Chapter 8 STORM DRAINAGE SYSTEMS

8.1 INTRODUCTION

This chapter provides guidance for the planning and design of pavement drainage systems. The complete system will be referred to as a storm drain system and will normally consist of curbs and/or gutters, inlets or catch basins, laterals or leads, trunk lines or mains, junction chambers, manholes, and ponds. Most aspects of storm drain design such as system planning, pavement drainage, gutter flow calculations, inlet spacing, pipe sizing, and hydraulic grade line calculations are included.

The design of a drainage system must address the needs of the traveling public as well as those of the local community through which it passes. The drainage system for a roadway traversing an urbanized region is more complex than for roadways traversing sparsely settled rural areas. This is often due to:

- the wide roadway sections, flat grades, both in longitudinal and transverse directions, shallow water courses, absence of side channels;
- the more costly property damage which may occur from ponding of water, or from flow of water through developed areas;
- the fact that the roadway section must not only carry traffic, but also act as a channel to convey water to a disposal point. Unless proper precautions are taken, this flow of water along the roadway will interfere with or possibly halt the passage of highway traffic.

The most serious effects of an inadequate roadway drainage system are:

- damage to surrounding or adjacent property, resulting from water overflowing the roadway curbs and entering such property,
- risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway and increased potential for accidents,
- weakening of base and subgrade due to saturation from frequent ponding of long duration.

8.1.1 Definition

A storm drain is defined as that portion of the storm drainage system that receives runoff from inlets and conveys the runoff to some point where it is then discharged into a channel, water body, or piped system. It consists of one or more pipes connecting one or more inlets. A storm drain may be a closed-conduit, open-conduit, or some combination of the two. The terminology "storm sewer" which has been in general use for many years, is gradually being replaced with the term "storm drain" to differentiate between sanitary sewers and storm drains. Storm drain will be used throughout this manual.

The purpose of a storm drain is to collect storm water runoff from the roadway and convey it to an outfall. Storm drain design generally consists of three major parts:

- system planning which includes data gathering and outfall location;
- pavement drainage which includes pavement geometrics and inlet spacing;
- location and sizing of the mains and manholes.

8.1.2 Concept Definitions

Following are discussions of concepts which will be important in a storm drainage analysis and design. These concepts will be used throughout the remainder of this chapter in dealing with different aspects of storm drainage analysis.

Bypass	Flow which goes past an inlet on grade and is carried in the street or channel to the next inlet downgrade. Is also referred to as carryover or runby.
Check Storm	The use of a lesser frequency event, such as a 50 year storm, to assess the flood hazard at critical locations where water can pond to appreciable depths is commonly referred to as a check storm or check event.
Combination Inlet	A drainage inlet is usually composed of a curb-opening inlet and a grate inlet, but may include a grate inlet and a slotted drain inlet.
Crown	The crown, sometimes known as soffit, is the top inside of a pipe.

Culvert	A culvert is a closed conduit whose purpose is to convey surface water under a roadway, railroad or other impediment. It may have one or two inlets connected to it to convey drainage from the median area.
Curb-Opening Inlet	A drainage inlet consisting of an opening in the roadway curb.
Drop Inlet	A drainage inlet with a horizontal or nearly horizontal opening.
Equivalent Cross Slope	An imaginary straight cross slope having conveyance capacity equal to that of the given compound cross slope.
Flanking Inlets	Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets are to intercept debris as the slope decreases and to act in relief of the inlet at the low point.
Flow (Q)	Flow refers to a quantity of water which is being conveyed .
Frontal Flow	The portion of the flow which passes over the upstream side of a grate.
Grate Inlet	A drainage inlet composed of a grate in the roadway section or at the roadside in a low point, swale or channel.
Grate Perimeter	The sum of the lengths of all sides of a grate, except that any side adjacent to a curb is not considered a part of the perimeter in weir flow computations.
Gutter	That portion of the roadway section adjacent to the curb which is utilized to convey stormwater runoff. A composite gutter section consists of the section immediately adjacent to the curb, usually 2.0′ at a cross-slope of say 0.06 ft/ft, and the parking lane, shoulder, or pavement at a cross-slope of a lesser amount, say 0.02 ft/ft. A uniform gutter section has one constant cross-slope.
Hydraulic Grade Line (HGL)	The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head).
Inlet	The term "inlets" refers to all types of inlets such as apron inlets, grate inlets, curb inlets, and slotted inlets. The term catch basin has been dropped from this manual and replaced with the appropriate type of inlet.
Inlet Efficiency	The ratio of flow intercepted by an inlet to total flow in the gutter.
Invert	The invert is the inside bottom of the pipe.
Lateral Line	A lateral line, sometimes referred to as a lead, has inlets connected to it but has no other storm drains connected. It is usually 15 inches or less in diameter and is tributary to the trunk line.
Manhole	Hydraulic structure that is included in a storm drain system to provide access to storm drain pipes for inspection and cleanout. Manhole structures are the same as inlet structures except for the castings and cover. Manhole structures are also referred to as access holes.
Pressure Head	Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.
Runby	Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade. Inlets can be designed to allow a certain amount of runby for one design storm and larger or smaller amounts for other storms. Runby is also referred to as bypass or carry-over.

Sag Point/ Major Sag Point	A low point in a vertical curve. A major sag point refers to a low point that can overflow only if water can pond to a depth of two feet or more.
Scupper	A vertical hole through a bridge deck for the purpose of deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.
Side-Flow Interception	Flow which is intercepted along the side of a grate inlet, as opposed to frontal interception.
Slotted Drain Inlet	A drainage inlet composed of a continuous slot built into the top of a pipe which serves to intercept, collect and transport the flow. Two types in general use are the vertical riser and the vane type.
Storm Drain	A storm drain is a closed conduit that conveys storm water that has been collected by inlets to an outfall. It generally consists of laterals or leads, and trunk lines or mains. Culverts connected to the storm drainage system are considered part of the system.
Splash-Over	Portion of the frontal flow at a grate which skips or splashes over the grate and is not intercepted.
Spread (T)	The width of stormwater flow in the gutter measured laterally from the roadway curb.
Trunk Line	A trunk line is the main storm drain line. Lateral lines may be connected at inlet structures or manholes. A trunk line is sometimes referred to as a "main".
Velocity Head	Velocity head is a measure of the kinetic energy of flowing water expressed as a height or head of water, ($V^2/2g$).

8.2 DESIGN CRITERIA

Design criteria establishes the standards by which a policy is placed into action. They form the basis for the selection of the final design configuration. Listed below are the design criteria which shall be considered for storm drain systems.

8.2.1 Policy

Policy is a set of goals that establish a definite course or method of action and are selected to guide and determine present and future decisions. Policy is implemented through design criteria established as standards for making decisions. The following policies are specific to storm drain systems.

Storm Drains

- Storm drain systems should have adequate capacity so that they can accommodate runoff that enters the system for the design frequency.
- Storm drain systems should be designed with future development in mind if it is appropriate.
- The storm drain system for a major vertical sag point that can't overflow elsewhere until the depth of water is two feet or greater, should have a greater level of flood protection to decrease the depth of ponding on the roadway and bridges.
- Where feasible, storm drains shall be designed to avoid existing utilities. The recommended minimum distance is 10 feet when the storm drain system is parallel to the waterline and 1 foot when the storm drain system crosses a waterline.
- Attention shall be given to the storm drain outfall design to insure that the potential for erosion is minimized.
- Drainage system design should be coordinated with the proposed staging of large construction projects in order to maintain an outlet throughout the construction project period.
- The placement and hydraulic capacities of storm drainage structures and conveyances should be designed to take into consideration potential damage to adjacent property and to minimize traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements, and available funds.
- Storm drain placement and capacity should be consistent with local storm water management plans.

Pavement Drainage

- Minimum pavement cross slope should be selected to ensure drainage of pavement. For multilane pavements there is often an 0.005 ft/ft increase in the cross slope for each additional lane beyond the first lane from the crown. Specific cross slope design criteria are provided in the Road Design Manual.
- In municipal areas, a minimum time of concentration of seven minutes is recommended for calculation of runoff from paved areas, all other areas should be calculated on a case by case basis.
- Curbs or dikes, inlets, or flumes are used where runoff from the pavement would erode fill slopes and/or to reduce right-of-way needed for shoulders, channels, etc.
- Where storm drains are necessary, pavement sections are usually curbed.

Gutter

- Gutter grades are recommended to be at least 0.5 percent for curbed pavement with a minimum grade of 0.35 percent.
- Composite gutter sections have greater hydraulic capacity for normal cross slopes than uniform gutter sections and are therefore preferred.
- Gutter flow calculations are necessary to relate the quantity of flow to the spread of water on the shoulder, parking lane, or pavement section.

Inlets

- Drainage inlets are sized and located to limit the spread of water on traffic lanes in accordance with the design criteria specified in Table 8.1.
- The width of water spread on the pavement at sags should not be substantially greater than the width of spread encountered on continuous grades.
- All grate inlets shall be bicycle safe when used on roadways that allow bicycle travel.
- Curb boxes are not recommended at locations where grate inlets on grade are utilized.
- Curb inlets are preferred to grate inlets at major sag locations because of their debris handling capabilities. When grate inlets are used at sag locations, assume they are half plugged with debris and size accordingly.
- Mn/DOT Standard Plate Number 4125 frame is preferred when combination grate/curb-opening inlets are utilized at sag points.
- In locations where significant ponding may occur, such as at underpasses or sag vertical curves in depressed sections, recommended practice is to place flanking inlets on each side of the low point inlet in the sag.

Manholes
- The maximum spacing of storm drain access structures whether manholes or inlets, should be approximately 400 feet for 12 inch through 54 inch diameter storm drains and approximately 600 to 800 feet for 60 inch and larger diameter storm drains.
- Minimum manhole diameter is limited by the maximum pipe size and the deflection angle of the pipes.

Pipes
- A minimum velocity of 3 fps is desirable in a storm drain in order to prevent sedimentation from occurring in the pipe.
- Water tight joints are used if velocities are high, or if the pipe is placed in the ground water table.

Bridge Decks
- Many bridges do not require any drainage structures. Equation 8.4 can be utilized to determine the maximum length of deck permitted without drainage structures.
- Zero gradients, sag vertical curves and superelevation transitions with flat pavement sections should be avoided on bridges.
- The minimum desirable longitudinal slope for bridge deck drainage is 0.5 percent.
- Runoff should be handled in compliance with applicable stormwater quality regulations.

Detention Storage
- Reduction of peak flows can be achieved by the storage of runoff in detention basins, storm drainage pipes, swales and channels, and other detention storage facilities. Stormwater can then be released to the downstream conveyance facility at a reduced flow rate.
- Detention storage should be considered in highway drainage design where:
  - existing downstream conveyance facilities are inadequate to handle peak flow rates from highway storm drainage facilities;
  - where the highway would contribute to increased peak flow rates and aggravate downstream flooding problems;
  - as a technique to reduce the right-of-way, construction, and operation costs of outfalls from highway storm drainage facilities;
  - or as a treatment technique to improve water quality by removing sediment and/or pollutants. (See the Storage Facilities chapter)

Roadside and Median Ditches
- Large amounts of runoff should be intercepted before it can reach the highway in order to minimize the deposition of sediment and other water transported debris on the roadway, and to reduce the amount of water carried in the gutter section.
- Slope median areas and inside shoulders toward a center depression to prevent runoff volume from running across the pavement.
- Surface channels should have adequate capacity for the design runoff volume and should be located and shaped in a manner that does not present a traffic hazard.
- Channels should have a vegetative lining when possible, based upon design velocity criteria. Appropriate linings may be necessary where vegetation will not control erosion.

8.2.2 Design Frequency and Allowable Spread

The design storm frequency selected for pavement drainage should be consistent with the frequency selected for other components of the storm drain system. The design frequency for pavement drainage is tied to the allowable water spread on the pavement and is related to ADT. Mn/DOT has established criteria for design frequency and water spread as shown in Table 8.1.

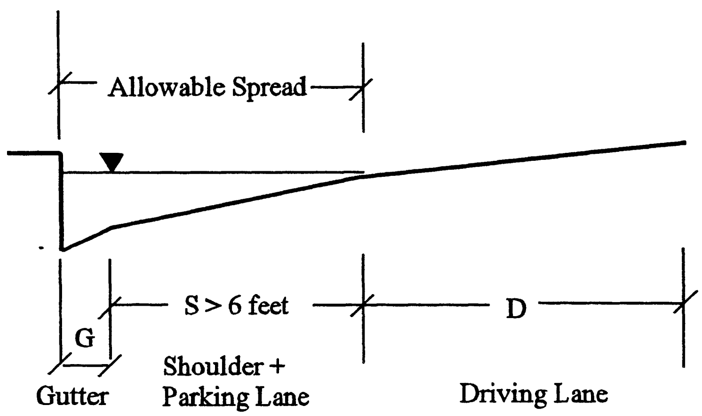
The factor that governs how much water can be tolerated in the curb and gutter section and on the adjacent roadway is known as water spread. Water is allowed to spread onto the roadway area within tolerable limits because it is usually not economically feasible to keep it within a narrow gutter width. The designer keeps track of the water spread by calculation, and when the allowable spread is reached, an inlet is proposed to intercept a portion of the flow. Placement of inlets is used to prevent gutter flow from exceeding the allowable water spread criteria.

Ramps, loops, turn lanes, acceleration and deceleration lanes should generally be designed for the same frequency as the mainline. Exceptions to the table values will be permitted for up to ½ driving lane for existing conditions. Where the speed limit is 35 miles per hour (MPH) or less and there are no shoulders or parking lanes, allowable spread can encroach up to ½ of the driving lane for short periods of time. High capacity inlets such as slotted vane drains should be considered at locations where excessive spread may cause inconvenience or safety hazards to the motorists.

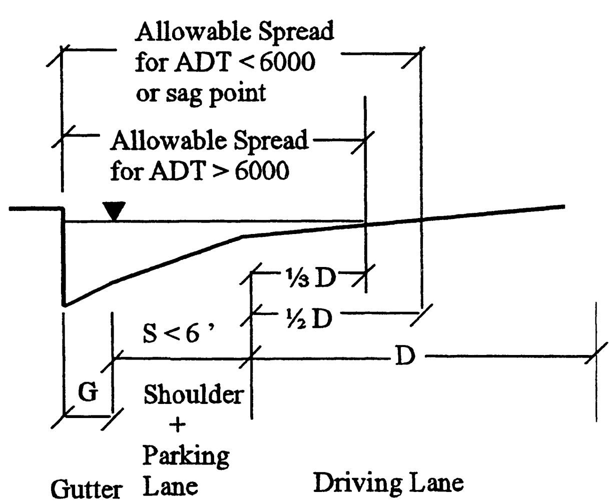
Table 8.1 Design Frequency for Storm Drains

PROJECTED 2-WAY ADT (vehicles per day, VPD)	DESIGN FREQUENCY (Year)	ALLOWABLE SPREAD[1]
> 6000	10	P, S, or ⅓ D
	50 Year at sag point[2]	P, S, or ½ D
2000-6000	10	P, S, or ½ D
1000-1999	5	P, S, or ½ D
< 1000	3	P, S, or ½ D

[1]P = Parking lane, S = Shoulder of 6′ or more, D = Driving lane if there is no shoulder.
[2]Sag Point refers to a true sag where flooding of 2' or more can occur.



Allowable spread when shoulder width is equal to or greater then 6 feet.



Allowable spread when the sum of the shoulder and parking lane is less than 6 feet.

8.3 SYSTEM PLANNING

System planning prior to commencing the design of a storm drain system is essential. The design of any storm drainage system involves the accumulation of basic data, familiarity with the project site, and a basic understanding of the hydrologic and hydraulic principles and drainage policy associated with that design.

The typical elements of the design process are listed below.

- Data collection
- Coordination with other agencies
- Preliminary sketch
- Inlet location and spacing
- Plan layout of storm drain system
  - locate main outfall
  - determine direction of flow
  - locate existing utilities
  - locate connecting mains
  - locate manholes
- Size the pipes
- Develop the energy and hydraulic grade line
- Prepare the plan
- Provide documentation

8.3.1 Required Data

The designer should be familiar with land use patterns, the nature of the physical development in the area(s) to be served by the storm drainage system, local stormwater management plans, and the ultimate pattern of drainage (both overland and by storm drains) to existing outfall locations. Furthermore, the designer should consider the location of the outfall and applicable water quality rules and regulations.

Onsite review and actual topographic survey data are often needed for storm drain system design. In addition, photogrammetric mapping has become an important method of obtaining the data required for drainage design, particularly for busy urban roadways with all the attendant urban development. Existing topographic maps, available from the United States Geological Survey (USGS), Natural Resource Conservation Service (NRCS), many municipalities, some county governments, and even private developers are also valuable sources of the kind of data needed for a proper storm drainage design.

Developers and governmental planning agencies should be consulted regarding drainage plans for the area in question. Often, in rapidly growing urban areas, the physical characteristics of an area to be served by a storm drainage system may change drastically in a very short time. In such cases, the designer is to anticipate these changes and consider them in the storm drainage design. Comprehensive Stormwater Management Plans and Floodplain Ordinances should be reviewed when they are available.

8.3.2 Cooperative Projects

Where a mutual economic benefit and a demonstrated need exists, Mn/DOT endorses cooperative storm drain projects with cities and municipalities. Early coordination with the governmental entity involved is necessary to determine the scope of the project. Each cooperative project must be initiated by a resolution adopted by the governing body of the municipality either requesting the improvements and/or indicating its willingness to share the cost of a state project, or indicating the municipality's intention to make certain improvements and requesting state cost participation in the municipal project. The policy regulating cooperative construction agreements is contained in the Mn/DOT Policy Manual under Cooperative Construction Projects With Municipalities. An explanation of the various methods of cost proration computation is contained therein.

8.3.3 Preliminary Considerations

Preliminary sketches or schematics, featuring the basic components of the intended design, are useful. Such sketches should indicate watershed areas and land use, existing drainage patterns, plan and profile of the roadway, street and driveway layout with respect to the project roadway, underground utility locations and elevations, overhead sign foundation locations, locations of proposed retaining walls, bridge abutments and piers, logical inlet and manhole locations, preliminary lateral and trunk line layouts, and a clear definition of the outfall location and characteristics. The preliminary sketch should be reviewed along with the traffic staging plans and soil recommendations to identify conflicts that could arise during construction staging. With this sketch or schematic, the designer is able to proceed with the detailed process of storm drainage design calculations, adjustments, and refinements. Unless the proposed system is very simple and small, the designer should not ignore a preliminary plan as described above. Upon completion of the design, documentation of the overall plan is facilitated by the preliminary schematic.

When designing storm drain systems, planning is needed to avoid utilities and deep cuts. Avoid unnecessary excavation to minimize adverse impacts upon utilities, reduce cost and potential negative environmental impacts.

In some cases, traffic must be maintained or temporary bypasses constructed and temporary drainage provided for during the construction phase. Further consideration should be given to the actual trunk line layout and its constructibility. For example, will the proposed location of the storm drain interfere with inplace utilities or disrupt traffic? Some instances may dictate a trunk line on both sides of the roadway with very few laterals while other instances may call for a single trunk line. Storm system layout is usually designed to minimize cost but may be controlled by other physical features.

It is generally not a good idea to decrease pipe size in a downstream direction regardless of the available pipe gradient because of potential plugging with debris. However for large pipes, 36 inches diameter and above, substantial savings can result if the gradient will allow pipe size reductions. In those cases, reductions can be made with smooth transitions such as commercially available reducers, at the discretion of the designer.

8.4 HYDROLOGY

The Rational method is the most common method in use for design of storm drains when momentary peak flow rate is desired. Its use should be limited to systems with drainage areas of 200 acres or less. Drainage systems involving detention storage and pumping stations require the development of a runoff hydrograph. Other hydrological methods include unit hydrograph procedures, or SCS TR-55 procedures. Refer to the Hydrology Chapter for a discussion of these methods.

8.4.1 Rational Method

The Rational Equation is written as follows:

$$Q = CIA = \left(\sum CA\right) I \tag{8.1a}$$

Where: Q = peak runoff rate (cfs)
C = runoff coefficient
I = rainfall intensity (inches/hour)
A = drainage area (acres)

Runoff Coefficient

The runoff coefficient is a dimensionless value representing characteristics of the watershed that affect how much of the rain will become runoff. Coefficient selection is based on land use and soil conditions. The weighted C value is to be based on a ratio of the drainage areas associated with each C value. The runoff coefficients for various types of surfaces are provided in Table 3.7.

$$Weighted\ C = \frac{A_1C_1 + A_2C_2 + \ldots + A_nC_n}{A_1 + A_2 + \ldots + A_n} \tag{8.1b}$$

Rainfall Intensity

Rainfall intensity (I) is an average rainfall intensity for a duration equal to the time of concentration and for a select recurrence interval. Rainfall intensity is the intensity of rainfall in inches per hour for a duration equal to the time of concentration. Intensity is a rate of rainfall over an interval of time such that intensity multiplied by duration equals total amount of rain. The value of I for various concentration times and recurrence frequencies is provided in Section 3.5.4.

Time of Concentration

The time of concentration is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. The designer is usually concerned about two different times of concentration, one for inlet spacing and the other for pipe sizing. There is a major difference between the two times.

- Inlet Spacing
  The time of concentration (t_c) for inlet spacing is the time for water to flow from the hydraulically most distant point of the drainage area to the inlet. Usually this is the sum of the time required for water to move across the pavement or overland back of the curb to the gutter, plus the time required for flow to move through the length of gutter to the inlet. For pavement drainage, when the total time of concentration to the upstream inlet is less than seven minutes, a minimum t_c of seven minutes should be used to estimate the intensity of rainfall. The time of concentration for the second downstream inlet and each succeeding inlet should be determined independently, the same as the first inlet. Runby travel time between inlets is not considered. In the case of a constant roadway grade and relatively uniform contributing drainage area, the time of concentration for each succeeding inlet could also be constant.

- Pipe Sizing
  The time of concentration for pipe sizing is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. It generally consists of two components, the time to flow to the inlet which can consist of overland and channel or gutter flow, and the time to flow through the storm drain to the point under consideration.

The preferred method of calculating overland flow time is the kinematic wave approach. Channel travel time of concentration can be developed using Manning's equation. Travel time within the storm drain pipes can be estimated by the relation:

$$t_t = \frac{L}{60V} \qquad \textbf{(8.2)}$$

Where: t_t = travel time (min)
L = length of pipe in which runoff must travel (ft)
V = estimated or calculated velocity (ft/s)
Velocity is based upon normal depth of flow for the design discharge.

To summarize, the time of concentration for any point on a storm drain is the inlet time for the inlet at the upper end of the line plus the time of flow through the storm drain from the upper end of the storm drain to the point in question. In general, where there is more than one source of runoff to a given point in the storm drainage system, the longest t_c is used to estimate the intensity (I). There could be exceptions to this generality, for example where there is a large inflow area at some point along the system, the t_c for that area may produce a larger discharge than the t_c for the summed area with the longer t_c. The designer should be cognizant of this possibility when joining drainage areas and determine which drainage area governs. To determine which drainage area controls, compute the peak discharge for each t_c and the one that produces the larger discharge controls. Note that when computing the peak discharge with the shorter t_c, not all the area from the basin with the longest t_c will contribute runoff.

$$A_c = A\left(\frac{t_{c1}}{t_{c2}}\right) \qquad \textbf{(8.3)}$$

Where: A_c = contributing area
A = area of the basin with longest time of concentration (t_{c2})
t_{c1} = smaller time of concentration
t_{c2} = larger time of concentration

8.4.2 Detention

When the Rational Method is utilized, detention is accommodated tô some extent by the process of building a design flow moving downstream through the storm drainage system as described later in this chapter. As the time of concentration is recalculated at each manhole by adding in the time of travel between manholes, a new lower rainfall intensity and new discharge is calculated. In a sense, the effect of detention by the carrier pipe itself is reflected in the design.

A more direct application of detention involves the use of detention ponds, usually located in the contributing watersheds and often immediately upstream of the entrance to the storm drainage system. By introducing detention ponds, the designer is able to attenuate the peak of the runoff hydrograph, thus reducing the immediate design discharge rate. Estimation of the effects of detention requires a reservoir routing procedure such as that presented in the Storage Facilities Chapter.

8.5 PAVEMENT DRAINAGE

Roadway features considered during gutter, inlet, and pavement drainage calculations include:

- longitudinal slope,
- cross slope,
- curb and gutter sections,
- roadside and median ditches,
- pavement texture, and
- bridge decks.

The pavement width, cross slope, and profile control the time it takes for storm water to drain to the gutter section. The gutter cross section and longitudinal slope control the quantity of flow which can be carried in the gutter section.

8.5.1 Longitudinal Slope

A minimum longitudinal gradient is more important for curbed pavement than for uncurbed pavement since the curbed section is susceptible to the spread of stormwater against the curb. Flat gradients on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge. To assure adequate drainage a minimum gutter grade of 0.35 percent is desirable.

To provide adequate drainage in sag vertical curves, a minimum slope of 0.3 percent is desirable within 50 feet of the level point in the curve. This is accomplished where the length of the curve divided by the algebraic difference in grades is equal to or less than 167. Although ponding is not usually a problem at crest vertical curves, on extremely flat curves a similar minimum gradient should be provided to facilitate drainage.

8.5.2 Cross Slope

Pavement cross slope should be adequate to provide proper drainage. The design rate of cross slope will vary with surface type, and road classification. The Mn/DOT Road Design Manual contains accepted values for pavement cross slopes.

On multi-lane divided highways, each roadway may be crowned separately or all lanes of each roadway may flow in one direction. Consider the following when selecting the cross slope direction:

- From a drainage and wet pavement safety standpoint, the preferred alternate is the crowned cross section with drainage both ways from the crown. This cross section will drain the pavement the quickest and the difference between the high and low points will be minimized. This alternate requires drainage design for both sides of the highway, and may complicate at-grade intersection traversability because of the several ups and downs of the cross section, although this can be lessened by transitioning to flatter cross slopes through the intersection. Therefore this alternate is most suited for divided highways with wide depressed medians and full or partial control of access.
- An uni-directional slope towards the outside edge will reduce traversability problems at at-grade intersections. It will require drainage design for one side of each roadway. With each lane contributing runoff the potential for hydroplaning is increased. During freeze-thaw periods, a safety problem can be created when snow, which has been plowed to the median, melts and drains across the travel lanes. Slippery conditions can be created when the melting runoff refreezes.

A careful check should be made of designs to minimize the number and length of flat pavement sections in super elevation transition areas where the cross slope is approaching zero. Since cross slope is the dominant factor in removing water from the pavement, the designer should be aware that hydroplaning is a potential hazard to be considered in these areas especially when combined with flat profile grades and wide pavement sections.

8.5.3 Pavement Texture

The pavement texture is an important consideration for roadway surface drainage. Although the hydraulic design engineer will have little control over the selection of the pavement type or its texture, it is important to know that pavement texture does have an impact on the buildup of water depth on the pavement during rain storms. A good macro texture provides a channel for water to escape from the tire-pavement interface and reduces the potential for hydroplaning.

A high level of macrotexture may be achieved by tining new portland cement concrete pavements while it is still in the plastic state. Retexturing of an existing portland cement concrete surface can be accomplished through pavement grooving. Both longitudinal and transverse grooving are very effective in achieving macrotexture in concrete pavement. Transverse grooving aids in surface runoff resulting in less wet pavement time. Combinations of longitudinal and transverse grooving provide the most adequate drainage for high speed conditions.

8.5.4 Curb and Gutter

Curbing at the outside edge of pavements is normal practice for low-speed, urban highway facilities. The curbs serve several purposes which include containing the surface runoff within the roadway and away from adjacent properties, preventing erosion, providing pavement delineation, and enabling the orderly development of property adjacent to the roadway.

The Road Design Manual is recommended for guidance in selecting a particular curb and gutter. A curb and gutter forms a triangular channel that can be an effective hydraulic conveyance facility which can convey runoff of a lesser magnitude than the design flow without interruption of the traffic. Where curbs are used, composite gutter sections can be effective at increasing gutter capacity and reducing spread on the pavement. When a design storm flow occurs, there is a spread or widening of the conveyed water surface and the water spreads to include not only the gutter width, but also parking lanes or shoulders, and portions of the traveled surface. This is the width the hydraulic engineer is most concerned about in curb and gutter flow, and limiting this width becomes a very important design criterion.

Where practicable, it is desirable to intercept runoff from cut slopes and other areas draining toward the roadway before it reaches the highway, in order to minimize the deposition of sediment and other debris on the roadway and to reduce the amount of water which must be carried in the gutter section.

8.5.5 Roadside and Median Ditches

Roadside ditches are commonly used with uncurbed roadway sections to convey runoff from the highway pavement and from areas which drain toward the highway. Due to right-of-way limitations, roadside ditches cannot be used on most urban arterials. They can be used in cut sections, depressed sections, and other locations where sufficient right-of-way is available and driveways or intersections are infrequent. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted in the ditch as appropriate. It is preferable to slope median areas and inside shoulders to a center swale, to prevent drainage from the median area from running across the pavement. This is particularly important for high-speed facilities, and for facilities with more than two lanes of traffic in each direction.

8.5.6 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. Deck drainage is often less efficient, because cross slopes are flatter, and small drainage inlets or scuppers have a higher potential for clogging by debris. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 0.5 percent. If a sag curve cannot be avoided, the sag should not occur directly over a water body unless it is a treatment pond. Deck drainage can generally not be discharged directly to a water body without providing a pond or other means of intercepting a hazardous spill. Runoff should be handled in compliance with applicable stormwater quality regulations.

The gutter spread should be checked to insure compliance with the design criteria in Section 8.2. A large number of bridges will not require any drainage structures at all. To determine the length of deck permitted to achieve the allowable spread, the following equation which is based on a uniform cross slope can be utilized:

$$L = \frac{24400\left(S_x^{1.67}\right)\left(S^{0.5}\right)\left(T^{2.67}\right)}{CnIW} \tag{8.4}$$

Where: S = Longitudinal slope (ft/ft)
S_x = cross slope (ft/ft)
W = width of drained deck (ft)
C = runoff coefficient
I = rainfall intensity (in/hr)
n = Manning's n
T = Allowable spread (ft)

The use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over embankments, slope pavement, slope protection, navigation channels, driving lanes, or railroad tracks. For bridges that require deck drains, it may be necessary to provide a drainage system on the bridge to convey water off the bridge.

8.5.7 Median and Median Barriers

Medians are commonly used to separate opposing lanes of traffic on divided highways. It is preferable to slope median areas and inside shoulders to a center depression to prevent drainage from the median area from running across the traveled pavement. Where median barriers are used and, particularly on horizontal curves with associated super-elevations, it is necessary to provide inlets and connecting storm drains to collect the water which accumulates against the barrier. Locating the center of the grate inlet 2.0′ from the edge of barrier as shown in Figure 8.1, has the advantage of allowing approximately 1′ for storage of snow and ice during snow removal operations.

8.5.8 Impact Attenuators

The location of impact attenuator systems should be reviewed to determine the need for drainage structures in these areas. With impact attenuator systems, it is often necessary to have a clear or unobstructed opening as traffic approaches the point of impact to allow a vehicle to impact the system head on. If the impact attenuator is placed in an area where superelevation or other grade separation occurs, grate inlets and/or slotted drains may need to be placed to prevent water from running through the clear opening and crossing the highway lanes or ramp lanes.

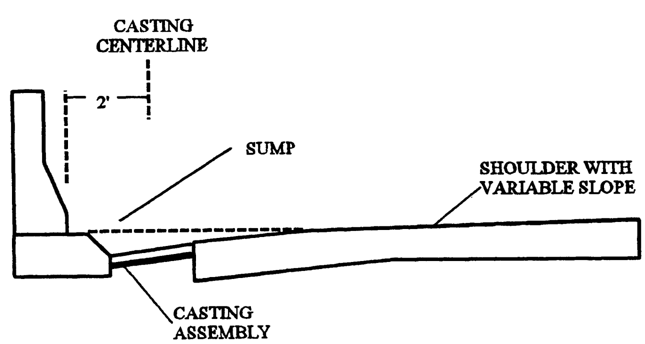


Figure 8.1 Median Barrier Inlet

8.6 GUTTER FLOW

Gutter flow calculations are necessary in order to relate the quantity of flow (Q) in the curbed channel to the spread of water on the shoulder, parking lane, or pavement section. Typical gutter section shapes include: uniform cross slope channels, composite gutter sections, and V shape gutter sections. Either nomographs (see Figures 8.2, 8.3 and 8.4), programable calculators or computer programs can be utilized to perform gutter flow calculations.

$$Q = \frac{0.56}{n} S_x^{1.67} S^{0.5} T^{2.67} \tag{8.5}$$

Where: Q = gutter flow (cfs)
S_x = cross-slope (ft/ft)
S = longitudinal slope (ft/ft)
T = spread (ft)
n = Manning's n

Table 8.2 Manning's n for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.012
Asphalt Pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, increase above values of n by	0.002

Source: HDS-3 (FHWA, 1961)

Two sets of procedures are provided, CONDITION 1 where gutter flow (Q) is known and spread (T) is calculated and CONDITION 2 where spread (T) is known and gutter flow (Q) is calculated.

8.6.1 Uniform Cross Slope Procedure

The nomograph in Figure 8.3 is used with the following procedures to find gutter capacity for uniform cross slopes:

CONDITION 1: Given gutter flow (Q), find spread (T).

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n.

Step 4 Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

CONDITION 2: Given spread (T), find gutter flow (Q).

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Q_n from the intersection of that line on the capacity scale.

Step 4 For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n (Q_n) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

8.6.2 Composite Gutter Sections Procedure

Figure 8.4 can be used to find the flow in a gutter section with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

CONDITION 1: Given gutter flow (Q), find spread (T).

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter section cross slope (S_w), gutter section width (W), Manning's n, total flow (Q), and a trial value of the curbed channel capacity above the gutter section (Q_s).

Step 2 Calculate the gutter flow (Q_w) in the gutter section over width, W, using the equation:

$$Q_w = Q - Q_s \qquad \textbf{(8.6)}$$

Where: Q_w = gutter flow (cfs)
Q = total flow (cfs)
Q_s = capacity above the gutter section (cfs)

Step 3 Calculate the ratios Q_w/Q and S_w/S_x and use Figure 8.4 to find an appropriate value of W/T.

Step 4 Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.

Step 5 Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4.

Step 6 Use the value of T_s from Step 5 along with Manning's n, longitudinal slope (S), and cross slope (S_x) to find the actual value of Q_s from Figure 8.3.

Step 7 Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

CONDITION 2: Given spread (T), find gutter flow (Q).

Step 1 Determine input parameters, including spread (T), spread above the gutter section (T_s), cross slope (S_x), longitudinal slope (S), gutter section cross slope (S_w), gutter section width (W), Manning's n, and depth of gutter flow (d).

Step 2 Use Figure 8.3 to determine the capacity of the curbed channel above the gutter section flow (Q_s). Use the procedure for uniform cross slopes - CONDITION 2, substituting T_s for T.

Step 3 Calculate the ratios W/T and S_w/S_x, and, from Figure 8.2, find the appropriate value of E_o (the ratio of Q_w/Q).

Step 4 Calculate the total flow using the equation:

$$Q = \frac{Q_s}{(1 - E_o)} \tag{8.7}$$

Where: Q = total curbed channel flow rate (cfs)
Q_s = flow capacity of the curbed channel above the gutter section (cfs)
E_o = ratio of frontal flow to total flow (Q_w/Q)

Step 5 Calculate the gutter flow (Q_w) in width (W), using Equation 8.6.

NOTE: Figure 8.3 can also be used to calculate the flow in a composite gutter section.

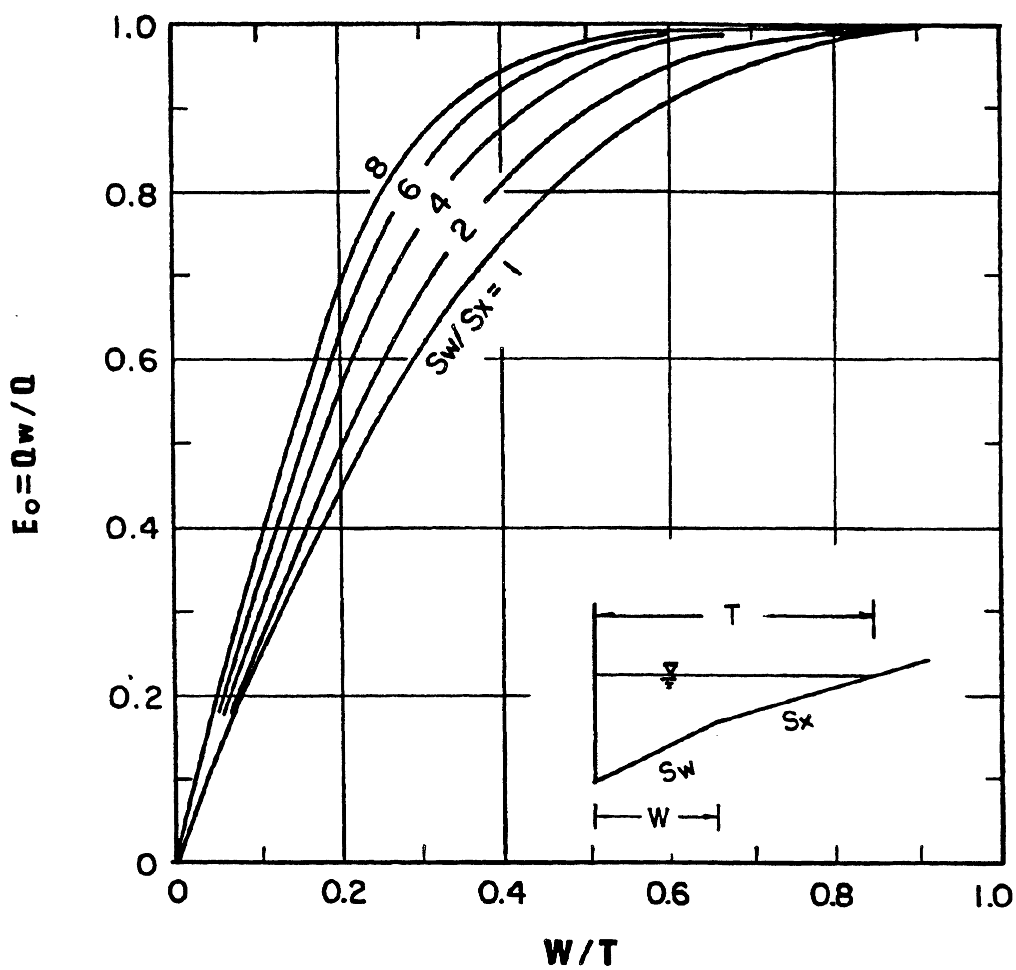


Figure 8.2 Ratio Of Frontal Flow To Total Gutter Flow
Source: HEC-12 (FHWA, 1984)

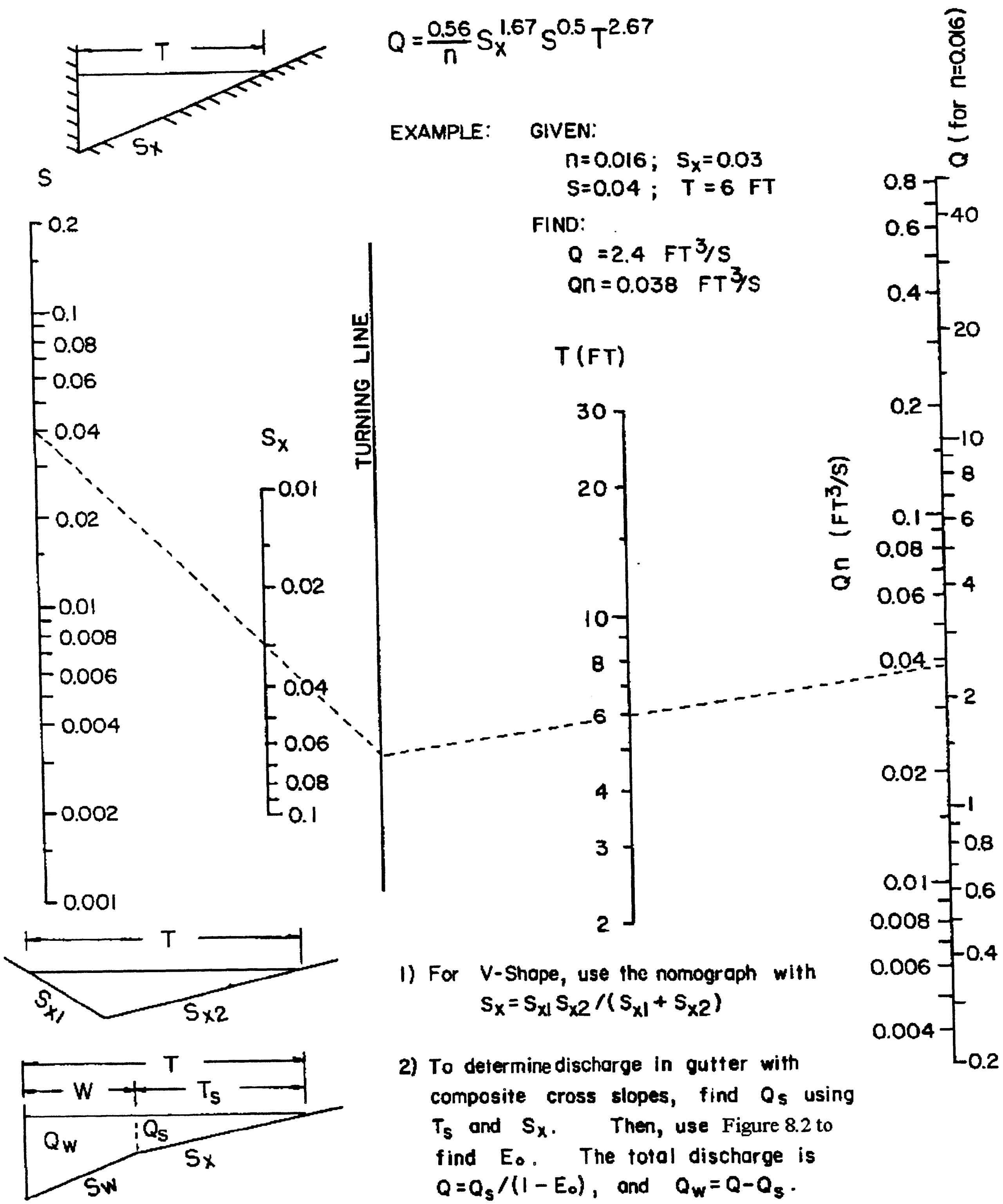


Figure 8.3 Flow in Triangular Gutter Sections
Source: HEC-12 (FHWA, 1984)

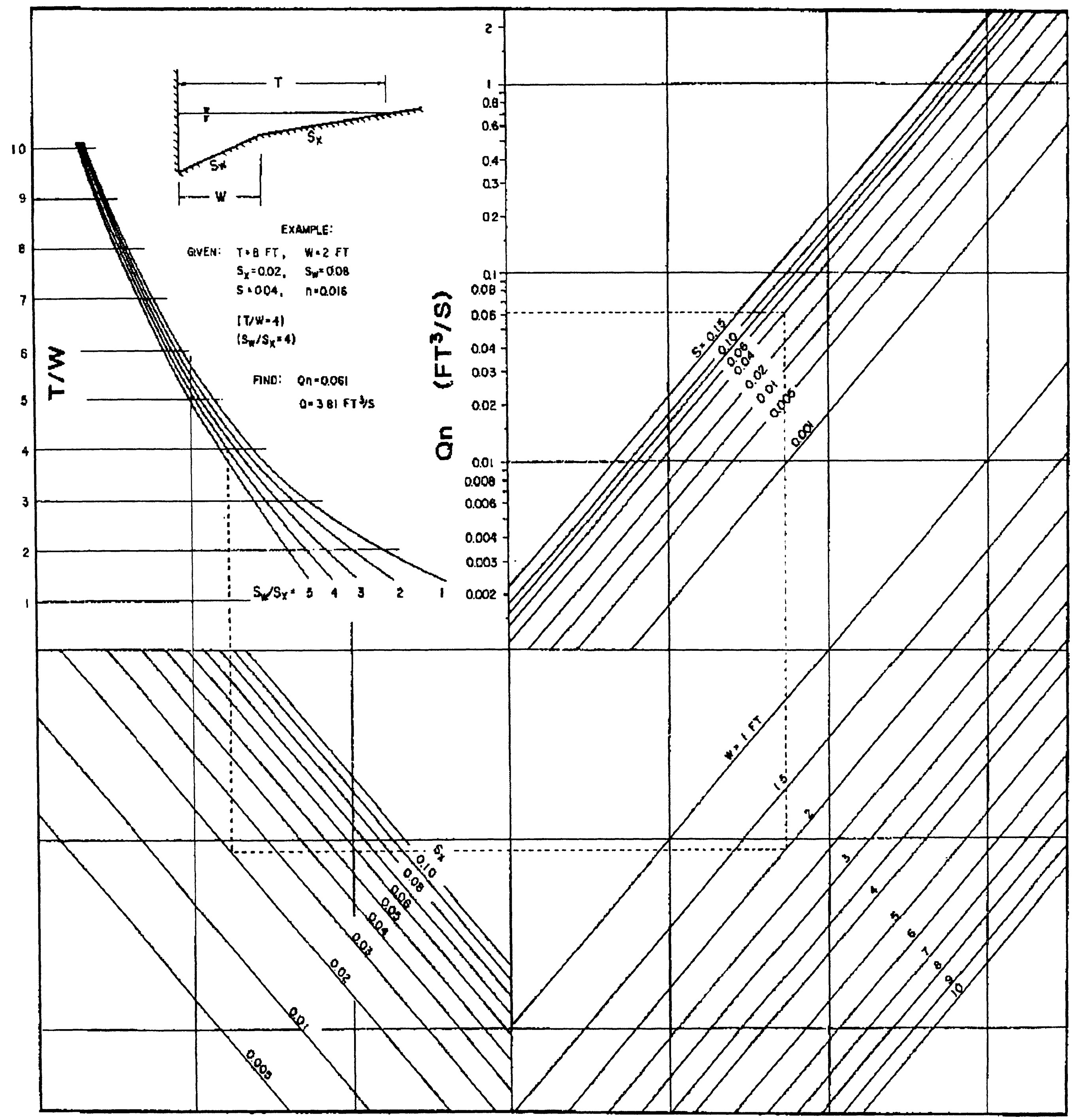


Figure 8.4 Flow in Composite Gutter Sections
Source: HEC-12 (FHWA, 1984)

8.6.3 V Type Gutter Sections Procedure

Figure 8.3 can also be used to solve V Type channel problems. The spread (T) can be calculated for a given flow (Q) or the flow can be calculated for a given spread. This method can be used to calculate flow conditions in the triangular channel adjacent to concrete median barriers. (See Figure 8.5)

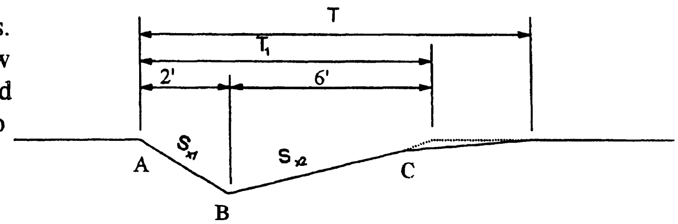


Figure 8.5 V Type Gutter Section

CONDITION 1: Given gutter flow (Q), find spread (T).

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), Manning's n, total flow (Q).

Step 2 Calculate S_x

$$S_x = \frac{S_{x1}S_{x2}}{\left(S_{x1} + S_{x2}\right)} \tag{8.8}$$

Where: S_x = cross slope (ft/ft)
S_{x1} = cross slope of section 1 (ft/ft)
S_{x2} = cross slope of section 2 (ft/ft)

Step 3 Solve for T_1 using the nomograph on Figure 8.3.
T_1 is a hypothetical width that is correct if it is contained within S_{x1} and S_{x2}.

Step 4 To find the actual spread, solve for depth at points B and C in Figure 8.5.

Step 5 Solve for the spread on the pavement.

Step 6 Find the actual total spread (T).

CONDITION 2: Given spread (T), find gutter flow (Q).

Step 1 Determine input parameters such as longitudinal slope (S), cross slope (S_x), Mannings n, and allowable spread.

Step 2 Calculate S_x using Equation 8.8.

Step 3 Using Figure 8.3 or Equation 8.5 to solve for Q.

8.7 INLETS

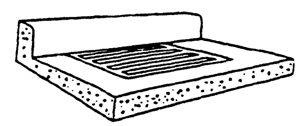
Inlets are drainage structures utilized to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts. Grate inlets should be bicycle safe unless located on highways where bicycles are not permitted. This section includes various types of inlets commonly in use and provides recommendations on the use of each type.

Table 8.3 lists the various types of inlets used by Mn/DOT. Comparisons are made between the various types for each of the inlet characteristics listed. This table should be reviewed prior to selecting a specific type of inlet or grate.

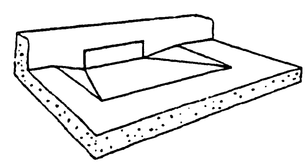
Table 8.3 Inlet Comparisons

Standard Plate	Curb Box	Bike Safe	Handle Debris	Sag Point	Max. Grade
4151 (811)	yes	yes	yes	yes[3]	2%
4152 (814)	no	yes	no	no	6%
4153 (815)	no[1]	no	yes	yes[3]	none
4154 (816)	no[1]	yes	yes	yes[3]	none
4021[2]	n/a	yes	yes	yes	0%
slotted inlet 3136, 3137, 3138	n/a	yes	yes	yes	n/a

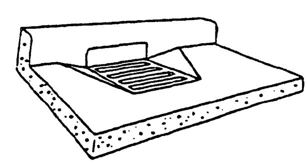
[1] Use curb box only when used at sag point.
[2] Use low point (LP) type only in sag point.
[3] Use with Frame 4125 at sag points.



Grate Inlet



Curb-Opening Inlet



Combination Inlet

Inlet Types

8.7.1 Inlet Types

Inlets used for the drainage of highway surfaces can be divided into four major classes.

Grate Inlets

These inlets consist of an opening in the gutter covered by one or more grates. They are best suited for use on continuous grades. Since they are susceptible to clogging with debris, the use of standard grate inlets alone at sag points should be limited to minor sag point locations without debris potential. Special design (oversize) grate inlets can be utilized at major sag points if sufficient capacity is provided for clogging. In this case, flanking inlets are definitely recommended. Grates should be bicycle safe where bike traffic is anticipated and should be structurally designed to handle the appropriate loads when subject to traffic.

Curb-Opening Inlets

These inlets are vertical openings in the curb covered by a top slab. They are best suited for use at sag points and flat grades since they can convey large quantities of water and debris. They are generally not recommended for use on steep continuous grades.

Combination Inlets

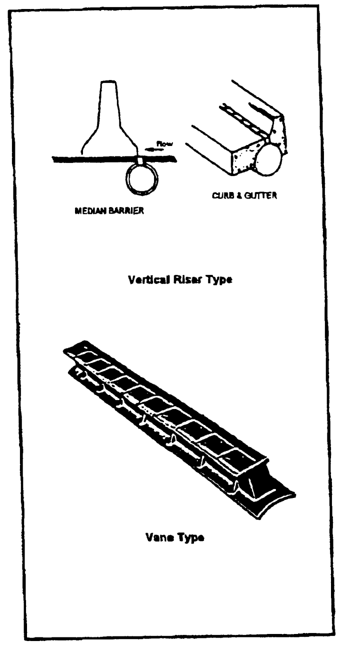
Various types of combination inlets are in use. Curb opening and grate combinations are in common use by Mn/DOT. Slotted inlets may also be used in combination with grates. In general, the vertical riser type is used for longitudinal placement upstream of the grate, while the vane type is used transversely to the grate. Engineering judgement is necessary to determine if the total capacity of the inlet is the sum of the individual components or a portion of each. The gutter grade, cross slope and proximity of the inlets to each other will be deciding factors. Combination grate and curb-opening inlets are desirable in sags because they can provide additional capacity in the event of plugging.

Slotted Drain Inlets
These inlets consist of a slotted opening with bars perpendicular to the opening. Slotted inlets function as weirs with flow entering from the side. They can be used to intercept sheet flow, collect gutter flow with or without curbs, modify existing systems to accommodate roadway widening or increased runoff, and reduce ponding depth and spread at grate inlets. The two types of slotted inlets in general use are the vertical riser type and the vane type.

8.7.2 Inlet Locations

Inlets are required at locations needed to collect runoff within the design controls specified in the design criteria (Section 8.2). In addition, there are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or runby. Examples of such locations are as follows:

- sag points in the gutter grade,
- upstream of median breaks, entrance/exit ramp gores, cross walks and street intersections,
- immediately upstream and downstream of bridges,
- immediately upstream of cross slope reversals,
- on side streets at intersections,
- at the end of channels in cut sections,
- behind curbs, shoulders, or sidewalks to drain low areas,
- where necessary to collect snow melt.



Slotted Inlets

Inlets should be placed in the correct position, typically in the gutterline. In superelevated roadways it can be challenging to place inlets so that spread criteria is met, and that pavement flow is not excessive. If possible, inlets should not be located in the path where pedestrians are likely to walk.

8.7.3 Inlet Spacing

Inlets are often required to collect runoff at locations with small or insignificant drainage areas, these should be plotted on the plan first. Next, it is best to start locating inlets from the crest of the profile and work down grade to the sag points. The location of the first inlet from the crest is found by determining the length of pavement and the area back of the curb sloping toward the roadway which will generate the design runoff. The design runoff can be computed as the maximum allowable flow in the curbed channel which will meet the design criteria as specified in Section 8.2. Where the contributing drainage area consists of a strip of land parallel to and including a portion of the highway, the first inlet can be calculated with an alternate form of the Rational Equation as follows:

$$L = \frac{43560 Q_t}{CIW} \tag{8.9}$$

Where: L = distance from the crest (ft)
Q_t = maximum allowable flow (cfs)
C = composite runoff coefficient for contributing drainage area
W = width of contributing drainage area (ft)
I = rainfall intensity for design frequency (in/hr)

If the drainage area contributing to the first inlet is irregular in shape, trial and error computations will be necessary to match a design flow with the maximum allowable flow. To space successive down grade inlets, it is necessary to compute the amount of flow which will be intercepted by the inlet (Q_i) and subtract it from the total gutter flow to compute the runby. The runby from the first inlet is added to the computed flow to the second inlet, the total of which must be less than the maximum allowable flow dictated by the criteria. Table 8.5 is an inlet spacing computation sheet which can be utilized to record the spacing calculations, alternatively inlet spacing can be performed using a computer application.

8.7.4 Grate Inlets on Grade

The capacity of a grate inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted.

On steep slopes, a portion of the frontal flow may tend to splash over the end of the grate. Mn/DOT grate 4151 is the most prone for splash over particularly on grades steeper than 2%. When using different grate lengths or types than those listed below, consult the manufacturer to get the inlet grate capacity (Neenah, 1987), or refer to HEC-12 (FHWA, 1984) for additional information on computing V_o. Although FHWA grates are not the same as the grates utilized by Mn/DOT two of them are sufficiently similar to be considered equivalent hydraulically.

Grate Type	Mn/DOT Grate Standard Plate No.	Equivalent FHWA Grate	Splash-Over Velocity, V_o [1]
Parallel Bar	4153	P-1-7/8	8 fps
Curved Vane	4152 and 4154	Curved Vane	6 fps
Reticuline	None	Reticuline	4 fps

[1] Splash over velocity for grates 2 foot in length.

Frontal Flow Ratio

The ratio quantifies how much of the total gutter flow runs directly over the front edge of the grate. The ratio of frontal flow to total gutter flow, E_o, for straight cross slope is expressed by Equation 8.10a. Figure 8.2 provides a graphical solution of E_o for either straight cross slopes or depressed gutter sections.

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \qquad \textbf{(8.10a)}$$

Side Flow Ratio

The ratio quantifies how much of the total gutter flow does not pass over the front edge of the grate, but passes along the side of the grate. The ratio of side flow, Q_s, to total gutter flow is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \qquad \textbf{(8.10b)}$$

Intercepted Frontal Flow Ratio

This ratio is equivalent to frontal flow interception efficiency. Figure 8.7 is a nomograph to solve for velocity in a triangular gutter section with known cross slope, longitudinal slope and spread. R_f cannot exceed 1.0. The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.09(V - V_o) \qquad \textbf{(8.10c)}$$

Intercepted Side Flow Ratio

The ratio is equivalent to the side flow interception efficiency of the grate. Figure 8.6 provides a graphical solution to Equation 8.10d. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by:

$$R_s = \frac{1}{\left[1 + \left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)\right]} \qquad \textbf{(8.10d)}$$

Where: E_o = ratio of frontal flow to total gutter flow
Q_w = flow in width W (cfs)
Q = total gutter flow (cfs)
W = width of depressed gutter or grate (ft)
T = total spread of water in the gutter (ft)
Q_s = ratio of side flow to total gutter flow
R_f = ratio of frontal flow intercepted to total frontal flow
V = velocity of flow in the gutter (fps)
V_o = gutter velocity where splash-over first occurs (fps)
R_s = ratio of side flow intercepted to total side flow
L = length of the grate (ft)
S_x = cross slope

To determine the efficiency of the grate at capturing gutter flow the frontal and side flow interception efficiencies are summed. The efficiency (E) of a grate is expressed as:

$$E = R_f E_o + R_s(1 - E_o) \tag{8.11}$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\left[R_f E_o + R_s(1 - E_o)\right] \tag{8.12}$$

Where: Q_i = flow intercepted by grate (cfs)
E = grate efficiency
Q = total gutter flow (cfs)
R_f = ratio of frontal flow intercepted to total frontal flow
E_o = ratio of frontal flow to total gutter flow
R_s = ratio of side flow intercepted to total side flow

Inlet interception capacity has been investigated by the FHWA. The grates tested in an FHWA research study are described in *Drainage of Highway Pavements*, HEC-12 (FHWA, 1984). These grates are not the same as the grates utilized by Mn/DOT, though two of them are sufficiently similar for on grade installations: FHWA P-1-7/8 and Mn/DOT grate 4153, Curved Vane and Mn/DOT 4152 or 4154.

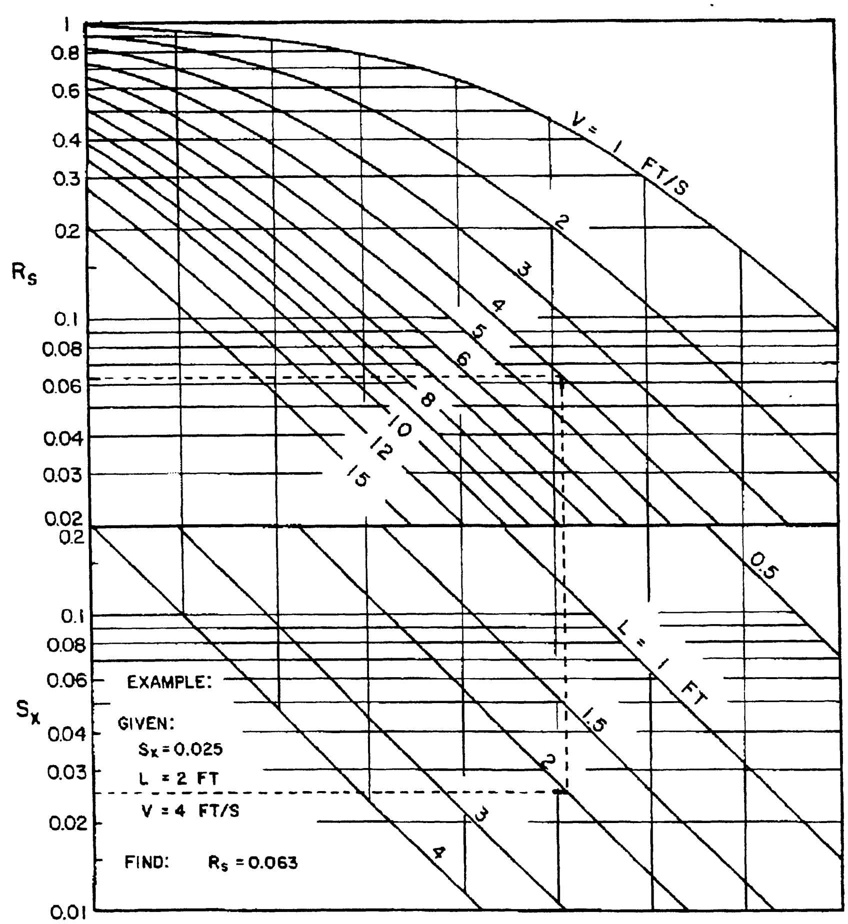


Figure 8.6 Grate Inlet Side Flow Interception Efficiency
Source: HEC-12 (FHWA, 1984)

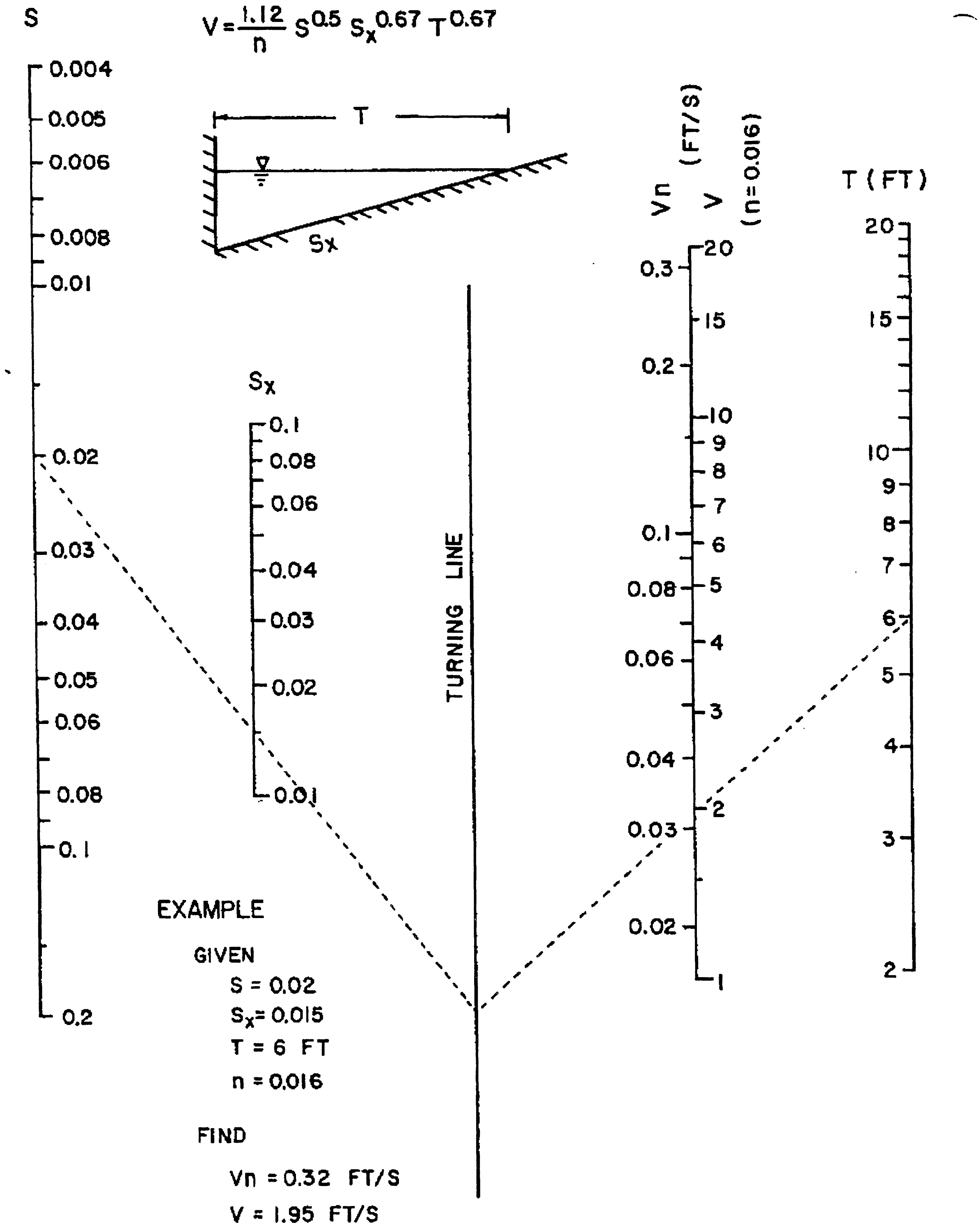


Figure 8.7 Velocity in Triangular Gutter Sections
Source: HEC-12 (FHWA, 1984)

8.7.5 Grate Inlets in Sag

Although curb opening inlets are generally preferred to grate inlets at a sag point, grate inlets can be used successfully. For minor sag points where debris potential is limited, grate inlets without a curb box can be utilized. An example of a minor sag point might be on a ramp as it joins a mainline. Curb boxes in addition to a grate are preferred at sag points where debris is likely such as on a city street. For major sag points such as on divided high speed highways, a curb opening inlet is preferable to a grate inlet because of its hydraulic capacity and debris handling capabilities. Where this is not practical such as adjacent to median barriers, special design grate inlets can be used successfully. In such instances, it is good practice to assume half the grate is plugged with debris. Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place a minimum of one flanking inlet on each side of the sag point inlet.

A grate inlet in a sag operates as a weir at smaller depths and as an orifice for greater depths. Between these depths, a transition from weir to orifice flow results in interception capacity less than that computed by either the weir equation or the orifice equation.

Weir

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

$$Q_i = C_w P d^{1.5} \tag{8.13}$$

Where: P = perimeter of grate disregarding the side against curb (ft)
C_w = weir coefficient (3.0)
d = depth of water above grate (ft)

Orifice

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

$$Q_i = C_o A (2gd)^{0.5} \tag{8.14}$$

Where: C_o = orifice coefficient (0.67)
A = clear opening area of the grate (ft^2)
g = acceleration due to gravity (32.2 ft/s^2)
d = depth of water above grate (ft)

Figure 8.8 is a plot of Equations 8.13 and 8.14 for various grate sizes. Figure 8.8 was developed with the assumptions that there was no curb box, sump or plugging. The sump is not included when computing depth of water above grate, d.

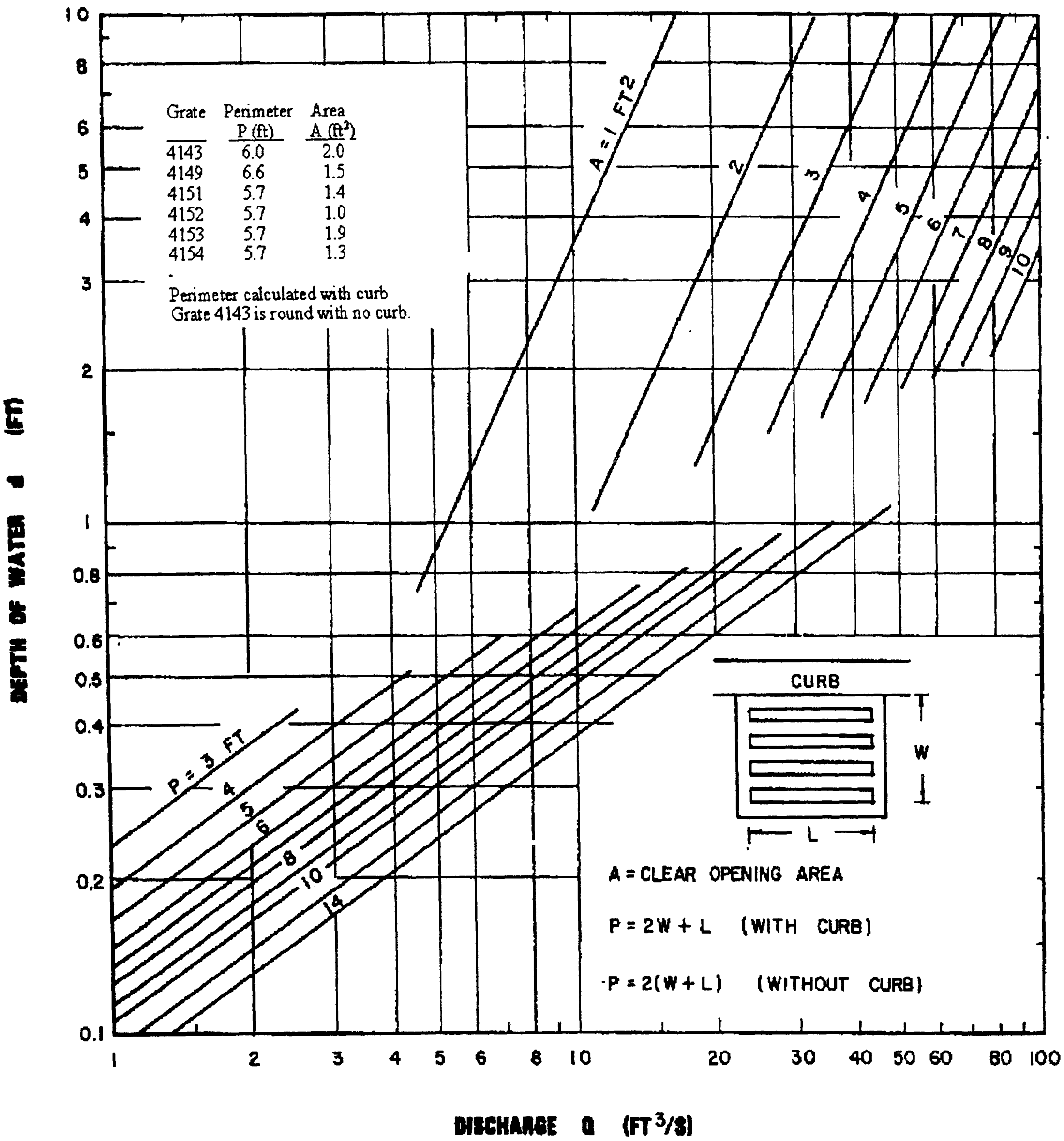


Figure 8.8 Grate Inlet Capacity In Sump Conditions
Source: HEC-12 (FHWA, 1984)

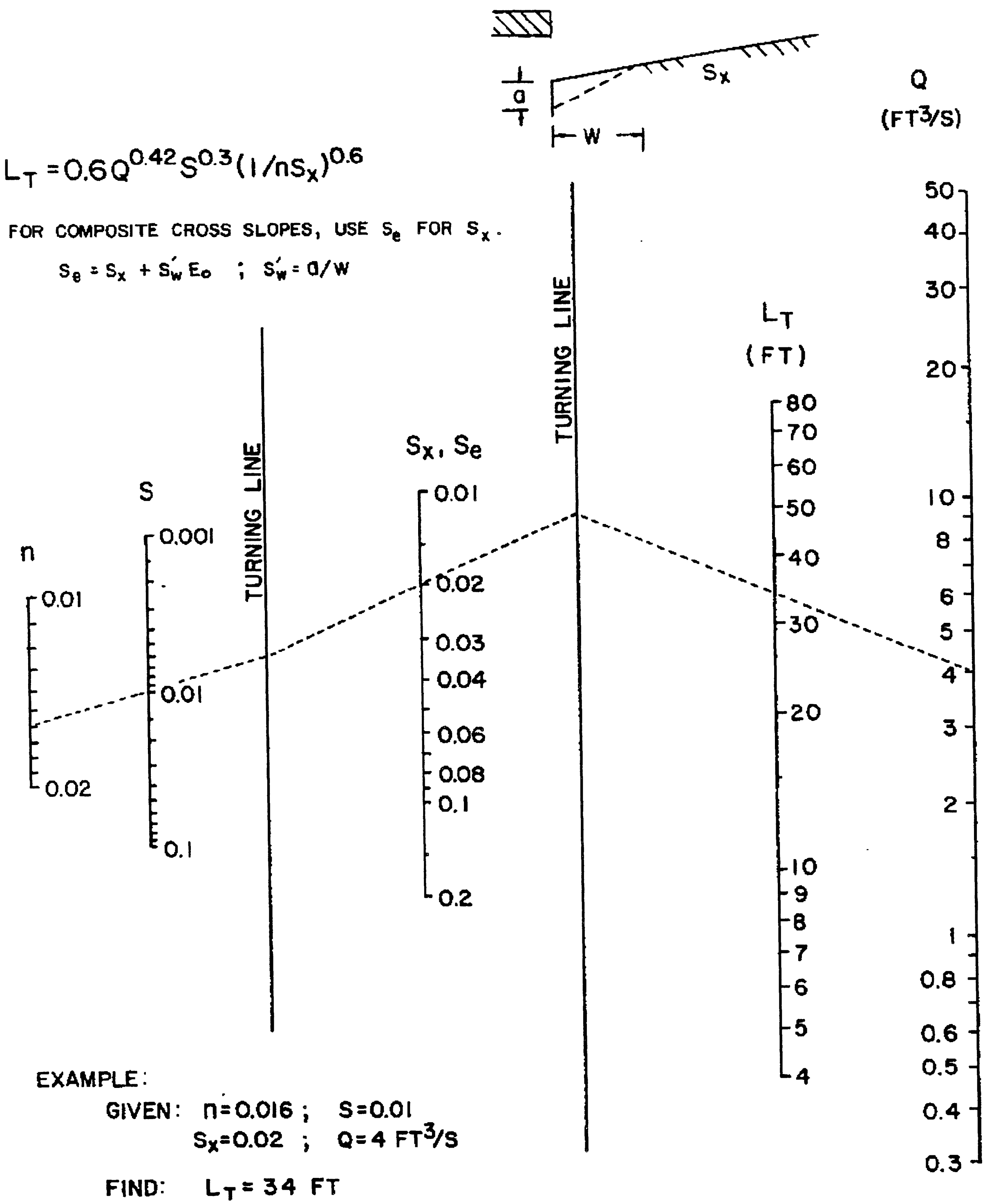


Figure 8.9 Curb Opening and Longitudinal Slotted Drain Inlet Length For Total Interception
Source: HEC-12 (FHWA, 1984)

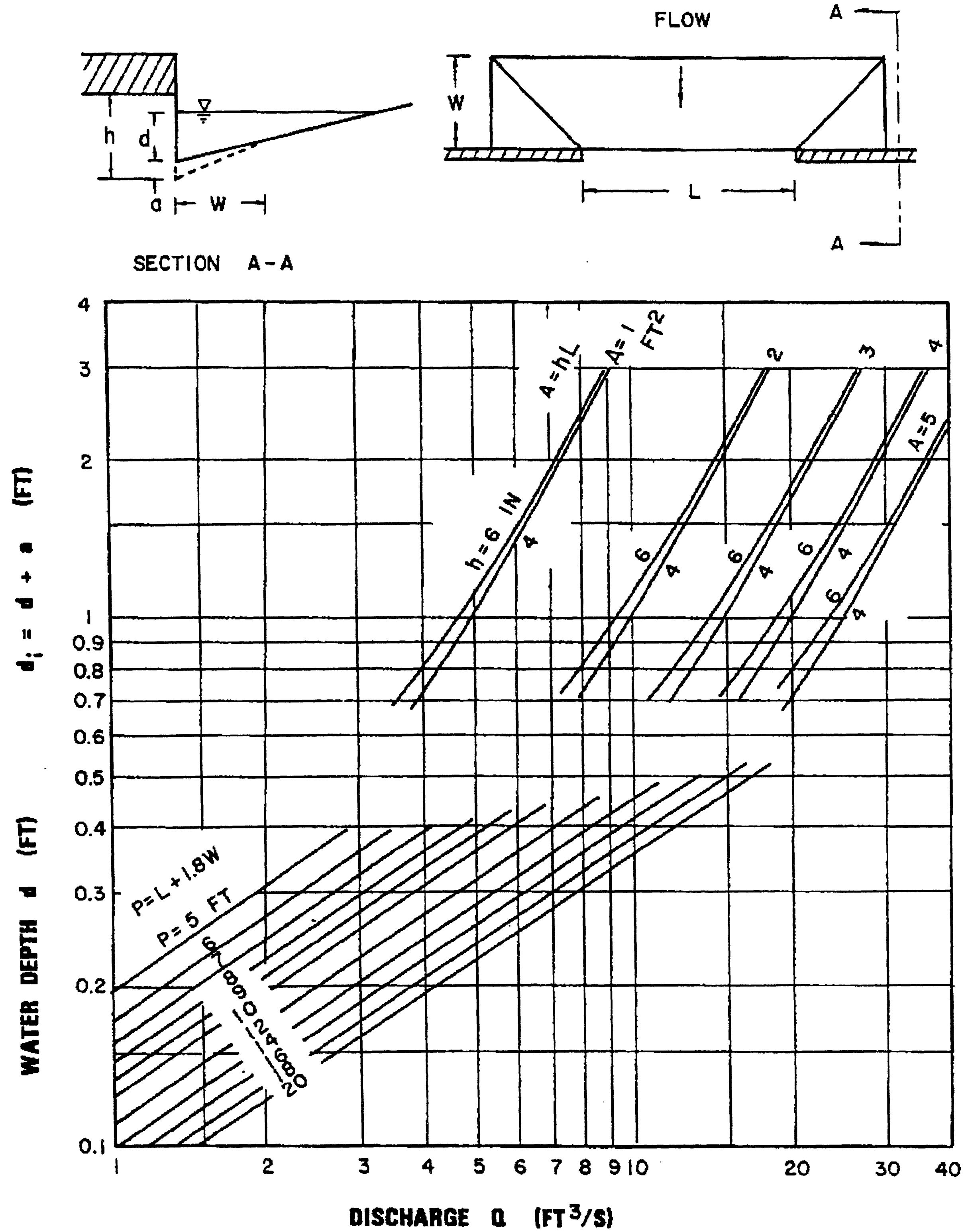


Figure 8.10 Depressed Curb-Opening Inlet Capacity In Sump Locations
Source: HEC-12 (FHWA, 1984)

8.7.6 Curb Inlets in Sag

The capacity of a curb-opening inlet in a sag depends on water depth at the curb, the curb opening length, and the height of the curb opening. The inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

Weir

Curb opening inlets operate as a weir when $d \leq h$. The equation for the interception capacity of a depressed curb opening inlet operating as a weir is:

$$Q_i = C_W\left(L + 1.8W\right)d^{1.5} \quad \textbf{(8.15a)}$$

The weir equation for curb-opening inlets without depression becomes:

$$Q_i = C_W L d^{1.5} \quad \textbf{(8.15b)}$$

Where: C_w = weir coefficient (2.3)
L = length of curb opening (ft)
W = width of depression (ft)
d = depth of water at curb measured from the normal cross slope gutter flow line (ft)

Orifice

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 h. Equation 8.16 is applicable to depressed and undepressed curb-opening inlets and the depth at the inlet includes any gutter depression. The interception capacity can be computed by:

$$Q_i = C_o A\left[2g\left(d_i - \frac{h}{2}\right)\right]^{0.5} \quad \textbf{(8.16)}$$

Where: C_o = orifice coefficient (0.67)
h = height of curb-opening orifice (ft)
A = clear area of opening, (ft^2)
d_i = depth at lip of curb opening (ft)
g = acceleration due to gravity (32.2 ft/s^2)

8.7.7 Slotted Inlets on Grade

Slotted inlets are effective pavement drainage inlets which have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations. They can be placed longitudinally in the gutter or transversely to the gutter. Slotted inlets should generally be connected into inlet structures so they will be accessible to maintenance forces in case of plugging or freezing.

Longitudinal Placement

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Slotted inlets may have economic advantages in some cases and could be very useful on widening and safety projects where right of way is narrow and existing drainage systems must be supplemented. In some cases, curbs can be eliminated as a result of utilizing slotted inlets.

The length of a slotted inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = KQ^{0.42}S^{0.3}\left(\frac{1}{nS_x}\right)^{0.6} \tag{8.17}$$

Where: K = coefficient (0.6)
L_T = slotted inlet length required to intercept 100% of the gutter flow (ft)
Q = gutter flow (cfs)
S = longitudinal slope
S_x = cross slope
n = Manning's n

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

$$E = 1-\left(1-\frac{L}{L_T}\right)^{1.8} \tag{8.18}$$

Where: L = slotted inlet length (ft)
E = slotted inlet efficiency
L_T = slotted inlet length required to intercept 100% of the gutter flow (ft)

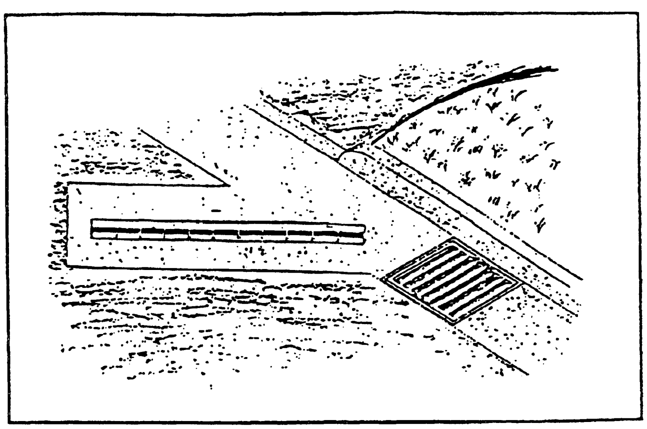
The length of inlet required for total interception by a slotted inlet in a composite section can be found by the use of an equivalent cross slope, S_e.

$$S_e = S_x + S'_w E_o \tag{8.19}$$

Where: S_x = pavement cross slope
S_w = gutter cross slope
S'_w = S_w - S_x
E_o = ratio of flow in the gutter section to total curbed channel flow, Q_w/Q

Transverse Placement

At locations where it is desirable to capture virtually all of the flow in the curbed section, a slotted vane drain can be installed in conjunction with a grate inlet. Tests have indicated that when the slotted vane drain is installed perpendicular to the flow, it will capture from 0 to 0.5 cfs per lineal foot of drain on longitudinal slopes of 0% to 6%. Capacity curves are available from the manufacturer. The ideal installation would utilize a grate inlet to capture the flow in the gutter and the slotted vane drain to collect the flow extending into the shoulder. Note that a slotted vane drain is shaped and rounded to increase inlet efficiency and should not be confused with a standard vertical riser type slotted inlet.



Slotted Vane Drain

8.7.8 Slotted Inlets in Sag

The use of slotted drain inlets in sag configurations is generally discouraged because of the propensity of such inlets to intercept debris in sags. However, there may be locations where it is desirable to supplement an existing low point inlet with the use of a slotted drain. Slotted inlets in sag locations perform as weirs to depths of about 0.2 feet, dependent on slot width and length. At depths greater than about 0.4 feet, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed by the following equation:

$$Q_i = 0.8LW(2gd)^{0.5} \tag{8.20}$$

Where: W = width of slot (ft)
L = length of slotted drain inlet (ft)
d = depth of water at slot (ft)
g = acceleration due to gravity (32.2 ft/s^2)

The interception capacity of slotted inlets at depths between 0.2 feet and 0.4 feet can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted inlet. Figure 8.11 provides solutions for weir flow and a plot representing data at depths between weir and orifice flow.

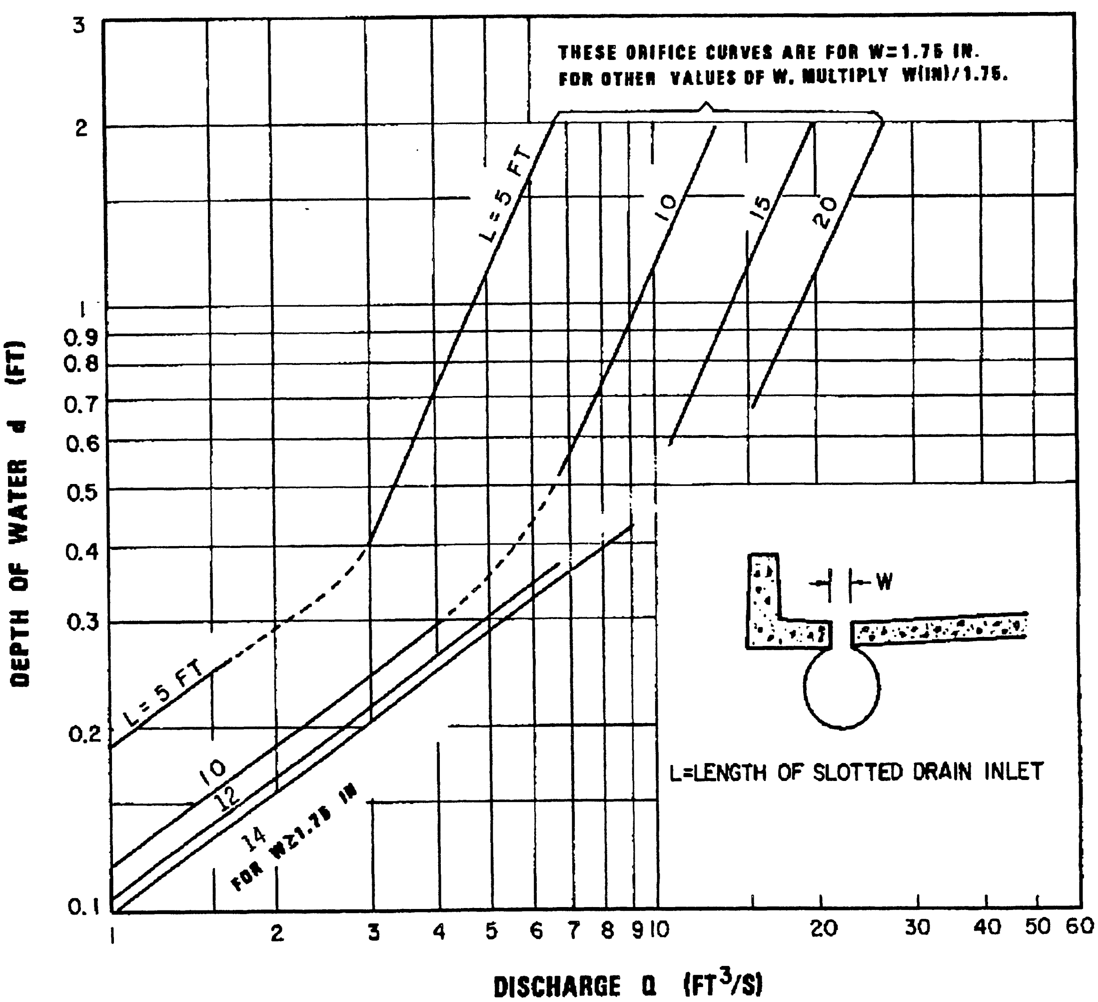


Figure 8.11 Slotted Drain Inlet Capacity In Sump Locations
Source: HEC-12 (FHWA, 1984)

8.7.9 Combination Inlet

The interception capacity of a combination grate/curb opening inlet on a continuous grade is not appreciably greater than that of a grate alone. Although Mn/DOT has used combination inlets for many years, we are now recommending that the curb box be dropped for all inlets placed on grade. The original intent of the curb box was to collect runoff in the event the grate became clogged with debris. However, experience has shown that the curb box tends to plug with debris and dirt, thus making it ineffective at capturing runoff and increasing maintenance costs. At minor sag points, combination inlets are recommended. The curb box is much less apt to plug and will provide some relief if the grate should become clogged. The capacity of a combination inlet in a sag is essentially the same as the grate alone in weir flow conditions unless the grate opening becomes clogged. In orifice flow, the capacity is equal to the total capacity of grate and curb opening.

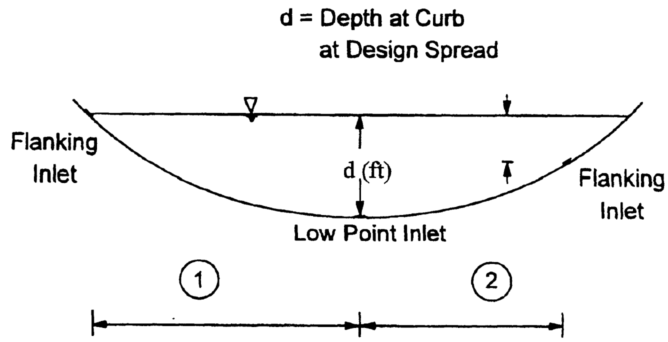


Figure 8.12 Flanking Inlets at Sag Point

8.7.10 Flanking Inlets

At major sag points where significant ponding may occur, such as underpasses or sag vertical curves in depressed sections, it is recommended practice to place a minimum of one flanking inlet on each side of the inlet at the sag point. The flanking inlets should be placed so they will limit spread on low gradient approaches to the sag and act in relief of the sag inlet if it should become clogged or if the design spread is exceeded. For major sag points, the flanking inlets are added as a safety factor, and not considered as intercepting flow to reduce the runby to the sag point. They are installed to assist the sag point inlet in the event of plugging.

Table 8.4 shows the spacing required for various depth at curb criteria and vertical curve lengths defined by K = L/A, where L is the length of the vertical curve in feet and A is the algebraic difference in approach grades. The AASHTO policy on geometrics specifies K values for various design speeds in miles per hour (MPH) and a maximum K of 167.

Table 8.4 Flanking Inlet Locations

Distance to flanking inlet in sag vertical curve locations using depth at curb criteria.												
Speed (MPH)	20	25	30	35	40	45	50	55	60	*	65	70
d (ft)↓ K→	20	30	40	50	70	90	110	130	160	167	180	220
0.1	20	24	28	32	37	42	47	51	57	58	60	66
0.2	28	35	40	45	53	60	66	72	80	82	85	94
0.3	35	42	49	55	65	73	81	88	98	100	104	115
0.4	40	49	57	63	75	85	94	102	113	116	120	133
0.5	45	55	63	71	84	95	105	114	126	129	134	148
0.6	49	60	69	77	92	104	115	125	139	142	147	162
0.7	53	65	75	84	99	112	124	135	150	153	159	176
0.8	57	69	80	89	106	120	133	144	160	163	170	188

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

Source: HEC-12, Table 5 (FHWA, 1984)

8.7.11 Inlet Spacing Computation Procedures

In order to design the location of the inlets for a given project, information such as a layout or plan sheet suitable for outlining drainage areas, road profiles, typical cross sections, grading cross sections, super elevation diagrams, and contour maps are necessary. The inlet computation sheet, Table 8.5 can be used to document the computations. Alternatively several computer applications are available which are capable of performing inlet spacing computations. The assumptions and procedures incorporated in a computer application must be carefully evaluated before the designer uses the application. A step by step procedure is as follows:

Step 1		Complete the blanks on top of the sheet to identify the job by S.P., route, date and your initials.
Step 2		Mark on the plan the location of inlets which are necessary even without considering any specific drainage area. See Section 8.7.2 Inlet Locations for additional information.
Step 3		Start at one end of the job, at one high point and work towards the low point, then space from the other high point back to the same low point.
Step 4		Select a trial drainage area approximately 300′ to 500′ below the high point and outline the area including any area that may come over the curb. (Use drainage area maps.) Where practical, large areas of behind the curb drainage should be intercepted before it reaches the highway.
Step 5	*Col 1* *Col 2*	Describe the location of the proposed inlet by number and station in Columns 1 & 2. Identify the curb and gutter type in the Remarks Column 19. A sketch of the cross section should be provided in the open area of the computation sheet.
Step 6	*Col 3*	Compute the drainage area in acres and enter in Column 3.
Step 7	*Col 4*	Select a C value from Tables 3.7 or compute a weighted value based on area and cover type as described in the Hydrology Chapter and enter in Column 4.
Step 8	*Col 5*	Compute a time of concentration for the first inlet. This will be the travel time from the hydraulically most remote point in the drainage area to the inlet. See additional discussion in Section 8.4 and the Hydrology Chapter. The minimum time of concentration should be 7 minutes. Enter value in Column 5.
Step 9	*Col 6*	Using the Intensity-Duration-Frequency curves, Section 3.5.4, select a rainfall intensity at the t_c for the design frequency. Enter in Column 6.
Step 10	*Col 7*	Calculate Q by multiplying Column 3 X Column 4 X Column 6. Enter in Column 7.
Step 11	*Col 8*	Determine the gutter slope at the inlet from the profile grade - check effect of superelevation. Enter in Column 8.
Step 12	*Col 9* *Col 13*	Enter cross slope adjacent to inlet in Column 9 and gutter width in Column 13. Sketch composite cross slope with dimensions.
Step 13	*Col 11*	For the first inlet in a series (high point to low point) enter Column 7 in Column 11 since no previous runby has occurred yet.
Step 14	*Col 12* *Col 14*	Using Figure 8.3 or 8.4 or a computer model, determine the spread T and calculate the depth d at the curb by multiplying T times the cross slope(s). Compare with the allowable spread as determined by the design criteria in Section 8.2.2. If Column 12 is less than the curb height and Column 14 is near the allowable spread, continue on to Step 16. If not OK, expand or decrease the drainage area to meet the criteria, and repeat Steps 5 through 16. Continue these repetitions until column 14 is near the allowable spread then proceed to step 15.
Step 15	*Col 15*	Calculate W/T and enter in Column 15.

Step 16	*Col 16*	Select the inlet type and dimensions and enter in Column 16.
Step 17	*Col 17*	Calculate the Q intercepted (Q_i) by the inlet and enter in Column 17. Utilize Figure 8.2, 8.3 or 8.4 to define the flow in the gutter. Utilize Figure 8.6, 8.7 and 8.8 and Equation 8.12 to calculate Q_i for a grate inlet and Figure 8.9 and 8.10 to calculate Q_i for a curb opening inlet.
Step 18	*Col 18*	Calculate the runby by subtracting Column 17 from Column 11 and enter into Column 18.
Step 19	*Col 1-4*	Proceed to the next inlet down grade. Select an area approximately 300′ to 400′ below the first inlet as a first trial. Repeat Steps 5 through 7 considering only the area between the inlets.
Step 20	*Col 5*	Compute a time of concentration for the second inlet downgrade based on the area between the two inlets.
Step 21	*Col 6*	Determine the intensity based on the time of concentration determined in Step 20 and enter it in Column 6.
Step 22	*Col 7*	Determine the discharge from this area by multiplying Column 3 X Column 4 X Column 6. Enter the discharge in Column 7.
Step 23	*Col 11*	Determine total gutter flow by adding Column 7 and Column 10. Column 10 is the same as Column 18 from the previous line.
Step 24	*Col 12* *Col 14*	Determine spread (T) based on total gutter flow (Column 11) by using Figure 8.3 or 8.4. If spread (T) in Column 14 exceeds the allowable spread, reduce the area and repeat Steps 19-23. If spread (T) in Column 14 is substantially less than the allowable spread, increase the area and repeat Steps 18-24.
Step 25	*Col 16*	Select inlet type.
Step 26	*Col 17*	Determine Q_i, See instruction in Step 17.
Step 27	*Col 18*	Calculate the runby by subtracting Column 17 from Column 7. This completes the spacing design for this inlet.
Step 28		Go back to Step 19 and repeat Step 19 through Step 27 for each subsequent inlet. If the drainage area and weighted "C" values are similar between each inlet, the values from the previous grate location can be reused. If they are significantly different, recomputation will be required.

INLET COMPUTATION SHEET

DATE__________ SP__________ ROUTE________

COMPUTED BY_________ SHEET____OF____

LOCATION		GUTTER DISCHARGE Design Frequency ______					GUTTER DISCHARGE Allowable Spread ______								INLET DISCHARGE			REMARKS
INLET NO.	STAT.	DRAIN AREA "A" (acres)	RUNOFF COEF. "C"	TIME OF CONC. "T_c" (min)	Rain Intens. "I" (in/hr)	Q=CIA (cfs)	GRADE "S_o" (ft/ft)	CROSS SLOPE S_x (ft/ft)	PREV. RUNBY (cfs)	TOTAL GUTTER FLOW (cfs)	DEPTH "d" (ft)	GUTTER WIDTH "W" (ft)	SPREAD "T" (ft)	W/T	INLET TYPE	INTER-CEPT "Q_i" (cfs)	RUNBY "Q_r" (cfs)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

Table 8.5 Inlet Spacing Computation Sheet

8.8 MANHOLES AND INLET STRUCTURES

Manholes (MH) are utilized to provide access to continuous underground storm drains for inspection and cleanout. Where feasible, grate inlets may be used in lieu of manholes for access so that the benefit of extra stormwater interception is achieved with minimal additional cost. Typical locations where manholes should be specified are:

- where two or more storm drains converge,
- at intermediate points along tangent sections,
- where pipe size changes,
- where an abrupt change in alignment occurs, and
- where an abrupt change of the grade occurs.

Manholes should not be located in traffic lanes; however, when it is impossible to avoid locating a manhole in a traffic lane, care should be taken to insure it is not in the normal wheel path. There are various types of manholes available which are listed in Table 8.6. Usually the type selected is dependent on the storm drain pipe size and depth of the manhole.

Table 8.6 Manhole and Inlet Structure Types

Standard Plate	Design Type	Manhole Diameter	Maximum Depth	Maximum Size Pipe	Minimum Size Pipe	Comments
4000	A	48"	variable	30 "	12"	Masonry field constructed, includes cone section and barrel.
4002	C	48"	5′	variable	12"	Masonry field constructed cone section, typically used with single pipe.
4003	N	30"	4′	18"	12"	Precast 30" catch basin, typically used with single pipe.
4005	F	48"	variable	27"	12"	Precast Type A and B cone sections
4006	G	48"	5′	21"	12"	Precast cone section, typically used with single pipe.
	H	27"	3′	15"	12"	Precast 27" catch basin, typically used with single pipe.
4008	I	24"	variable	diameter	24"	Sectional concrete pipe (Tee) with 24" diameter riser.
4009	J	48"	N/A	diameter	42"	Sectional concrete pipe (Tee) with 48" diameter riser and Type A cone section.
4020	-	48" to 120"	variable	variable	12"	Manholes larger than 120" need to be evaluated on a case by case basis.
4024	SD	48"	4'	24"	12"	Precast 48" shallow depth catch basin with cover. Maximum Depth measured to bottom of cover

8.8.1 Height

Precast manhole sections can be made for installation of pipes up to 60 inch diameter. Larger sizes are limited by the precast segment height of 8 feet. Manholes with installation of larger pipe sizes will require either field construction (cast-in-place or masonry), tee structures or a special precast design.

Pay heights should be computed according to the methodology provided in the Minnesota Department of Transportation Standard Specification for Construction.

8.8.2 Spacing

The maximum spacing of access structures whether manholes or inlets should be approximately 400 feet for 12 inch through 54 inch diameter storm drains and approximately 600 to 800 feet for 60 inch and above. Where self cleaning velocities of at least 3 feet per second (fps) are assured, the distance between access points is not as critical as for storm drains on very flat grades where sedimentation could be a problem. A minimum velocity of 3 feet per second (fps) is recommended where feasible.

8.8.3 Sizing

When determining the minimum manhole size required for various pipe sizes and locations, two conditions must be met.

- The manhole or inlet structure must be large enough to accept the maximum pipe as shown in Table 8.7.
- A minimum leg width between pipe holes of 6" measured on the inside of the manhole must be maintained. Knowing the relative locations of any two pipes, apply Equation 8.21 and determine minimum manhole size. If there are more then two pipes, check every combination of pipes to determine the most critical pair.

$$\frac{180\left(P_1 + P_2 + 12\right)}{\pi D \Delta} \leq 1 \qquad \textbf{(8.21)}$$

Where: P_1 = manhole perimeter removed for pipe 1 hole (inches)
P_2 = manhole perimeter removed for pipe 2 hole (inches)
Δ = angle between the pipe center lines (degrees)
D = inner diameter of the manhole (inches)
look up P_1 and P_2 in Table 8.7

Manholes must be sized to ensure constructability. Consider any special features such as flexible pipe connections that may result in the need to up size a manhole. For instance, connections sometimes used with plastic pipe may require additional space. Complex configurations, where numerous pipes enter a single structure, particularly with unusual pipe configurations or varying inverts will need to be evaluated on a case by case basis. The values provided in Table 8.7 are based on pre-cast structures, It may be possible to fit slightly larger pipes into a cast-in-place or masonry structure.

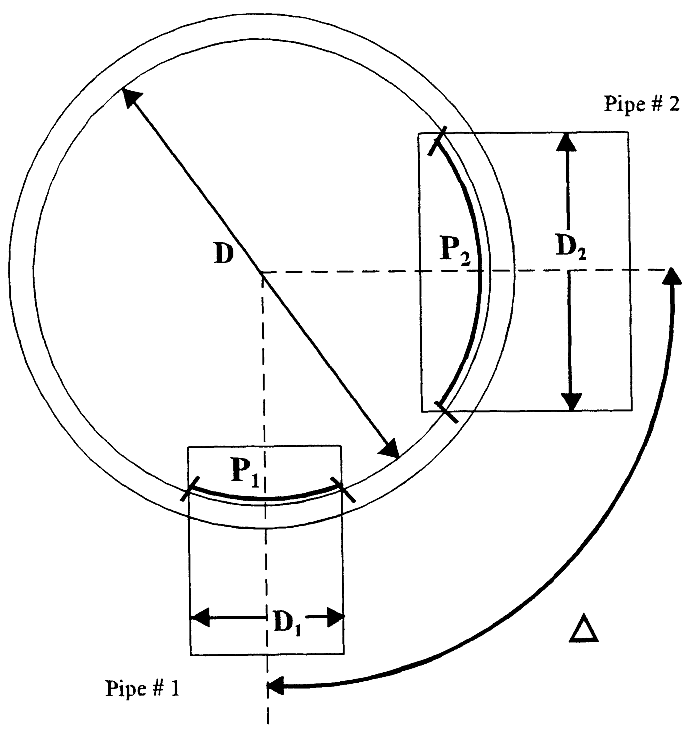


Figure 8.13 Manhole Diagram

Table 8.7 Minimum Manhole Size and Pre-cast Manhole Perimeter Removed to Install Pipe

Pipe Inner Diameter (inches)	Required hole size for C wall concrete pipe (inches)	Minimum Manhole Inner Diameter (inches)	Manhole Perimeter, P (inches) Removed to Install Pipe													
			MH I.D. 27"	MH I.D. 30"	MH I.D. 48"	MH I.D. 54"	MH I.D. 60"	MH I.D. 66"	MH I.D. 72"	MH I.D. 78"	MH I.D. 84"	MH I.D. 90"	MH I.D. 96"	MH I.D. 102"	MH I.D. 108"	MH I.D. 120"
12	20	27[1]	22.52	21.89	20.63	20.49	20.39	20.32	20.27	20.23	20.19	20.17	20.15	20.13	20.12	20.09
15	24	27[1]	29.56	27.82	25.13	24.87	24.69	24.56	24.47	24.40	24.34	24.29	24.26	24.23	24.20	24.16
18	26	30[2]		31.45	27.48	27.13	26.89	26.72	26.60	26.51	26.43	26.38	26.33	26.29	26.26	26.21
21	30	48			32.41	31.81	31.42	31.14	30.94	30.79	30.68	30.59	30.51	30.45	30.40	30.32
24	34	48			37.78	36.78	36.15	35.72	35.41	35.18	35.00	34.87	34.75	34.66	34.59	34.47
27	38	48[3]			43.85	42.15	41.15	40.49	40.03	39.69	39.43	39.23	39.07	38.94	38.83	38.67
30	42	54				48.12	46.52	45.53	44.84	44.35	43.98	43.70	43.47	43.29	43.14	42.91
33	46	54				55.05	52.42	50.90	49.90	49.20	48.68	48.28	47.97	47.72	47.52	47.21
36	48	60					55.64	53.75	52.54	51.70	51.09	50.63	50.27	49.98	49.74	49.38
42	55	66						65.02	62.59	61.04	59.96	59.17	58.57	58.09	57.71	57.13
48	64	72							78.83	75.06	72.76	71.20	70.05	69.18	68.50	67.50
54	70	84									82.75	80.20	78.44	77.15	76.16	74.74
60	78	90										94.36	91.05	88.80	87.16	84.91
66[4]	84	102												100.48	97.85	94.46
72[4]	90	108													110.11	104.84
78[4]	98	120														116.42

Notes: Pipes larger then 60" and manholes larger then 120", will need to be evaluated on a case by case basis.

[1] Design H minimum size based on one pipe connection

[2] Design N minimum size based on one pipe connection

[3] One exception is that the Design SD, Precast Shallow Depth Catch Basin is a 48" structure where the maximum pipe size is 24" Inner Diameter.

[4] MH for pipes larger then 60" require field construct, Tee structures, or special precast design method.

8.9 STORM DRAINS

After the preliminary locations of inlets, connecting pipes, and outfalls with tailwater have been determined, the next logical step is the computation of the rate of discharge to be carried by 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 the storm drain connects with other drains or the outfall. At manholes where the pipe size is increased, it is recommended the pipe invert in the manhole be lowered to match crowns or at least 80% of the difference in pipe sizes.

The rate of discharge at any point in the storm drain is not the sum of the design inlet flow rates of all inlets above that storm drain section. It is generally less than this total. The time of concentration is most influential and as it 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. The Hydrology Chapter contains a detailed 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. The exceptions are depressed sections and underpasses where ponded water can be removed only through the storm drain system. In these situations, a 50 year frequency design should be used to design the storm drain which drains the sag point. The main storm drain which drains 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.

8.9.1 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:
location of storm drains, direction of flow, location of manholes, location of existing utilities such as water, gas, or underground cables.

Step 3 Determine drainage areas and runoff coefficients, and a time of concentration to the first inlet. Using an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity. Calculate the discharge by multiplying A x C x I.

Step 4 Size the pipe to convey the discharge by varying the slope and pipe size as necessary. The storm drain systems are normally designed for full gravity flow conditions using the design frequency discharges.

Step 5 Calculate travel time in the pipe to the next inlet or manhole by dividing pipe length by the velocity. This travel time is added to the time of concentration for a new time of concentration and a new rainfall intensity at the next entry point.

Step 6 Calculate the new area (A) and multiply by the runoff coefficient (C), add to the previous (CA), multiply by the new rainfall intensity to determine the new discharge. Determine the size of pipe and slope necessary to convey the discharge.

Step 7 Continue this process to the storm drain outlet. Utilize the equations and/or nomographs to accomplish the design effort.

Step 8 Check the design by calculating the hydraulic grade line (HGL) as described in Section 8.10. The design procedure should include the following:

- Storm drain design computation can be made on forms as illustrated in Table 8.11.
- All computations and design sheets should be clearly identified. The designer'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.

8.9.2 50 Year Sag Point

As indicated above, the storm drain which drains a major sag point should be sized to accommodate the runoff from a 50 year frequency rainfall. This can be done by actually computing the runby occurring at each inlet during a 50 year rainfall and accumulating it at the sag point. The inlet at the sag point as well as all storm drain pipes leading from the sag point to the outlet must be sized to accommodate this additional runby within the criteria established. Another method which is approximate, assumes that during a 50 year rainstorm, the on grade inlets will intercept the 10 year flow and the runby will consist of the difference between the 50 year and the 10 year runoff. A step by step procedure for this approximate method follows.

Step 1 Total the CA contributing to the sag point inlet.

Step 2 From the IDF curve, determine the rainfall intensity for both I_{50} and I_{10} for the time of concentration computed in the storm drain pipe at the sag point. Subtract I_{10} from I_{50}.

Step 3 Multiply the total CA by the difference of I_{50} - I_{10}. This is the 50 year runby. Size the sag inlet to accommodate this additional flow.

Step 4 Convert the 50 year runby to an equivalent CA by dividing it by I_{10} in the pipe at the sag point.

Step 5 Add the equivalent CA to the total CA.

Step 6 Design the pipe from the sag point for the Q resulting from the CA in Step 5 multiplied by I_{10} and continue down line adding CA from additional inlets.

8.9.3 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:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{8.22a}$$

In terms of discharge, the above formula becomes:

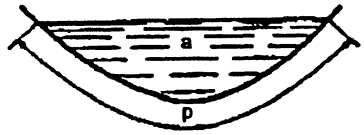
$$Q = VA = \frac{1.486}{n} AR^{2/3} S^{1/2} \tag{8.22b}$$

For storm drains flowing full, the above equations become:

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \qquad Q = \frac{0.463}{n} D^{8/3} S^{1/2} \tag{8.22c}$$

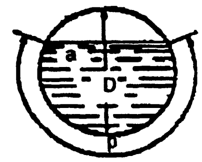
Where: V = mean velocity of flow (ft/s)
S = the slope of the energy grade line
R = hydraulic radius (ft)
area of flow divided by the wetted perimeter (A/P)
n = Manning's roughness coefficient
Q = rate of flow (cfs)
A = cross sectional area of flow (ft^2)
D = diameter of pipe (ft)

Figure 8.14 has been provided to assist in the solution of the Manning's equation for part full flow in storm drains.



a = Cross-sectional area of waterway
p = wetted perimeter
R = a/p = Hydraulic radius

Section of Any Channel



For pipes full or half full
R = D/4

Section of Circular Pipe

V = Average or mean velocity in m/s
Q = a V = Discharge of pipe or channel in m³/s
n = Coefficient of roughness of pipe or channel surface
S = Slope of hydraulic gradient (water surface in open channels or pipes not under pressure, same as slope of channel or pipe invert only when flow is uniform in constant section)

Hydraulic Elements of Channel Sections

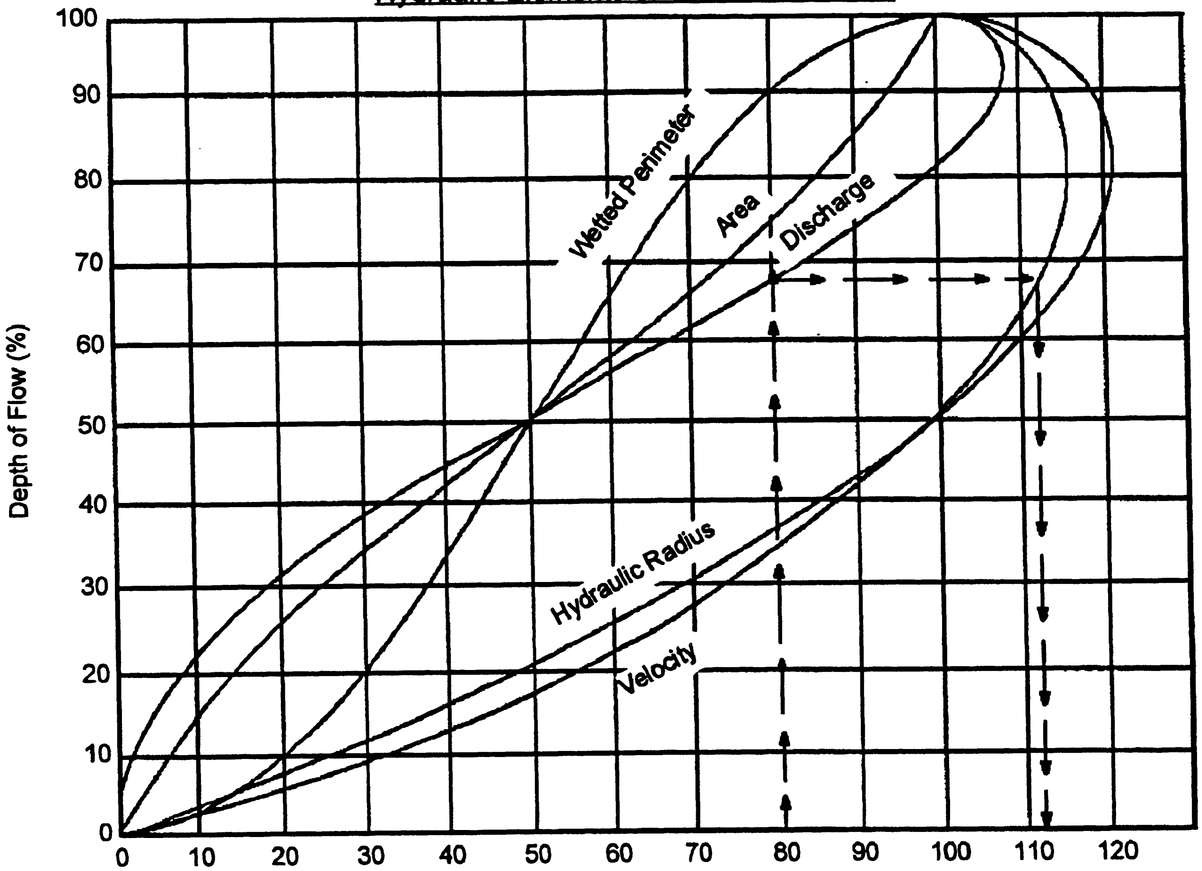


Figure 8.14 Values Of Hydraulic Elements Of Circular Section for Various Depths of Flow
Source: HEC-22 (FHWA 1996)

8.9.4 Minimum Grades

All storm drains should be designed such that velocities of flow will not be less than 3 feet per second at full flow or lower. For very flat grades the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system should be checked to be sure there is sufficient velocity in all of the drains to deter settling of particles. Minimum slopes required for a velocity of 3 fps can be calculated by the Manning formula or by using values given in Table 8.8.

$$S = \frac{(nV)^2}{2.208R^{4/3}} \tag{8.23}$$

Where: S = slope of the energy grade line
n = Manning's n
V = mean velocity of flow (fps)
R = hydraulic radius (ft)
area of flow divided by the wetted perimeter (A/P)

Table 8.8 Minimum Grades

Minimum Grades to Ensure 3 fps for Full Flow (ft/ft)		
Pipe Size (in)	Q Full (cfs)	Grade (ft/ft)
12	2.36	.0037
15	3.68	.0028
18	5.30	.0022
21	7.22	.0018
24	9.43	.0015
27	11.93	.0013
30	14.73	.0011
33	17.82	.00097
36	21.21	.00086
42	28.86	.00070
48	37.70	.00059
54	47.71	.00050
60	58.90	.00044
66	71.27	.00038
72	84.82	.00034

8.9.5 Curved Alignment

Curved storm drains are permitted where necessary. Long radius bend sections are available and are the preferable means of changing direction in pipes 48" 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.

8.10 HYDRAULIC GRADE LINE

The hydraulic grade line (HGL) is the last important feature to be established relating to the hydraulic design of storm drains. This gradeline aids the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise when the system is operating during a flood of design frequency.

In general, if the HGL is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the HGL is below the crown of the pipe, open channel flow calculations are appropriate. Storm drain systems can often alternate between pressure and open channel flow conditions from one section to another. A special concern with storm drains designed to operate under pressure flow conditions is that inlet surcharging and possible manhole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made.

The detailed methodology employed in calculating the HGL through the system begins at the system outfall with the tailwater elevation. If the outfall is an existing storm drain system, the HGL calculation must begin at the outlet end of the existing system, and proceed upstream through this inplace system, then upstream through the proposed system to the upstream inlet. The same considerations apply to the outlet of a storm drain as to the outlet of a culvert. Usually it is helpful to compute the energy grade line (EGL) first, then the velocity head ($V^2/2g$) is subtracted to obtain the HGL.

8.10.1 Tailwater

For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth. To determine the EGL, begin with the tailwater elevation or $(d_c+D)/2$, whichever is higher, add the velocity head for full flow and proceed upstream to compute all losses such as exit losses, friction losses, junction losses, bend losses, and entrance losses as appropriate.

An exception to the above might be a very large outfall with low tailwater when a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the tailwater, whichever is higher.

When estimating tailwater depth on the receiving stream, the prudent designer will consider the joint or coincidental probability of two events occurring at the same time. For the case of a tributary stream or a storm drain, its relative independence may be qualitatively evaluated by a comparison of its drainage area with that of the receiving stream. A short duration storm which causes peak discharges on a small basin may not be critical for a larger basin. Also, it may safely be assumed that if the same storm causes peak discharges on both basins, the peaks will be out of phase. Table 8.9 will aid in the evaluation of joint probabilities.

Table 8.9 Joint Probability Analysis

Area Ratio	Frequencies for Coincidental Occurrence			
	10 Year Design		100 Year Design	
	Mainstream	Tributary	Mainstream	Tributary
10,000 to 1	1 10	10 1	2 100	100 2
1,000 to 1	2 10	10 2	10 100	100 10
100 to 1	5 10	10 5	25 100	100 25
10 to 1	10 10	10 10	50 100	100 50
1 to 1	10 10	10 10	100 100	100 100

Source: US Army Corps of Engineers, Norfolk District, 1974

8.10.2 Exit Loss

The exit loss is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as an endwall, the exit loss is:

$$H_o = 1.0\left(\frac{V^2}{2g} - \frac{V_d^2}{2g}\right) \tag{8.24}$$

Where: V = average outlet velocity (fps)
V_d = channel velocity downstream of outlet (fps)
g = acceleration due to gravity (32.2 ft/sec^2)

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.

8.10.3 Bend Loss

The bend loss coefficient is minor but can be evaluated using the formula:

$$h_b = 0.0033\Delta\left(\frac{V_o^2}{2g}\right) \tag{8.25}$$

Where: Δ = angle of curvature in degrees
V_o = velocity (ft/s)
g = acceleration due to gravity (32.2 ft/sec^2)

8.10.4 Pipe Friction Losses

The friction slope is the hydraulic gradient in ft/ft for that run. The friction loss is simply the hydraulic gradient multiplied by the length of the run in feet. The head losses due to friction may be determined by the formula:

$$H_f = S_f L \tag{8.26}$$

Energy losses from pipe friction may be determined by rewriting the Manning Equation with terms as previously defined:

$$S_f = \left[\frac{Qn}{1.486AR^{2/3}}\right]^2 \tag{8.27}$$

The Manning's equation can also be written to determine friction losses for storm drains as follows:

$$H_f = 2.87\frac{n^2V^2L}{D^{4/3}} = \frac{29n^2L}{R^{4/3}}\left(\frac{V^2}{2g}\right) \tag{8.28}$$

Where: H_f = total head loss due to friction (ft)
S_f = friction slope of hydraulic grade line (ft/ft)
L = length of pipe (ft)
Q = rate of flow (cfs)
n = Manning's roughness coefficient
A = cross sectional area of flow (ft^2)
R = hydraulic radius (ft)
V = mean velocity (ft/s)
g = acceleration due to gravity (32.2 ft/sec^2)
D = diameter of pipe (ft)

8.10.5 Manhole Losses

The head loss encountered in going from one pipe to another through a manhole is commonly represented as being proportional to the velocity head at the outlet pipe. A head loss coefficient, K is used to signify this constant of proportionality. K is calculated by multiplying correction factors together, Equation 8.29. The head loss coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection., the energy loss is approximated as $K(V_o^2/2g)$. Experimental studies have determined that the K value can be approximated as follows:

$$K = K_o C_D C_d C_Q C_p C_B \tag{8.29}$$

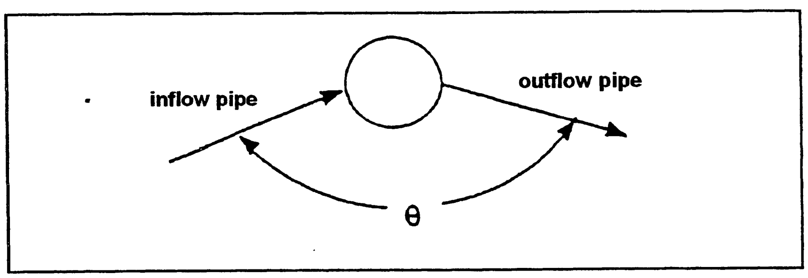
Where: K = adjusted loss coefficient
K_o = initial head loss coefficient based on relative manhole size
C_D = correction factor for pipe diameter (pressure flow only)
C_d = correction factor for flow depth (non-pressure flow only)
C_Q = correction factor for relative flow
C_B = correction factor for benching
C_p = correction factor for plunging flow

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.

$$K_o = 0.1\left(\frac{b}{D_o}\right)(1-\sin\theta) + 1.4\left(\frac{b}{D_o}\right)^{0.15}\sin\theta \tag{8.30}$$

Where: θ = the angle between the inflow and outflow pipes (degrees)
b = manhole diameter (ft)
D_o = outlet pipe diameter (ft)



Pipe Diameter

A change in head loss due to differences in pipe diameter is only significant in pressure flow situations when the depth in the manhole to outlet pipe diameter ratio, d/D_o, is greater than 3.2. Therefore, it is only applied in such cases.

$$C_D = \left(\frac{D_o}{D_i}\right)^3 \tag{8.31}$$

Where: D_i = incoming pipe diameter (ft)
D_o = outgoing pipe diameter (ft)

Flow Depth

The correction factor for flow depth is significant only in cases of free surface flow or low pressures, when d/D_o ratio is less than 3.2 and is only applied in such cases. Water depth in the manhole is approximated as the level of the hydraulic gradeline at the upstream end of the outlet pipe. The correction factor for flow depth, C_d, is calculated by the following:

$$C_d = 0.5\left(\frac{d}{D_o}\right)^{0.6} \qquad \textbf{(8.32)}$$

Where: d = water depth in manhole above outlet pipe (ft)
D_o = outlet pipe diameter (ft)

Relative Flow

The correction factor for 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. As can be seen from the Equation 8.33, 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. It is computed as follows:

$$C_Q = (1 - 2\sin\theta)\left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1 \qquad \textbf{(8.33)}$$

Where: C_Q = correction factor for relative flow
θ = the angle between the inflow and outflow pipes (degrees)
Q_i = flow in the inflow pipe (cfs)
Q_o = flow in the outlet pipe (cfs)

Plunging Flow

This correction factor corresponds to the effect of another inflow pipe, plunging into the manhole, on the inflow pipe for which the head loss is being calculated. The correction factor is only applied when h > d.. The correction factor for plunging flow, C_p, is calculated by the following:

$$C_p = 1 + 0.2\left[\frac{h}{D_o}\right]\left[\frac{(h-d)}{D_o}\right] \qquad \textbf{(8.34)}$$

Where: C_p = correction for plunging flow
h = vertical distance of plunging flow from center of outlet pipe (ft)
D_o = outlet pipe diameter (ft)
d = water depth in manhole (ft)

Benching

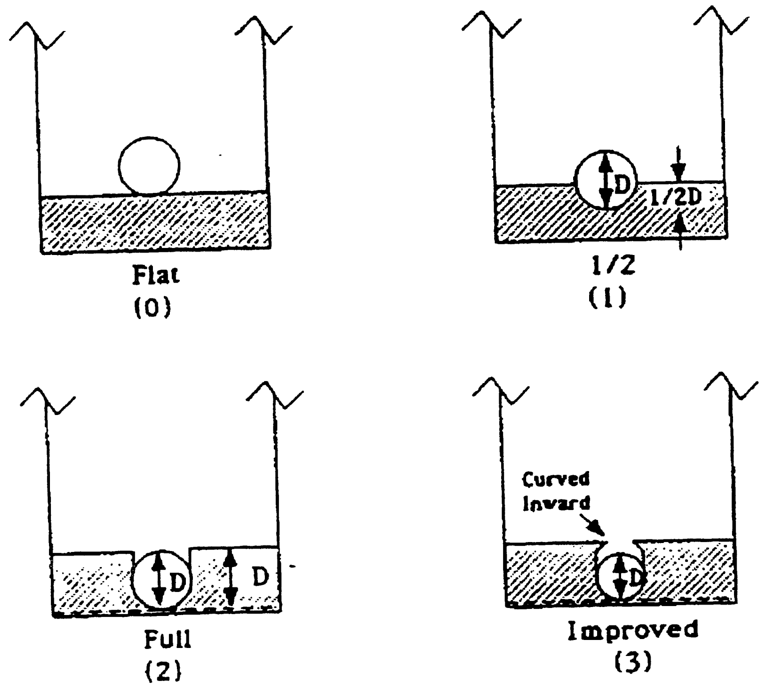
Benching tends to direct flows through the manhole, resulting in reductions in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed. The correction for benching in the manhole, C_B, is obtained from Table 8.10.

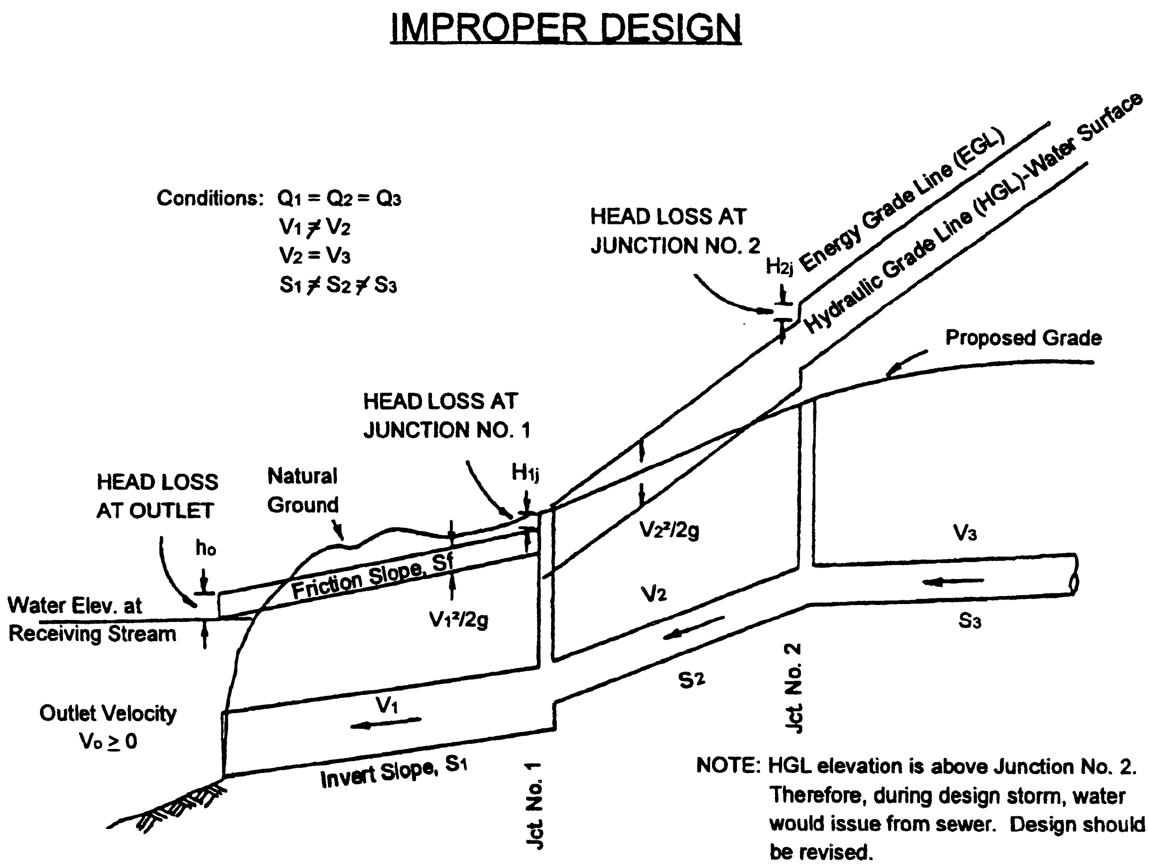
Table 8.10 Correction for Benching

Bench Type	Correction Factors, C_B	
	Submerged[1]	Unsubmerged[2]
Flat floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Type J	0.40	0.02

[1] pressure flow, $d/D_o > 3.2$

[2] free surface flow, $d/D_o < 1.0$





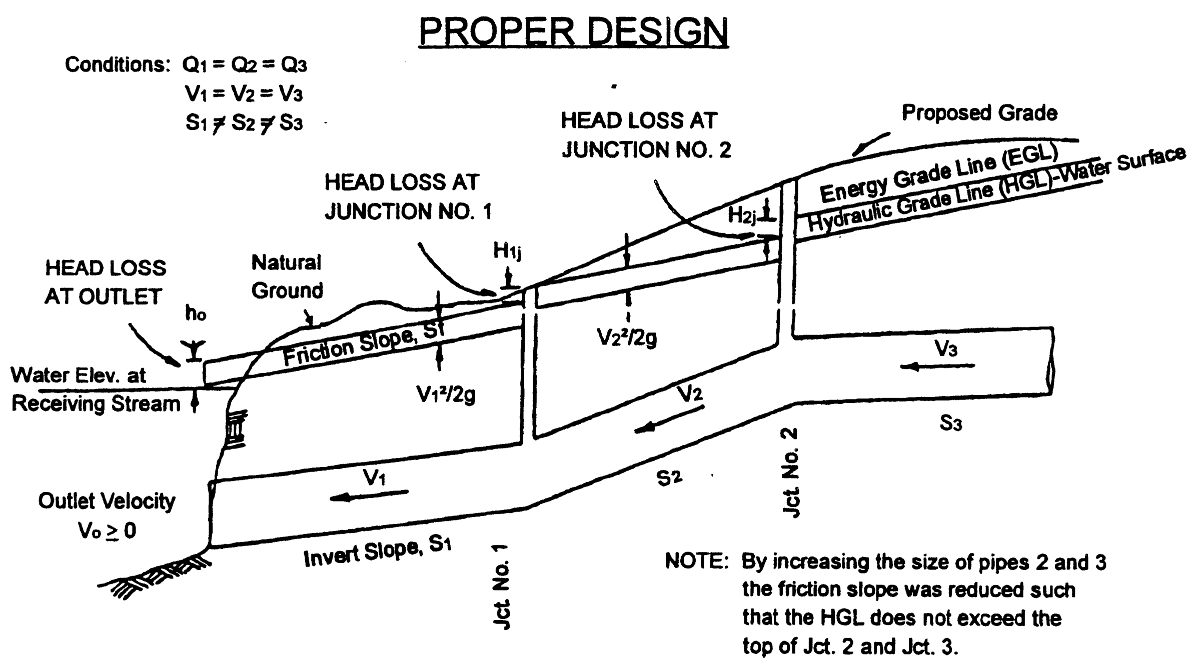


Figure 8.15 Use Of Energy Losses In Developing A Storm Drain System
Source: Model Drainage Manual - Metric Edition, (AASHTO, 1999)

8.10.6 Hydraulic Grade Line (HGL) Design Procedure

The equations and charts necessary to manually calculate the location of the hydraulic gradeline are included in this chapter. Any computer program used for design of storm drains should include a HGL analysis and a pressure flow simulation. A step by step procedure is given to manually compute the HGL. Table 8.11 can be used to document the procedure.

If the HGL is above the pipe crown at the next upstream manhole, pressure flow calculations are indicated; if it is below the pipe crown, then open channel flow calculations should be used at the upstream manhole. The process is repeated throughout the storm drain system. If all HGL elevations are acceptable then the hydraulic design is adequate. If the HGL exceeds an inlet elevation, then adjustments to the trial design must be made to lower the water surface elevation.

See Figure 8.15 for a sketch depicting the use of energy losses in developing a storm drain system.

Step 1 Enter in Column 1 the station for the junction immediately upstream of the outflow pipe. HGL computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2 Enter in Column 2 the tailwater elevation if the outlet will be submerged during the design storm otherwise refer to the tailwater discussion in Section 8.10.1 for procedure.

Step 3 Enter in Column 3 the diameter (D_o) of the outflow pipe.

Step 4 Enter in Column 4 the design discharge (Q_o) for the outflow pipe.

Step 5 Enter in Column 5 the length (L_o), of the outflow pipe.

Step 6 Enter in Column 6 the outlet velocity of flow (V_o).

Step 7 Enter in Column 7 the velocity head, $V_o^2/2g$.

Step 8 Enter in Column 8 the exit loss, H_o, as computed by Equation 8.24.

Step 9 Enter in Column 9 the friction slope (SF_o) in ft/ft of the outflow pipe. This can be determined by using the Equation 8.27. Note: Assumes full flow conditions.

Step 10 Enter in Column 10 the friction loss (H_f) which is computed by multiplying the length (L_o) in Column 5 by the friction slope (SF_o) in Column 9. On curved alignments, calculate bend losses by using Equation 8.25 adding the answer to the friction loss.

Step 11 Enter in Column 11 the initial head loss coefficient, K_o, based on relative manhole size as computed by Equation 8.30.

Step 12 Enter in Column 12 the correction factor for pipe diameter, C_D, as computed by Equation 8.31.

Step 13 Enter in Column 13 the correction factor for flow depth, C_d, as computed by Equation 8.32. Note this factor is only significant in cases where the d/D_o ratio is less than 3.2.

Step 14 Enter in Column 14 the correction factor for relative flow, C_Q, as computed by Equation 8.33.

Step 15 Enter in Column 15 the correction factor for plunging flow, C_p, as computed by Equation 8.34. The correction factor is only applied when h > d.

Step 16 Enter in Column 16 the correction factor for benching, C_B, as determined in Table 8.10.

Step 17 Enter in Column 17 the value of K as computed by Equation 8.29.

Step 18 Enter in Column 18 the value of the total manhole loss, $K(V_o^2/2g)$.

Step 19 If the tailwater submerges the outlet end of the pipe, enter in Column 19 the sum of Column 2 (TW elevation) and Column 7 (exit loss) to get the EGL at the outlet end of the pipe. If the pipe is flowing full, but the tailwater is low, the EGL will be determined by adding the velocity head to $(d_c + D)/2$.

Step 20 Enter in Column 20 the sum of the friction head (Column 10), the manhole losses (Column 18), and the energy grade line (Column 19) at the outlet to obtain the EGL at the inlet end. This value becomes the EGL for the downstream end of the upstream pipe.

Step 21 Determine the HGL (Column 21) throughout the system by subtracting the velocity head (Column 7) from the EGL (Column 20).

Step 22 Check to make certain that the HGL is below the level of allowable high water at that point. If the HGL is above the finished grade elevation, water will exit the system at this point for the design flow.

- The above procedure applies to pipes that are flowing full, as should be the condition for design of new systems. If a part full flow condition exists, the EGL is located one velocity head above the water surface.

Station	TW	D_o	Q_o	L_o	V_o	$V_o^2/2g$	H_o	SF_o	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V_o^2/2g)$	EGL_o 2+7	EGL_i 10+18+19	HGL EGL-7	TOC Elev.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

Table 8.11 Hydraulic Grade Line Computation Form

8.11 REFERENCES

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