6.C.6 Open Channel Flow Travel Time

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation but may change with respect to stream reach.

Manning's equation is

$$V = (r^{2/3}\, s^{1/2})/n \quad (V = (1.49 r^{2/3}\, s^{1/2})/n) \tag{6.C.6}$$

where: V = average velocity, m/s (ft/s)
r = hydraulic radius, m (ft) (equal to a/p_w)
a = cross sectional flow area, m^2 (ft^2)
p_w = wetted perimeter, m (ft)
s = slope of the hydraulic grade line (watercourse slope), m/m (ft/ft)
n = Manning's roughness coefficient (see Appendix A of Chapter 8, Culverts and Table 7-1 of Chapter 7, Channels)

After average velocity is computed using equation 6.C.6, T_t for the channel segment can be estimated using equation 6.C.3.

6.C.7 Reservoir Or Lake Flow Travel Time

Sometimes it is necessary to compute a T_t for a watershed having a relatively large body of water in the flow path. In such cases, T_t is computed to the upstream end of the lake or reservoir, and for the body of water the travel time is computed using the equation:

$$V_w = (gD_m)^{0.5} \tag{6.C.7}$$

Where: V_w = the wave velocity across the water, m/s (ft/s)
g = 9.81 m/s^2 (32.2 ft/s^2)
D_m = mean depth of lake or reservoir, m (ft)

Generally, V_w will be high 2.44 - 9.14 m/s (8-30 ft/s).

One must not overlook the fact that equation 6.C.7 only provides for estimating travel time across the lake and for the inflow hydrograph to the lake's outlet. It does not account for the travel time involved with the passage of the inflow hydrograph through spillway storage and the reservoir or lake outlet. This time is generally much longer and is added to the travel time across the lake. The travel time through lake storage and its outlet can be determined by the storage routing procedures in the Storage Chapter. Equation 6.C.7 can be used for swamps with much open water, but where the vegetation or debris is relatively thick (less than about 25% open water), Manning's equation is more appropriate. For additional discussion of equation 6.C.7 see King's Handbook of Hydraulics, fourth edition, page 8-50, or Elementary Mechanics of Fluids, by Hunter Rouse, John Wiley and Sons, Inc., 1946, page 142.

After wave velocity is computed using equation 6.C.7, T_t for the reservoir or lake can be estimated using equation 6.C.3.

6.C.8 Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 91.4 m (300 ft). Equation 6.C.2 was developed for use with the four standard rainfall intensity-duration relationships.
- In watersheds with storm drains, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm drains generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert.

11.7 Design Frequency And Spread

The major considerations for selecting a design frequency and spread include highway classification, because it defines and reflects public expectations for finding water on the pavement surface. Ponding should be prevented on the traffic lanes of high-speed, high-volume highways, where it is not expected.

Highway speed is another major consideration, because at speeds greater than 70 km/h, (45 mi/h) even a shallow depth of water on the pavement can cause hydroplaning. Design speed is recommended for use in evaluating hydroplaning potential. When the design speed is selected, consideration should be given to the likelihood that legal posted speeds may be exceeded. It is clearly unreasonable and not cost effective to provide the same level of protection for low speed facilities as for high speed facilities.

Other considerations include inconvenience, hazards and nuisances to pedestrian traffic and buildings adjacent to roadways which are located within the splash zone. These considerations should not be minimized and, in some locations (such as commercial areas), may assume major importance.

The design criteria for various types of Connecticut roadways are outlined in Table 11-2.

Table 11-2 Pavement Drainage Design Criteria

ROADWAY	ADT	SPEED km/hr (mi/hr)	DESIGN FREQUENCY yr	ALLOWABLE DESIGN SPREAD
State Arterial Highways and Expressways	≥ 3000 ≥ 3000 < 3000	≥ 80 (≥ 50) ≤ 70 (≤ 45) ---	10 10 10	shoulder ½ of lane ½ of lane
Sag Condition	any	any	50*	all except one lane width
State Collector Highways and State-owned service Roads	≥ 3000 ≥ 3000 < 3000	≥ 80 (≥ 50) ≤ 70 (≤ 45) ---	10 10 10	shoulder ½ of lane ½ of lane
Sag Condition	any	any	25*	all except one lane width
Town Roads	≥ 3000 < 3000	any	10 5	½ of lane ½ of lane
Sag Condition	≥ 3000 < 3000	any	25 10	all except one lane width
One Lane Ramps	any	any	10	0.3m (1 ft) of lane
Ramps > one lane	any	any	10	1m (3 ft) of lane

* Sag condition is defined as sag vertical curves where the water cannot escape over berms and down an embankment. The procedure is to design the drainage inlets and storm system for a 10 year frequency and then to impose the higher frequency storm on the inlets and storm system. If the higher frequency storm closes the facility to traffic then additional inlets or the storm system will have to be changed.

11.11 Storm Drains

11.11.1 Introduction

After the preliminary locations of inlets, connecting pipes and outfalls with tailwaters have been determined, the next logical step is the computation of the rate of discharge to be carried by each reach of the storm drain, and the determination of the size and gradient of pipe required to convey this discharge. This is done by starting at the upstream reach, calculating the discharge and sizing the pipe, then proceeding downstream, reach by reach to the point where the storm drain connects with other drains or the outfall. For manholes where the pipe size is increased, the downstream crown should be lower than the upstream crown by the amount of the energy loss in the manhole.

The rate of discharge at any point in the storm drain is not necessarily the sum of the inlet flow rates of all inlets above that section of storm drain. It is generally less than this total. The time of concentration is most influential and as the time of concentration grows larger, the rainfall intensity to be used in the design grows smaller. In some cases, where a relatively large drainage area with a short time of concentration is added to the system, the peak flow may be larger using the shorter time even though the entire drainage area is not contributing. The prudent designer will be alert for unusual conditions and determine which time of concentration controls for each pipe segment. See Section 11.5.5 for a discussion on time of concentration.

For ordinary conditions, storm drains should be sized on the assumption that they will flow full or practically full under the design discharge but will not flow under pressure head. The Manning's formula is recommended for capacity calculations. In locations such as depressed roadway sections and underpasses where ponded water can be removed only through the storm drain system, a higher design frequency should be analyzed to ensure the roadway stays open to traffic (see Table 11-2 for design criteria). The main storm drain downstream of the depressed section should be designed by computing the hydraulic grade line and keeping the water surface elevations below the grates and/or established critical elevations for the check storm.

11.11.2 General Guidelines

The following items must be considered during the design of a storm drain system.

- Storm drains shall be designed for "just-full" condition. The head waters in structures shall be limited to 0.3 meters (1 ft) below the top of grate, taking into consideration the possible effect of headwater in the next downstream structure.
- Underdrain pipes of 100 and 150 mm (4 in and 6 in) size should be laid in straight segments or gradual curves if possible. Where bends of underdrain are necessary to enter a structure they should be no greater than 30 degrees.
- Long skew crossings of storm drain laterals under pavement should be avoided.
- All roadway drainage, including the side and slope ditches shall be carried to a suitable outlet, preferably an existing stream. Where outletting to an existing stream is impractical, or where no stream is available, appropriate drainage rights must be obtained.
- The discharge of effluent from sanitary sewers, cesspools, septic tanks, discharge of cooling water or industrial wastes into a State maintained roadway drainage system will not be permitted.
- Private connections to State drainage systems are only allowed after issuance of an encroachment permit accompanied by a special connection agreement.

- Roadway drainage shall not be outletted into existing drainage systems which are privately owned or those maintained by towns or cities except in the case where an independent outlet is not feasible due to excessive cost or other reasons. Where outletting into such a system, an agreement must be entered into with the municipality. A deeded right to drain must be secured from owners of private systems.
- All existing metal pipes to be abandoned under the travelway are to be removed. Concrete pipes to be abandoned should be plugged at the ends.
- State drainage systems shall not be outletted into municipal systems which carry both storm water and sanitary sewage, nor will any such municipal system carrying both storm water and sanitary sewage be outletted into State systems.
- Diversion of watershed area should be avoided if possible. However, in all cases where drainage is diverted from one watershed area to another, as is frequently the case in incised highways, the designer shall note the diversions in the computations and on the preliminary plans to better allow the reviewers and right of way negotiators to make proper provisions for the lawful disposal of the drainage from this area at the outlet locations.
- Utility conflicts may require design changes. New installations should be kept at least 0.3 meters (1 ft) from any utilities.
- The pertinent plans and computations for drainage systems on a project which originate or terminate on an adjacent project shall be furnished for review by the designer of the project being reviewed. The area used for runoff computation shall be shown on topographical maps also to be supplied.
- Each outlet must be carefully designed with erosion protection as needed and carried down steep slopes to lesser slopes where outlet erosion will not occur. Riprap shall be designed at all outlets not flowing over exposed rock or into deep watercourses or ponds. (See Section 11.14.)
- Storm drainage systems will be designed for the watershed which naturally drains to it. In many urban areas the existing drainage systems are inadequate and it is impossible to provide inlet capacity for the overflow, however, the trunk line system should be designed to allow the municipality to upgrade their contributing system at a future date.
- Minimum size pipe for storm drainage is 300 mm (12 in).
- Slotted drain shall be outletted into catch basins.

11.11.3 Outlets

All proposed storm drains have an outlet point where the flow is discharged. The designer should consider at least the following aspects that may affect the hydraulic design of a storm drainage system.

- The flowline elevation of the outfall should be equal to, or higher than the recipient. If this is not the case, excavation may be required to ensure positive gravity flow, or in severe cases pump stations may be required.
- Where practical, the outlet should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for erosion.
- When the outlet is located in a manner to allow the discharge to impinge on the opposite bank of a channel, that bank should be evaluated to determine the need for riprap.

11.11.4 Bridge Deck Outlets

The design of deck drain outlets should be such as to prevent the discharge of drainage water against any portion of the structure or on moving traffic below, and to prevent erosion at the outlet of the downspout. Deck drainage may be connected to conduits leading to stormwater outfalls at ground level. Water in a roadway gutter section should be intercepted prior to the bridge. Scuppers are not to be designed with freefall outlets except over water or where it can be demonstrated that the flow will not cause damage or become a nuisance.

Regardless of the outfall type, a design of the pipe, which is usually a vertical connection, must be performed. The depth of water required to convey the flow in the pipe should be maintained below the bottom of the grate. This will allow the flow to be intercepted, in most cases, without causing splashover. The following formula is used to determine the required pipe size. The minimum pipe size shall be 200 mm (8 in).

$$Q_I = 0.6\ A\ (2gH)^{0.5} \qquad \textbf{(11.14)}$$

Where:

Q_I = Flow intercepted by inlet and for the pipe design m^3/s (ft^3/s)
0.6 = Orifice coefficient
A = Area of the pipe outletting inlet m^2 (ft^2)
H = Depth of flow over pipe m (ft)
g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)

Any debris entering the inlet must pass through the outlet pipe to the disposal point. Therefore, the outlet pipes must be designed to be self-cleaning and fittings that trap debris should be avoided. The following criteria should be followed in the design of pipes for bridge drainage.

- Inlets should be placed near bridge piers - Long runs of pipe to reach the pier will not function well and should be avoided.
- The minimum slope should be the maximum slope achievable, but in no case less than 8%.
- When elbows must be used, use long radius elbows. Use elbows of 45 degrees or less.
- Use smooth walled pipe which is resistant to corrosion and has watertight joint capable of withstanding the internal pressure imposed by backflushing.
- If discharge is free fall under the bridge, the pipes should be carried at least 75 mm (3 in) below the bottom of adjacent girders.
- Cleanouts should be provided at key points, considering the maintenance equipment available to access them.
- Outfall from bridge drainage should be located so that flow will not be interrupted in a manner that will preclude the conveyance of the debris.

11.11.5 Storm Drainage System Design Procedures

The design of storm drainage systems is generally divided into the following operations:

Step 1 Determine inlet location and spacing as outlined earlier in this chapter.

Step 2 Prepare plan layout of the storm drainage system establishing the following design data:

a. Location of storm drains.
b. Direction of flow.
c. Location of manholes.
d. Location of existing utilities such as water, gas, sanitary sewer, electric, communication facilities, etc.

Step 3 Perform storm drainage computations to determine flows in the drainage system and the required pipe sizes for "just-full conditions" (smallest available pipe size not flowing at full capacity for the design discharge). Use the Storm Drainage Computation Sheet, Table 11-8 or 11-8.1, described below, to facilitate the computational procedure.

- Columns 1 and 2 – Starting at the highest point in the system and proceeding downstream, enter the station and offset of the drainage structures (catch basin, manhole, junction, etc.) for each pipe segment.

- Column 3 – Determine the time of concentration for surface flow (runoff) to reach inlet. If a gutter flow analysis was performed, this information would be taken from the Gutter Flow Analysis form.

- Column 4 – Determine the time required for the design flow to pass through the pipe segment. Calculated by dividing the pipe length (13) by the velocity (19).

- Column 5 – Determine the accumulated time which is the time of concentration effective at the upstream end of the pipe segment. The longest time is to be used. This can be overland flow to an inlet, accumulation of time in pipe or a branch line entering a system.

- Column 6 – Determine the A X C entering the inlet or catch basin or enter information previously determined from the Gutter Flow Analysis.

- Column 7 – Determine the total A X C which is the sum of <u>all</u> the A X C entering the drainage structure and conveyed by the pipe segment.

- Column 8 – Determine the Rainfall Intensity (Chapter 6, Appendix B) based on the Accumulated Time (5).

- Column 9 – Determine the Total Flow in system which is the product of the Total A X C in system (7) and the Rainfall Intensity (8).

- Columns 10, 11, 12, 13, 14, 15 and 16 – Enter the information for the selected pipe: Size; Type; Manning's Roughness Coefficient - "n" (See Chapter 8, Appendix A for recommended values); Length; Invert Elevation, Upstream; Invert Elevation, Downstream; Slope (expressed to the nearest thousandth).

- Column 17 – Determine the discharge which can be carried by pipe of size and type specified, flowing full.

- Column 18 – Determine the full flow velocity based on the full capacity discharge (17).

- Column 19 – Determine the velocity obtained in pipe of size, type and slope specified, at design discharge (9).

Step 4 Complete the design by calculating the hydraulic grade line (HGL) as described in Section 11.12. If all HGL elevations meet the 0.3m (1 ft) freeboard from top of grate or rim requirement, then the hydraulic design is adequate. If the HGL exceeds this 0.3m (1 ft) freeboard, then adjustments to the design must be made to lower the water surface elevation and computations revised accordingly.

Step 5 All computations and design sheets should be clearly identified. The engineer's initials and date of computations should be shown on every sheet. Voided or superseded sheets should be so marked. The origin of data used on one sheet but computed on another should be given.

11.11.6 Check Storm for Sag Point

As indicated above, the storm drain which drains a major sag point should be sized to accommodate the runoff from a check storm frequency rainfall. This can be done by actually computing the bypass occurring at each inlet during a check storm rainfall and accumulating it at the sag point. The inlet at the sag point as well as the storm drain pipe leading from the sag point must be sized to accommodate this additional bypass within the criteria established. In order to design the pipe leading from the sag point, it may be helpful to convert the additional bypass created by the check storm rainfall into an equivalent CA which can be added to the design CA. This equivalent CA can be approximated by dividing the check storm bypass by 0.00278 X I_{10} for metric or I_{10} for English units in the pipe at the low point.

11.11.7 Hydraulic Capacity

The most widely used formula for determining the hydraulic capacity of storm drains for gravity and pressure flows is the Manning's formula and it is expressed by the following equation:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad \left(V = \frac{1.49}{n} R^{2/3} S^{1/2}\right) \qquad \mathbf{(11.15)}$$

Where: V = mean velocity of flow, m/s (ft/s)
n = Manning's roughness coefficient (see Appendix A of Chapter 8, Culverts)
R = hydraulic radius, m (ft) = area of flow divided by the wetted perimeter (A/WP)
S = the slope of the energy grade line, m/m (ft/ft)

In terms of discharge, the above formula becomes:

$$Q = V\,A = \frac{1}{n} A\ R^{2/3}\ S^{1/2} \qquad \left(Q = \left(\frac{1.49}{n}\right) A R^{2/3} S^{1/2} \right) \qquad \textbf{(11.16)}$$

Where: Q = rate of flow, m^3/s (ft^3/s)
A = cross sectional area of flow, m^2 (ft^2)

For storm drains flowing full, the above equations become:
Where: D = diameter of pipe, m(ft)

$$V = \frac{0.397}{n} D^{2/3}\ S^{1/2} \qquad Q = \frac{0.312}{n}\ D^{8/3}\ S^{1/2}$$

$$\left(V = \frac{0.592}{n} D^{2/3} S^{1/2} \right) \quad \left(Q = \frac{0.464}{n} D^{8/3} S^{1/2} \right) \qquad \textbf{(11.17)}$$

The nomograph solution of Manning's formula for full flow in circular storm drains is shown on Figure 11-9, and Figure 11-10. Figure 11-11 has been provided to assist in the solution of the Manning's equation for part full flow in storm drains.

11.11.8 Curved Alignment

Curved storm drains are permitted where necessary. Long radius bend sections are available from many suppliers and are the preferable means of changing direction in pipes 1200 mm (48 in) and larger. Short radius bend sections are also available and can be utilized if there isn't room for the long radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. Utilizing large manholes solely for changing direction may not be cost effective on large size storm drains.