficients for Selected Natural Stream Channels With Vegetated Banks in New York", U.S. Geological Survey Open-File Report 93-161, William Coon, U. S. Geological Survey, Earth Science Information Center, Open-File Reports Section, Box 25286, MS 517, Denver Federal Center, Denver, CO., 80225.
15. "Evaluation of Six Methods for Estimating Magnitude and Frequency of Peak Discharges of Urban Streams in New York", USGS Water-Resources Investigations Report 84-4350, David A. Stedfast, U. S. Geological Survey, Open-File Reports Section, Box 25425, Denver Federal Center, Denver, CO., 80225.
16. "Fish Passage Through Culverts." FHWA-FL-90-006, 1990, C. Baker, and F. Votapka, Forest Service Technology and Development Center, San Dimas, California
17. HEC-HMS "Hydrologic Modeling System", Hydrologic Engineering Center, U. S. Army Corps of Engineers, 609 Second Street, Davis, CA., 95616.
18. "Highway Drainage Guidelines", 2007, American Association of State Highway and Transportation Officials, 444 North Capitol St., N.W., Suite 249, Washington, D.C., 20001.
19. Federal Highway Administration Hydraulic Design Series and Hydraulics Engineering Circular are available electronically at:

http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm

- Hydraulic Design Series No. 2, "Highway Hydrology".
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 - Hydraulic Design Series No. 4, "Introduction to Highway Hydraulics".
 - Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts".
 - Hydraulic Engineering Circular No. 9, "Debris Control Structures Evaluation and Countermeasures"
 - Hydraulic Engineering Circular No. 12, "Drainage of Highway Pavements".
 - Hydraulic Engineering Circular No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels".
 - Hydraulic Engineering Circular No. 15, "Design of Roadside Channels with Flexible Linings".
 - Hydraulic Engineering Circular No. 22, Urban Drainage Design Manual.
20. "Manual of Administrative Procedures", Audit and Civil Rights Division, New York State Department of Transportation, 50 Wolf Road, Albany, N.Y., 12232.
 21. "Metal-Loss Rates Of Uncoated Steel and Aluminum Culverts In New York", Research Report 115, Office of Technical Services, New York State Department of Transportation, 50 Wolf Road, Albany, N.Y., 12232.
 22. "Maximum Known Stages and Discharges of New York Streams, 1865-1989, with Descriptions of Five Selected Floods, 1913-85", 1993, R. Lumia, and P. M. Murray, U. S. Geological Survey, Open-File Reports-ESIC, Box 25425, Denver, CO., 80225.
 23. "AASHTO Drainage Manual", 2014, American Association of State Highway and

Transportation Officials, 444 North Capitol St., N.W., Suite 249, Washington, D.C., 20001.

24. "New York State Standards and Specifications for Erosion and Sediment Control", November 2016, Division of Water, New York State Department of Environmental Conservation, 625 Broadway, Albany, N.Y., 12233.
25. "NYSDOT Guidelines for the Adirondack Park", August 2008, Office of Environment, New York State Department of Transportation, 50 Wolf Road, Albany, N.Y., 12232.
26. "NOAA Technical Memorandum NWS HYDRO-35", June 1977, R. Frederick, V. Myers, E. Auciello, National Technical Information Service, Springfield, VA., 22161.
27. "Policy and Standards for Entrances to State Highways", Plan Sales Unit, New York State Department of Transportation, 50 Wolf Road, Albany, N.Y., 12232.
28. "Magnitude and Frequency of Floods in New York", USGS Scientific Investigations Report 2006-5112, 2006, R. Lumia, U. S. Geological Survey, Books and Open-File Reports Section, Box 25425, Denver, CO., 80225.
29. "Technical Paper No. 40", May 1961, U.S. Department of Commerce, Weather Bureau, Washington, D.C.
30. "Urban Hydrology for Small Watersheds", Technical Release 55, June 1986, United States Department of Agriculture, Soil Conservation Service, Washington, D.C
31. "Water Resources Data, New York, Water Year 2004, Volume 1, Eastern New York Excluding Long Island", 2006, United States Geological Survey Water Data Report NY-05-1, National Technical Information Service, Springfield, VA 22161
32. "Water Resources Data, New York, Water Year 2004, Volume 2, Long Island", 2005, United States Geological Survey Water Data Report NY-04-2, National Technical Information Service, Springfield, VA 22161
33. "Water Resources Data, New York, Water Year 2005, Volume 3, Western New York", 2005, United States Geological Survey Water Data Report NY-05-3, National Technical Information Service, Springfield, VA 22161

8.12.2 Topics Presented in the "Highway Drainage Guidelines" and the "AASHTO Drainage Manual"

8.12.2.1 Highway Drainage Guidelines

The "[Highway Drainage Guidelines](#)" consists of fifteen chapters as follows and includes a glossary of terms.

<u>Chapter</u>	<u>Title</u>
Chapter 1	Hydraulic Considerations in Highway Planning and Location
Chapter 2	Hydrology
Chapter 3	Erosion and Sediment Control in Highway Construction
Chapter 4	Hydraulic Design of Culverts
Chapter 5	The Legal Aspects of Highway Drainage
Chapter 6	Hydraulic Analysis and Design of Open Channels
Chapter 7	Hydraulic Analysis for the Location and Design of Bridges
Chapter 8	Hydraulic Aspects in Restoration and Upgrading of Highways
Chapter 9	Storm Drain Systems
Chapter 10	Evaluating Highway Effects on Surface Water Environments
Chapter 11	Highways Along Coastal Zones and Lakeshores
Chapter 12	Stormwater Management
Chapter 13	Training and Career Development of Hydraulic Engineers
Chapter 14	Culvert Inspection, Material Selection, and Rehabilitation
Chapter 15	Guidelines for Selecting and Utilizing Hydraulics Engineering Consultants

8.12.2.2 AASHTO Drainage Manual

The "AASHTO [Drainage Manual](#)" consists of twenty-two chapters as follows and includes a glossary of terms.

<u>Chapter</u>	<u>Title</u>
Chapter 1	Introduction
Chapter 2	Legal Aspects
Chapter 3	Data Collection
Chapter 4	Documentation
Chapter 5	Software [Reserved]
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 Dissipators
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