TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER THIRTY-SIX

PAVEMENT/STORM DRAINAGE SYSTEMS

36-1.0 OVERVIEW

36-1.01 Introduction

This Chapter provides guidance on storm-drain design and analysis. The quality of the final in-place system reflects the attention accorded to every aspect of the design as well as that accorded to the construction and maintenance of the facility. The aspects of storm-drain design such as system planning, pavement drainage, gutter-flow calculations, inlet spacing, pipe sizing, and hydraulic grade line calculations are discussed herein.

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 a roadway traversing a sparsely-settled rural area. This is due to the following:

1. wide roadway sections and flat grades, both in the longitudinal and transverse directions, shallow water courses, absence of side channels;

2. more costly property damage which may occur from ponding of water or from flow of water through a built-up area; and

3. the roadway section must carry traffic but also act as a channel to convey the 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.

36-1.02 Inadequate Drainage

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

1. damage to surrounding or adjacent property resulting from water overflowing the roadway curbs and entering such property;

2. risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway; and

3. weakening of base and subgrade due to saturation from frequent ponding of long duration

36-2.0 POLICY AND GUIDELINES

36-2.01 Introduction [Rev. Jan. 2011]

A highway storm-drainage facility collects stormwater runoff and conveys it through the roadway right of way to adequately drain the roadway and minimize the potential for flooding and erosion to properties adjacent to the right of way. A storm-drainage facility consist of curbs, gutters, storm drains, side ditches or open channels (as appropriate), or culverts. The placement and hydraulic capacity of a storm-drainage facility should be designed to consider damage to adjacent property and to secure as low a degree of risk of traffic interruption due to flooding as is consistent with the importance of the road, the design traffic service requirements, and available funds. Stormwater-pollution-prevention requirements should be considered and addressed in the design process of each storm-drainage system.

The following is a summary of the policies for pavement drainage system design and analysis.

36-2.02 Bridge Deck

A zero gradient, sag vertical curve, or superelevation transition with a flat pavement section should be avoided on a bridge. The desirable longitudinal grade for bridge-deck drainage is 0.5% or steeper, especially for new construction. A flatter grade will be tolerated where it is not physically or economically desirable to satisfy this criterion. A bridge may not require drainage facilities. The quantity and quality of runoff should be maintained as required by applicable stormwater regulations. See Chapter Thirty-three for additional information.

36-2.03 Curbs, Inlets, and Turnouts

Curbs, inlets, or turnouts are used where runoff from the pavement can erode fill slopes, or where reduction of the right of way needed for shoulders, side ditches, or open channels, etc., is desirable. Where storm drains are necessary, the pavement section should be curbed.

36-2.04 Design Frequency

The design flood frequency for roadway drainage is related to the allowable water spread on the pavement and design speed. This design criterion is discussed in Section 36-7.0.

36-2.05 Detention Storage

Reduction of peak flow can be achieved through the storage of runoff in a detention basin, storm drainage pipe, swale, side ditch, open channel, or other detention storage facility. Stormwater can then be released to the downstream conveyance facility at a reduced flow rate. The concept should be considered where existing downstream conveyance facilities are inadequate to handle peak flow rates from a highway storm-drainage facility. A developer may not be permitted to increase runoff over existing conditions, thus necessitating a detention storage facility. Additional benefits include the reduction of downstream pipe sizes and the improvement of water quality by removing sediment or pollutants. For additional information, see Chapter Thirty-five.

36-2.06 Gutter-Flow Calculations

Gutter-flow calculations are necessary to relate the quantity of flow to the spread of water on a shoulder, parking lane, or pavement section. A composite gutter section has a greater hydraulic capacity for a normal cross slope than a uniform gutter section, and is therefore preferred. See Section 36-8.0 for additional information and procedures.

36-2.07 Hydrology

The Rational Method is the most common method in use for the design of a storm drain if the momentary peak flow rate is desired. Its use should be limited to a system with a drainage area of 200 acres or less. A minimum time of concentration of 5 min is acceptable. The Rational method is described in Chapter Twenty-nine.

36-2.08 Inlets

The term refers to each type, such as a grate inlet, curb inlet, or slotted inlet. A drainage inlet is sized and located to limit the spread of water on traffic lanes to a tolerable width for the design storm in accordance with the design criteria specified in Section 36-7.0. The width of water spread on the pavement at a sag should not be substantially greater than the width of spread encountered on a continuous grades.

A grate inlet or depression-of-curb-opening inlet should be located outside the through traffic lanes to minimize the shifting of a vehicle attempting to avoid it. A grate inlet should be bicycle-safe if used on a roadway that allows bicycle travel. If a grate inlet is used at a sag location, a double curved vane grate should be utilized to compensate for plugging that can occur.

Where significant ponding can occur, such as at an underpass or sag vertical curve in a depressed section, flanking inlets should be placed on each side of the inlet at the low point in the sag. See Section 36-9.03 for a discussion on the location of inlets.

36-2.09 Manholes

The maximum spacing of access structures whether manholes, junction boxes, or inlets should be approximately 400 ft. Figure 36-11B is useful in determining the relationship between manhole diameter, maximum pipe size, and deflection angle as defined in Figure 36-13B.

36-2.10 Roadside or Median Ditch

A large amount of runoff should be intercepted before it reaches the highway to minimize the deposition of sediment or other debris on the roadway, and to reduce the amount of water which must be carried in the gutter section. A median area or inside shoulder must be sloped to prevent runoff from the median area from flowing across the pavement. A surface channel should have adequate capacity for the design runoff and should be located and shaped to not present a traffic hazard. Where permitted by the design velocity, a channel should have a vegetative lining. An appropriate lining may be necessary where vegetation will not control erosion. See Chapter Thirty for detailed hydraulic information on a channel.

36-2.11 Storm Drain

A storm drain is defined as a closed-conduit system. It consists of that portion of the storm-drainage system that receives runoff from inlets and conveys the runoff to a point where it is then discharged into a side ditch or water body. At least one end is connected to a manhole, inlet, catch basin, or similar structure. A pipe which is connected to an inlet located in a paved median, grassed median, or lawn area is considered a storm-drain structure. A storm drain should have adequate capacity so that it can accommodate runoff that enters the system. It should be designed considering future development if appropriate. The storm-drain system for a sag vertical curve should have a higher level of flood protection to decrease the depth of potential ponding on the roadway or bridge. Where feasible, the storm drain should be designed to avoid existing utilities. The storm-drain outfall should be designed to ensure that the potential for erosion is minimized. The drainage-system design should be coordinated with the proposed staging of a large construction project to maintain an outlet throughout the construction project.

A storm-sewer trunk line should be located behind the curb or, if not practical, under the roadway without being located in the wheel path.

Design the main and all laterals as a system. The system must not operate under pressure for the design storm. The hydraulic grade line must not exceed a manhole, catch basin, or inlet rim elevation for the check storm.

The placement and capacity should be consistent with local stormwater management plans. A minimum pipe size of 12 in. with a minimum velocity of 2.5 ft/s is desirable to prevent sedimentation from occurring in the pipe.

36-2.12 System Planning

System planning prior to commencing the design of a storm-drain system is essential. The basic requirements are discussed in Section 36-5.0, and include the general design approach, type of data required, information on initiating a cooperative agreement with a municipality, the importance of a preliminary sketch, and some special considerations.

36-2.13 Storm-Drainage Agreement Policy

A storm-drainage agreement is required if a new or reconstructed INDOT drainage facility is designed to accommodate stormwater from a sewer controlled by a local public agency (LPA). This is applicable regardless of whether the shared drainage facility is constructed within or outside of INDOT right of way.

Where INDOT constructs a drainage facility outside the limits of the right of way to provide adequate drainage for a highway, I.C. 8-23-6-2 allows INDOT to assess a proportionate share of the cost of constructing the drainage facility outside the right of way to beneficiaries of the drainage structure. Therefore, a municipality or other beneficiary that connects to an INDOT drainage structure outside the limits of the right of way can be assessed a share of the cost of the drainage structure in proportion to the amount of drainage discharged. The proportionate share is calculated as follows:

$$A_B = C_F \left(\frac{Q_{OR}}{Q_T} \right)$$

Where: A_B = Amount of assessment to beneficiary
C_F = Cost of drainage facility
Q_{OR} = Discharge from storm sewer draining from outside INDOT R/W
Q_T = Total discharge of drainage facility

The remainder of the cost will be paid by INDOT.

By common law, INDOT also has the authority to seek a contribution from the LPA if stormwater from outside the INDOT right of way discharges into a drainage facility within the INDOT right of way. For example, if a municipality wishes to make a direct discharge into an INDOT trunkline storm drain, INDOT's policy will be to request a storm-drainage agreement for the trunkline sewer construction. The proportionate share will also be determined from Equation 36-2.1. If the discharge is in the form of sheet flow onto INDOT right of way, INDOT will not seek a contribution from the municipality involved. INDOT is not legally required to accept sheet-flow runoff from outside the right of way, but will do so as a matter of public policy.

If a particular situation involving sheet flow onto right of way is sufficiently significant to warrant a storm-drainage agreement, the LPA should agree to the necessary local contribution as a condition for initiating the State highway improvement. Such an agreement cannot be forced upon an LPA, but must be pre-arranged through negotiations between the LPA and the Planning Division or the Office of Environmental Services' Environmental Policy Team. However, this may occur as late as the design phase.

A situation may arise if INDOT storm-sewer construction results in a request for stormwater detention or a county assessment for the reconstruction of a regulated drain. See Section 28-3.07. If the situation also involves INDOT conveying city or town stormwater, INDOT should seek a storm-sewer cost-sharing contribution from the city or town. The procedure for determining the appropriate contribution by the city or town will be as described above. INDOT cannot site I.C. 8-23-6-2 as authority to pass on a portion of a county drainage assessment to the city or town. Only a county drainage board has the authority to levy a drainage assessment on a municipality or private-property owner if a regulated drain is involved.

A county drainage assessment does not require a formal agreement to be legally binding on INDOT. However, if an assessment includes a monetary contribution which relieves INDOT from providing stormwater detention mandated by the county, the conditions of the assessment should be formalized in a storm-drainage agreement.

The need for a storm-drainage agreement should be identified during the preliminary-plans development. Detailed information necessary for the preparation of the formal agreement should be coordinated with the municipality prior to INDOT design approval. The preliminary cost estimate of the trunkline sewer and the exact ratio to be used in determining the municipality's share should be verbally agreed to with the municipality. The ratio may be based on the sewer's cross-sectional area if the discharge of the municipality's storm sewer cannot be reasonably determined. The municipality should be notified in writing of the approximate cost of its share so that it can arrange financing.

After design approval, the formal storm-drainage agreement will be written to bind the LPA and the State. The Legal Services Division will prepare this document. The agreement must be signed by all parties concerned before the project may be scheduled for a letting.

36-2.14 Compatibility of Drainage Structure and Casting

Figure 36-2A shows which casting may be used with a given type of catch basin, inlet, or manhole. The information shown in the figure is complementary to that shown on the related INDOT *Standard Drawings*. In developing a drainage plan, the designer should refer to the figure to ascertain structure and casting compatibility. If the designer desires to use a structure-casting combination other than that permitted in the figure, he or she should contact the Production Management Division's Hydraulics Team.

36-3.0 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this *Manual*, the symbols shown in Figure 36-3A will be used. The symbols were selected because of their wide use in storm-drainage publications.

36-4.0 CONCEPT DEFINITIONS

The following are the concepts which should be considered in a storm-drainage analysis or design. The concepts will be used throughout this Chapter in addressing the different aspects of storm drainage analysis.

1. Check Storm or Check Event. The use of a less-frequent event, such as a 50-year storm, to assess a hazard at a critical location where water can pond to an appreciable depth.

2. Combination Inlet. A drainage inlet composed of a curb-opening inlet and a grate inlet.

3. Crown or Soffit. The inside top of a pipe.

4. Culvert. A drainage structure which extends through the embankment on both ends for the purpose of conveying surface water under a roadway. It may have one or two inlets connected to it to convey drainage from the median area.

5. Curb Opening. A drainage inlet consisting of an opening in the roadway curb.

6. Drop Inlet. A drainage inlet with a horizontal or nearly-horizontal opening.

7. Equivalent Cross Slope. An imaginary straight cross slope having conveyance capacity equal to that of the given compound cross slope.

8. Flanking Inlet. An inlet placed upstream and on each side of another inlet at the low point in a sag vertical curve. The purpose of a flanking inlet is to intercept debris as the slope decreases and to act in relief of the inlet at the low point.

9. Flow. The quantity of water which is flowing.

10. Frontal Flow. The portion of the flow which passes over the upstream side of a grate.

11. Grate Inlet. A drainage inlet composed of a grate in the roadway section or at the roadside in a low point, swale, or channel.

12. Grate Perimeter. The sum of the lengths of all sides of a grate, except that a side adjacent to a curb is not considered a part of the perimeter in weir-flow computations.

13. 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, of 24 in. width at a cross-slope of 2.5%, and the parking lane, shoulder, or pavement at a cross slope of 2.0%. A uniform gutter section has one constant cross slope. See Section 36-8.0 for additional information.

14. Hydraulic Grade Line. The locus of elevations to which water can rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head).

15. Inlet Efficiency. The ratio of flow intercepted by an inlet to total flow in the gutter.

16. Invert. The inside bottom of a pipe.

17. Lateral Line or Lead. This has inlets connected to it but has no other storm drains connected to it. It is 15 in. diameter or less and is a tributary to the trunkline.

18. Pressure Head. The height of a column of water that can exert a unit pressure equal to the pressure of the water.

19. Runby, Bypass, or Carryover. Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade. An inlet can be designed to allow a certain amount of runby for one design storm, or a larger or smaller amount for another storm.

20. Sag Point or Major Sag Point. A low point in a vertical curve. A major sag point is a low point that can overflow only if water can pond to a depth of 1.5 ft or more.

21. Scupper. A vertical hole through a bridge deck for the purpose of deck drainage. This can also be a horizontal opening in a curb or barrier.

22. Side-Flow Interception. Flow which is intercepted along the side of a grate inlet, as opposed to frontal interception.

23. 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. The types in use are slotted drain pipe and slotted vane-drain pipe. A slotted-drain inlet is used in conjunction with a single-grate inlet for cleanout access.

24. Storm Drain. Each pipe that is installed in conjunction with at least one inlet, catch basin, or manhole. A grassed-median inlet, lawn inlet, lawn catch basin, or pipe catch basin is considered to be a storm drain.

25. Splash-Over. The portion of frontal flow at a grate which skips or splashes over the grate and is not intercepted.

26. Spread. The width of stormwater flow in the gutter measured laterally from the roadway curb.

27. Trunkline or Main. The main storm-drain line. A lateral line may be connected at an inlet structure or manhole.

28. Velocity Head. Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water ($V^2/2g$).

36-5.0 SYSTEM PLANNING

36-5.01 Introduction

The design of a 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.

36-5.02 General Design Approach

The design of a storm-drain system is a process which evolves as a project develops. The primary ingredients to this process are listed below in the sequence by which they may be carried out. All of the individuals who contribute to this process cannot be listed, because the list varies for each project. However, the Hydraulics Engineer's role is as follows:

1. data collection (see Section 36-5.03);
2. coordination with other agencies (Section 36-5.04);
3. preliminary sketch (Section 36-5.05);
4. inlet locations and spacing (Sections 36-9.0 and 36-10.0);
5. plan layout of storm-drain system as follows:
    a. locate main outfall;
    b. determine direction of flow;
    c. locate existing utilities;
    d. locate connecting mains; and
    e. locate manholes;
6. size the pipes (Section 36-12.0);
7. review hydraulic grade line (Section 36-13.0);
8. prepare the plan; and
9. provide documentation (Chapter Twenty-eight).

36-5.03 Required Data [Rev. Jan. 2011]

The designer should be familiar with land-use patterns, the nature of the physical development of the area to be served by the storm-drainage system, the stormwater management plans for the area, and the ultimate pattern of drainage (both overland and by storm drains) to an existing outfall location. There should be an understanding of the nature of the outfall because it has a significant influence on the storm-drainage system. Water-quality requirements should always be considered, particularly in an environmentally-sensitive area.

Actual surveys of these and other features are the most reliable means of gathering the required data. Photogrammetric mapping has become a method of obtaining the large amounts of data required for drainage design, particularly for a busy urban roadway with attendant urban development. Existing topographic maps, available from the U. S. Geological Survey, the Natural Resources Conservation Service, municipalities, county governments, or private developers, are also valuable sources of the kind of data needed for a proper storm-drainage design. Governmental planning agencies should be consulted regarding plans for the area in question. The physical characteristics of a rapidly-growing urban area to be served by a storm-drainage system may change drastically in a short time. The designer must anticipate these changes and consider them in the storm-drainage design.

Comprehensive stormwater-management plans or floodplain ordinances should be reviewed if they are available.

<u>36-5.04 Preliminary Sketch</u>

A preliminary sketch or schematic, showing the basic components of the intended design, is useful. Such a sketch should indicate watershed areas and land use, existing drainage patterns, plan and profile of the roadway, street or drive layout with respect to the project roadway, underground utility locations and elevations, locations of proposed retaining walls, bridge abutments and piers, logical inlet and manhole locations, preliminary lateral and trunkline layouts, and a definition of the outfall location and characteristics. The sketch should be reviewed with the traffic-staging plans and soils recommendations for an area which is incompatible with required construction staging. With the 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 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.

<u>36-5.05 Special Considerations</u>

Consideration and planning should be directed toward avoidance of utilities and deep cuts. Traffic may be maintained or a temporary bypass may be constructed, and temporary drainage may be provided for during the construction phase. Further consideration should be given to the actual trunkline layout and its constructability. The proposed location of the storm drain may interfere with existing utilities or disrupt traffic. A trunkline may be required on each side of the roadway with few laterals, or only a single trunkline may be required. Such features are a function of economy but may be controlled by other physical features.

Pipe size should not be decreased in a downstream direction regardless of the available pipe gradient because of potential plugging with debris.

36-6.0 PAVEMENT DRAINAGE

<u>36-6.01 Introduction</u>

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

1. longitudinal and cross slopes;
2. curb and gutter sections;
3. roadside and median ditches; and
4. bridge deck.

The pavement width, cross slope, and profile grade control the time required for stormwater 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.

36-6.02 Roadway Longitudinal Slope

A minimum longitudinal grade should be considered for a curbed pavement because of the spread of stormwater against the curb. A flat grade on an uncurbed pavement can also lead to a spread problem if vegetation is allowed to build up along the pavement edge.

The desirable minimum gutter grade for a curbed pavement is 0.5%, and the desirable minimum is 0.3%. A minimum grade in curbed sections can be maintained in flat terrain by rolling the longitudinal-gutter profile. On an uncurbed roadway, the minimum longitudinal grade is 0%.

To provide adequate drainage in a sag vertical curve, a minimum slope of 0.3% should be maintained within 50 ft 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 170. Although ponding is not a problem at a crest vertical curve, a similar minimum grade should be provided to facilitate drainage.

36-6.03 Cross Slope

The selection of pavement cross slope is a compromise between motorist comfort and safety (i.e., flatter cross slope) and drainage (i.e., steeper cross slope). Chapters Forty-five and Fifty-three provide INDOT criteria on cross slope for the traveled way, shoulder, and curb offset. The slope will vary according to the following:

1. facility of two-lanes, or facility of 4 or more lanes;
2. urban or rural location;
3. functional classification of the facility;
4. new construction or reconstruction, or 3R work; and
5. curbed or uncurbed facility.

See Chapters Forty-five and Fifty-three to determine the applicable roadway cross slope.

36-6.04 Pavement Texture

The pavement texture should be considered for roadway surface drainage. Although the designer will have little control over the selection of the pavement type or its texture, the pavement texture does have an impact on the buildup of water depth on the pavement during a storm. A high level of macrotexture provides a channel for water to escape from the tire-pavement interface and thus reduces the potential for hydroplaning.

A high level of macrotexture may be achieved by tining a new portland-cement-concrete pavement surface while it is still in the plastic state. Retexturing of an existing portland-cement-concrete surface can be accomplished through pavement grooving or cold milling. Longitudinal or transverse grooving is effective in achieving macrotexture in concrete pavement. Transverse grooving aids in surface runoff resulting in less wet pavement time. A combination of longitudinal and transverse grooving provides the most adequate drainage for high-speed conditions.

36-6.05 Curb and Gutter

A curb at the outside edge of a pavement is common for a low-speed, urban highway facility. It contains the surface runoff within the roadway and away from adjacent properties, prevents erosion, provides pavement delineation, and enables the orderly development of property adjacent to the roadway. See Section 45-1.0 for a discussion on curb types and usage.

A curb and gutter forms a triangular channel that can be an efficient hydraulic conveyance facility to convey runoff of a lesser magnitude than the design flow without interruption to traffic. If 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 of concern for curb-and-gutter flow. This width should be limited as discussed in Section 36-7.0.

Where practical, it is desirable to intercept runoff from a cut slope or other area draining toward the roadway before it reaches it, to minimize the deposition of sediment or other debris on the roadway and to reduce the amount of water which must be carried in the gutter section. A shallow swale section at the edge of the roadway pavement or shoulder offers advantages over a curbed section where curbs are not needed for traffic control. The advantages include a lesser hazard to traffic than a near-vertical curb, and hydraulic capacity that is not dependent on spread on the pavement. A swale section without a curb is appropriate where a curb has been used to prevent water from eroding a fill slope.

36-6.06 Roadside or Median Ditch

A roadside ditch is used with an uncurbed roadway section to convey runoff from the highway pavement and from areas which drain toward the highway. Due to right-of-way limitations, a roadside ditch should not be used on an urban arterial. It can be used in a cut section, depressed section, or other location where sufficient right of way is available, and drives or intersections are infrequent. Where practical, the flow from an area draining toward a curbed highway pavement should be intercepted in the ditch as appropriate.

A median area or inside shoulder should be sloped to a center swale to prevent drainage from the median area from flowing across the pavement. This should be considered for a high-speed facility, or for one with more than two lanes of traffic in each direction.

Chapter Thirty discusses the hydraulic design of a channel.

36-6.07 Bridge Deck

Drainage of a bridge deck is similar to that for a curbed roadway section. However, it can be less efficient because cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets or scuppers have a higher potential for clogging by debris. Bridge-deck construction requires a constant cross slope. Because of the difficulties in providing and maintaining an adequate deck-drainage system, gutter flow from the roadway should be intercepted before it reaches a bridge. Runoff should be collected by inlets, although gutter turnouts may be used for a minor flow. Runoff should also be handled in compliance with applicable stormwater-quality regulations. Deck drainage can be carried several spans to the bridge end for disposal.

The gutter spread should be checked to ensure compliance with the design criteria described in Section 36-7.0. A flat grade or sag vertical curve is not allowed on a bridge on a new alignment. The desirable longitudinal slope for bridge-deck drainage is 0.5% or steeper. A flatter grade will be tolerated where it is not physically or economically desirable to satisfy the above criteria.

A bridge deck may not require drainage structures. To determine the length of deck permitted without drainage structures and without exceeding the allowable spread, see Chapter Thirty-three.

36-6.08 Shoulder Gutter or Curb

A shoulder gutter or sloping curb may be appropriate to protect a fill slope from erosion caused by water from the roadway pavement. It should be considered for a 2:1 fill slope higher than 20 ft. It should also be considered for a 3:1 fill slopes higher than 20 ft if the roadway grade is steeper than 2%. Where permanent vegetation cannot be established, the height criterion should be reduced to 10 ft regardless of the grade. Inspection of the existing and proposed site conditions and contact with maintenance and construction personnel should be made by the designer to determine if vegetation will survive. An erosion-control blanket can be effective to facilitate the establishment of vegetation.

A shoulder gutter or curb, or a riprap turnout should be utilized at a bridge end where concentrated flow from the bridge deck would otherwise flow down the fill slope. The section of gutter should be long enough to include the transitions. A shoulder gutter or riprap turnout is not required on the high side of a superelevated section or adjacent to a barrier wall on a high fill.

36-6.09 Median or Median Barrier

A median is used to separate opposing lanes of traffic on a divided highway. It is preferable to slope a median area or inside shoulder to a center depression to prevent stormwater in the median area from flowing across the traveled way. Where a median barrier is used, or on a horizontal curve with associated superelevations, it is necessary to provide inlets and connecting storm drains to collect the water which accumulates against the barrier. A slotted drain adjacent to the median barrier or weep holes in the barrier can also be used for this purpose.

36-6.10 Impact Attenuator

The location of an impact-attenuator system should be reviewed to determine the need for a drainage structure. It is 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 where superelevation or other grade separation occurs, a grate inlet or a slotted drain may be needed to prevent water from flowing through the clear opening and crossing the highway lanes or ramp lanes. A curb, curb-type structure, or swale cannot be used to direct water across the clear opening because vehicular vaulting can occur once the attenuator system is impacted.

36-7.0 STRUCTURE-SIZING PROCESS

The following is a summary of the hydraulic processes for sizing a storm-drain system or slotted-drain inlet.

36-7.01 Storm-Drain System

36-7.01(01) Design Frequency and Spread

The design-storm frequency for pavement drainage should be consistent with the frequency selected for other components of the storm-drain system. For pavement drainage, the design frequency must include both the recurrence interval of the rainfall and the allowable spread of water in the gutter. See Figure 36-7A for INDOT practices.

The factor that governs how much water can be tolerated in the curb-and-gutter section and on the adjacent roadway is water spread. Water is allowed to spread onto the roadway area within tolerable limits because it is not economically feasible to limit it within a narrow gutter width.

The spread should be held to the specified width for the design frequency. For a storm of greater magnitude, the spread can be allowed to utilize most of the pavement as an open channel. For a curb-and-gutter section with 4 or more lanes, or a gutter-section roadway with no parking, it is not practical to avoid travel-lane flooding where the longitudinal grade is 0.2% to 1%. However, flooding should not extend beyond the lane adjacent to the gutter or shoulder for design conditions. INDOT design criteria for allowable water spread are shown in Figure 36-7A.

The median-inlet spacing for an Interstate route or other divided highway is also based on an allowable-spread width. Runoff collected by inlets in a grass or paved median must not encroach beyond the inside traveled way edge for the storm frequency shown in Figure 36-7A.

36-7.01(02) Inlet Spacing

Curb-inlet spacing must be in accordance with accepted engineering practice. The designer must contact the Hydraulics Team if the intended calculation method is acceptable. Gutter flow that bypasses curb inlets installed on grade must be accounted for at a downstream structure. Flanking inlets should be provided at a sag location to mitigate ponding problems resulting from grate clogging.

After calculating the required spacing, actual inlet locations must be determined. Section 36-10.0 provides the Department’s hydraulic calculations for inlet spacing. Section 36-9.03 provides criteria for inlet locations independent of hydraulic calculations. Each curb inlet and its associated lateral line must be included in the system modeling required for the design- and check-storm evaluation discussed below.

1. Design Storm. Each storm-drain structure must be designed so that Q_{10} passes through each structure via gravity. See Section 36-12.0.

2. Check Storm. The storm-drain network must accommodate the Q_{50} storm event. The system may operate under pressure, but the Hydraulic Grade Line (HGL) must remain below the rim elevation at each system manhole, inlet, catch basin, or similar structure. See Section 36-13.0.

The design process for a storm-drain structure does not require two sets of hydraulic calculations, because each pipe material acceptable for use as a storm drain has a smooth-interior designation. Therefore, computer modeling or hand calculations for storm-drain pipe sizing can be based on a Manning's *n* value of 0.012.

36-7.01(03) Pipe Size, Cover, and Velocity

The minimum pipe size that can be used for a storm drain structure is 12 in. dia. or 1.11 ft^2. The cover provided over a storm-drain structure must be at least 1 ft and not greater than 100 ft. The minimum full-flow velocity for a storm drain structure is 2.5 ft/s, and the recommended maximum velocity is 6.5 ft/s. A storm-drain outlet structure also requires an energy dissipator to mitigate potential erosion. The dissipator riprap-gradation requirements are identical to those outlined for a culvert structure. See Chapter Thirty-four. Contact the Hydraulics Team for additional instructions if the required riprap gradation is prohibited due to clear zone or other issues.

If a satisfactory pipe type cannot be identified for a storm-drain structure, the only acceptable specialty-structure type is a precast-reinforced concrete box section. If a suitable precast reinforced-concrete box section size cannot be determined, contact the Hydraulics Team for additional instructions.

36-7.02 Slotted-Drain Pipe or Slotted-Vane-Drain Pipe

The design requirements for this structure type depend on the structure application. See Sections 36-9.02 and 36-10.0 for a discussion on the hydraulic design of a slotted-drain inlet. The following provides the applications and the associated design requirements.

1. Superelevated Traveled-Way-Edge Installation (Slotted-Drain Pipe). If installed adjacent to the edge of a superelevated section, the slotted-drain pipe sizing will be based on a 50-year storm frequency for an Interstate facility, or a 10-year storm frequency for another type of facility. The pipe sizing must be in accordance with accepted practices described in recognized engineering publications. See Section 36-10.06.

2. Gutter Installation at Sag Curb Inlet (Slotted-Drain Pipe). The design-storm requirement for this installation is identical to that for a storm drain. The length and size of pipe required must be determined in accordance with accepted practices described in recognized engineering publications. See Section 36-10.05.

3. Storm-Drain Structure. A slotted-drain pipe or slotted-vane-drain pipe installed as a component of a storm-drain system must adequately intercept sheet flow and also accommodate all upstream runoff collected by the storm-drain system. The structure is first

sized in accordance with the storm-drain sizing-procedure outlined in Section 36-7.01, except that Manning's n = 0.024 for slotted-drain pipe. The pipe size obtained from the process described above must be checked for adequacy for interception of sheet flow. The sheet-flow-interception design-storm frequency will be Q_{50} for an Interstate facility, or Q_{10} for another type of facility.

4. Culvert Structure. The sizing of a slotted-drain pipe with corrugated-interior designation or a slotted-vane-drain pipe with smooth-interior designation for a culvert application is also a two-step process. The structure is first sized as a culvert in accordance with the requirements for culvert sizing (see Chapter Thirty-one). After the appropriate culvert size is determined, it is necessary to verify whether the structure is adequate for intercepting sheet flow at the site.

If the required slotted-drain pipe or slotted-vane-drain pipe size exceeds the maximum size shown on the INDOT *Standard Drawings*, contact the Hydraulics Team for additional instructions.

36-7.03 Pipe Extension

The sizing of a pipe extension for a storm-drain structure is as follows.

1. Match Existing Pipe Size and Interior Designation. If practical, the pipe extension should be the same size and material as the existing pipe. However, at this stage, it is only necessary to identify the required interior designation for the extension.

2. Perform Appropriate Hydraulic Analysis. The hydraulic analysis must verify that all storm-drain design criteria described above are satisfied.

If the extended structure satisfies all of the required design criteria, the structure-sizing process is complete. If the extended structure does not satisfy the required design criteria, the designer must reevaluate whether the existing structure can be replaced with a new structure. If it is not practical to replace the existing pipe because of the construction method, traffic maintenance, or other concern, contact the Hydraulic Team for further instructions.

36-7.04 Sanitary Sewer and Water Utility Coordination

Coordination with each utility should begin as soon as possible once it is determined that the proposed construction will impact existing utility facilities. For an INDOT-route project, the coordination will be administered through the Utilities Team. For a project not on an INDOT route, the designer should contact each affected utility as soon as possible.

Preliminary inlet spacing and trunkline design determinations should be incorporated into the Preliminary Field Check Plans, as required for early coordination with the utility companies.

Final storm-drain design determinations should be incorporated into the Hearing Plans so that final utility coordination can begin upon design approval.

If it is determined that utility relocation work will be included in the contract, the designer must verify that all elements of the utility construction are included in the contract documents. For example, the INDOT *Standard Specifications* do not include material or testing requirements for sanitary-sewer or water-main pipe. Therefore, if construction of these facilities is required, the designer is responsible for including all applicable requirements in the contract via special provisions. If the utility has specific casting, manhole, or other facility requirements that differ from those included in the INDOT *Standard Specifications* or *Standard Drawings*, these requirements must be included in the contract via plan details or special provisions.

See Chapter Ten for more information on utility accommodation.

36-8.0 GUTTER-FLOW CALCULATIONS

36-8.01 Introduction

Gutter-flow calculations are necessary to relate the quantity of flow, *Q*, in a curbed channel to the spread of water on a shoulder, parking lane, or pavement section. Equations can be utilized to solve for a uniform cross-slope channel, composite gutter section, or V-shaped gutter section. Figure 36-8D can also be used to solve for a composite-gutter section. A computer program, such as the FHWA HEC 12 program, can be used to solve for this, or to determine inlet capacity. A composite gutter section has a greater hydraulic capacity for normal cross slopes than a uniform gutter section, and is therefore preferred. The following provides example problems for each gutter section.

36-8.02 Manning's *n* For Pavement

Figure 36-8A provides the value of Manning's *n* for a street or pavement gutter.

36-8.03 Uniform-Cross-Slope Procedure

Gutter capacity for a uniform cross slope can be determined from the equation as follows:

$$Q = \frac{0.56 S_X^{1.67} S^{0.5} T^{2.67}}{n} \qquad \text{(Equation 36-8.1)}$$

Where:

Q = flow in the gutter (ft^3/s)
S_X = cross slope
S = longitudinal slope
T = water spread, ft
n = Manning's n (see Figure 36-8A)

If the gutter geometrics are known, Q or T can be determined if one of these is known. Figure 36-8B illustrates the parameters shown in Equation 36-8.1.

36-8.04 Composite-Gutter-Section Procedure

To solve for composite gutter flow, use Equation 36-8.1, Equation 36-8.2, Equation 36-8.3 and Figure 36-8C as illustrated in the following procedure. Figure 36-8C can be used to determine the flow in a gutter section with width, W, less than the total spread, T. These calculations are used for evaluating a composite gutter section or frontal flow for a grate inlet.

1. Condition 1. Find spread, given flow.

   a. Determine input parameters, including longitudinal slope, S, cross slope, S_X, depressed-section slope, S_W, depressed-section width, W, Manning's n, gutter flow, Q, and a trial value of the gutter capacity above the depressed section, Q_S.

      Example: $S = 0.01$; $S_X = 0.02$; $S_W = 0.06$; $W = 2.0$ ft; $n = 0.016$; $Q = 2.0\ ft^3/s$. Try $Q_S = 0.706\ ft^3/s$.

   b. Calculate the gutter flow, Q_W, in W, using the equation as follows:

      $$Q_W = Q - Q_S \qquad \text{(Equation 36-8.2)}$$

      Therefore, $Q_W = 2.0 - 0.706 = 1.294\ ft^3/s$

   c. Calculate the ratios Q_W/Q and S_W/S_X and use Figure 36-8C to find an appropriate value of W/T:

      $Q_W/Q = 1.294/2.0 = 0.65$. $S_W/S_X = 0.06/0.02 = 3$.
      From Figure 36-8C, $W/T = 0.27$.

d. Calculate the spread, T, by dividing the depressed-section width, W, by the value of W/T from Step 1.c., as follows:

$$T = 2.0/0.27 = 7.41 \text{ ft}$$

e. Find the spread above the depressed section, T_S, by subtracting W from the value of T obtained in Step 1.d., as follows: $T_S = 7.41 – 2.0 = 5.41$ ft

f. Use the value of T_S from Step 1.e., Manning's n, S, and S_X to find the actual value of Q_S from Equation 36-8.1 as follows: $Q_S = 0.494$ ft^3/s

g. Compare the value of Q_S from Step 1.f. to the trial value from Step 1.a. If the values are not comparable, select a new value of Q_S and return to Step 1.a.

Compare 0.494 to 0.706. It is too low. Try $Q_S = 0.812$. Therefore 2.0 – 0.812 = 1.188, and 1.188/2.0 = 0.6. From Figure 36-8C, $W/T = 0.23$. Therefore $T = 2.0/0.23 = 8.70$ ft, and $T_S = 8.70 – 2.0 = 6.7$ ft. From Equation 36-8.1, $Q_S = 0.812$ ft^3/s. Therefore OK.

Answer: Spread, $T = 8.70$ ft.

2. Condition 2. Find gutter flow, given spread.

a. Determine input parameters, including spread, T, spread above the depressed section, T_S, cross slope, S_X, longitudinal slope, S, depressed-section slope, S_W, depressed-section width, W, Manning's n, and depth of gutter flow, d.

Example: Allowable spread, $T = 10.17$ ft; $W = 2.0$ ft; $T_S = 10.17 – 2.0 = 8.17$ ft; $S_X = 0.04$; $S = 0.005$ ft/ft; $S_W = 0.06$; $n = 0.016$; $d = 0.$f3 ft

b. Use Equation 36-8.1 to determine the capacity of the gutter section above the depressed section, Q_S. Use the procedure for uniform cross slope, Condition 2, substituting T_S for T. From Equation 36-8.1, $Q_S = 3$ ft^3/s.

c. Calculate the ratios W/T and S_W/S_X and, from Figure 36-8C, find the appropriate value of E_O, the ratio of Q_W/Q. $W/T = 2.0/10.17 = 0.2$. $S_W/S_X = 0.06/0.04 = 1.5$. From Figure 36-8C, $E_O = 0.46$.

d. Calculate the total gutter flow using the equation as follows:

$$Q = \frac{Q_S}{1 - E_O} \qquad \text{(Equation 36-8.3)}$$

Where: Q = gutter flow rate, ft^3/s

Q_S = flow capacity of the gutter section above the depressed section, ft^3/s
E_O = ratio of frontal flow to total gutter flow, Q_W/Q

Therefore, $Q = 3.00/(1 – 0.46) = 5.55\ ft^3/s$.

e. Calculate the gutter-flow width, W, using Equation 36-8.2 as follows:

$$Q_W = Q - Q_S = 5.55 - 3.00 = 2.55\ ft^3/s$$

Figure 36-8D can also be used to calculate the flow in a composite-gutter section.

36-8.05 V-Type Gutter Section Procedure

Equation 36-8.1 can also be used to solve for a V-type channel. 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 approximate flow conditions in the triangular channel adjacent to a median barrier. It assumes the effective flow is confined to the V-section with spread, T_1.

1. Condition 1. Given flow, Q, find spread, T.

   a. Determine input parameters, including longitudinal slope, S, cross slope, $S_X = S_{X1}S_{X2}/(S_{X1} + S_{X2})$, Manning's n, total flow, Q. Example: $S = 0.01$, $S_{X1} = 0.06$, $S_{X2} = 0.04$, $S_{X3} = 0.015$, $n = 0.016$, $Q = 2.0\ ft^3/s$, shoulder = 6.1 ft. See Figure 36-8E.

   b. Calculate S_X as follows:

$$S_X = \frac{S_{X1}S_{X2}}{S_{X1} + S_{X2}} = \frac{(0.06)(0.04)}{0.06 + 0.04} = 0.024$$

   c. Solve for T_1 using Equation 36-8.1. T_1 is a hypothetical width that is correct if it is contained within S_{X1} and S_{X2}. From Equation 36-8.1, $T_1 = 8.5$ ft. However, because the shoulder width of 6.1 ft is less than 8.5 ft, S_{X2} is 0.04 and the pavement cross slope S_{X3} is 0.015, T will actually be greater than 8.5 ft. Therefore, 8.5 – 2.0 = 6.5 ft, > 4.0 ft. Therefore, the spread is greater than 8.5 ft.

   d. To find the actual spread, solve for depth at points B and C.

   Point B: 6.5 ft at 0.04 = 0.26 ft. Point C: 0.26 ft – (4.0 ft at 0.04) = 0.10 ft

   e. Solve for the spread on the pavement. Pavement cross slope = 0.015.

$T_{0.015} = 0.10/0.015 = 6.67$ ft

f. Find the actual total spread, T. $T = 6.10 + 6.67 = 12.77$ ft

2. Condition 2. Given spread, T, find flow, Q.

a. Determine input parameters, longitudinal slope, S, cross slope, $S_X = S_{X1}S_{X2}/(S_{X1} + S_{X2})$, Manning's n, and allowable spread. Example: $n = 0.016$, $S = 0.015$, $S_{X1} = 0.06$, $S_{X2} = 0.04$, $T = 6.10$ ft

b. Calculate S_X as follows:

$$S_X = \frac{S_{X1}S_{X2}}{S_{X1} + S_{X2}} = \frac{(0.06)(0.04)}{0.06 + 0.04} = 0.024$$

c. Using Equation 36-8.1, solve for Q as follows:

For $T = 6.10$ ft, $Q = 1.0$ ft^3/s

36-9.0 INLETS

36-9.01 General

An inlet is a drainage structure which is utilized to collect surface water through a grate or curb opening and convey it to a storm drain or a direct outlet to a culvert. A grate inlet should be bicycle-safe unless located on a highway where bicycles are not permitted.

36-9.02 Types

Inlets used for the drainage of a highway surface can be divided into three major classes. These classes are discussed as follows. See the INDOT *Standard Drawings* for details on the inlet types used by the Department.

36-9.02(01) Grate Inlet

This consists of an opening in the gutter covered by one or more grates. It is best suited for use on a continuous grade. The grate is susceptible to clogging with debris and, thus, should be supplemented with a curb box and additional grate capacity to allow for partial clogging at a sag point. Flanking inlets are recommended at a major sag point. The grate should be bicycle safe where bicycle traffic is anticipated. It should be structurally designed to handle the appropriate loads if subject to traffic. The width of each inlet casting should match the width of the gutter. See Section 36-10.0 for additional discussion.

36-9.02(02) Combination Inlet

Various types of combination inlet are in use. A curb-box and grate combination is common with the curb opening adjacent to the grate. A slotted inlet is used in combination with a grate, located either longitudinally upstream of the grate, or transversely adjacent to the grate. Engineering judgment 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. A combination inlet may be desirable in a sag because it can provide additional capacity if plugged.

36-9.02(03) Slotted-Drain Inlet

INDOT uses the slotted-drain-pipe inlet on a mainline roadway, and the slotted-vane-drain-pipe inlet on a drive. The slotted-drain is used to intercept sheet flow at the roadway edge. It can also be installed in a concrete gutter in conjunction with a curb inlet at a sag location. The slotted-vane-drain is used to intercept sheet flow on an urban drive. A slotted-drain inlet is used as a component of a storm-drainage system.

The slotted-drain pipe consists of a horizontal metal pipe with a continuous vertical riser and a slotted opening with bars perpendicular to the opening. The slotted-vane drain consists of a gray-iron casting which is placed on top of a horizontal PVC pipe encased in a low-grade concrete. Each type functions as a weir with flow entering from the side. It can be used to intercept sheet flow, collect gutter flow with or without a curb, modify an existing system to accommodate roadway widening or increased runoff, or reduce ponding depth and spread at a grate inlet.

36-9.03 Inlet Location

An inlet is required where needed to collect runoff within the design controls specified in the design criteria (Section 36-7.0). An inlet may be necessary where it contributes little to the drainage area. Such a location should be shown on the plans prior to performing computations regarding discharge, water spread, inlet capacity, or run-by. Location examples are as follows:

1. sag point in the gutter grade;
2. upstream of a median break, entrance- or exit-ramp gore, crosswalk, or street intersection;

3. immediately upstream and downstream of a bridge;
4. immediately upstream of a cross-slope reversal;
5. on a side street at an intersection;
6. at the end of a channel in a cut section;
7. behind a curb, shoulder, or sidewalk to drain a low area; or
8. where necessary to collect snowmelt.

An inlet should not be located in the path where a pedestrian is likely to walk.

36-10.0 INLET SPACING

36-10.01 General

A number of inlets are required to collect runoff at a location with little regard for contributing drainage area as discussed in Section 36-9.0. These should be plotted on the plan first. Locate inlets starting from the crest and working downgrade to the sag point. The location of the first inlet from the crest can be established 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 satisfy the design criteria described in Section 13-7.0. Where the contributing drainage area consists of a strip of land parallel to and including a portion of the highway, the location of the first inlet can be calculated as follows:

$$L = \frac{43\,560Qt}{CWi} \qquad \text{(Equation 36-10.1)}$$

Where:

	L	=	distance from the crest, ft
	Q_t	=	maximum allowable flow, ft^3/s
	C	=	composite runoff coefficient for contributing drainage area
	W	=	width of contributing drainage area, ft
	i	=	rainfall intensity for design frequency, in/h

If the drainage area contributing to the first inlet from the crest is irregular in shape, trial and error will be necessary to match a design flow with the maximum allowable flow. Equation 36-10.1 is an alternative form of the Rational Equation.

To space successive downgrade 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 run-by. The run-by 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. Figure 36-10K is an inlet-spacing computation sheet which can be utilized to record the spacing calculations. An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

36-10.02 Grate Inlet On Grade

The capacity of a grate inlet depends upon its geometry, 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 a gutter inlet and a curb-opening inlet. At a low velocity, all of the water flowing in the section of gutter occupied by the grate, termed frontal flow, is intercepted by the grate inlet. A small portion of the flow along the length of the grate, termed side flow, is intercepted. On a steep slope, a portion of the frontal flow may tend to splash over the end of the grate. Figure 36-10A can be used to determine splashover velocity for a curved vane grate or reticuline grate. Data is not available for other grate types used by INDOT. An estimate of splashover velocity for a grate with a rectangular opening, such as the alternative grate for casting type 12, 13, or 14, is approximately 2.0 ft/s less than the splashover velocity for a reticuline grate.

INDOT recommends the curved vane grate for a curb-and-gutter application. Section 36-17.0 provides a hydraulic capacity chart for the curved vane grate inlet used by INDOT. The chart is based on a roadway cross section used by the Department. For another inlet type and roadway cross section, the procedure for determining the hydraulic performance is described below.

FHWA has developed computer software, HY12, which will analyze the flow in a gutter and the interception capacity of a grate inlet, curb-opening inlet, slotted-drain inlet, or combination inlet on a continuous grade. Both uniform and composite cross-slopes can be analyzed. The program can analyze a curb-opening, slotted-drain, or grate inlet in a sag. Enhanced versions by private vendors have made the program more user-friendly and have improved its usefulness. Not all INDOT grate configurations have been included in the HEC 12 program. The curved vane grate and the reticuline grate used in the program are similar to the INDOT grates and can be used by inputting the appropriate size. Other grate types, such as INDOT casting type 12, 13, or 14, are not included in HEC 12. However, grate-inlet-capacity curves are available from manufacturers and are recommended for use.

The ratio of frontal flow to total gutter flow, E_O, for a straight cross slope can be determined from the equation as follows:

$$E_O = \frac{Q_W}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \qquad \text{(Equation 36-10.2)}$$

Where: Q = total gutter flow, ft^3/s

Q_W = flow in width W, ft^3/s

W = width of depressed gutter or grate, ft
T = total spread of water in the gutter, ft

Figure 36-8C provides a graphical solution for E_O for a straight cross slope or a depressed-gutter section.

The ratio of side flow, Q_S, to total gutter flow is as follows:

$$\frac{Q_S}{Q} = 1 - \frac{Q_W}{Q} = 1 - E_O \qquad \text{(Equation 36-10.3)}$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the equation as follows:

$$R_f = 1 - 0.09(V - V_O) \qquad \text{(Equation 36-10.4)}$$

Where: V = velocity of flow in the gutter, ft/s
V_O = gutter velocity where splashover first occurs, ft/s

This ratio is equivalent to frontal-flow-interception efficiency. Figure 36-10A provides a solution for Equation 36-10.4 which reflects grate length, bar configuration, and gutter velocity at which splashover occurs. The gutter velocity needed to use Figure 36-10A is total gutter flow divided by the area of flow.

The following equation may be used to solve for velocity in a triangular gutter section with known cross slope, slope, and spread.

$$V = \frac{1.12 S^{0.5} S_X^{0.67} T^{0.67}}{n} \qquad \text{(Equation 36-10.5)}$$

Where: V = velocity of flow in gutter, ft/s
S = longitudinal slope of gutter
S_X = cross slope
T = water spread, ft

Figure 36-10B illustrates the gutter cross section to which Equation 36-10.5 applies.

The ratio of side flow intercepted to total side flow, R_S, or side flow interception efficiency, is expressed as follows:

$$R_S = \frac{SL^{2.3}}{SL^{2.3} + 0.15V^{1.8}} \qquad \text{(Equation 36-10.6)}$$

Where: V = velocity of flow in gutter, ft/s
L = length of grate, ft
S_X = cross slope

Figure 36-10C provides a solution to Equation 36-10.6.

The efficiency, E, of a grate is expressed as follows:

$$E = R_f E_O + R_S(1 - E_O) \qquad \text{(Equation 36-10.7)}$$

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

$$Q_i = EQ = Q[R_f E_O + R_S(1 - E_O)] \qquad \text{(Equation 36-10.8)}$$

* * * * * * * * * *

Example 36-10.1

Given: Urban non-freeway; 4-lanes, undivided with crown at centerline
Drainage area: 200 ft residential strip, $C = 0.4$, $S = 0.005$
12 ft lane with 0.02 cross slope and 2 ft gutter at 0.025 cross slope
10-year design, IDF Curve for Indianapolis in Chapter Twenty-nine
Allowable spread $T = 8.0$ ft, $n = 0.016$
$S_O = 0.01$, $S_X = 0.02$, $S_W = 0.025$
Use curves and equations
Use INDOT standard grate types 10 and 11, 16 in. x 36 in.

Find: Maximum allowable flow, Q_T
Q_i intercepted by 16 in. x 36 in. vane grate
Q_r run-by
Location of first and second inlets from crest of hill

See Figure 36-10C(1) for sketch.

Solution:

Step 1. Solve for Q_S using Equation 36-8.1 as follows:

$$Q_S = \frac{0.56}{0.016}(0.02)^{1.67}(0.01)^{0.5}(8.0 - 2.0)^{2.67} = 0.6087 \text{ ft}^3\text{/s}$$

$Q_S = 0.6087$ ft^3/s

Step 2. Use Figure 36-8C to find E_O as follows:

$$\frac{W}{T}=\frac{2.0}{8.0}=0.25; \quad E_O=0.55=\frac{Q_W}{Q}; \quad \frac{S_W}{S_X}=\frac{0.025}{0.02}=1.25$$

Step 3. Find total maximum allowable flow, Q_T,as follows:

$$Q_T=\frac{Q_S}{1-E_O}=\frac{0.6087}{1-0.55}=1.35 \text{ ft}^3\text{/s}$$

Step 4. Determine V from Equation 36-10.5 as follows:

$$V=\frac{1.12}{0.016}(0.01)^{0.5}(0.02)^{0.67}(8)^{0.67}=2.05 \text{ ft/s}$$

Step 5. Determine Q_i from Equation 36-10.8 as follows:

$$Q_i=1.35[(1.0)(0.55)+0.35(1-0.55)]=0.955 \; ft\text{/s}$$

Step 6. From Figure 36-10A, R_f = 1.0; from Figure 36-10C, R_S = 0.35.

Step 7. $Q_r = Q_T - Q_i$. Therefore, Q_r = 1.35 – 0.955 = 0.395.

Step 8. Locate first inlet from crest, use Equation 36-10.1. To find i in the equation, first solve for t_C. From Figure 29-7D, for a residential area, the following apply.

100-ft strip
C = 0.4
S = 0.5%, overland flow
t_C = 15 min
Gutter flow estimated at V = 2.03 ft/s from Step 4.

Try L = 330 ft. Therefore, $t_C=\frac{330}{(2.03)(60)}-2.7$ min.

Total t_C = 15 + 2.7 = 17.7 min
From Figure 29-8C (IDF curve), i = 4.4 in./h
Solve for weighted C value as follows:

$$C=\frac{(100)(0.4)+(26)(0.9)}{126}=0.50$$

Solve for L as follows:

$$Q_i=EQ=Q_T[R_f E_O+R_S(1-E_O)]$$

$$L = \frac{43\,560(1.35)}{(0.50)(4.4)(126)} = 212\ \text{ft. No Good.}$$

Try i = 200 ft/h and recalculate L as follows:

$$L = \frac{43\,560(1.35)}{(0.50)(4.48)(126)} = 208\ \text{ft. OK.}$$

Therefore, place the first inlet 200 ft from the crest.

9. Step 9. Locate the second inlet.

$Q_T = 1.35\ ft^3/s$, $Q_r = 0.395\ ft^3/s$, $Q_{allowable} = 1.35 - 0.395 = 0.955\ ft^3/s$.
Assuming similar drainage area and t_C, i = 4.4 in/h.

$$L = \frac{43\,560(0.955)}{(0.50)(4.4)(126)} = 150\ \text{ft}$$

Therefore, place the second inlet 150 ft from the first inlet.

* * * * * * * * * *

36-10.03 Grate Inlet In Sag

36-10.03(01) Standard Practice

Standard practice is to install two curved vane grates, types 10 or 11, on one frame casting at the sag point. Each vane grate is positioned to receive water from each upstream direction. A curb box is combined with the grate to provide relief if the grate is plugged with debris. The curb box is ignored in the hydraulic-capacity calculations.

A grate inlet in a sag operates as a weir up to a depth of about 0.5 ft and as an orifice for a depth greater than 1.5 ft. Between these depths, a transition from weir- to orifice-flow occurs. The capacity of a grate inlet operating as a weir is as follows:

$$Q_i = CPd^{1.5} \qquad \text{(Equation 36-10.9)}$$

Where: P = perimeter of grate excluding bar widths and side against curb, ft
C = 3.0
d = depth of water at curb measured from the normal cross slope gutter flow line, ft

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

$$Q_i = CA(2gd)^{0.5} \qquad \text{(Equation 36-10.10)}$$

Where: C = orifice coefficient, 0.67
A = clear opening area of the grate, ft^2
g = acceleration due to gravity, 32.2 ft/s^2

Figure 36-10D is a plot of Equations 36-10.9 and 36-10.10 for various grate sizes. The effect of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either the weir or orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

* * * * * * * * * *

Example 36-10.2

The following example illustrates the use of Figure 36-10D.

Given: A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point.

Q = 1.0 ft^3/s, design storm	Q_r = 0.35 ft^3/s
Q = 1.25 ft^3/s, check storm	Q_r = 0.53 ft^3/s
S_X = 0.02 ft/ft	T = 8.0 ft, design
$d = TS_X$ = 0.16 ft	n = 0.016

Use grate types 10 and 11 (16 in. x 36 in.)

Find: Grate size for design Q and depth at curb for check Q. Check spread at S = 0.003 on approaches to the low point.

Solution: Try one grate type 10 and one grate type 11.

1. Design Storm.

   a. P = 16 + 36 + 16 = 68 in. = 5.67 ft

   b. Using Equation 36-10.9, solve for allowable Q as follows:

$$Q = (3.0)(5.67)(0.16)^{1.5} = 1.089 \qquad \text{OK}$$

In accordance with INDOT policy, one grate type 10 and one grate type 11, each with a curb box, will be placed at the sag point. The curb boxes will not be analyzed hydraulically, but are available for drainage if the inlet becomes plugged.

2. Check Storm.

a. $P = 2(16) + 2(36) = 104$ in. ≈ 8.67 ft

b. Determine d as follows:

$$d^{1.5} = \frac{Q}{3.0P} = \frac{1.089}{(3.0)(8.67)} = 0.42. \quad d = 0.119 \text{ ft} \qquad \text{OK}$$

INDOT practice is to provide a grade of 0.3% within 50 ft of the level point in a sag vertical curve. Check T at $S = 0.003$ for the design and check the flow as follows:

c. Use Equation 36-8.1. Determine T for the design storm as follows:

$$T^{2.67} = \frac{(0.35)(0.016)}{0.56(0.02)^{1.67}(0.003)^{0.5}} = 125.5 \quad T = 6.11 \text{ ft} \qquad \text{OK}$$

d. Determine T for the check storm as follows:

$$T^{2.67} = \frac{(0.53)(0.016)}{0.56(0.02)^{1.67}(0.003)^{0.5}} = 7.99. \quad T = 7.14 \text{ ft} \qquad \text{OK}$$

Thus, a standard castings type 10 and 11 are adequate to intercept the design flow at a spread which does not exceed the design spread. The standard INDOT practice of placing two grates with curb boxes will intercept a check storm within the design criteria and allow for some plugging with debris.

* * * * * * * * * *

36-10.03(02) Flanking Inlets

At a major sag point where significant ponding can occur, such as an underpass or sag vertical curve in a depressed section, a minimum of one flanking inlet should be placed on each side of the inlet at the sag point. The flanking inlets should be placed so that they will limit spread on a low-grade approach to the level point and act in relief of the sag inlet if it becomes clogged. Figure 36-10E shows the spacing required for depth-at-curb criteria and vertical curve length 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 INDOT geometrics specify a maximum K value for the design speed, and a maximum K of 170 considering drainage on a curbed facility.

* * * * * * * * * *

<u>Example 36-10.3</u>

Given: Data from Example 36-10.2:

Speed = 57 mi/h, K = 40 S_X = 0.02 T = 8.0 ft, design

Find: Location of flanking inlets so that they will function in relief of the inlet at the low point where the depth at the curb exceeds the design depth.

Solution: Allowable depth, d, at curb = 8.0 ft at 0.02 = 0.16 ft

Spacing to flanking inlet, x = 64.3 ft, from Figure 36-10E by interpolation.

* * * * * * * * * *

<u>36-10.04 Slotted Inlet</u>

36-10.04(01) Divided Facility with Median Barrier

Snow accumulation adjacent to a concrete barrier on the inside or outside shoulder can present a drainage problem. Therefore, Department practice is to use a slotted drain in conjunction with inlet type H-5 or HA-5 as follows.

1. <u>Tangent Section</u>. Use at every third inlet.

2. <u>Low Side of Superelevated Curve</u>. Use at all inlet sites.

3. <u>Sag Vertical Curve</u>. Use three, centered on the low point.

See the INDOT *Standard Drawings* for more-detailed information.

36-10.04(02) High-Side Shoulder

A slotted-drain pipe is used at locations as follows:

1. high-side shoulder of a superelevated section;
2. the high-side shoulder slopes toward the traveled way;
3. high-traffic-volume freeway; or
4. roadway that is either curbed or uncurbed.

See the INDOT *Standard Drawings* for more-detailed information.

36-10.05 Slotted Inlet on Grade

A slotted inlet, which uses a vertical riser, is an effective pavement drainage inlet which has a variety of applications. It can be used on a curbed or uncurbed section, and offers little interference to traffic operations. It can be placed longitudinally in the gutter or transversely to the gutter. A slotted inlet should be connected into an inlet structure so that it will be accessible to maintenance forces upon plugging or freezing.

36-10.05(01) Longitudinal Placement

Flow interception by a slotted-drain pipe and a curb-opening inlet 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. A slotted inlet can have economic advantages and can be useful in a widening or safety project where right of way is narrow and existing inlet capacity must be supplemented. A curb can be eliminated as a result of utilizing a slotted inlet.

The length of a slotted-drain pipe required for total interception of gutter flow on a pavement section with a straight cross slope is expressed as follows:

$$L_T = KQ^{0.42}S^{0.3}\left(\frac{1}{nS_X}\right)^{0.6} \qquad \text{(Equation 36-10.11)}$$

Where: $K = 0.60$
L_T = slotted inlet length required to intercept 100% of gutter flow, ft

Figure 36-10H illustrates the gutter cross section to which Equation 36-10-11 applies.

The INDOT standard slotted-drain-pipe slot width is 1-¾ in. and the length is 20 ft. The efficiency of a slotted inlet shorter than the length required for total interception is expressed as follows:

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} \qquad \text{(Equation 36-10.12)}$$

Where L = slotted-inlet length, ft

Figure 36-10 I provides a solution of Equation 36-10.12.

The length of inlet required for total interception by a slotted inlet in a composite section can be determined by using an equivalent cross slope, S_e, as follows:

$$S_e = S_X + S'_W E_O \qquad \text{(Equation 36-10.13)}$$

Where: S_X = pavement cross slope
S_W = gutter cross slope
$S'_W = S_W - S_X$
E_O = ratio of flow in the depressed gutter to total gutter flow, Q_W/Q (see Figure 36-8C)

The same equations are used for a slotted inlet or a curb-opening inlet. The following example illustrates the use of this procedure.

* * * * * * * * * *

<u>Example 36-10.4</u>

Given: Longitudinal placement of slotted inlet adjacent to curb.

$S_O = 0.01$ Allowable spread = 10.0 ft $n = 0.016$ $W = 2.0$ ft
Uniform cross slope, $S_X = 0.02$
Composite cross slope, $S_X = 0.02$, $S_W = 0.025$

Find:
(1) Maximum allowable Q
Q_i for a 20.0 ft slotted inlet on a straight cross slope.
(2) Maximum allowable Q
Q_i for a 10.0 ft slotted inlet on a composite cross slope.

Solution: (1) Determine maximum allowable Q from Equation 36-8.1 as follows:

$$Max\ Q = \frac{0.56}{0.016}(0.02)^{1.67}(0.01)^{0.5}(10)^{2.67} = 2.38\ \text{ft}^3/\text{s}$$

Determine L_T from Equation 36-10.11 as follows:

$$L_T - 0.60(2.38)^{0.42}(0.01)^{0.3}\left[\frac{1}{(0.016)(0.02)}\right]^{0.6} = 27.12 \text{ ft}$$

Therefore, $\dfrac{L}{L_T} = \dfrac{20}{27.12} = 0.74$

From Figure 36-10I, E = 0.91.

$Q_i = EQ$ = (0.91)(2.38) = 2.17 ft³/s intercepted.

(2) Determine Q_S from Equation 36-8.1 as follows:

$$Q_S = \frac{0.560}{0.016}(0.02)^{1.67}(0.01)^{0.5}(10.0-2.0)^{2.67} = 1.312 \text{ ft}^3\text{/s}$$

$$\frac{W}{T} = \frac{2.0}{10} = 0.2. \quad \frac{S_W}{S_X} = \frac{0.025}{0.02} = 1.25.$$

From Figure 36-8C, E_O = 0.46.

$$MaxQ = \frac{1.312}{1-0.46} = 2.43 \text{ ft}^3\text{/s}$$

$S'_W = S_W - S_X = 0.025 - 0.02 = 0.005$
$S_e = S_X + S'_W E_O = 0.02 + (0.005)(0.46) = 0.022$

Determine L_T from Equation 36-10.11 as follows:

$$L_T - 0.60(2.38)^{0.42}(0.01)^{0.3}\left[\frac{1}{(0.016)(0.022)}\right]^{0.6} = 25.6 \text{ ft}$$

Therefore, $\dfrac{L}{L_T} = \dfrac{20}{25.6} = 0.78$

From Figure 36-10I, E = 0.92.

$Q_i = EQ$ = (0.92)(2.38) = 2.19 ft³/s intercepted.

* * * * * * * * * *

36-10.05(02) Transverse Placement of Slotted Vane Drain

At a drive where it is desirable to capture virtually all of the flow (e.g., in a drive sloped toward the roadway), a slotted-vane drain can be installed in conjunction with a grate inlet. Tests have indicated that, if the slotted-vane drain is installed perpendicular to the flow, it will capture approximately 1.589 ft^3/s per running foot of drain on a longitudinal slope of 0% to 6%. Capacity curves are available from the manufacturers. The ideal installation utilizes a grate inlet to capture the flow in the gutter and the slotted-vane drain to collect the flow extending into the shoulder. A slotted-vane drain is shaped and rounded to increase inlet efficiency and should not be confused with a vertical-riser-type slotted inlet (i.e., a slotted-drain pipe).

36-10.06 Slotted Inlet In Sag Location

Except adjacent to a concrete barrier (Section 36-10.04), the use of a slotted-drain inlet in a sag configuration is discouraged because of the propensity of such an inlet to collect debris. However, there may be a location where it is desirable to supplement an existing low point inlet with the use of a slotted drain. A slotted inlet in a sag location performs as a weir to a depth of about 0.2 ft, dependent on slot width and length. At a depth greater than about 0.4 ft, it performs as an orifice. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed from the equation as follows:

$$Q_i = 0.8\,L\,W\,(2gd)^{0.5} \qquad \text{(Equation 36-10.14)}$$

Where:
- W = width of slot, ft
- L = length of slot, ft
- d = depth of water at slot, ft
- g = acceleration due to gravity, 32.2 ft/s^2

For a slot width of 1-¾ in., the above equation becomes the following:

$$Q_i = 0.94Ld^{0.5} \qquad \text{(Equation 36-10.15)}$$

The interception capacity of a slotted inlet at a depth between 0.2 ft and 0.4 ft can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of slotted inlet. Figure 36-10J provides solutions for weir flow and a plot representing data at depth between weir and orifice flow.

36-10.07 Inlet-Spacing Computations

To determine the locations of the inlets for a given project, information such as a layout or plan sheet suitable for outlining drainage area, road profile, typical cross section, grading cross section, superelevation diagram, and contour maps is necessary. The inlet computation sheet (Figure 36-10K) should be used to document the computations. The procedure is as follows.

1. Complete the blanks on top of the sheet to identify the job by project, route, date, and initials.

2. Mark on the plan the location of inlets which are necessary without considering specific drainage area. See Section 36-9.03 for additional information.

3. Start at one end of the project, at one high point, and work toward the low point, then space from the other high point back to the same low point.

4. Select a trial drainage area of approximately 300 ft to 500 ft below the high point and, using a drainage-area map, outline the area including drainage that may come over the curb. Where practical, a large area of behind-curb drainage should be intercepted before it reaches the highway. See Section 36-6.05.

5. Describe the location of the proposed inlet by number and station in columns 1 and 2. Identify the curb and gutter type in column 19. A sketch of the cross section should be provided in the open area of the computation sheet.

6. Compute the drainage area in acres and enter in column 3.

7. Select a C value from the appropriate table in Chapter Twenty-nine and enter in column 4.

8. 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 Chapter Twenty-nine. The minimum time of concentration should be 5 min. Enter value in column 5.

9. Using the Intensity-Duration-Frequency curves from Chapter Twenty-nine, select a rainfall intensity at the t_C for the design frequency. Enter in column 6.

10. Calculate Q by multiplying the values in columns 3, 4, and 6. Enter in column 7.

11. Determine the gutter slope at the inlet from the profile grade. Check the effect of superelevation. Enter in column 8.

12. Enter the cross slope adjacent to the inlet in column 9 and the gutter width in column 13. Sketch the composite cross slope and include dimensions.

13. For the first inlet in a series (high point to low point), enter the value from column 7 in column 11 because no previous run-by has occurred yet.

14. Using Equation 36-8.1 or the available computer model, determine the spread, T, and enter it in column 14. Calculate the depth d at the curb by multiplying T by the cross slopes, and enter in column 12. Compare with the allowable spread as determined by the design criteria described in Section 36-7.0. If the value in column 15 is less than the curb height, and the value in column 14 is near the allowable spread, continue on to Step 16. If not acceptable, expand or decrease the drainage area to satisfy the criteria and repeat Steps 5 through 14. Continue until the value in column 14 is near the allowable spread, and then proceed to Step 15.

15: Calculate W/T and enter in column 15.

16. Select the inlet type and dimensions and enter in column 16.

17. Calculate the intercepted flow, Q_i, and enter in column 17. Use Equation 36-8.1 and Figure 36-8C or Figure 36-8D to define the flow in the gutter. Use Figures 36-8C, 36-10A, and 36-10C, and Equation 36-10.8 to calculate Q_i for a grate inlet, and Equation 36-10.11 to calculate Q_i for a curb-opening inlet. See Section 36-10.02 for a grate-inlet example.

18. Calculate the run-by by subtracting the value in column 17 from that in column 11 and enter into column 18 and also into column 10 on the next line if an additional inlet exists downstream.

19. Proceed to the next inlet downgrade. Select an area approximately 300 ft to 400 ft below the first inlet as a first trial. Repeat Steps 5 through 7 considering only the area between the inlets.

20. Compute a time of concentration for the second inlet downgrade based on the area between the two inlets.

21. Determine the intensity based on the time of concentration determined in Step 20 and enter it in column 6.

22. Determine the discharge from this area by multiplying the values in columns 3, 4, and 6. Enter the discharge in column 7.

23. Determine total gutter flow by adding the values in columns 7 and 10, and enter in column 11. The value in column 10 is the same as that in column 18 from the previous line.

24. Determine T based on total gutter flow from column 11 by using Equation 36-8.1 or Figure 36-8D and enter in column 14. If T exceeds the allowable spread, reduce the area and repeat Steps 19 through 24. If T is substantially less than the allowable spread, increase the area and repeat Steps 19 through 24.

25. Select inlet type and dimensions and enter in column 16.

26. Determine Q_i and enter in column 17. See Step 17.

27. Calculate the run-by by subtracting the value in column 17 from that in column 7 and enter in column 16. This completes the spacing design for this inlet.

28. Return to Step 19 and repeat Steps 19 through 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.

36-11.0 MANHOLES

36-11.01 Location

A manhole is utilized to provide entry to a continuous underground storm drain for inspection and cleanout. An inlet box with a grate may be used in lieu of a manhole on the upper end of a storm-drain run to provide access to the system. In this manner, stormwater interception can be achieved with minimal additional cost. The locations where a manhole should be specified are as follows:

1. where two or more storm drains converge;
2. at an intermediate points along a tangent section;
3. where the pipe size changes;
4. where an abrupt change in alignment occurs; and
5. where an abrupt change of the grade occurs.

A manhole should not be located in a traffic lane. However, if this is impossible, it should not be in the normal vehicle wheel path. Where practical, a manhole should be located off the roadway.

36-11.02 Spacing

The spacing should be a maximum of 400 ft.

36-11.03 Types

The types of manholes used by INDOT are listed in Figure 36-11A. The type selected is dependent on the storm-drain pipe size and depth of the manhole.

36-11.04 Sizing

In determining the minimum round manhole size required for a given trunkline pipe size and location, the criteria to be satisfied are as follows:

1. The manhole or inlet structure must be large enough to accept the maximum pipe size shown in Figure 36-11B. In addition to accommodating the maximum pipe size, ensure that not too many pipes enter the manhole to threaten its structural capacity.

2. Knowing the relative locations of two pipes, compute the following:

$$K = \frac{R_1 + T_1 + R_2 + T_2 + 14.2\,in}{\Delta} \qquad \text{(Equation 36-11.1)}$$

Where: K is the in./deg (see Figure 36-11B)
R_1 and T_1 are the interior radius and wall thickness of Pipe No. 1, in.
R_2 and T_2 are the interior radius and wall thickness of Pipe No. 2, in.
Δ = angle between the pipes, deg

* * * * * * * *

Example 36-11.1

Given: Pipe No. 1 dia. = 54 in., Pipe No. 2 dia. = 48 in.
Δ = 140 deg

Solution:

$$\frac{27\,in + 5.6\,in + 24\,in + 5.08\,in + 14.2\,in}{140°}$$

K = 0.542 in./deg

The table indicates the minimum manhole barrel to be 66 in. For the 1650-barrel, the table indicates a maximum pipe size of 48 in. Because the maximum pipe size in the example is 54 in., a 72-in. manhole must be used.

For this example, spacing is not critical and the pipe size governs. If Δ is 115 deg or less, the spacing is critical and a larger manhole barrel is required. If pipes are located at substantially different elevations, pipes may not conflict and the above analysis is unnecessary.

See Figure 36-11C for pipe layout.

36-12.0 STORM DRAINS

36-12.01 Introduction

The design frequency for storm-drain design is 10 years utilizing gravity-flow techniques. The trunkline should be checked only utilizing HGL techniques for the 50-year storm.

After the preliminary locations of inlets, connecting pipes, and outfalls with tailwater have been determined, the next step is the computation of the rate of discharge to be carried by each reach of the storm drain and the determination of the size and grade of pipe required to convey this discharge. This is done by starting at the upstream reach, calculating the discharge and sizing the pipe, then proceeding downstream, reach by reach to the point where the storm drain connects with other drains or the outfall. At a manhole where the pipe size is increased, the pipe crowns should match where grades permit.

The rate of discharge at a point in the storm drain is not necessarily the sum of the inlet flow rates of all inlets above that section of storm drain. It is less than this total. The time of concentration is most influential and, as the time of concentration grows larger, the rainfall intensity to be used in the design grows smaller. 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, though the entire drainage area is not contributing. Flows may arrive at a manhole at which no additional flows enter. Although the time of concentration at this point is technically longer, the flow rate in the downstream pipe should not be reduced. The designer should be aware of unusual conditions and should determine which time of concentration controls for each pipe segment. See Chapter Twenty-nine for a discussion on time of concentration.

For ordinary conditions, a storm drain should be sized based on the assumption that it will flow full or practically full under the design discharge but will not flow under pressure head. The Manning’s formula is recommended for capacity calculations. In a depressed section or underpass where ponded water can be removed only through the storm-drain system, a higher design frequency, 50 years, should be considered to design the storm drain which drains the sag point. See Section 36-10.08 for a discussion on the location of flanking inlets. The main storm drain downstream of the depressed section should be designed by computing the hydraulic grade line and keeping the water-surface elevations below the grates or the established critical elevations for the design storm.

36-12.02 Design Procedure

The storm-drainage system design procedure is as follows.

1. Determine inlet locations and spacing as outlined earlier herein.
2. Prepare a plan layout of the storm-drainage system establishing the design data as follows:
   a. location of storm drains;
   b. direction of flow;
   c. location of manholes; and
   d. locations of existing utilities such as water, gas, sanitary sewer, or underground cables.
3. Determine drainage area, runoff coefficient, and a time of concentration to the first inlet. Using an Intensity-Duration-Frequency (IDF) curve for 10-year recurrence interval, determine the rainfall intensity. Calculate the discharge as $1.008ACI$.
4. Size the pipe to convey the discharge by varying the slope and pipe size as necessary. A storm-drain system is designed for full gravity flow conditions using the design-frequency discharges.
5. Calculate travel time in the pipe to the next inlet or manhole by dividing the 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.
6. Calculate the new area, A, and multiply by the runoff coefficient, C. Add to the previous A time C product. Multiply by 1.008 and the new rainfall intensity, I, to determine the new discharge. Determine the size of pipe and slope necessary to convey the discharge.
7. Continue this process to the storm-drain outlet. Utilize the equations or nomographs to accomplish the design.

8. Complete the design by calculating the hydraulic grade line as described in Section 36-13.0 for the trunkline only for the 50-year recurrence interval. The design procedure should include the following.

   a. Storm-drain design computations can be made on forms as illustrated in Figure 36-12F. An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

   b. All computations and design sheets should be identified. The designer's initials and date of computations should be shown on each sheet. Voided or superseded sheets should be so marked. The origin of data used on one sheet but computed on another should be identified.

36-12.03 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 run-by 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 the storm-drain pipe leading from the sag point must be sized to accommodate this additional run-by within the criteria established. See Section 36-7.0. To design the pipe leading from the sag point, convert the additional run-by created by the 50-year rainfall into an equivalent CA which can be added to the design CA. This equivalent CA can be approximated by dividing the 50-year run-by by 1.008 x I_{10} in the pipe at the sag point.

A separate system may be designed to prevent the above-ground system from draining into the depressed area. This concept may be more costly but may be justified. Another method is to design the upstream system for a 50-year design to minimize the run-by to the sag point. Each method must be evaluated on its own merits and the impacts and risk of flooding a sag point must be assessed.

36-12.04 Hydraulic Capacity

The formula for determining the hydraulic capacity of a storm-drain for gravity and pressure flow is the Manning's formula, expressed by the equation as follows:

$$V = \frac{1.486}{n} R^{0.67} S^{0.5} \qquad \text{(Equation 36-12.1)}$$

Where: V = mean velocity of flow, ft/s
n = Manning's roughness coefficient

R = hydraulic radius, ft. R = area of flow divided by the wetted perimeter
S = slope of the energy grade line

In terms of discharge, the above formula becomes the following:

$$Q = VA = \frac{1.486}{n} AR^{0.67} S^{0.5} \qquad \text{(Equation 36-12.2)}$$

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

For a storm drain flowing full, the above equations become the following:

$$V = \frac{0.59}{n} SD^{0.67} \qquad Q = \frac{0.463}{n} S^{0.5} D^{2.67} \qquad \text{(Equation 36-12.3)}$$

Where D = diameter of pipe, ft

The nomograph solution of Manning's formula for full flow in a circular storm drain is shown in Figure 36-12A, Figure 36-12B, and Figure 36-12C. Figure 36-12D has been provided to assist in the solution of Manning's formula for part full flow in a storm drain.

36-12.05 Minimum Grade

A storm drain should be designed such that the velocity of flow will not be less than 2.67 ft/s at design flow. For a flat grade, the components should be designed so that the flow velocity will increase progressively throughout the length of the pipe system. The storm-drainage system should be checked to ensure that there is sufficient velocity in all of the drains to deter settling of particles. The minimum slope, *S*, required for a velocity of 2.67 ft/s can be calculated as shown below, or the value shown in Figure 36-12E can be used.

$$S = \frac{(nV)^2}{R^{1.33}} \qquad \text{(Equation 36-12.4)}$$

The maximum velocity of flow should not exceed 8.3 ft/s.

36-13.0 HYDRAULIC GRADE LINE

36-13.01 Introduction

The hydraulic grade line (HGL) is the last feature to be established regarding the hydraulic design of a storm drain. This grade line aids the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise if the system is operating from a flood of design frequency. INDOT policy is that the maximum HGL is at the top of the inlet or manhole for Q_{50} flow.

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. A concern with a storm drain 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 planned, including evaluation of the potential for excessive and inadvertent flooding created if 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. A storm-drain system can often alternate between pressure and open-channel flow conditions from one section to another.

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 the in-place 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. See Figure 31-5D for a sketch of a culvert outlet which depicts the difference between the HGL and the energy grade line (EGL). The EGL should be computed first, then the velocity head, $V^2/2g$, should be subtracted to obtain the HGL.

36-13.02 Tailwater

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 is a very large outfall with low tailwater for which a water-surface profile calculation is appropriate to determine the location where the water surface will intersect the top of the barrel and full-flow calculations can begin. The downstream water-surface elevation is based on critical depth or the tailwater, whichever is higher.

In estimating tailwater depth on the receiving stream, the designer will consider the joint or coincidental probability of two events occurring at the same time. For 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 discharge on a small basin may not be critical for a larger basin. If the same storm causes peak discharge on both basins, the peaks will be out of phase. To aid in the evaluation of joint probabilities, see to Figure 36-13A.

36-13.03 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 determined as follows:

$$H_o = 1.0\left(\frac{V^2}{2g} - \frac{V_d^2}{2g}\right) \qquad \text{(Equation 36-13.1)}$$

Where: V = average outlet velocity, ft/s
V_d = channel velocity downstream of outlet, ft/s

If $V_d = 0$ as in a reservoir, the exit loss is one velocity head. At a location with a flap gate at the outlet to prevent water from backing up into the system, an additional loss caused by the flap gate may need to be added. A manufacturer should be consulted for information. For part full flow where the pipe outlets into a channel with moving water, the exit loss may be reduced to virtually zero.

36-13.04 Bend Loss

The bend loss coefficient for storm-drain design is minor, but can be evaluated using the formula as follows:

$$H_b = \frac{0.0033\Delta V_o^{\,2}}{2g} \qquad \text{(Equation 36-13.2)}$$

Where Δ = angle of curvature, deg.

36-13.05 Pipe-Friction Loss

The friction slope is the energy gradient for that run. The friction loss is the energy gradient multiplied by the length of the run in feet. Energy loss from pipe friction can be determined from the Manning's formula with terms as previously defined, as follows:

$$S_f = \left(\frac{Qn}{AR^{0.67}} \right)^2 \qquad \text{(Equation 36-13.3)}$$

The head loss due to friction can be determined from the formula as follows:

$$H_f = LS_f \qquad \text{(Equation 36-13.4)}$$

The Manning's formula can be used to determine friction loss as follows:

$$H_f = \frac{2.87 n^2 V^2 L}{D^{1.33}} \qquad \text{(Equation 36-13.5)}$$

$$H_f = \left(\frac{64.4 n^2 L}{R^{1.33}} \right) \left(\frac{V^2}{2g} \right) \qquad \text{(Equation 36-13.6)}$$

Where:

- H_f = total head loss due to friction, ft
- n = Manning's roughness coefficient
- D = diameter of pipe, ft
- L = length of pipe, ft
- V = mean velocity, ft/s
- R = hydraulic radius, ft
- g = acceleration due to gravity, 32.2 ft/s^2
- S_f = slope of hydraulic grade line

36-13.06 Manhole Losses

The head loss encountered in flowing from one pipe to another through a manhole is represented as being proportional to the velocity head at the outlet pipe. Using K to identify this constant of proportionality, the energy loss is approximated as $K(V_o^2/2g)$. Experimental studies have determined that K can be approximated as follows:

$$K = K_O \, C_D \, C_d \, C_Q \, C_p \, C_B \qquad \text{(Equation 36-13.7)}$$

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

1. Relative Manhole Size. K_O is estimated as a function of the relative manhole size and the angle of deflection between the inflow and outflow pipes. See Figure 36-13B.

$$K_O = 0.1\left(\frac{b}{D_O}\right)(1-\sin\theta)+1.4\sin\theta\left(\frac{b}{D_O}\right)^{0.15} \qquad \text{(Equation 36-13.8)}$$

Where: θ = angle between the inflow and outflow pipes, deg
b = manhole diameter, in.
D_O = outlet-pipe diameter, in.

2. Pipe Diameter. A change in head loss due to differences in pipe diameter is significant only in a pressure-flow situation where the depth in manhole to outlet pipe diameter ratio, d/D_O, is greater than 3.2.

$$C_D = \left(\frac{D_O}{D_i}\right)^3 \qquad \text{(Equation 36-13.9)}$$

Where: D_i = incoming pipe diameter, in.
D_O = outgoing pipe diameter, in.

3. Flow Depth. The correction factor for flow depth is significant only in free-surface flow or low pressure, where the d/D_O ratio is less than 3.2. Water depth in the manhole is approximated as the level of the hydraulic-grade line at the upstream end of the outlet pipe. The correction factor for flow depth, C_d, is calculated as follows:

$$C_d = 0.5\left(\frac{d}{D_O}\right)^{0.6} \qquad \text{(Equation 36-13.10)}$$

Where: d = water depth in manhole above outlet pipe, ft
D_O = outlet pipe diameter, ft

4. 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. It is computed as follows:

$$C_Q = (1-2\sin\theta)\ \text{x}\ \left(1-\frac{Q_i}{Q_O}\right)^{0.75}+1 \qquad \text{(Equation 36-13.11)}$$

Where: C_Q = correction factor for relative flow
θ = angle between the inflow and outflow pipes, deg

Q_i = flow in the inflow pipe, ft^3/s
Q_O = flow in the outlet pipe, ft^3/s

As can be seen from the equation, 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. To illustrate this effect, consider the manhole shown in the sketch and assume the following two discharge situations to determine the impact of pipe 2 entering the access hole.

$$C_{Q3-1} = (1 - 2\sin 180)\left(\frac{1-3.18}{4.24}\right)^{0.75} + 1 = 1.35$$

a. For discharge situation 1, Q_1 = 3.18 ft^3/s, Q_2 = 1.06 ft^3/s, and Q_3 = 4.24 ft^3/s. Therefore, C_Q = 1.35.

b. For discharge situation 2, Q_1 = 1.06 ft^3/s, Q_2 = 3.18 ft^3/s, and Q_3 = 4.24 ft^3/s. Therefore, C_Q = 1.81.

See Figure 36-13B(1) for relative-flow effect.

5. Plunging Flow. The correction factor for plunging flow, C_p, is calculated as follows:

$$C_p = 1 + 0.2\left(\frac{h}{D_O}\right) x \left[\frac{(h-d)}{D_O}\right] \qquad \text{(Equation 36-13.12)}$$

Where:
C_p = correction for plunging flow
h = vertical distance of plunging flow from flowline of incoming pipe to the center of outlet pipe, ft
D_O = outlet pipe diameter, ft
d = water depth in manhole, ft

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet, plunging into the manhole, on the inflow pipe for which the head loss is being calculated. Using the notations in the above sketch for the example, C_p is calculated for pipe No. 1 if pipe No. 2 discharges plunging flow. The correction factor is applied only if $h > d$.

6. Benching. The correction for benching in the manhole, C_B, is obtained from Figure 36-13C. Benching tends to direct flow through the manhole, resulting in reductions in head loss. For a flow depth between the submerged and unsubmerged conditions, a linear interpolation is performed.

7. Summary. To estimate the head loss through a manhole from the outflow pipe to a particular inflow pipe, multiply the above correction factors together to get the head loss coefficient, *K*. This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

36-13.07 Hydraulic-Grade-Line Design Procedure

The equations and charts necessary to manually calculate the location of the hydraulic gradeline are included herein. The HYDRA computer program in the HYDRAIN system is recommended for design of a storm drain. It will include a HGL analysis and a pressure-flow simulation. A step-by-step procedure is described to manually compute the HGL. Figure 36-13D can be used to document the procedure. An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

If the HGL is above the pipe crown at the next upstream manhole, pressure-flow calculations should be performed. If it is below the pipe crown, 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, the hydraulic design is adequate. If the HGL exceeds an inlet elevation, adjustments to the trial design must be made to lower the water-surface elevation.

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.

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 36-13.02 for the procedure.

3. Enter in column 3 the diameter of the outflow pipe, D_O.

4. Enter in column 4 the design discharge for the outflow pipe, Q_O.

5. Enter in column 5 the length of the outflow pipe, L_O.

6. Enter in column 6 the outlet velocity of flow, V_O.

7. Enter in column 7 the velocity head, $V_O^2/2g$.

8. Enter in column 8 the exit loss, H_O.

9. Enter in column 9 the friction slope of the outflow pipe, SF_O. This can be determined by using Equation 36-13.3. This assumes full-flow conditions.

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. For a curved alignment, calculate curve loss by using the formula $H_b = 0.0033\theta V_O^2/2g$, where θ = angle of curvature, deg, and add to the friction loss.

11. Enter in column 11 the initial head loss coefficient, K_O, based on relative manhole size as computed from Equation 36-13.8.

12. Enter in column 12 the correction factor for pipe diameter, C_D, as computed from Equation 36-13.9.

13. Enter in column 13 the correction factor for flow depth, C_d, as computed from Equation 36-13.10. This factor is significant only where the d/D_O ratio is less than 3.2.

14. Enter in column 14 the correction factor for relative flow, C_Q, as computed from Equation 36-13.11.

15. Enter in column 15 the correction factor for plunging flow, C_p, as computed from Equation 36-13.12. This correction factor is applied only if $h > d$.

16. Enter in column 16 the correction factor for benching, C_B, as determined from Figure 36-13C.

17. Enter in column 17 the value of K as computed from Equation 36-13.7.

18. Enter in column 18 the value of the total manhole loss, $KV_O^2/2g$.

19. If the tailwater submerges the outlet end of the pipe, enter in column 19 the sum of the TW-elevation value in column 2 and the exit-loss value in column 7 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 $(d_C + D)/2$.

20. Enter in column 20 the sum of the friction-head value from column 10, the manhole-losses value from column 18, and the energy-grade-line value from 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.

21. Determine the HGL for column. 21 throughout the system by subtracting the velocity-head value from column 7) from the EGL value from column 20.

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 a pipe that is flowing full, as should be the condition for the design of a new system. If a part full flow condition exists, the EGL is located one velocity head above the water surface.

Figure 36-13E provides a summary of energy losses which should be considered. Figure 36-13F illustrates the proper and improper use of energy losses in developing a storm-drain system.

36-14.0 UNDERDRAINS

See Section 52-10.0 for INDOT's design criteria and application for underdrains.

36-15.0 COMPUTER PROGRAMS

To assist with storm-drain-system design, a microcomputer software module has been developed for the computation of the hydraulic grade line. The computer program, called HYDRA, is part of the HYDRAIN system. HYDRA can be used to check design adequacy and to analyze the performance of a storm-drain system under assumed inflow conditions. Section 36-16.0 provides an example problem using the HYDRA computer model.

If another commercial package is used, it must be capable of computing the trunkline size for gravity flow at a 10-year event and performing the hydraulic-grade-line computation for the 50-year event. Hand calculations which satisfy these requirements are also acceptable.

For slotted-drain pipe and slotted-vane-drain pipe, use manufacturers' publications with capture-rate information for a sag or on-grade installation.

36-16.0 EXAMPLE PROBLEM

The following is an example problem of inlet and storm-drain computations worked manually and using microcomputer software. The inlet computations utilized HEC 12, available from McTRANS. The storm-drain calculations utilized HYDRA.

Given: Sketch of roadway segment with inlets located as shown in Figure 36-16A. The drainage area is as indicated on the Inlet Computation sheet.

12-ft travel lane with S_X = 0.02, and 2.0-ft gutter with S_W = 0.025
10-year design, IDF curve for Indianapolis
Allowable spread, T = 6.0 ft; n = 0.016; S_O = 0.012
Curved vane grates type 10 and 11, size 16 in x 36 in

Find: Check spread at inlet locations. Design storm-drain size and slope. Calculate hydraulic grade line.

Solution:

1. Inlet Computations.

   a. Hand calculations documented in Figure 36-16B, Inlet Computation Sheet.

   b. Analysis computed by HEC 12 computer program and documented on printout labeled HEC 12, after Figure 36-16B.

2. Storm-Drain Calculations.

   a. Hand calculations for pipe sizing in Figure 36-16C, Storm-Drain Computation Sheet.

   b. HYDRA calculations for pipe sizing and Hydraulic-Grade Line shown on HYDRA printout, after Figure 36-16C.

36-17.0 INLET-CAPACITY CHART

Because of its frequency of usage by INDOT, Figure 36-17A provides a hydraulic-capacity chart for the curved vane grate, frame casting types 10 and 11. See the INDOT *Standard Drawings*. The inlet-capacity chart has been produced based on the following assumptions.

1. Grate dimensions: 36 in x 16 in
2. S_X = roadway slope = 0.02
3. S_W = gutter-pan slope = 0.025
4. W = gutter width = 2 ft
5. n = 0.016
6. S = longitudinal slope = 0.5% to 7%
7. Q = gutter flow = 0.35 ft^3/s to 8.48 ft^3/s

The assumed roadway conditions for S_X, S_W, and W are those that occur on a curbed facility. Figure 36-17A allows the user to determine the intercepted flow, Q_i, for a given longitudinal slope, S, and total gutter flow, Q. For example:

S = 1%
Q = 4.23 ft^3/s

Figure 36-17A yields Q_i = 1.87 ft^3/s.

36-18.0 REFERENCES

1. American Association of State Highway and Transportation Officials, Volume 9, *Highway Drainage Guidelines, Storm Drainage Systems*, 1992.

2. Federal Highway Administration, *Bridge Deck Drainage Guidelines*, FHWA Report No. RD-014, December 1986.

3. Federal Highway Administration, *Design of Bridge Deck Drainage*, Hydraulic Engineering Circular No. 21, 1993.

4. Federal Highway Administration, *Design of Urban Highway Drainage - The State of The Art*, FHWA-TS-79-225, 1979.

5. Federal Highway Administration, *Drainage of Highway Pavements*, Hydraulic Engineering Circular No. 12, 1984.

6. Federal Highway Administration, *Pavement and Geometric Design Criteria For Minimizing Hydroplaning*, FHWA Report No. RD-79-31, December 1979.

7. Dah-Chen Woo, *Bridge Drainage System Needs Criteria*, U.S. Department of Transportation, Public Roads, Vol.52, No. 2, September 1988.

Str.	Type	Casting Types												
		2	3	4	5	6	7	8	10	12	12A	13	14	15
Catch Basins	A	X	X					X						
	D					X								
	E						X							
	J								X					
	K								X					
	S												X	
	W [1]	X	X					X						
Inlets	A	X	X					X						
	B													X
	C													X
	D					X								
	E						X							
	F						X							
	G						X							
	H, HA				X									
	J								X					
	M								X					
	N									X				
	P										X			
	R											X		
	S												X	
	T												X	
Manholes	A	X		X				X						
	B	X		X				X						
	C [2]	X		X				X						
	D	X		X				X						
	E	X		X				X						
	F	X		X				X						
	G	X		X				X						
	H	X		X				X						
	J	X		X				X						
	K	X		X				X						
	L	X		X				X						
	M	X		X				X						
	N	X		X				X						

Notes: [1] May be substituted for catch basin type A.
[2] May be substituted for manhole type A or B.

COMPATIBILITY OF DRAINAGE STRUCTURES AND CASTINGS

Figure 36-2A

Symbol	Definition	Units
A	Area of cross section	ft^2
A	Watershed area	ac
a	Depth of depression	in.
C	Runoff coefficient or coefficient	-
d	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
E_O	Ratio of frontal flow to total gutter flow Q_w/Q	-
h	Height of curb opening inlet	ft
H	Head loss	ft
I	Rainfall intensity	in./h
K	Coefficient	-
L	Length of curb opening inlet	ft
L	Pipe length	ft
L	Pavement width	ft
L	Length of runoff travel	ft
n	Roughness coefficient in Manning formula	-
P	Perimeter of grate opening, neglecting bars and side against curb	ft
P	Tire pressure	lb/ft^2
Q	Rate of discharge in gutter	ft^3/s
Q_i	Intercepted flow	ft^3/s
Q_S	Gutter capacity above the depressed section	ft^3/s
Q_T	Total flow	ft^3/s
Q_W	Gutter capacity in the depressed section	ft^3/s
R_h	Hydraulic radius	ft
S or S_X	Pavement cross slope	ft/ft
S	Crown slope of pavement	ft/ft
S or S_L	Longitudinal slope of pavement	ft/ft
S_W	Depressed section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
t_C	Time of concentration	min
T_D	Tire tread depth	in
T_S	Spread above depressed section	ft
TXD	Pavement texture depth	in.
V	Vehicle speed	mph
V	Velocity of flow	ft/s
W	Width of depression for curb opening inlets	ft
W_d	Rotational velocity on dry surface	rpm
WD	Water depth	in.
W_W	Rotational velocity on flooded surface	rpm
y	Depth of flow in approach gutter	ft
Z	T/d, reciprocal of the cross slope	-

SYMBOLS AND DEFINITIONS

Figure 36-3A

Type of Facility	Design Frequency	Allowable Spread, *T*
Freeway	50 years	Edge of travel lane
Non-Freeway, ≥ 4 Lanes	10 years	Across one-half travel lane
Two-Lane Facility	10 years	4 ft onto travel lane
Bridge Deck, Non-Freeway		
V ≥ 50 mi/h	10 years	Edge of travel lane
V ≤ 50 mi/h	10 years	3 ft onto travel lane
Ramp, Roadway or Bridge		
V ≥ 50 mi/h	10 years	Edge of travel lane
V ≤ 50 mi/h	10 years	3 ft onto travel lane

DESIGN FREQUENCY AND ALLOWABLE WATER SPREAD

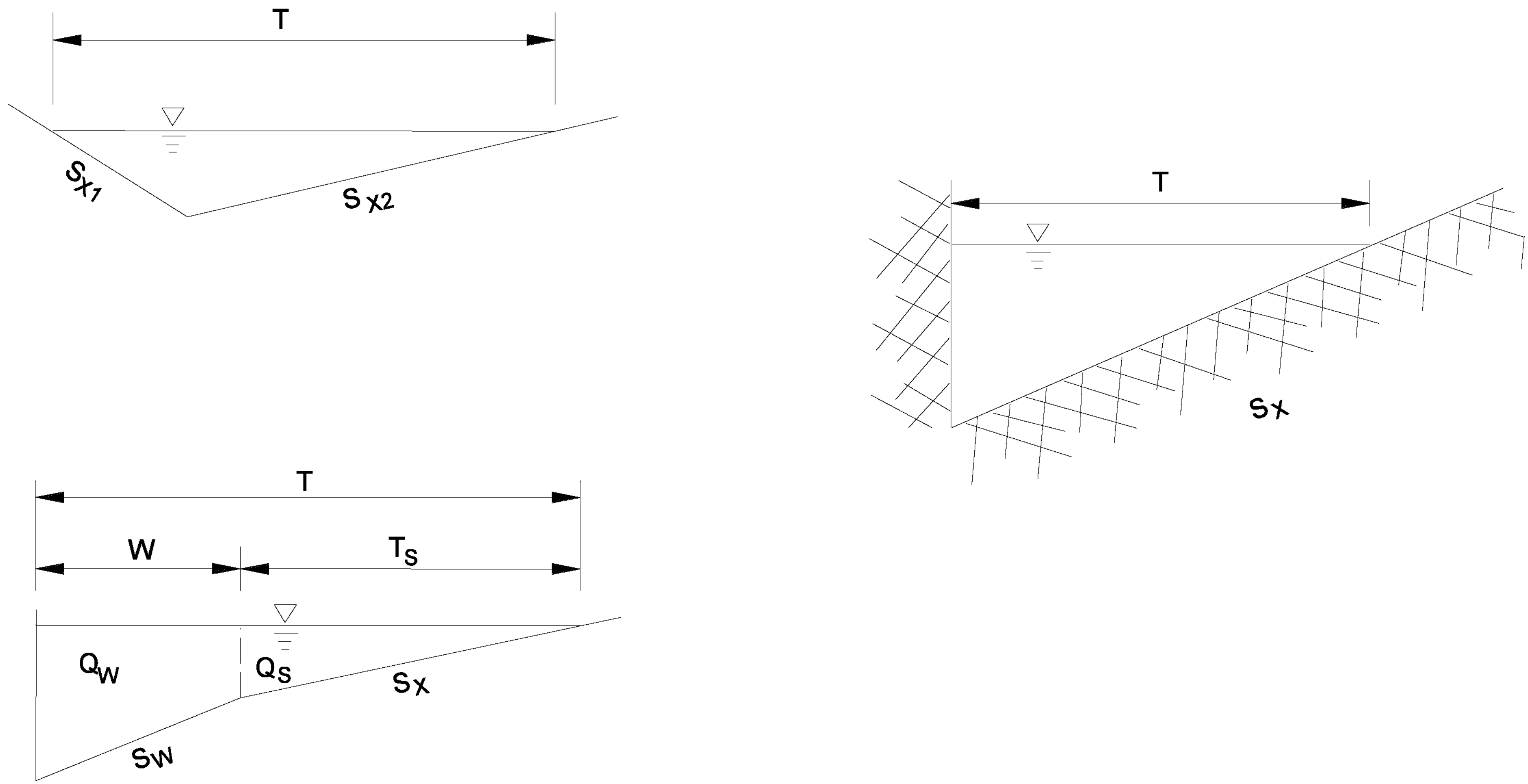
Figure 36-7A

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-asphalt pavement	
Smooth	0.013
Rough	0.015
Concrete pavement	
Float finish	0.014
Broom finish	0.016

Notes: 1. *For a gutter with a small slope where sediment may accumulate, increase* n *value by 0.002.*
2. *Reference: USDOT, FHWA, HDS-3 (1961)*

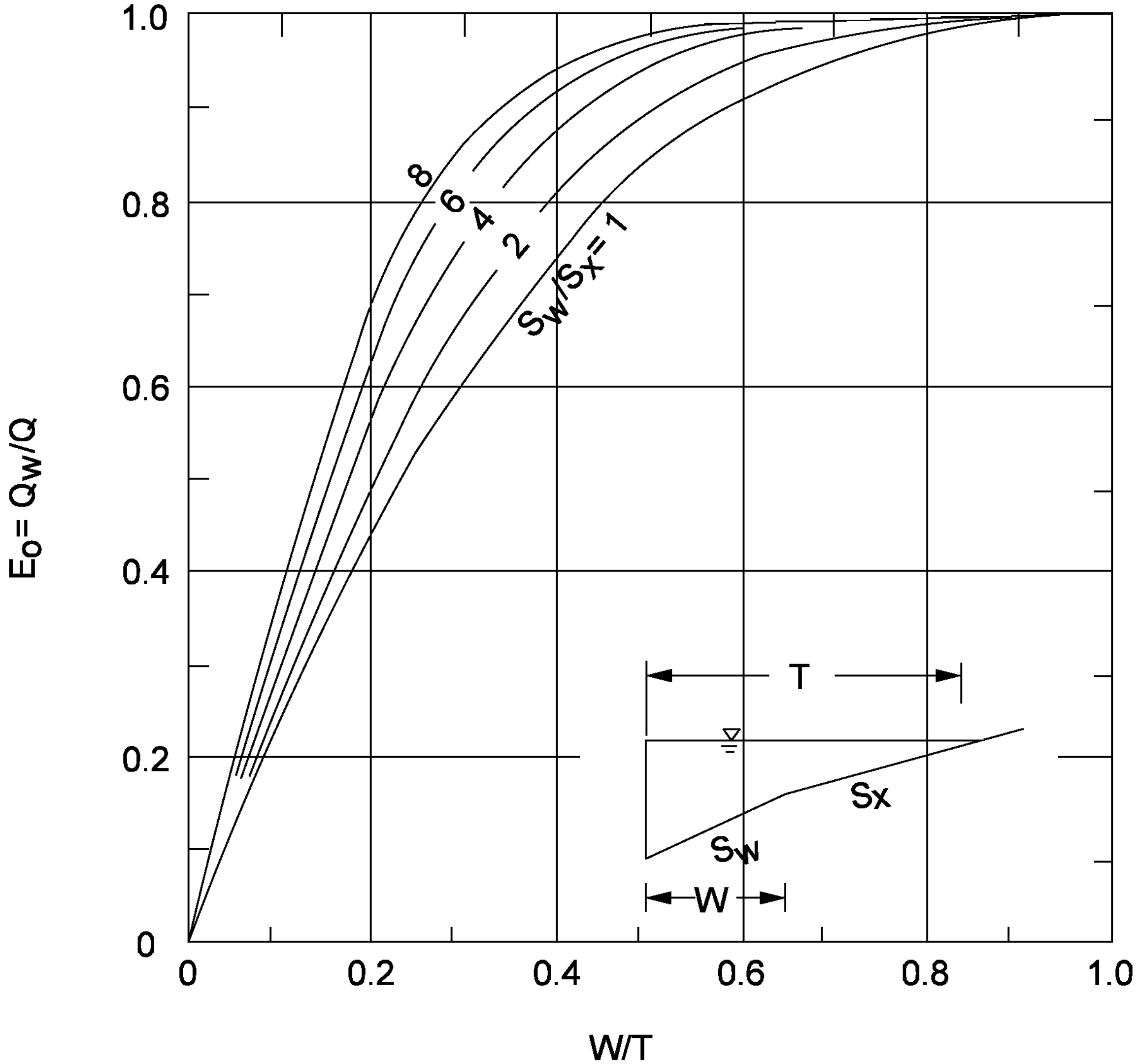
MANNING'S *n* FOR STREET OR PAVEMENT GUTTER

Figure 36-8A

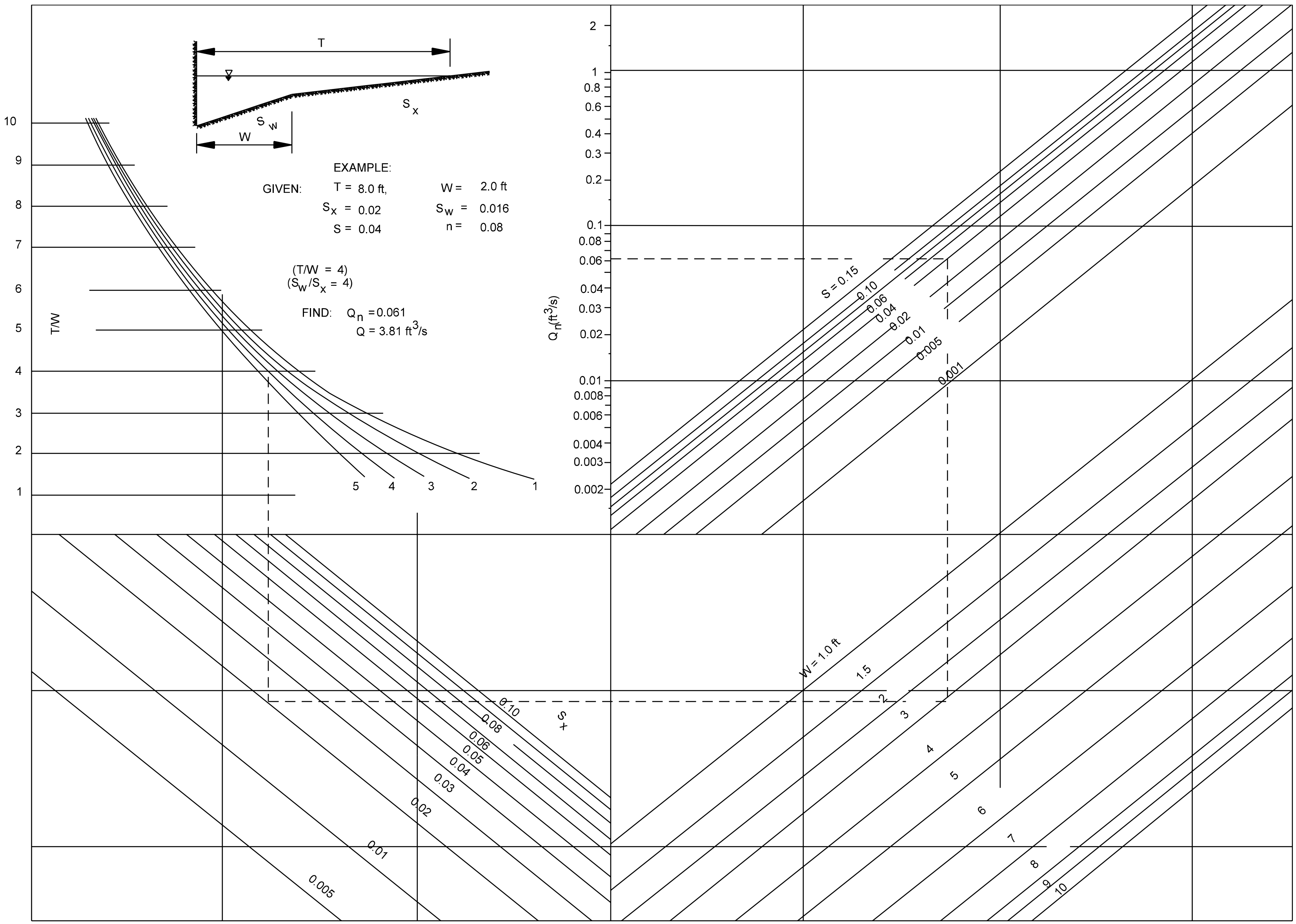


FLOW IN TRIANGULAR GUTTER SECTIONS

Figure 36-8B

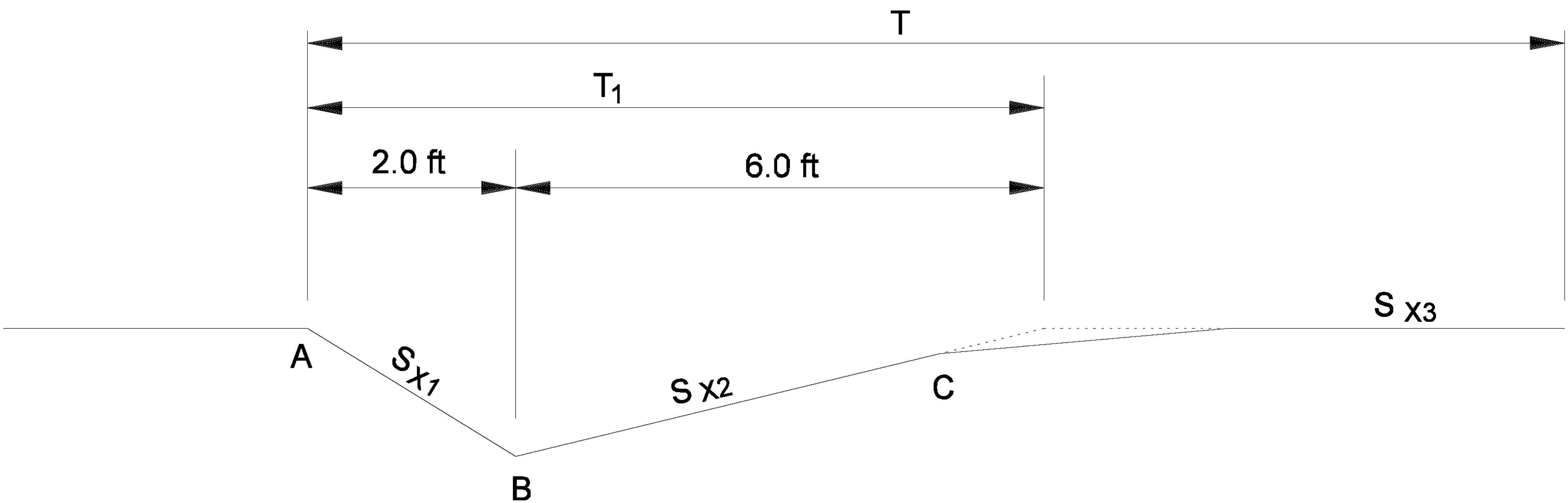


RATIO OF FRONTAL FLOW TO TOTAL GUTTER FLOW
Figure 36-8C



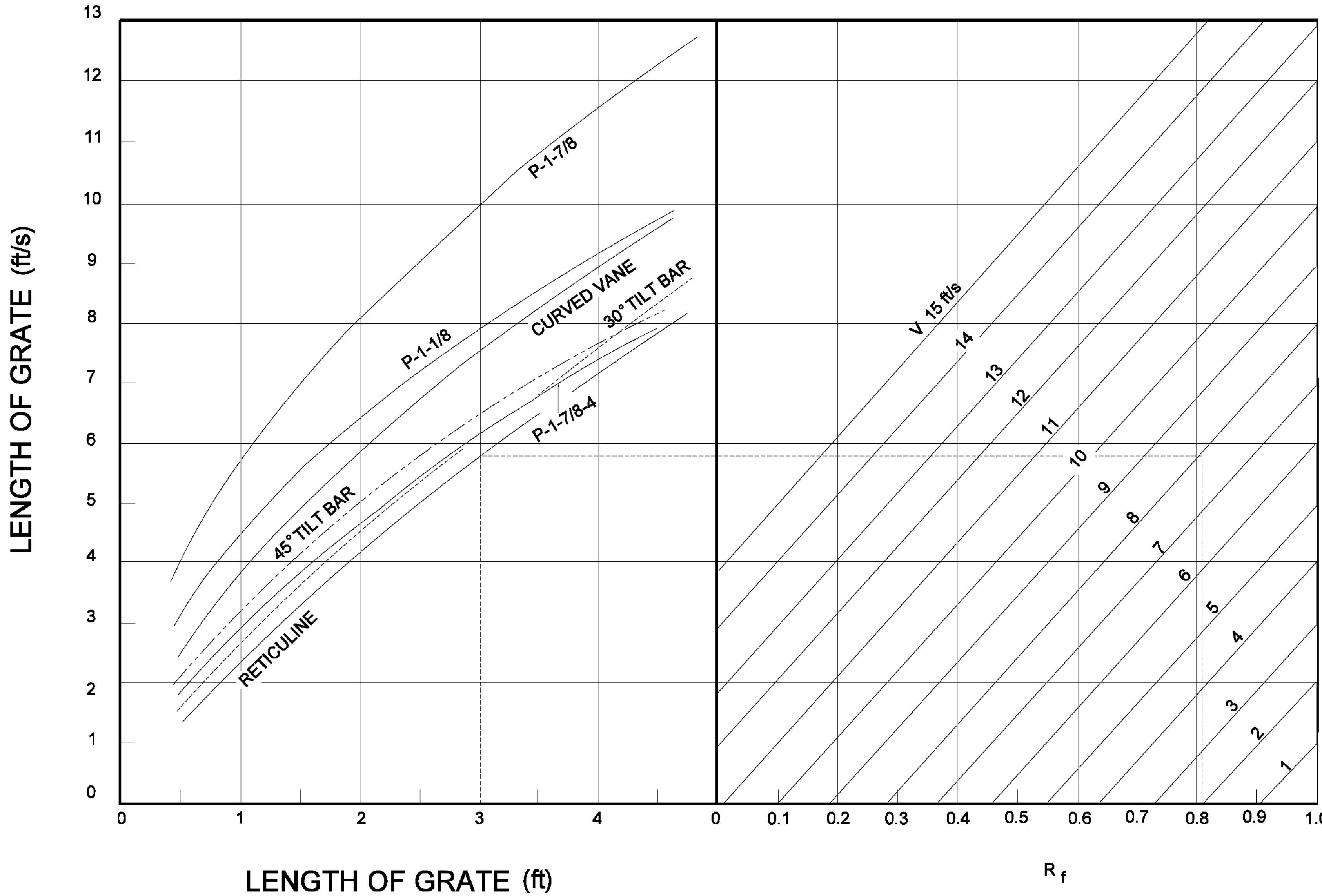
FLOW IN COMPOSITE GUTTER SECTIONS

Figure 36-8D



V-TYPE GUTTER

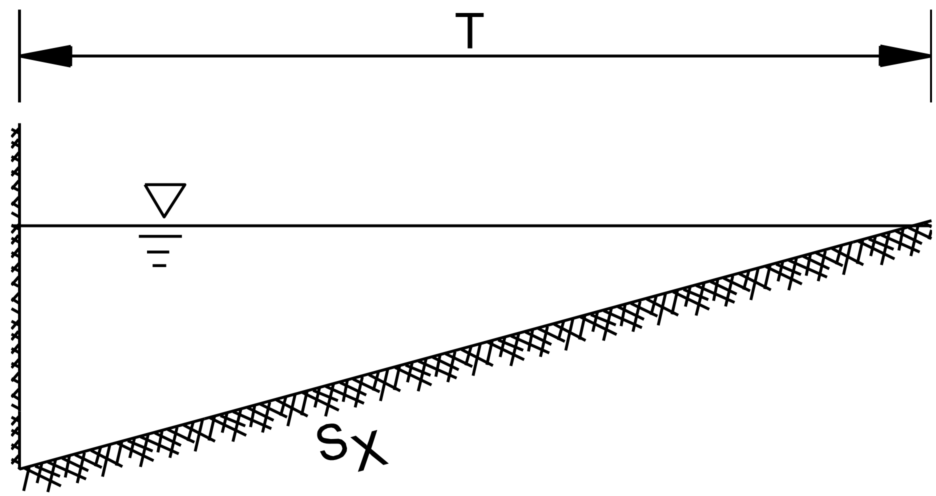
Figure 36-8E



Note: For grate types other that curved vane and reticuline. refer to the manufacture's data for nhydraulic characteristics.

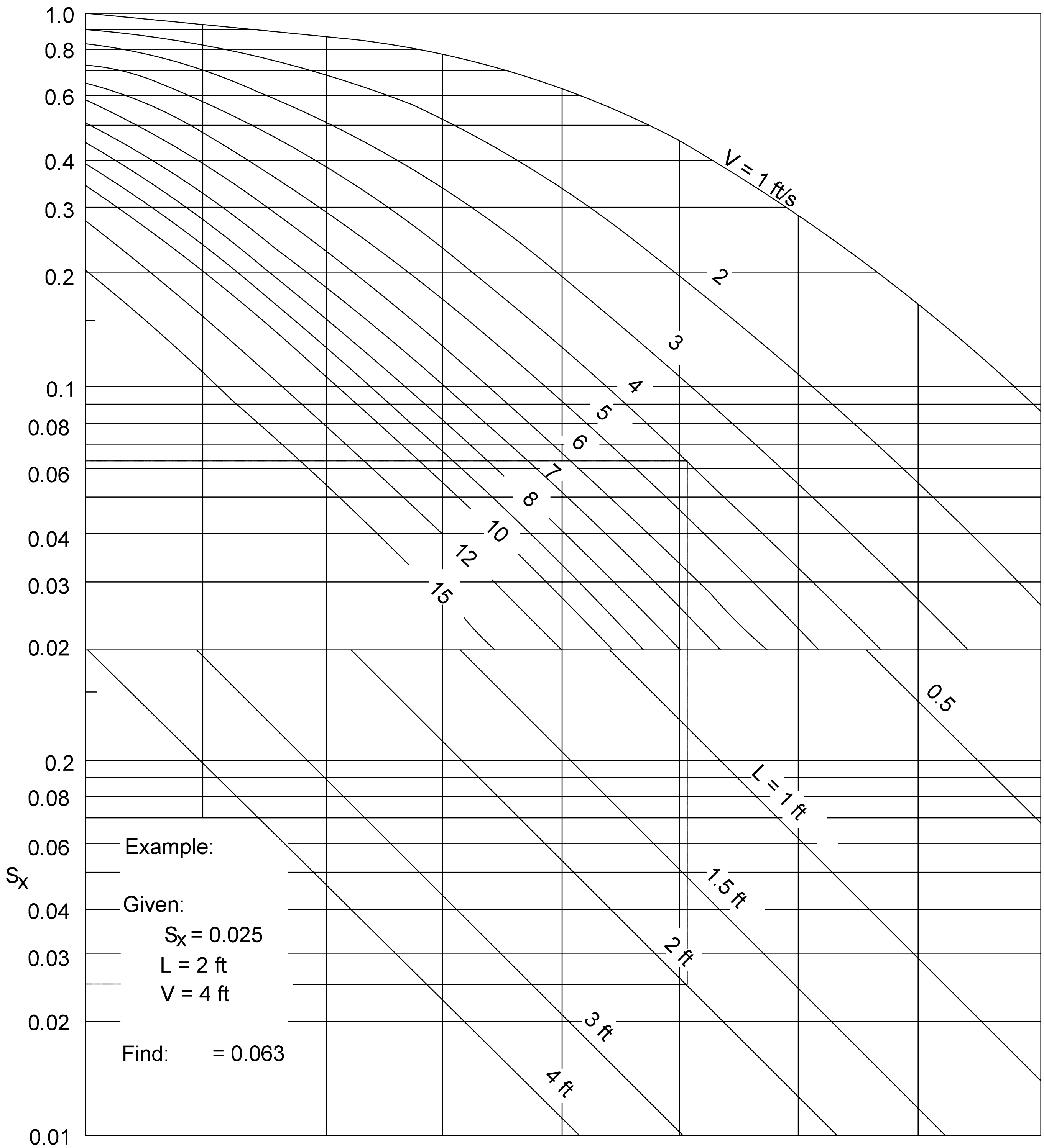
GRATE INLET FRONTAL FLOW INTERCEPTION EFFICIENCY

Figure 36-10A



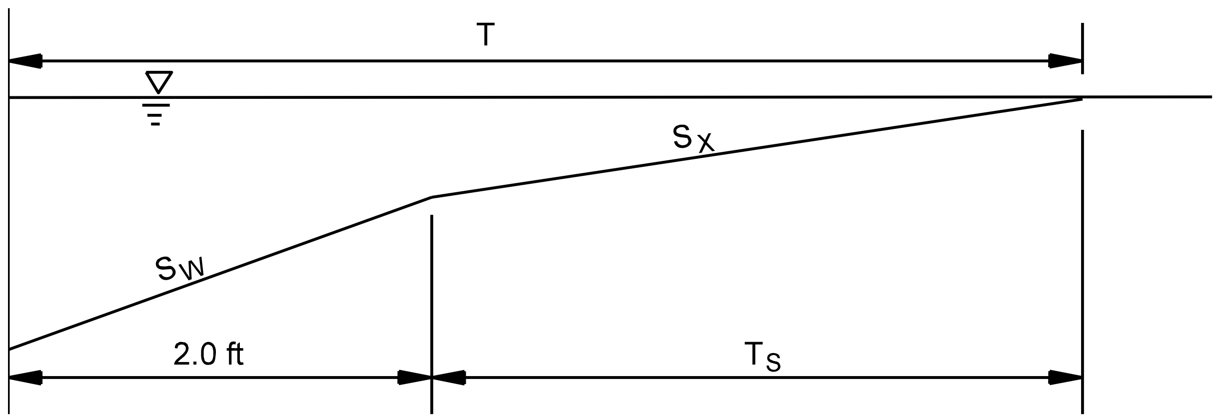
GUTTER CROSS SECTION
(Equation 36-10.5)

Figure 36-10B



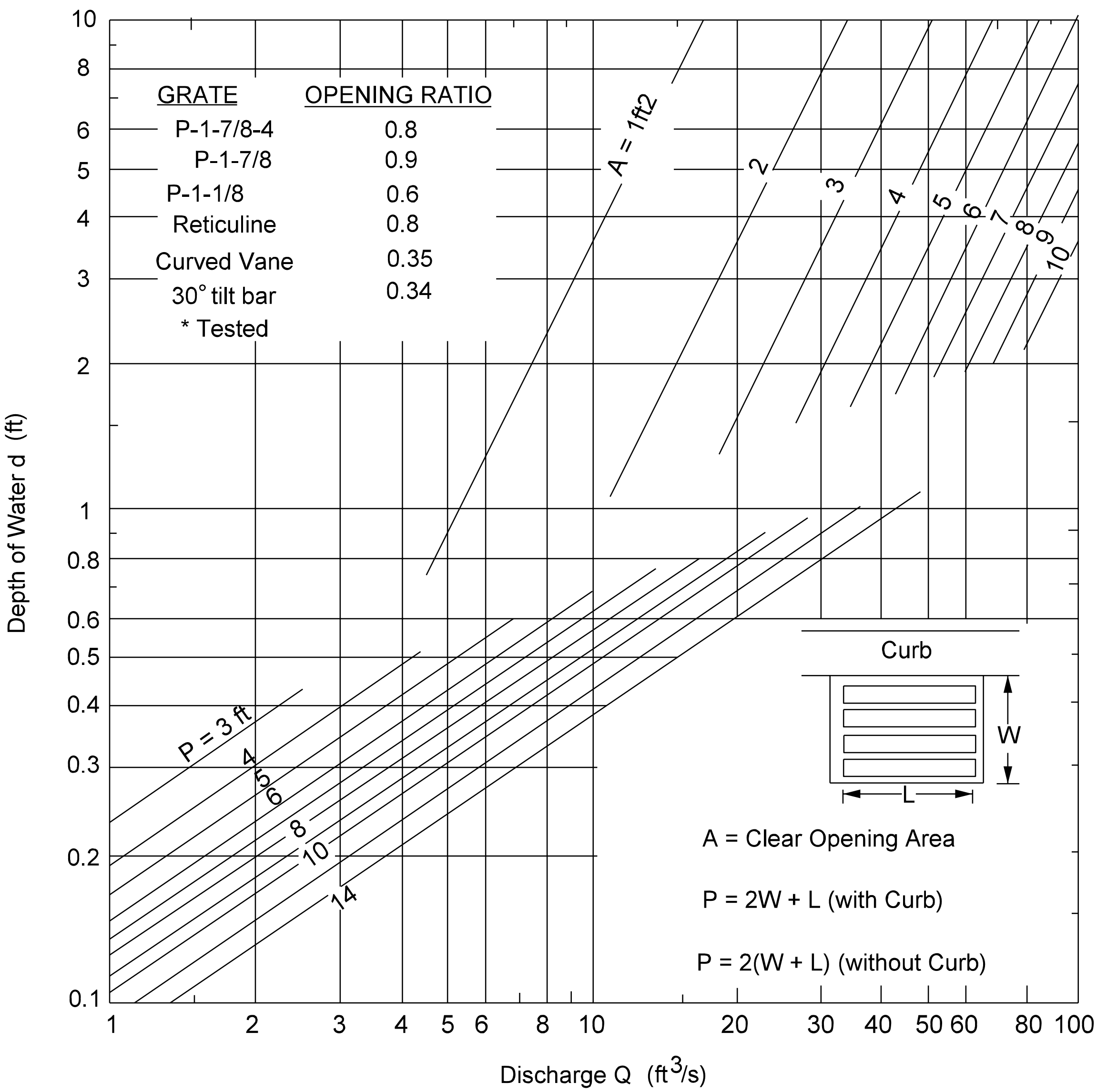
GRATE INLET SIDE FLOW INTERCEPTION EFFICIENCY

Figure 36-10C



EXAMPLE PROBLEM SKETCH

Figure 36-10C(1)



GRATE INLET CAPACITY IN SUMP CONDITIONS

Figure 36-10D

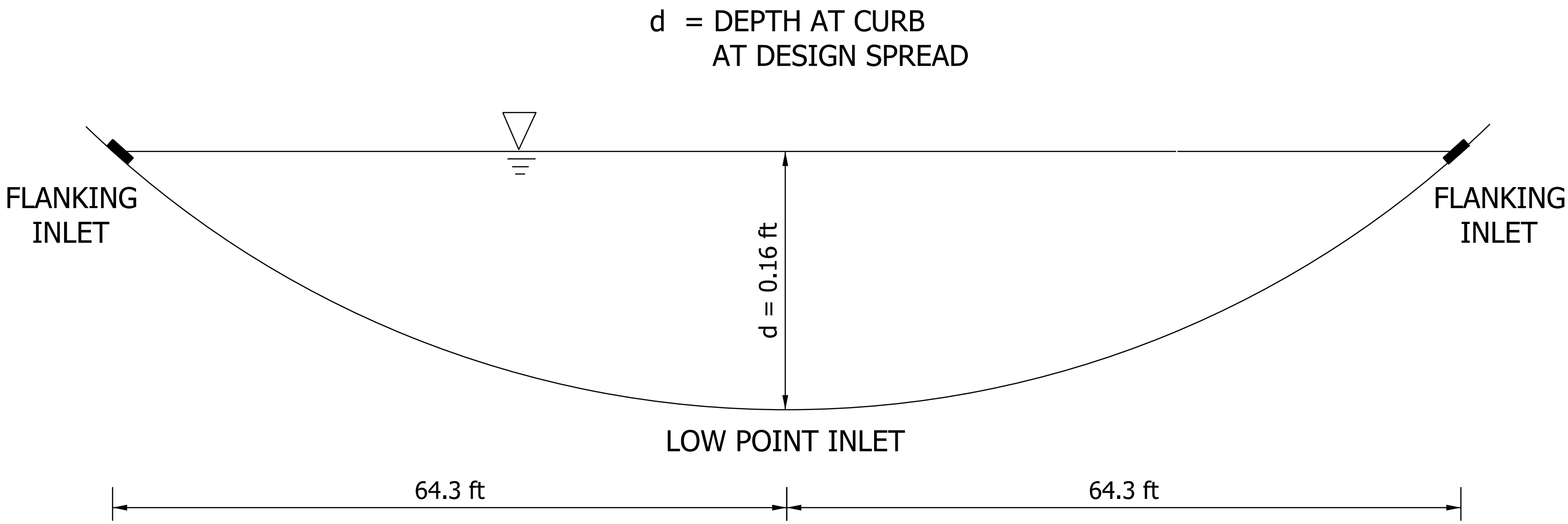
Speed (mph)	20	25	30	35	40	45	50	55	60	62	65	70
d↓ *K*→	20	30	40	50	70	90	110	130	160	167	180	220
0.1	20	24	28	32	37	42	47	51	57	58	60	66
0.2	28	35	40	45	53	60	66	72	80	82	85	94
0.3	35	42	49	55	65	73	81	88	98	100	104	115
0.4	40	49	57	63	75	85	94	102	113	116	120	133
0.5	45	55	63	71	84	95	105	114	126	129	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 to flanking inlet, ft, and
   d = depth at curb, ft
   Maximum K for drainage = 170 (ft/%A) for a curbed facility.
2. $K = L/A$, where L = length of vertical curve, ft, and A = algebraic difference in approach grades, %.
   Reference: *HEC 12* (modified).
3. See Figure 36-10E(1) for Example 36-10.3 details.

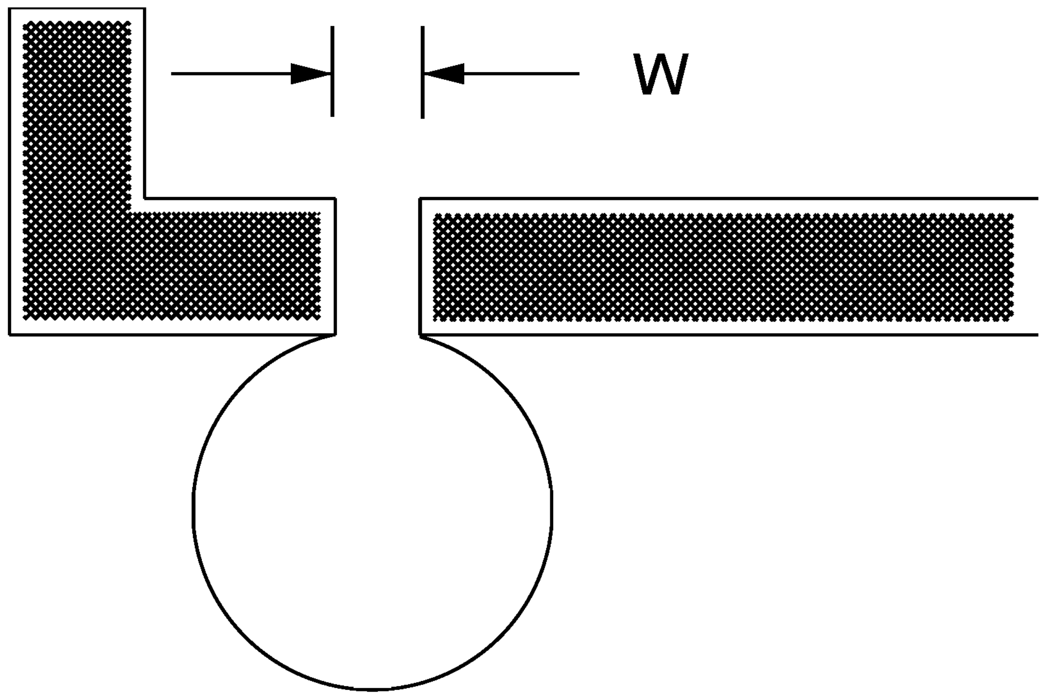
FLANKING-INLET LOCATIONS

Figure 36-10E



EXAMPLE PROBLEM 36-10.3
FLANKING INLETS AT SAG POINT

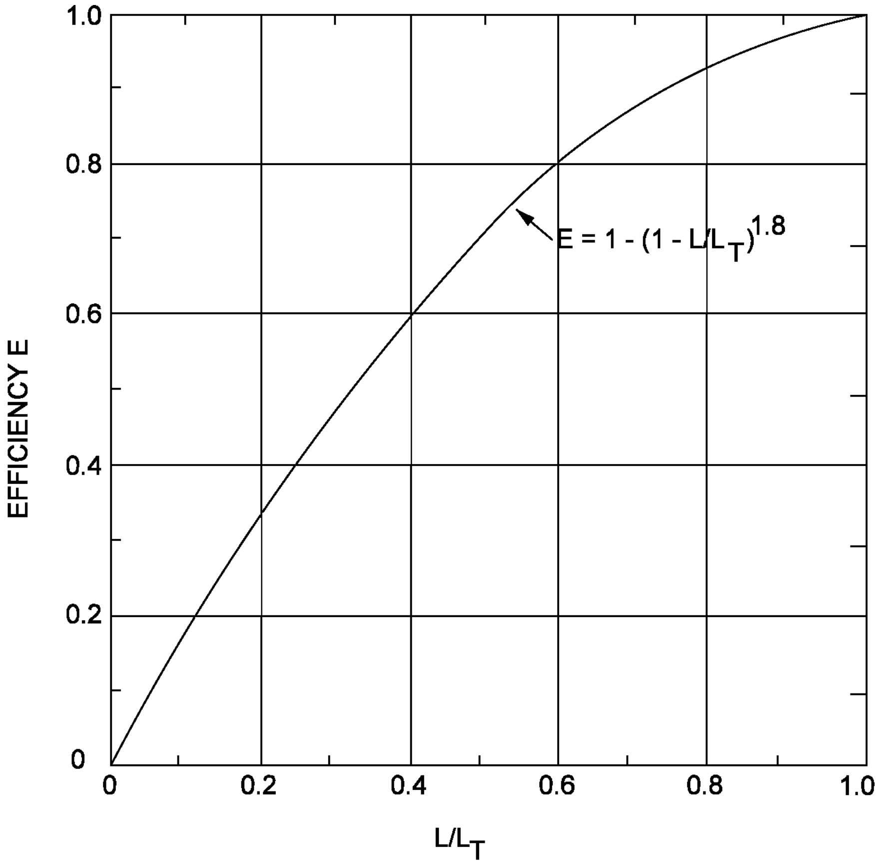
Figure 36-10E(1)



L = Length of Slotted Drain Inlet

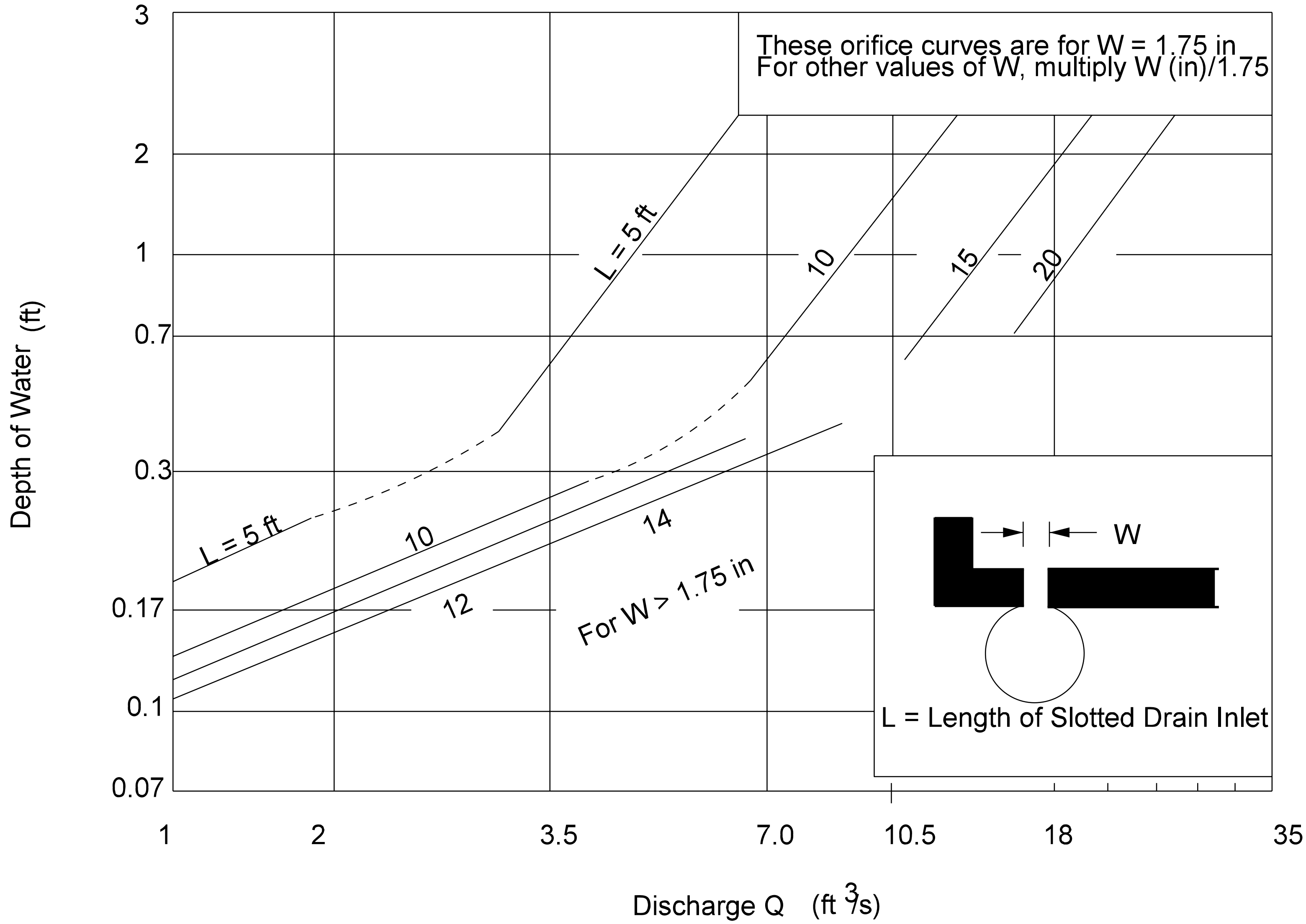
GUTTER CROSS SECTION
(Equation 36-10.11)

Figure 36-10H



CURB-OPENING AND SLOTTED DRAIN INLET INTERCEPTION EFFICIENCY

Figure 36-10I



SLOTTED DRAIN INLET CAPACITY IN SUMP LOCATIONS

Figure 36-10J

ROUTE: __________ PROJECT: __________________________ COMPUTED BY: __________________________ DATE: ___________ SHEET ____ OF ____																		
LOCATION		GUTTER DISCHARGE Design Frequency: _________					GUTTER DISCHARGE Allowable Spread: _________								INLET DISCHARGE			REMARKS
Inlet No.	Station	Drain Area A (ac)	Run-off Coef. C	Time of Conc. T_C (min.)	Rain Inten-sity, I (in./h)	Q = .002 – 78 x CIA (cfs)	Grade S_o (ft/ft)	Cross Slope S_x (ft/ft)	Prev. Run-by (cfs)	Total Gutt. Flow (cfs)	Depth d T/W (ft)	Gutt. Width W (ft)	Sprd. T (ft)	W/T	Inlet Type	Inter-cept Q_i (cfs)	Run-by Q_r (cfs)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

INLET SPACING COMPUTATION SHEET

Manhole Type	Manhole Inside-Dia. Dimension (in.)	Maximum Trunkline Pipe Size (in.)	Minimum Trunkline Pipe Size (in.)
A	48 dia	24	12
B	36 dia	18	12
C	49 dia	24	12
D	58 x 74	42	27
E	80 x 74	60	48
F	108 x 74	84	66
G	136 x 74	108	90
H	49 dia	36	24
J	62 dia	36	24
K	74 dia	48	36
L	98 dia	54	48
M	104 dia	72	54
N	110 dia	84	72

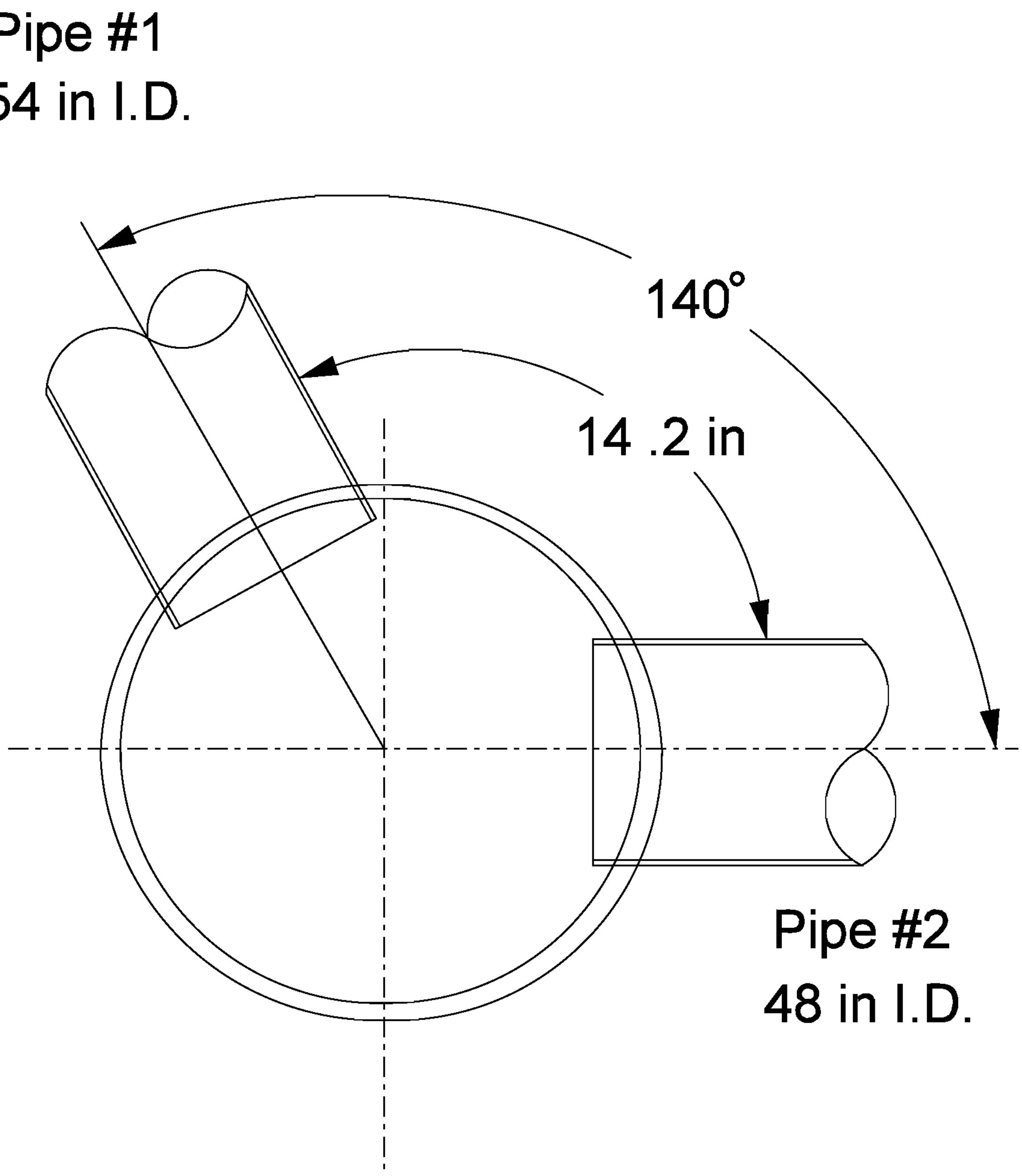
MANHOLE TYPES

Figure 36-11A

MH Dia. (in.)	*K* (in./deg)	Maximum Pipe Size (in.)
27	0.24	15
42	0.38	27
48	0.43	30
54	0.48	36
60	0.53	42
66	0.59	48
72	0.64	54
78	0.69	60
84	0.74	66
90	0.80	72
96	0.85	72
102	0.90	78
108	0.96	84

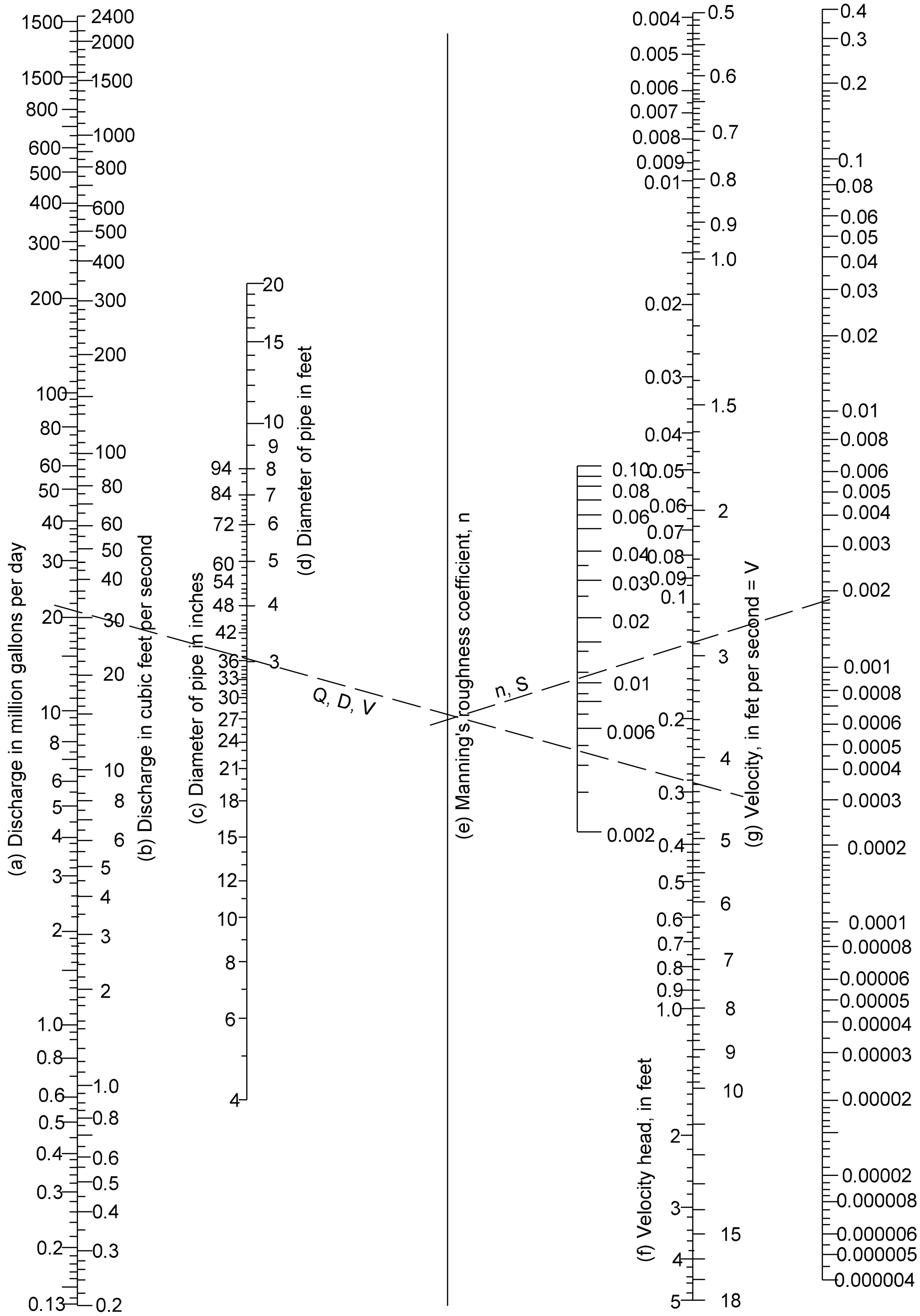
MANHOLE SIZING

Figure 36-11B



PIPE LAYOUT FOR EXAMPLE PROBLEM 36-11.1

Figure 36-11C

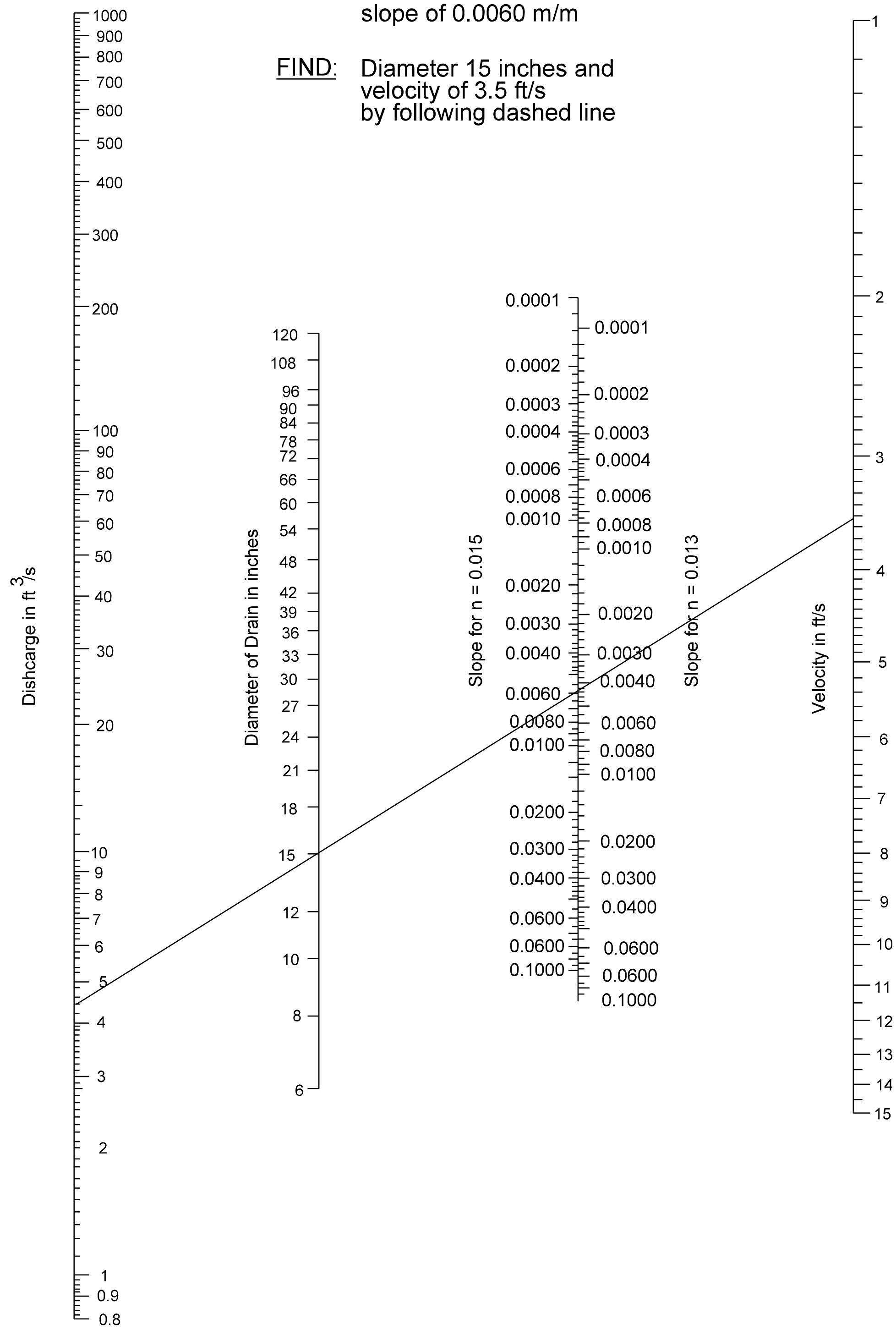


MANNINGS FORMULA FOR FLOW IN STORM DRAINS

Figure 36-12A

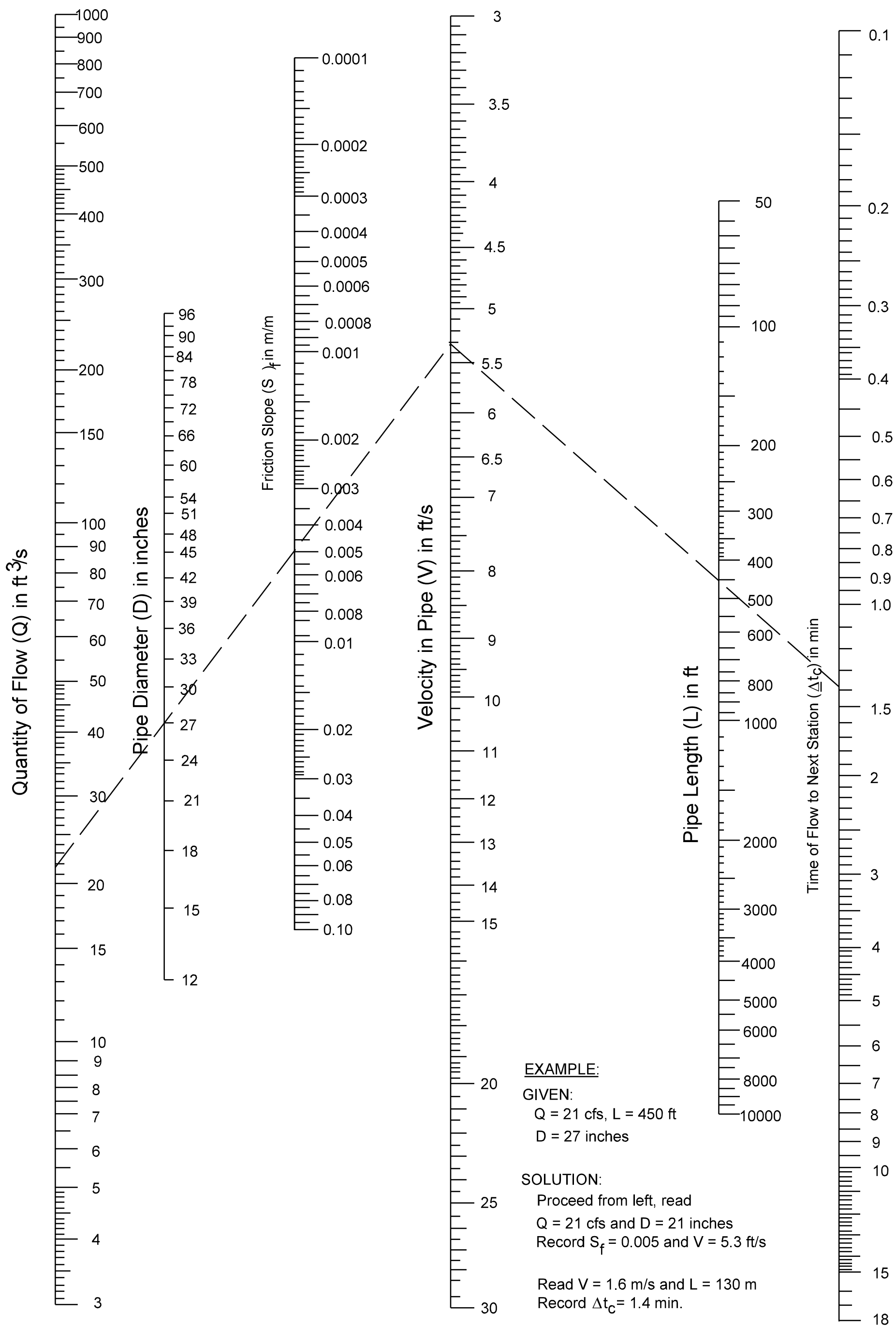
<u>EXAMPLE:</u> Given discharge Q = 4.4 ft^3/s
friction factor n = 0.015
slope of 0.0060 m/m

<u>FIND:</u> Diameter 15 inches and
velocity of 3.5 ft/s
by following dashed line



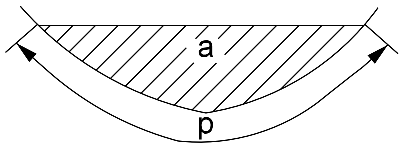
NOMOGRAPH FOR COMPUTING REQUIRED SIZE
OF CIRCULAR DRAIN FOR FULL FLOW
(n = 0.013 or n = 0.015)

Figure 36-12B



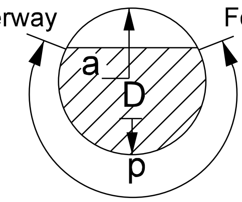
CONCRETE PIPE FLOW NOMOGRAPH

Figure 36-12C



Section of Any Channel

A = Cross-sectional area of waterway
P = wetted perimeter
R = A/P = Hydraulic Radius

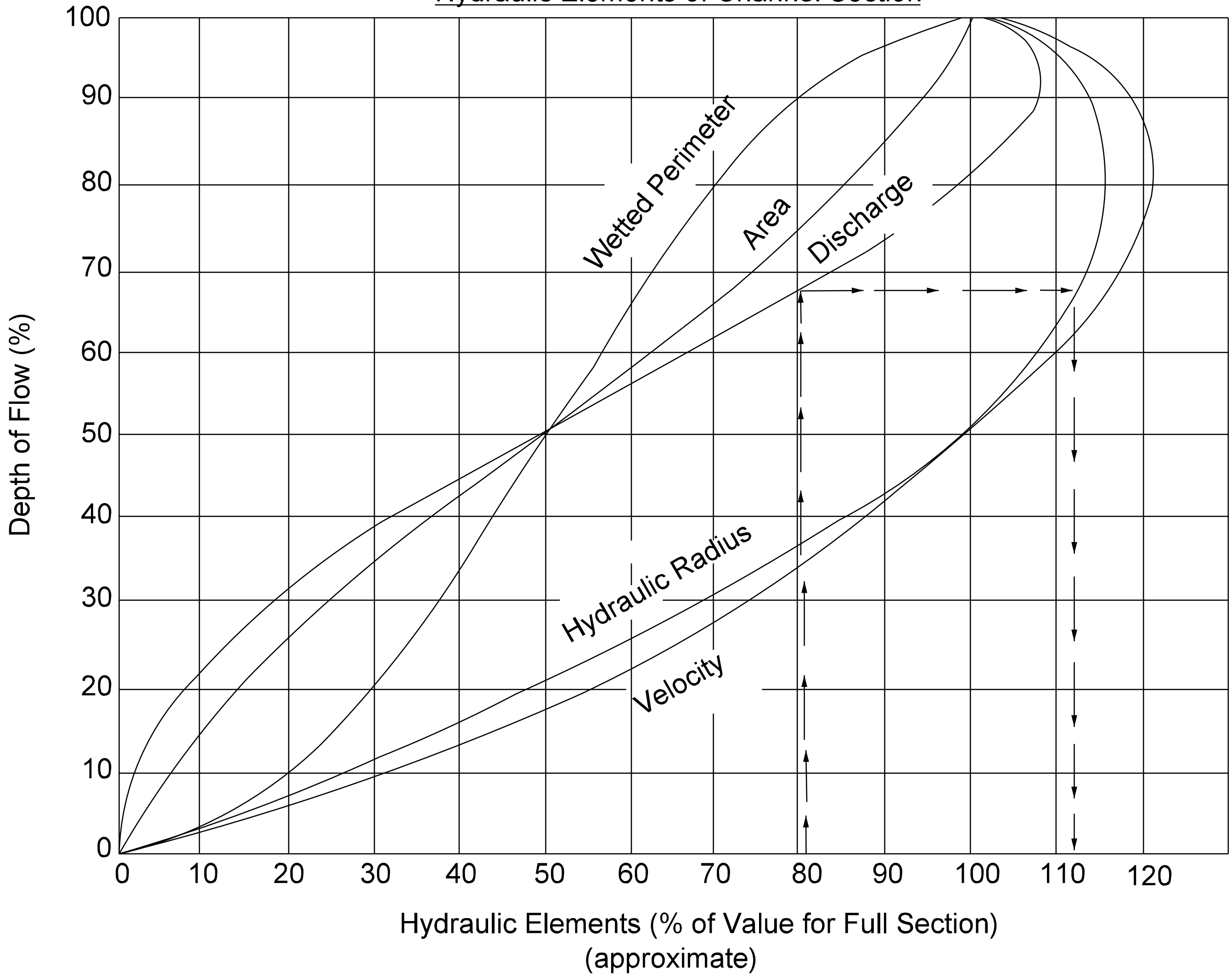


For pipes full or half full
R = D/4

Section of Circular Pipe

V = Average or mean velocity in ft/s
Q =AV = Discharge of pipe or channel in ft^3/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 Section



VALUES OF HYDRAULIC ELEMENTS OF CIRCULAR SECTION FOR VARIOUS DEPTHS OF FLOW

Figure 36-12D

Pipe Size, in	Full-Pipe Flow, ft^3/s	Slope *
12	2.12	0.0030
15	3.14	0.0022
18	4.48	0.0017
21	6.14	0.0014
24	8.12	0.0012
27	10.17	0.0010
30	12.57	0.00087
33	15.11	0.00076
36	18.05	0.00068
42	24.47	0.00055
48	31.96	0.00046
54	40.29	0.00039
60	49.86	0.00034
66	60.39	0.00030
72	72.25	0.00027

* n = *0.012, coefficient of pipe roughness assumed by INDOT for storm drain*

MINIMUM SLOPE NECESSARY TO ENSURE 3.0 ft/s IN STORM DRAIN FLOWING FULL

Figure 36-12E

Route ___________ Section _____________ County ________________________
Computed _____________________ Date ____________
Checked ______________________ Date ____________

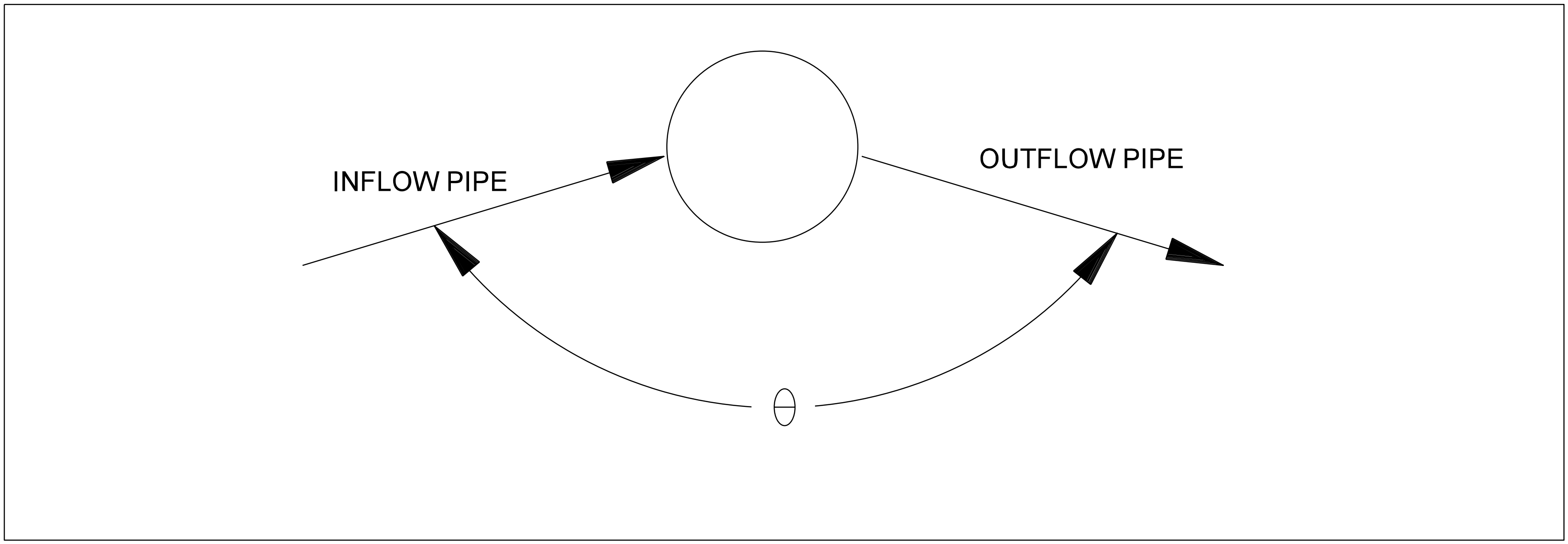
Station	Length (ft)	Drainage Area A (ac)		Runoff Coefficient C	(A)(C)		Flow Time (min)		Rainfall Intensity I (in./h)	Total Runoff 0.00278CIA = Q (cfs)	Diameter Pipe (in.)	Capacity Fill (cfs)	Velocity (ft/s)		Invert Elev.		Manhole Invert Drop	Slope of Drain (ft/ft)
		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

STORM DRAIN COMPUTATION SHEET

AREA RATIO	FREQUENCY FOR COINCIDENTAL OCCURRENCE			
	10-Year Design		100-Year Design	
	MAIN STREAM	TRIBUTARY	MAIN STREAM	TRIBUTARY
10 000 to 1	1 10	10 1	2 100	100 2
1000 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

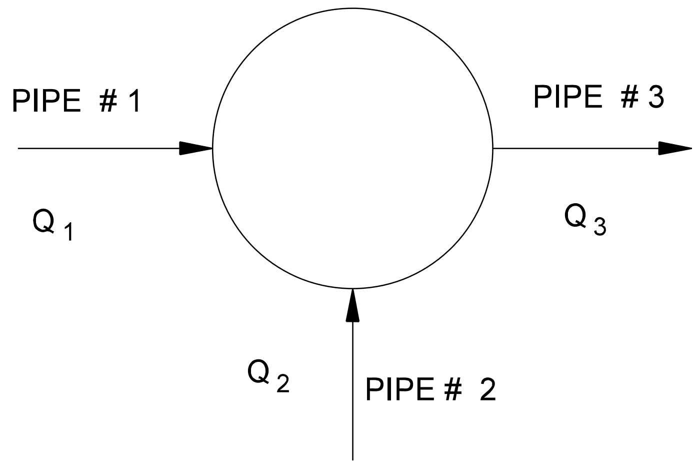
JOINT PROBABILITY ANALYSIS

Figure 36-13A



DEFLECTION ANGLE

Figure 36-13B



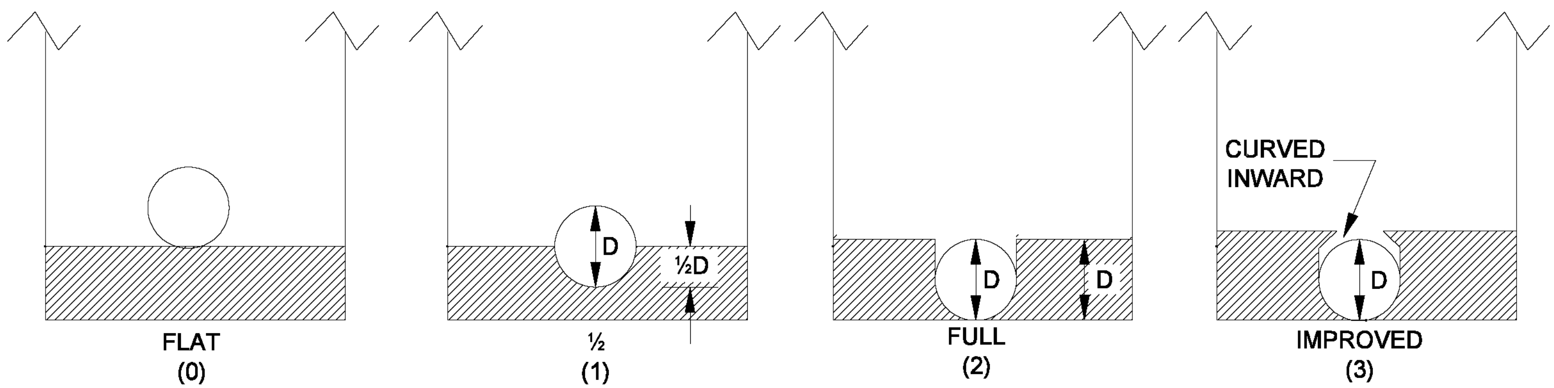
RELATIVE FLOW EFFECT

Figure 36-13B(1)

Bench Type	Correction Factors, C_B	
	Submerged*	UnSubmerged**
Flat Floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Improved	0.40	0.02

*pressure flow, $d/D_0 > 3.2$

**free surface flow, $d/D_0 < 1.0$



SCHEMATIC REPRESENTATION OF BENCHING TYPES

CORRECTION FOR BENCHING
Figure 36-13C

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)

HYDRAULIC GRADE LINE COMPUTATION FORM

$$H_{tm} = \frac{V^2}{2g}$$

TERMINAL JUNCTION LOSSES
(at beginning of run)

Where: g = gravitational constant, 32.2 ft/s^2

$$H_e = 0.5\,\frac{V^2}{2g}$$

ENTRANCE LOSSES
(for structure at end of run)
Assuming square - edge

$$H_{j1} = \frac{V^2}{2g}\ \text{(Outflow)}$$

JUNCTION LOSSES

Use only where flows are identical to above; otherwise, use H_{j2} Equation.

$$H_{j2} = \frac{Q_4 V_4^2 - Q_1 V_1^2 - Q_2 V_2^2 + K Q_1 V_1^2}{2gQ_4}$$

JUNCTION LOSSES
(After FHWA)

Total losses to include H_{j2} plus losses for changes in direction of less than 90º (H_b).

Where:	K	=	Bend loss factor
	Q_3	=	Vertical dropped-in flow from an inlet
	V_3	=	Assumed to be zero

$$H_b = \frac{KV_1^2}{2g}$$

BEND LOSSES
(Change in direction of flow)

Where K	Degree of Turn (A) in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSS (H_f)

$H_f = (S_f)\ (L)$

Where:	H_f	=	Friction head, ft
	S_f	=	Friction slope, ft/ft
	L	=	Length of conduit, ft

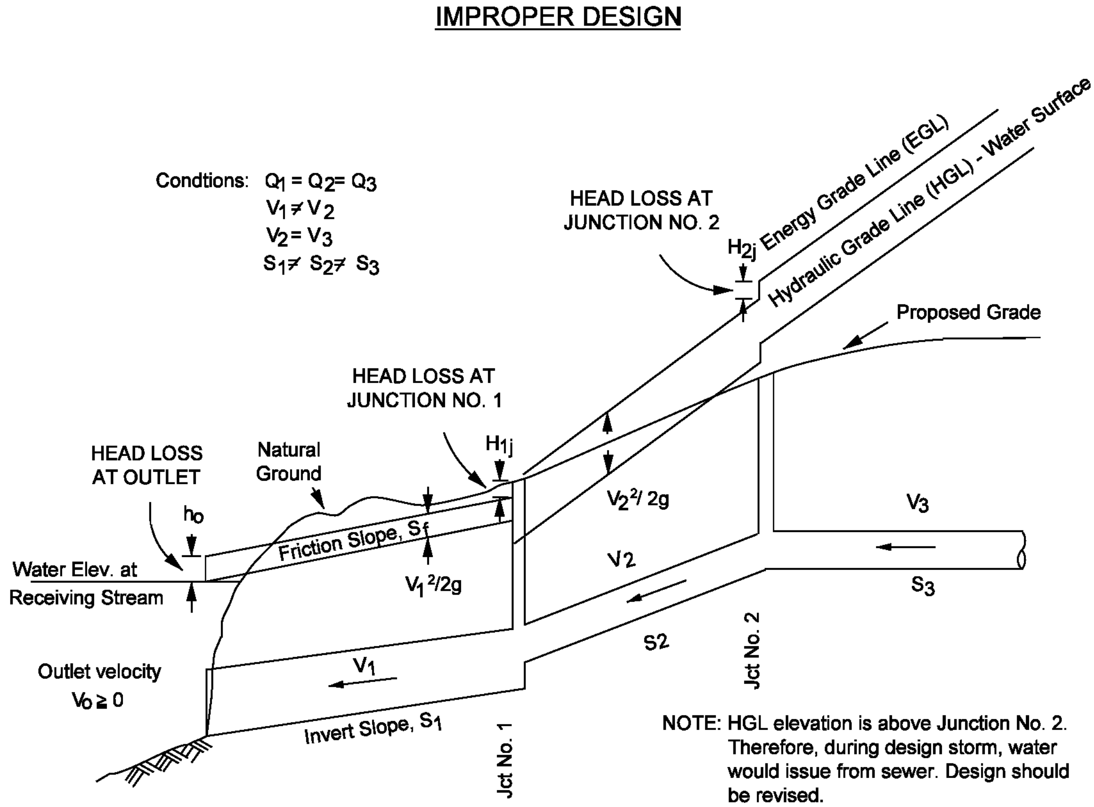
$$S_f = \left(\frac{Qn}{AR^{2/3}}\right)^2$$

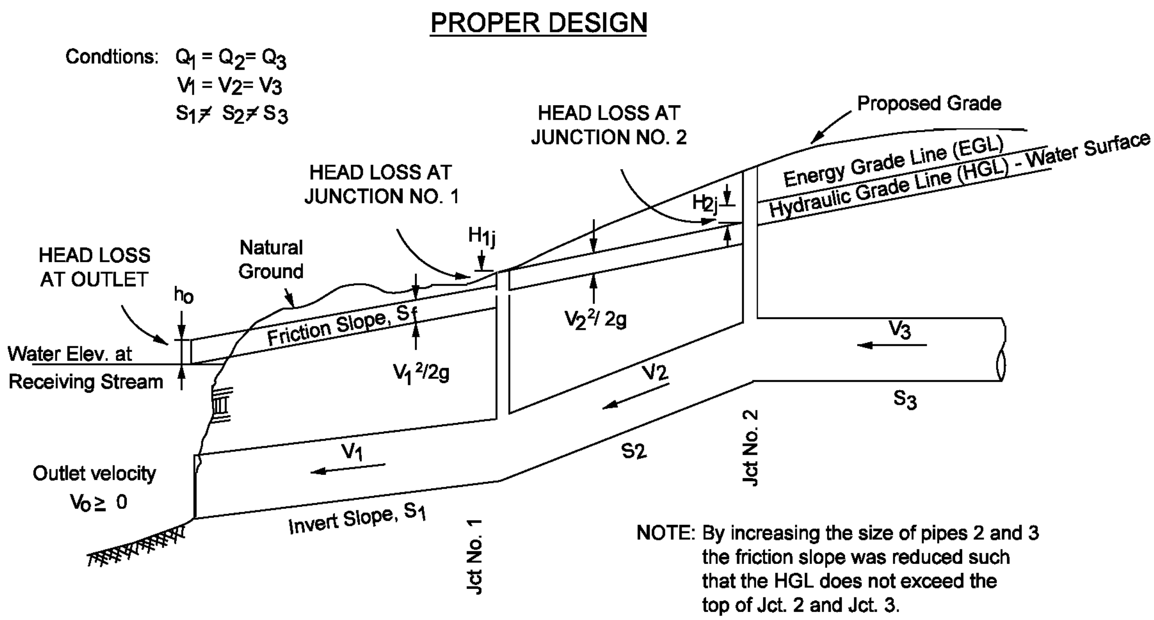
Where:	Q	=	Discharge of conduit, ft^3/s
	n	=	Manning's coefficient of roughness (use 0.013 for R.C. Pipes)
	A	=	Area of conduit, ft^2
	R	=	Hydraulic radius of conduit (D/4 for round pipe), ft

TOTAL ENERGY LOSSES AT EACH JUNCTION

$$H_T = H_{tm} + He + (H_{j1} \text{ or } H_{j2}) + H_b + H_f$$

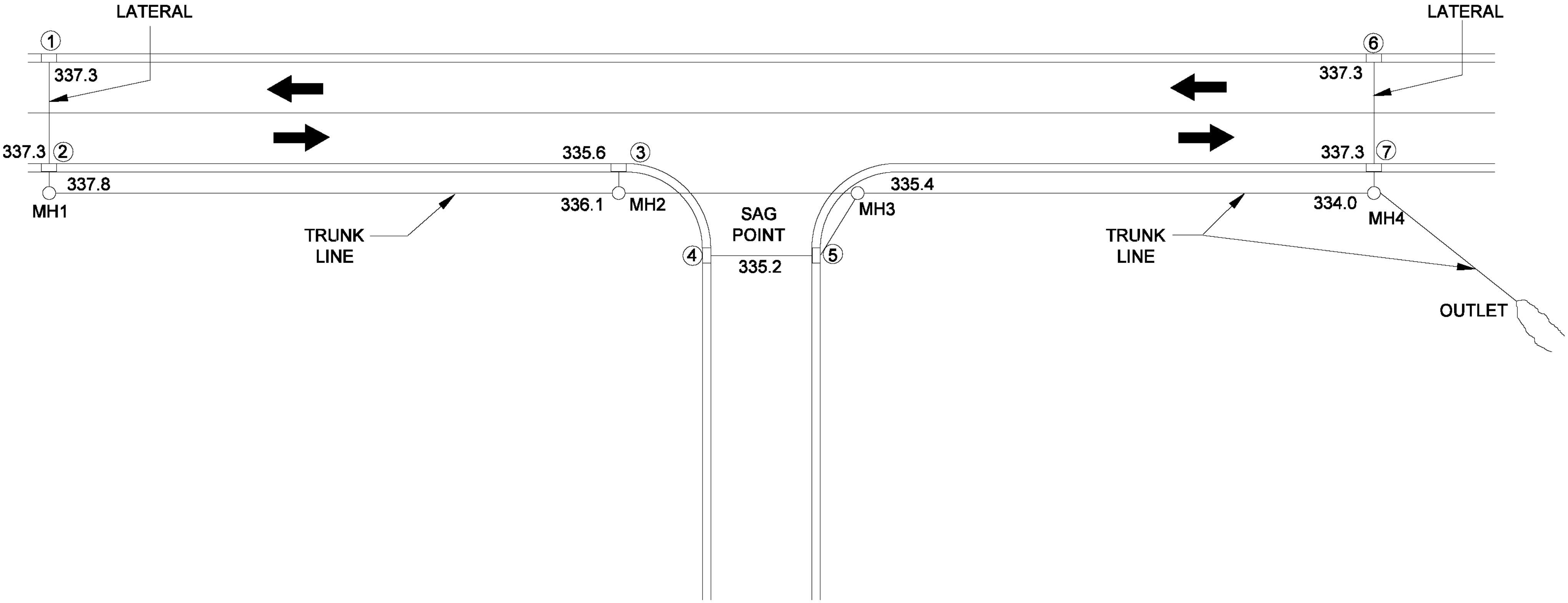
SUMMARY OF ENERGY LOSSES
Figure 36-13E





USE OF ENERGY LOSSES IN DEVELOPING A STORM DRAIN SYSTEM

Figure 36-13F



EXAMPLE PROBLEM

Figure 36-16A

INLET COMPUTATION SHEET													DATE_________PROJECT_________ROUTE______ COMPUTED BY________________SHEET_____OF____					
LOCATION		GUTTER DISCHARGE DESIGN FREQUENCY 10					GUTTER DISCHARGE ALLOWABLE SPEED 6.0 ft (Roadway) 7.0 ft (Street)								INLET DISCHARGE			RE-MARKS
INLET No.	STAT.	DRAIN AREA "A" (acres)	RUNOFF COEF "C"	TIME OF C ONC. "T_C" (min)	Rain Intensity "I" (in/h)	Q=.002-78CIA (ft^3/s)	GRADE "S_O" (ft/ft)	CROSS SLOPE S_x (ft/ft)	PREV. RUNBY (ft^3/s)	TOTAL GUTTER FLOW (ft^3/s)	DEPTH "d" T/W (ft)	GUTTER WIDTH "W" (ft)	SPREAD "T" (ft)	W/T	INLET TYPE	INTER-CEPT "Q_i" (ft^3/s)	RUNBY "Q_r" (ft^3/s)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
1	10+00	0.125 0.100	0.4 0.9	10	5.28	0.706	0.012	0.02	0	0.706	0.133	2.00	6.00	0.33	10 & 11	0.530	0.177	
2	10+00	0.125 0.100	0.4 0.9	10	5.28	0.706	0.012	0.02	0	0.706	0.133	2.00	6.00	0.33	10 & 11	0.530	0.177	
3	10+42.7	0.050 0.150	0.9 0.4	11.2	5.08	0.530	0.012	0.02	0.177	0.706	0.133	2.00	6.00	0.33	10 & 11	0.530	0.177	
4	Street	0.100 0.125	0.9 0.9	7	6.00	1.200	Sag	0.02	0	1.200	0.140	1.32	6.67		Double 10 & 11	1.200	0	
5	Street	0.100 0.100	0.9 0.9	7	6.00	1.059	Sag	0.02	0.177	1.236	0.143	1.32	6.67		Double 10 & 11	1.236	0	Sag Point
6	11+00	0.100	0.9	11.5	5.04	0.424	0.012	0.02	0.177	0.600	0.123	1.32	5.73		10 & 11	0.459	0.141	Sag Point
7	11+00	0.050 0.250	0.9 0.4	11.5	5.04	0.706	0.012	0.02	0	0.706	0.133	1.32	6.00		10 & 11	0.530	0.177	

INLET SPACING COMPUTATION SHEET
(Example Problem)
Figure 36-16B

```
***************************************
*****     ROADWAY DRAINAGE DESIGN    *****
***************************************
```

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 1

STATION: 10+00

DRAINAGE AREA: .09 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	S (m/m)	Sx (m/m)	n	Q (m^3/s)	T (m)
Composite	0.0120	0.0200	0.016	0.020	1.827

GUTTER FLOW

W (m)	Sw (m/m)	a (mm)	Eo	d (mm)	V (m/s)
0.397	0.0250	N/A	0.679	39.611	0.582

INLET INTERCEPTION

Inlet Type	L (m)	W (m)	E	Qi (m^3/s)	Qb (m^3/s)
Curved Vane	0.879	0.397	0.782	0.015	0.005

HIGHWAY DRAINAGE DESIGN EXAMPLE

Figure 36-16B(1)

****
***** ROADWAY DRAINAGE DESIGN *****
****

DESIGNER: INDOT   DATE: 09-02-1998

PROJECT: Example   PROJECT NO.:

INLET NO.: 2   STATION: 10+00

DRAINAGE AREA: .09 Hectares   DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	S (m/m)	Sx (m/m)	n	Q (m^3/s)	T (m)
Composite	0.0120	0.0200	0.016	0.020	1.827

GUTTER FLOW

W (m)	Sw (m/m)	a (mm)	Eo	d (mm)	V (m/s)
0.397	0.0250	N/A	0.679	39.611	0.582

INLET INTERCEPTION

Inlet Type	L (m)	W (m)	E	Qi (m^3/s)	Qb (m^3/s)
Curved Vane	0.879	0.397	0.782	0.015	0.005

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

****
***** ROADWAY DRAINAGE DESIGN *****
****

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 3

STATION: 10+42.7

DRAINAGE AREA: .08 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	S (m/m)	Sx (m/m)	n	Q (m^3/s)	T (m)
Composite	0.0120	0.0200	0.016	0.020	1.827

GUTTER FLOW

W (m)	Sw (m/m)	a (mm)	Eo	d (mm)	V (m/s)
0.397	0.0250	N/A	0.679	39.611	0.582

INLET INTERCEPTION

Inlet Type	L (m)	W (m)	E	Qi (m^3/s)	Qb (m^3/s)
Curved Vane	0.879	0.397	0.782	0.015	0.005

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

```
***************************************************
*****    FHWA URBAN DRAINAGE DESIGN PROGRAMS   *****
*****         ROADWAY DRAINAGE DESIGN          *****
***************************************************
```

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 4

STATION: Street

DRAINAGE AREA: .17 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	Sx (m/m)	Sw (m/m)	n	W (m)	a (mm)
Composite	0.020	0.025	0.016	0.397	0.00

INLET INTERCEPTION

Inlet Type * Sag *	L (m)	W (m)	T (m)	d (m)	Qi (m^3/s)
Curved Vane	1.760	0.397	2.004	0.042	0.034

HIGHWAY DRAINAGE DESIGN EXAMPLE (continued)

Figure 36-16B(1)

```
*************************************************
*****    FHWA URBAN DRAINAGE DESIGN PROGRAMS    *****
*****          ROADWAY DRAINAGE DESIGN          *****
*************************************************
```

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 5

STATION: Street

DRAINAGE AREA: .18 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	Sx (m/m)	Sw (m/m)	n	W (m)	a (mm)
Composite	0.020	0.025	0.016	0.397	0.00

INLET INTERCEPTION

Inlet Type * Sag *	L (m)	W (m)	T (m)	d (m)	Qi (m^3/s)
Curved Vane	1.760	0.397	2.043	0.043	0.035

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

```
*****************************************
*****     ROADWAY DRAINAGE DESIGN   *****
*****************************************
```

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 6

STATION: 11+00

DRAINAGE AREA: .04 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	S (m/m)	Sx (m/m)	n	Q (m^3/s)	T (m)
Composite	0.0120	0.0200	0.016	0.017	1.715

GUTTER FLOW

W (m)	Sw (m/m)	a (mm)	Eo	d (mm)	V (m/s)
0.397	0.0250	N/A	0.708	37.358	0.560

INLET INTERCEPTION

Inlet Type	L (m)	W (m)	E	Qi (m^3/s)	Qb (m^3/s)
Curved Vane	0.879	0.397	0.807	0.013	0.004

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

```
****************************************
*****     ROADWAY DRAINAGE DESIGN     *****
****************************************
```

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 7

STATION: 11+00

DRAINAGE AREA: .12 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	S (m/m)	Sx (m/m)	n	Q (m^3/s)	T (m)
Composite	0.0120	0.0200	0.016	0.020	1.827

GUTTER FLOW

W (m)	Sw (m/m)	a (mm)	Eo	d (mm)	V (m/s)
0.397	0.0250	N/A	0.679	39.611	0.582

INLET INTERCEPTION

Inlet Type	L (m)	W (m)	E	Qi (m^3/s)	Qb (m^3/s)
Curved Vane	0.879	0.397	0.782	0.015	0.005

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

Computed ________ Date ________ Route ____________________

Section ____________________

Checked ________ Date ________ County ____________________

Station		Length (ft)	Drainage Area A (acres)		Runoff Coefficient C	A x C		Flow Time (min)		Rainfall Intensity I (in/h)	Total Runoff 0.00278CIA = Q (ft^3/s)	Diameter Pipe (in)	Capacity Fill (ft^3/s)	Velocity (ft/s)		Inert Elev.		Manhole Invert Drop	Slope of Drain (ft/ft)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		
1	2	24.3	0.125		0.4	0.05													
			0.100	0.225	0.9	0.9	0.14	10		5.28	0.745	12	3.53	5.67	5.00	333.33	333.10		0.01
2	MH1	3.33	0.125		0.4	0.050													
			0.100	0.45	0.9	0.09	0.28	10.2		5.28	1.48	12	3.53	5.67	5.00	333.10	333.06		0.01
MH1	MH2	143.3		0.45			0.28	10.2		5.28	1.48	12	4.24	6.67	6.00	333.06	331.33		0.012
3	MH2	3.33	0.05 0.15	0.20	0.9 0.4	0.045 0.6	0.105	11.2		5.08	0.533	12	3.53	5.67	5.00	331.60	331.57		0.01
MH2	MH3	73.33		0.65			0.385	11.2		5.08	1.96	12	4.24	6.67	6.00	331.33	330.47		0.012
4	5	24.33	0.300		0.4	0.12													
			0.125	0.425	0.9	0.113	0.232	10		5.28	1.22	12	2.83	4.00	3.67	330.80	330.67		0.005
5	MH3	30.0	0.325		0.4	0.13													
			0.125	0.875	0.9	0.113	0.475	10.1		5.28	2.51	12	3.53	4.67	4.67	330.67	330.47		0.007
MH3	MH4	116.7		1.525			1.09	11.4		5.08	5.54	15	8.12	7.00	6.76	330.20	328.80		0.012
6	7	24.3	0.100		0.9		0.09	11.5		5.04	0.454	12	2.83	4.00	2.67	329.33	329.20		0.005
7	MH4	3.33	0.050		0.9	0.045													
			0.250	0.30	0.4	0.100	0.145	11.5		5.04	0.731	12	2.83	4.00	3.33	329.20	329.17		0.005
MH4	Outlet	81.7		1.825			1.23	11.7		5.04	6.20	18	8.48	5.33	5.00	328.53	328.13		0.005

STORM DRAIN COMPUTATION SHEET
FOR EXAMPLE PROBLEM

Figure 36-16C

```
     ******  HYDRA  ******* (Version 6.0) *****              Date **-04-98
                                                              Page No   1

   Example Storm Drain Analysis  -  Rational Method


 +++ Commands Read From File C:\HYDRA\EXSTO.HDA
   JOB
   SWI 2
   PDA 0.012 300  .914 .610 0.8 .0030
   RAI 5 170 10 132 15 112 20 100 25 90 30 81
                              IDF CURVE

 .17E+03*..............................................................
        .                                                             .
        .                                                             .
        .                                                             .
 .13E+03.         *                                                   .
        .                                                             .
        .                   *                                         .
        .                                                             .
        .                                                             .
 .85E+02.                             *                               .
        .                                       *                     .
        .                                                             .
        .                                                             .
        .                                                             .
 .43E+02.                                                             .
        .                                                             .
        .                                                             .
        .                                                             .
        .                                                             .
 .00E+00*..............................................................
         5.       9.       12.      16.      19.      23.      26.      3

                         PLOT-DATA (TIME Vs.VALUE)
----------------------------------------------------------------------------
     5.   170.00    25.    90.00    0.     .00    0.     .00    0.     .00
    10.   132.00    30.    81.00    0.     .00    0.     .00    0.     .00
    15.   112.00    0.       .00    0.     .00    0.     .00    0.     .00
    20.   100.00    0.       .00    0.     .00    0.     .00    0.     .00
----------------------------------------------------------------------------
   HGL 1
   REM Begin Mainline 1 to OUTLET
   NEW Start Lateral 1 to MH1
   REM Link 1  Lateral 1 to 2
   STO .05 .4 10
   STO .04 .9 10
   PIP 7.3 101.2 101.2 100.00 99.93 300
 +++ Tc =   10.0 min
 +++ CA =     .1
 +++ Link #  1,  Flow depth =    .093 m
```

HYDRA EXAMPLE

Figure 36-16D

Example Storm Drain Analysis - Rational Method

```
  PNC 1 2 .92 0 0 1
  REM Link 2 Lateral 2 to MH1
  STO .05 .4 10.2
  STO .04 .9 10.2
  PIP 1.0 101.2 101.35 99.93 99.92
+++ Tc =  10.2 min
+++ CA =    .1
+++ Link #  2,  Flow depth =    .132 m
  PNC 2 21 1.22 90 0 1
  REM LINK 3 MAINLINE MH1 to MH2
  PIP 43.0 101.35 100.83 99.92 99.40 300
+++ Tc =  10.2 min
+++ CA =    .1
+++ Link #  3,  Flow depth =    .123 m
  PNC 21 22 1.22 180 0 1
  HOL 3
  NEW Lateral 3 to MH2
  REM Link 4 Lateral 3 to MH2
  STO .02 .9 11.2
  STO .06 .4 11.2
  PIP 1.0 100.68 100.83 99.48 99.47 300
+++ Tc =  11.2 min
+++ CA =    .0
+++ Link #  4,  Flow depth =    .078 m
  PNC 3 22 1.22 0 0 1
  REM Link 5 MH2 to MH3
  REC 3
  PIP 22.0 100.83 100.62 99.40 99.13 300
+++ Tc =  11.2 min
+++ CA =    .2
+++ Link #  5,  Flow depth =    .144 m
  PNC 22 23 1.22 180 0 1
  NEW Lateral 4 to 5
  REM Link 6 Street Inlets 4 to 5
  STO .12 .4 10
  STO .05 .9 10
  PIP 7.3 100.56 100.56 99.24 99.20 300
+++ Tc =  10.0 min
+++ CA =    .1
+++ Link #  6,  Flow depth =    .141 m
  PNC 4 5 .76 0 0 1
  REM Link 7 Street Inlet 5 to MH 3
  HOL 5
  STO .13 .4 10.1
  STO .05 .9 10.1
  PIP 9.0 100.56 100.62 99.20 99.14 300
+++ Tc =  10.1 min
+++ CA =    .2
+++ Link #  7,  Flow depth =    .201 m
```

HYDRA EXAMPLE
(continued)

Figure 36-16D

```
  Example Storm Drain Analysis  -  Rational Method

  PNC 5 23 1.22 135 0 1
  REM Link 8 MH3 to MH4
  REC 5
  PIP 35.0 100.62 100.20 99.06 98.64 375
+++ Tc =  10.2 min
+++ CA =     .3
+++ Link #  8,  Flow depth =     .188 m
  PNC 23 24 1.22 180 0 1
  NEW Lateral 6 to 7
  REM Link 9 Inlets 6 to 7
  STO .04 .9 11.5
  PIP 7.3 100.00 100.00 98.80 98.76 300
+++ Tc =  11.5 min
+++ CA =     .0
+++ Link #  9,  Flow depth =     .084 m
  PNC 6 7 .92 0 0 1
  REM Link 10 Inlet 7 to MH4
  HOL 7
  STO .02 .9 11.5
  STO .10 .4 11.5
  PIP 1.0 100.00 100.20 98.76 98.75 300
+++ Tc =  11.7 min
+++ CA =     .1
+++ Link # 10,  Flow depth =     .117 m
  PNC 7 24 1.22 180 0 1
  REM Outfall Link:  MH4 to Outlet
  REC 7
  PIP 24.5 100.2 98.95 98.56 98.44 450
+++ Tc =  11.7 min
+++ CA =     .1
+++ Cover at lower manhole     .023 m
+++ Link # 11,  Flow depth =     .140 m
  PNC 24 25 0 135 2
  END
 END OF INPUT DATA.
```

HYDRA EXAMPLE
(continued)

Figure 36-16D

Example Storm Drain Analysis - Rational Method

*** Start Lateral 1 to

Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
1	7	300	100.000 99.930	.010	1.200 1.270	.875	1.131 1.455	.020 .103	0.
2	1	300	99.930 99.920	.010	1.270 1.430	.945	1.389 1.486	.040 .105	0.
3	43	300	99.920 99.400	.012	1.430 1.430	1.105	1.490 1.634	.040 .116	0.

Length = 51. m Total length = 51. m
Cost = 0. Total Cost = 0.

*** Lateral 3 to MH2

Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
4	1	300	99.480 99.470	.010	1.200 1.360	.875	1.046 1.486	.015 .105	0.
5	22	300	99.400 99.130	.012	1.430 1.490	1.105	1.615 1.646	.054 .116	0.

Length = 23. m Total length = 74. m
Cost = 0. Total Cost = 0.

HYDRA EXAMPLE
(continued)

Figure 36-16D

****** HYDRA ******* (Version 6.0) *****

Date **-04-98
Page No 5

Example Storm Drain Analysis - Rational Method

*** Lateral 4 to 5

Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
6	7	300	99.240 99.200	.005	1.320 1.360	.995	1.061 1.100	.034 .078	0.
7	9	305	99.200 99.140	.007	1.360 1.480	1.030	1.351 1.226	.069 .089	0.
8	35	375	99.060 98.640	.012	1.560 1.560	1.154	1.875 1.889	.102 .209	0.

Length = 51. m Total length = 59. m
Cost = 0. Total Cost = 0.

*** Lateral 6 to 7

Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
9	7	300	98.800 98.760	.005	1.200 1.240	.875	.807 1.100	.012 .078	0.
10	1	300	98.760 98.750	.010	1.240 1.450	.915	1.312 1.486	.032 .105	0.
11	24	450	98.560 98.440	.005	1.640 .510	.023	1.074 1.363	.045 .217	0.

HYDRA EXAMPLE
(continued)

Figure 36-16D

```
****** HYDRA ******* (Version 6.0) *****          Date **-14-98
                                                  Page No    6
```

Example Storm Drain Analysis - Rational Method

Hydraulic Gradeline Computations

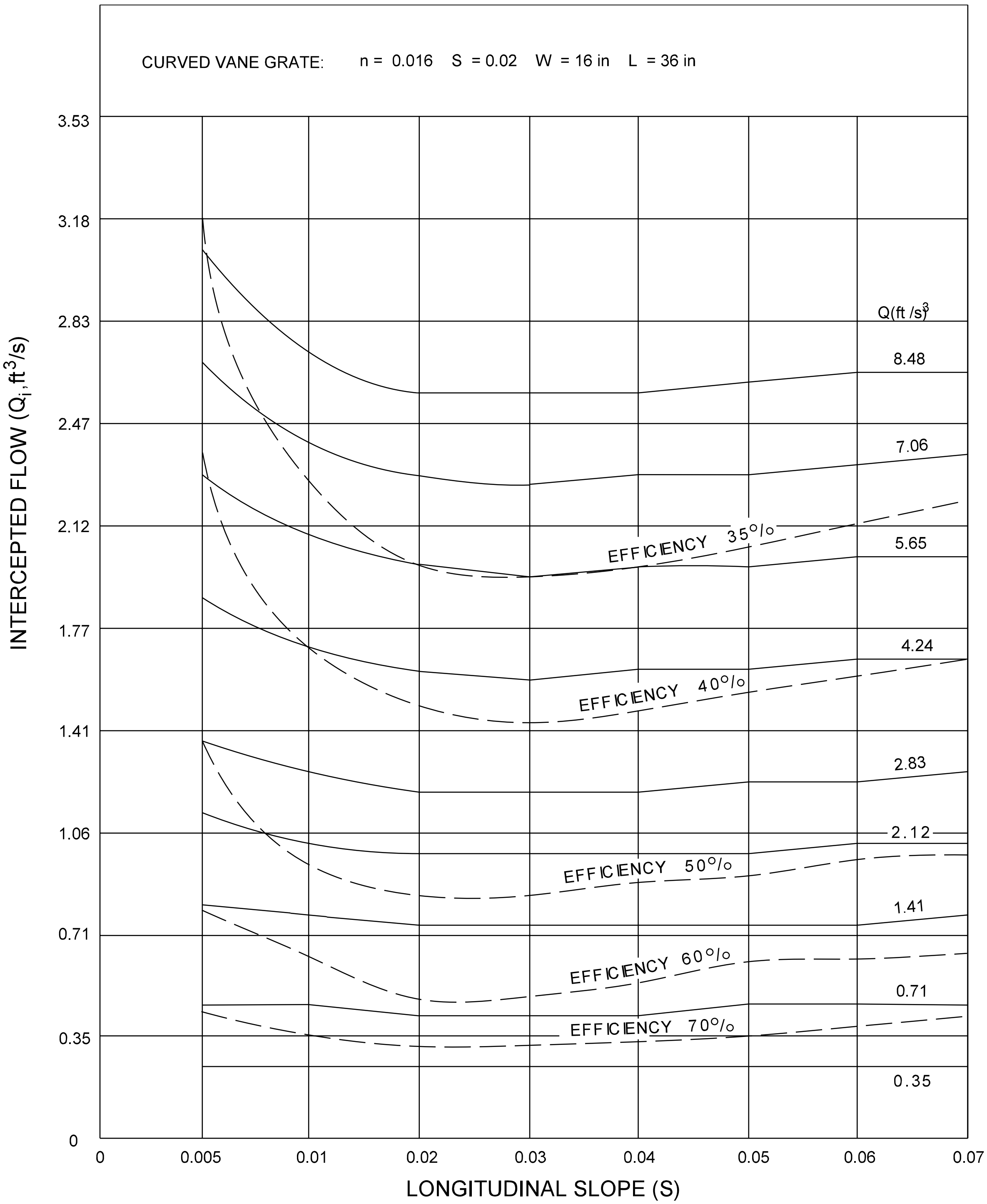
Link #	Down-stream Node #	Hydraulic Gradeline Elevation	Crown Elev.	Possible Surcharge	Ground Elev.	Super-crit.?	Manhole Depth	Loss Coef
1	2	100.075	100.235	N	101.200	Y	.138	.15
2	21	100.059	100.225	N	101.350	Y	.130	.15
3	22	99.564	99.705	N	100.830	Y	.153	.17
4	22	99.553	99.775	N	100.830	Y	.153	.17
5	23	99.272	99.435	N	100.620	Y	.188	.05
6	5	99.415	99.505	N	100.560	Y	.208	.15
7	23	99.341	99.445	N	100.620	Y	.188	.05
8	24	98.824	99.021	N	100.200	Y	.139	.03
9	7	98.887	99.065	N	100.000	Y	.120	.15
10	24	98.863	99.055	N	100.200	Y	.139	.03
11	25	98.578	98.897	N	98.950	Y	.000	.00

Link #	Terminal Node #	Hydraulic Gradeline Elevation	Ground Elevation	Loss Coef.
1	1	100.188	101.200	1.50
4	3	99.704	100.680	1.50

NORMAL END OF HYDRA

HYDRA EXAMPLE
(continued)

Figure 36-16D



INLET CAPACITY CHART
(Curved Vane Grate)

Figure 36-17A