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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 usually reflects the attention given to every aspect of the design as well as that accorded to the construction and maintenance of the facility. Most aspects of storm drain design such as system planning, pavement drainage, gutter flow calculations, inlet spacing, pipe sizing and hydraulic grade line calculations are included.

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

1. the wide roadway sections, flat grades, both in longitudinal and transverse directions, shallow water courses, absence of side channels;
2. the more costly property damage which may occur from ponding of water or from flow of water through built-up areas; and
3. the fact that 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

Highway storm drainage facilities collect stormwater runoff and convey it through the roadway right-of-way in a manner which adequately drains the roadway and minimizes the potential for flooding and erosion to properties adjacent to the right-of-way. Storm drainage facilities consist of curbs, gutters, storm drains, side ditches or open channels (as appropriate) and culverts. The placement and hydraulic capacities of storm drainage facilities should be designed to take into consideration damage to adjacent property and to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements and available funds.

Following is a summary of policies which should be followed for pavement drainage system design and analysis.

36-2.02 Bridge Decks

Zero gradients, sag vertical curves and superelevation transitions with flat pavement sections should be avoided on bridges. The desirable longitudinal grade for bridge deck drainage is 0.5% or greater, especially on new construction; flatter grades will be tolerated where it is not physically or economically desirable to meet this criteria. Many bridges do not require any drainage facilities. Quantity and quality of runoff should be maintained as required by applicable stormwater regulations. See Chapter Thirty-three “Bridge Deck Drainage” for additional information.

36-2.03 Curbs, Inlets and Turnouts

Curbs, inlets and turnouts are used where runoff from the pavement would erode fill slopes and/or where reduction of the right-of-way needed for shoulders, side ditches or open channels, etc., is desirable. Where storm drains are necessary, pavement sections are usually 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 criteria is included in Section 36-7.0.

36-2.05 Detention Storage

Reduction of peak flows can be achieved by the storage of runoff in detention basins, storm drainage pipes, swales and side ditches or open channels and other detention storage facilities. Stormwater can then be released to the downstream conveyance facility at a reduced flow rate. The concept should be considered at locations where existing downstream conveyance facilities are inadequate to handle peak flow rates from highway storm drainage facilities. In many locations developers are not permitted to increase runoff over existing conditions, thus necessitating detention storage facilities. Additional benefits may include the reduction of downstream pipe sizes and the improvement of water quality by removing sediment and/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 the shoulder, parking lane or pavement section. Composite gutter sections have a greater hydraulic capacity for normal cross slopes than uniform gutter sections and are therefore preferred. Refer to 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 storm drains when the momentary peak flow rate is desired. Its use should be limited to systems with drainage areas of 80 ha or less. A minimum time of concentration of five minutes is generally acceptable. The Rational method is described in Chapter Twenty-nine "Hydrology."

36-2.08 Inlets

The term "inlets" refers to all types of inlets such as grate inlets, curb inlets and slotted inlets. Drainage inlets are sized and located to limit the spread of water on traffic lanes to tolerable widths 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 sags should not be substantially greater than the width of spread encountered on continuous grades.

Grate inlets and depression of curb opening inlets should be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid them. All grate inlets shall be bicycle safe when used on roadways that allow bicycle travel. In general, when grate inlets are used at sag locations, a double curved vane grate should be utilized to compensate for plugging that might occur.

In locations where significant ponding may occur, such as at underpasses or sag vertical curves in depressed sections, it is recommended practice to place flanking inlets on each side of the inlet at the low point in the sag. Review 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 120 m. 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 and Median Ditches

Large amounts of runoff should be intercepted before it reaches the highway to minimize the deposition of sediment and other debris on the roadway and to reduce the amount of water which must be carried in the gutter section. Median areas and inside shoulders must be sloped to prevent runoff from the median area from running across the pavement. Surface channels should have adequate capacity for the design runoff and should be located and shaped in a manner that does not present a traffic hazard. Where permitted by the design velocities, channels should have a vegetative lining. Appropriate linings may be necessary where vegetation will not control erosion. See Chapter Thirty for detailed hydraulic information on channels.

36-2.11 Storm Drains

Storm drains are defined as closed conduit systems and consist of that portion of the storm drainage system that receives runoff from inlets and conveys the runoff to some point where it is then discharged into a side ditch or water body. At least one end is connected to a manhole, inlet, catch basin or similar structure. Pipes connected to inlets located in paved medians, grassed medians or lawn areas are considered storm drain structures. Storm drains should have adequate capacity so that they can accommodate runoff that enters the system. They should be designed with future development in mind if it is appropriate. The storm drain system for a major vertical sag curve should have a higher level of flood protection to decrease the depth of potential ponding on the roadway and bridges. Where feasible, the storm drains shall be designed to avoid existing utilities. Attention shall be given to the storm drain outfalls to ensure that the potential for erosion is minimized. The drainage system design should be coordinated with the proposed staging of large construction projects to maintain an outlet throughout the construction project.

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

The placement and capacities should be consistent with local storm water management plans. A minimum pipe size of 300 mm and a minimum velocity of 0.8 m/s is desirable in the storm drain 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 basics required 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 Drain Agreements Policy

A storm sewer agreement is required whenever a new or reconstructed INDOT drainage facility is designed to accommodate the storm water from a sewer controlled by a local governmental agency. This is applicable regardless of whether the shared drainage facility is constructed within or outside of INDOT right-of-way.

When INDOT constructs drainage facilities outside the limits of the right-of-way to provide adequate drainage of the highway, I.C. 8-23-6-2 allows INDOT to assess a proportionate share of the cost of constructing the drainage facilities outside the right-of-way to beneficiaries of the drainage structure. This means that 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:

$$\left[\begin{array}{c}\textit{Amount of}\\ \textit{Assessment to}\\ \textit{Beneficiary}\end{array}\right] = \left[\frac{\begin{array}{c}\textit{Discharge from Storm Sewer}\\ \textit{Draining from Outside}\\ \textit{INDOT R/W}\end{array}}{\begin{array}{c}\textit{Total Discharge of the}\\ \textit{Drainage Facility}\end{array}}\right]\left[\begin{array}{c}\textit{Cost of}\\ \textit{Drainage}\\ \textit{Facility}\end{array}\right] \qquad \text{(Equation 36-2.1)}$$

The remainder of the cost will be paid by INDOT.

By common law, INDOT also has the authority to seek a contribution from the local government any time storm water 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 trunk line storm drain, INDOT's policy will be to request a cost-sharing agreement for the trunk line sewer construction. The proportionate share will also be determined by Equation 36-2.1. If the discharge is in the form of sheet flow onto INDOT right-of-way, INDOT will generally 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 in most cases as a matter of good public policy.

If a particular situation involving sheet flow onto right-of-way is sufficiently significant to warrant a cost-sharing agreement, the local government should agree to the necessary local contribution as a condition for initiating the State highway improvement. Such an agreement cannot be forced upon a municipality but must be pre-arranged through negotiations between the local agency and the Division of Planning and Programming or the Engineering Assessment Section within the Division of Pre-Engineering and Environment. However, this may occur as late as the design phase.

A situation may arise where INDOT storm sewer construction results in a request for storm water 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 storm water, INDOT should seek a storm sewer cost-sharing contribution from the city. The procedure for determining the appropriate contribution by the city will be as described in the preceding two paragraphs. INDOT cannot site I.C. 8-23-6-2 as authority to pass on any portion of a county drainage assessment to the city. Only county drainage boards have the authority to levy drainage assessments on municipalities or private property owners when regulated drains are involved.

County drainage assessments do not require a formal agreement to be legally binding on INDOT. However, where an assessment includes a monetary contribution which relieves INDOT from providing storm water detention mandated by the county, the conditions of the assessment should be formalized in an agreement.

In most cases, the need for a storm sewer agreement should be identified during the preliminary plan 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 trunk line sewer and the exact ratio to be used in determining the city's share should be verbally agreed to with the city. The ratio may be based on the sewer cross sectional area if the discharge of the city storm sewer cannot be reasonably determined. The city should be notified in writing of the approximate cost of their share so that they can arrange financing.

After design approval, the formal storm sewer arrangement will be written binding the City and the State. The INDOT Legal Division usually 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 Structures and Castings

Figure 36-2A shows which castings may be used with given types of castings, inlets, or manholes. The information shown in the figure is complementary to that shown on the related INDOT *Standard Drawings*. When 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 those permitted in the figure, he or she should contact the Design Division's Hydraulics Unit.

36-3.0 SYMBOLS AND DEFINITIONS

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

36-4.0 CONCEPT DEFINITIONS

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

1. Check Storm. The use of a less frequent event, such as a 50-year storm, to assess hazards at critical locations where water can pond to appreciable depths is commonly referred to as a check storm or check event.

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

3. Crown. The crown, sometimes known as the soffit, is the top inside of a pipe.

4. Culvert. A culvert is 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 Inlets. Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets is to intercept debris as the slope decreases and to act in relief of the inlet at the low point.

9. Flow. Flow refers to a 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 any 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, usually 610 mm at a cross-slope of 2.5%, and the parking lane, shoulder or pavement at a cross slope of a lesser amount, typically 2.0%. A uniform gutter section has one constant cross slope. See Section 36-8.0 for additional information.

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

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

16. Invert. The invert is the inside bottom of the pipe.

17. Lateral Line. A lateral line, sometimes referred to as a lead, has inlets connected to it but has no other storm drains connected. It is usually 375 mm or less in diameter and is tributary to the trunk line.

18. Pressure Head. Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

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

20. Sag Point/Major Sag Point. A low point in a vertical curve. A major sag point refers to a low point that can overflow only if water can pond to a depth of 0.5 m or more.

21. Scupper. A vertical hole through a bridge deck for the purpose of deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.

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. Two types in general use are slotted drain pipe and slotted vane drain pipe. Slotted drain inlets are often used in conjunction with a single grate inlet for clean-out access.

24. Storm Drain. Storm drains include all pipes which are installed in conjunction with inlets, catch basins or manholes. The grassed median inlet (Types N and P), lawn inlet (Type E), and lawn catch basin (Type E and Pipe Catch Basins) pipes are considered storm drains.

25. Splash-Over. 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. Trunk Line. A trunk line is the main storm drain line. Lateral lines may be connected at inlet structures or manholes. A trunk line is sometimes referred to as a "main."

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 any storm drainage system involves the accumulation of basic data, familiarity with the project site, and a basic understanding of the hydrologic and hydraulic principles and drainage policy associated with that design.

36-5.02 General Design Approach

The design of a storm drain system is generally a process which evolves as a project develops. The primary ingredients to this process are listed below in a general sequence by which they may be carried out. This *Manual* will not attempt to name all the players of this process because it may vary for each project; however, the Hydraulics Engineer will play a major role 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 location and spacing (Sections 36-9.0 & 36-10.0).
5. Plan layout of storm drain system:
    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.
9. Provide documentation (Chapter Twenty-eight).

36-5.03 Required Data

The designer should be familiar with land use patterns, the nature of the physical development of the area(s) 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 some existing outfall location. Furthermore, there should be an understanding of the nature of the outfall because it usually has a significant influence on the storm drainage system. In environmentally sensitive areas, there may be water quality requirements to consider as well.

Actual surveys of these and other features are the most reliable means of gathering the required data. Photogrammetric mapping has become one of the most important methods of obtaining the large amounts of data required for drainage design, particularly for busy urban roadways with all the attendant urban development. Existing topographic maps, available from the U. S. Geological Survey, the Natural Resources Conservation Service, many municipalities, some county governments and even private developers, are also valuable sources of the kind of data needed for a proper storm drainage design. Governmental planning agencies should be consulted regarding plans for the area in question. Often, in rapidly growing urban areas, the physical characteristics of an area to be served by a storm drainage system may change drastically in a very short time. In such cases, the designer must anticipate these changes and consider them in the storm drainage design. Comprehensive Stormwater Management Plans and Floodplain Ordinances should be reviewed when they are available.

36-5.04 Preliminary Sketch

Preliminary sketches or schematics, featuring the basic components of the intended design, are useful to the designer. Such sketches should indicate watershed areas and land use, existing drainage patterns, plan and profile of the roadway, street and driveway layout with respect to the project roadway, underground utility locations and elevations, locations of proposed retaining walls, bridge abutments and piers, logical inlet and manhole locations, preliminary lateral and trunk line layouts and a clear definition of the outfall location and characteristics. This sketch should be reviewed with the traffic staging plans and soils recommendations for areas which are incompatible with required construction staging. With this sketch or schematic, the designer is able to proceed with the detailed process of storm drainage design calculations, adjustments and refinements.

Unless the proposed system is very simple and small, the designer should not ignore a preliminary plan as described above. Upon completion of the design, documentation of the overall plan is facilitated by the preliminary schematic.

36-5.05 Special Considerations

Consideration and planning should be directed toward avoidance of utilities and deep cuts. In some cases, traffic must be maintained or temporary bypasses constructed and temporary drainage provided for during the construction phase. Further consideration should be given to the actual trunk line layout and its constructibility. For example, will the proposed location of the storm drain interfere with existing utilities or disrupt traffic? Some instances may dictate a trunk line on both sides of the roadway with very few laterals while other instances may call for a single trunk line. Such features are usually a function of economy but may be controlled by other physical features.

It is generally not a good practice to decrease pipe size in a downstream direction regardless of the available pipe gradient because of potential plugging with debris.

36-6.0 PAVEMENT DRAINAGE

36-6.01 Introduction

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 decks.

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

36-6.02 Roadway Longitudinal Slope

A minimum longitudinal gradient is important for a curbed pavement because of the spread of stormwater against the curb. Flat gradients on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge.

The desirable minimum gutter grades for curbed pavements is 0.5% and the absolute minimum is 0.3%. Minimum grades in curbed sections can be maintained in very flat terrain by rolling the longitudinal gutter profile. On uncurbed roadways, the minimum longitudinal gradient is 0.0%.

To provide adequate drainage in sag vertical curves, a minimum slope of 0.3% should be maintained within 15 m 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 51 (m/%). Although ponding is not usually a problem at crest vertical curves, on extremely flat curves a similar minimum gradient should be provided to facilitate drainage.

36-6.03 Cross Slope

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

1. two-lane or multilane facilities,
2. urban or rural location,
3. functional classification of the facility,
4. new construction/reconstruction or 3R work, and
5. curbed or uncurbed facilities.

Refer to Chapters Forty-five and Fifty-three to determine the applicable roadway cross slopes.

36-6.04 Pavement Texture

The pavement texture is an important consideration for roadway surface drainage. Although the hydraulic design engineer will have little control over the selection of the pavement type or its texture, it is important to know that pavement texture does have an impact on the buildup of water depth on the pavement during rain storms. A good 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 new Portland cement concrete pavements while it is still in the plastic state. Re-texturing of an existing Portland cement concrete surface can be accomplished through pavement grooving and cold milling. Both longitudinal and transverse grooving are very effective in achieving macrotexture in concrete pavement. Transverse grooving aids in surface runoff resulting in less wet pavement time. Combinations of longitudinal and transverse grooving provide the most adequate drainage for high-speed conditions.

36-6.05 Curb and Gutter

Curbing at the outside edge of pavements is normal practice for low-speed, urban highway facilities. They serve several purposes which include containing the surface runoff within the roadway and away from adjacent properties, preventing erosion, providing pavement delineation and enabling the orderly development of property adjacent to the roadway. See Section 45-1.0 for a detailed 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. When a design storm flow occurs, there is a spread or widening of the conveyed water surface and the water spreads to include, not only the gutter width, but also parking lanes or shoulders and portions of the traveled surface. This is the width which concerns the hydraulics engineer the most for curb and gutter flow. Limiting this width becomes a very important design criterion. This is discussed in further detail in Section 36-7.0.

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

36-6.06 Roadside and Median Ditches

Roadside ditches are commonly used with uncurbed roadway sections to convey runoff from the highway pavement and from areas which drain toward the highway. Due to right-of-way limitations, roadside ditches cannot be used on most urban arterials. They can be used in cut sections, depressed sections and other locations where sufficient right-of-way is available and driveways or intersections are infrequent. Where practical, the flow from major areas draining toward curbed highway pavements should be intercepted in the ditch as appropriate.

It is preferable to slope median areas and inside shoulders to a center swale to prevent drainage from the median area from running across the pavement. This is particularly important for high-speed facilities and for facilities with more than two lanes of traffic in each direction.

Chapter Thirty discusses the hydraulic design of channels.

36-6.07 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. However, it is often 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 usually requires a constant cross slope. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. Runoff should be collected by inlets, although gutter turnouts may be used for extremely minor flows in some areas. Runoff should also be handled in compliance with applicable stormwater quality regulations. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

The gutter spread should be checked to ensure compliance with the design criteria in Section 36-7.0. Flat gradients and sag vertical curves are not normally allowed on bridges on new alignment. The desirable longitudinal slope for bridge deck drainage is 0.5% or greater. Flatter grades will be tolerated where it is not physically or economically desirable to meet the above criteria.

Many bridges will not require any drainage structures at all. 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 and/or Curbs

Shoulder gutter and/or mountable curbs may be appropriate to protect fill slopes from erosion caused by water from the roadway pavement. They should be considered on 2:1 fill slopes higher than 6 m. They should also be considered on 3:1 fill slopes higher than 6 m if the roadway grade is steeper than 2%. In areas where permanent vegetation cannot be established, the height criteria should be reduced to 3 m regardless of the grade. Inspection of the existing/proposed site conditions and contact with maintenance and construction personnel shall be made by the designer to determine if vegetation will survive. Erosion control blankets can be an effective tool to facilitate the establishment of vegetation.

Shoulder gutter and/or curbs or, more commonly, riprap turnouts shall be utilized at bridge ends where concentrated flow from the bridge deck would otherwise run down the fill slope. This section of gutter should be long enough to include the transitions. Shoulder gutters or riprap turnouts are not required on the high side of superelevated sections or adjacent to barrier walls on high fills.

36-6.09 Median/Median Barriers

Medians are commonly used to separate opposing lanes of traffic on divided highways. It is preferable to slope median areas and inside shoulders to a center depression to prevent drainage from the median area from running across the traveled way. Where median barriers are used and, particularly on horizontal curves with associated superelevations, it is necessary to provide inlets and connecting storm drains to collect the water which accumulates against the barrier. Slotted drains adjacent to the median barrier and in some cases weep holes in the temporary barriers can also be used for this purpose.

36-6.10 Impact Attenuators

The location of impact attenuator systems should be reviewed to determine the need for drainage structures in these areas. With impact attenuator systems, it is necessary to have a clear or unobstructed opening as traffic approaches the point of impact to allow a vehicle to impact the system head on. If the impact attenuator is placed in an area where superelevation or other grade separation occurs, grate inlets and/or slotted drains may be needed to prevent water from running through the clear opening and crossing the highway lanes or ramp lanes. Curb, curb-type structures or swales cannot be used to direct water across this clear opening because vehicular vaulting could occur when the impact attenuator system is impacted.

36-7.0 STRUCTURE SIZING PROCESS

Section 36-7.0 presents a summary of the hydraulic process for sizing storm drain systems and slotted drain inlets. Where applicable, the discussion references other sections in Chapter Thirty-six.

36-7.01 Storm Drain Systems

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 typical practices.

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

In general, the spread should be held to the specified width for design frequencies. For storms of greater magnitude, the spread can be allowed to utilize "most" of the pavement as an open channel. For multi-lane curb and gutter sections, or guttered roadways with no parking, it is not practical to avoid travel lane flooding when longitudinal grades are flat (0.2% to 1%). However, flooding should not exceed the lane adjacent to the gutter (or shoulder) for design conditions. INDOT design criteria for allowable water spread is presented in Figure 36-7A.

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

36-7.01(02) Inlet Spacing

Curb inlet spacing calculations must be in accordance with accepted engineering practice. The designer must contact the Hydraulics Unit if there is any question on whether the intended calculation method is acceptable. Gutter flow that bypasses curb inlets installed on grade must be accounted for at downstream structures. In addition, flanking inlet(s) should be provided at sag locations to mitigate ponding problems resulting from grate clogging.

After calculating the required spacing, actual inlet locations must be determined. Section 36-10.0 presents the Department's detailed hydraulic calculations for inlet spacing, and Section 36-9.03 presents criteria for inlet locations independent of hydraulic calculations. All curb inlets and associated laterals must be included in all system modeling required for the design and check storm evaluation discussed below.

1. Design Storm. All storm drain structures 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 all system manholes, inlets, catch basins and similar structures. See Section 36-13.0.

The design process for storm drain structures does not require two sets of hydraulic calculations, because all pipe materials acceptable for use as storm drains have 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 Sizes/Cover/Velocity

The minimum pipe size used for storm drain structures is 300 mm or 0.10 m^2. The cover provided over a storm drain structure must be at least 0.30 m and no greater than 30.5 m. The minimum full-flow velocity for storm drain structures is 0.8 m/s, and the recommended maximum velocity is 2.0 m/s. Storm drain outlet structures also require energy dissipators to mitigate potential erosion. The dissipator riprap gradation requirements are identical to those outlined for culvert structures. See Chapter Thirty-four. Contact the Hydraulics Unit for additional instructions if the required riprap gradation is prohibited due to clear zones or other issues.

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

36-7.02 Slotted Drain Pipe/Slotted Vane Drain Pipe

The design requirements for these structure types depend on the structure application. See Sections 36-9.02 and 36-10.0 for a detailed discussion on the hydraulic design of slotted drain inlets. The following presents commonly used applications and the associated design requirements.

1. Superelevated Traveled Way Edge Installations (Slotted Drain Pipe). When installed adjacent to the edge of superelevated sections, the slotted drain pipe sizing will be based on a 50-year storm frequency for Interstate facilities and a 10-year storm frequency for all other roadways. The pipe sizing must be in accordance with accepted practices contained in recognized engineering publications. See Section 36-10.06.

2. Gutter Installations at Sag Curb Inlets (Slotted Drain Pipe). The design storm requirements for these installations are identical to those for storm drains. The length and size of pipe required must be determined in accordance with accepted practices contained in recognized engineering publications. See Section 36-10.05.

3. Storm Drain Structure. 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. Therefore, the sizing process involves two steps for these structures. 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 above process

must be checked for adequacy for interception of sheet flow. The sheet flow interception design storm frequency will be Q_{50} for Interstate facilities and Q_{10} for all other roadways.

4. Culvert Structure. The sizing of a slotted drain pipe (corrugated interior designation) or a slotted vane drain pipe (smooth interior designation) for a culvert application is also a two-step process. The structure is first sized as a culvert in accordance with all 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 in the appropriate standard drawing, contact the Hydraulics Unit for additional instructions.

36-7.03 Pipe Extensions

The sizing of pipe extensions for storm drain structures will be in accordance with the following:

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 outlined previously are met.

If the extended structure meets all required design criteria, then the structure sizing process is complete. If the extended structure does not meet 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 concerns, contact the Hydraulic Units for further instructions.

36-7.04 Sanitary Sewer and Water Utility Coordination

Coordination with utilities should begin as soon as possible once it is determined that the proposed construction will impact existing utility facilities. On INDOT projects, the coordination will be administered through the Utilities Unit. For projects not on INDOT routes, the designer should contact all affected utilities as soon as possible.

If it is determined that utility relocation elements 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* contain no 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 provision(s). If the utility has specific casting, manhole or other facility requirements that differ from those contained 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 the curbed channel to the spread of water on the shoulder, parking lane or pavement section. Equations can be utilized to solve uniform cross slope channels, composite gutter sections and V-shape gutter sections. Figure 36-8D may also be used to solve composite gutter section problems. Computer programs, such as the FHWA HEC 12 program, is also very useful for this computation and inlet capacity. Composite gutter sections have a greater hydraulic capacity for normal cross slopes than uniform gutter sections and are therefore preferred. The following Sections present example problems for each gutter section.

36-8.02 Manning's n For Pavements

Figure 36-8A provides typical values for Manning's n for street and pavement gutters.

36-8.03 Uniform Cross Slope Procedure

The following equation can be used to solve for gutter capacity for uniform cross slopes:

$$Q = \frac{0.377}{n} S_X^{1.67} S^{0.5} T^{2.67} \qquad \text{(Equation 36-8.1)}$$

Where:

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

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

36-8.04 Composite Gutter Sections Procedure

To solve composite gutter flow problems, 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 find the flow in a gutter section with width (W) less than the total spread (T). These calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

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: $S = 0.01$; $S_X = 0.02$; $S_W = 0.06$; $W = 0.6$ m; $n = 0.016$; $Q = 0.057\ m^3/s$; try $Q_S = 0.020\ m^3/s$

   b. Calculate the gutter flow in W (Q_W), using the equation:

      $Q_W = Q - Q_S$ $\quad Q_W = 0.057 - 0.020 = 0.037\ m^3/s$ $\quad$ (Equation 36-8.2)

   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:

      $Q_W/Q = 0.037/0.057 = 0.65$ $\quad S_W/S_X = 0.06/0.02 = 3$. 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 c:

      $T = 0.6/0.27 = 2.22$ m

e. Find the spread above the depressed section (T_S) by subtracting W from the value of T obtained in step d. $T_S = 2.22 - 0.6 = 1.62$ m

f. Use the value of T_S from step e, Manning's n, S and S_X to find the actual value of Q_S from Equation 36-8.1; $Q_S = 0.014$ m^3/s.

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

Compare 0.014 to 0.020 "no good." Try $Q_S = 0.023$; then $0.057 - 0.023 = 0.034$, and $0.034/0.057 = 0.6$; from Figure 36-8C, $W/T = 0.23$, then $T = 0.6/0.23 = 2.61$ m and $T_S = 2.61 - 0.6 = 2.01$ m. From Equation 36-8.1, $Q_S = 0.023$ m^3/s. OK

Answer: Spread T = 2.61 m

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 $T = 3.05$ m; $W = 0.6$ m; $T_S = 3.05 - 0.6 = 2.44$ m; $S_X = 0.04$; $S = 0.005$ m/m; $S_W = 0.06$; $n = 0.016$; $d = 0.13$ m

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 slopes, Condition 2, substituting T_S for T. From Equation 36-8.1, $Q_S = 0.085$ m^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). $W/T = 0.6/3.05 = 0.2$; $S_W/S_X = 0.06/0.04 = 1.5$; From Figure 36-8C, $E_O = 0.46$.

d. Calculate the total gutter flow using the equation:

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

Where:
- Q = gutter flow rate, m^3/s
- Q_S = flow capacity of the gutter section above the depressed section, m^3/s
- E_O = ratio of frontal flow to total gutter flow (Q_W/Q)

$$Q = 0.085 / (1 - 0.46) = 0.157 \text{ m}^3/\text{s}$$

e. Calculate the gutter flow in width (W), using Equation 36-8.2:

$Q_W = Q - Q_S = 0.157 - 0.085 = 0.072\ m^3/s$

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

36-8.05 V-Type Gutter Sections Procedures

Equation 36-8.1 can also be used to solve V-Type channel problems. The spread (T) can be calculated for a given flow (Q) or the flow can be calculated for a given spread. This method can be used to calculate approximate flow conditions in the triangular channel adjacent to concrete median barriers. It assumes the effective flow is confined to the V channel 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: $S = 0.01$, $S_{X1} = 0.06$, $S_{X2} = 0.04$, $S_{X3} = 0.015$, $n = 0.016$, $Q = 0.057\ m^3/s$, shoulder = 1.83 m. See Figure 36-8E.

   b. Calculate S_X:

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

   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, $T_1 = 2.55$ m; however, because the shoulder width of 1.83 m is less than 2.55 m, S_{X2} is 0.04 and the pavement cross slope S_{X3} is 0.015, T will actually be greater than 2.55 m; 2.55 - 0.6 = 1.95 m which is > 1.22 m; therefore, the spread is greater than 2.55 m.

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

   Point B: 1.95 m @ 0.04 = 0.078 m. Point C: 0.078 m - (1.22 m @ 0.04) = 0.029 m

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

   $T_{0.015} = 0.029/0.015 = 1.93$ m

   f. Find the actual total spread (T). T = 1.83 + 1.93 = 3.76 m

2. Condition 2. Given Spread (T), Find Flow (Q).

   a. Determine input parameters such as 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 = 1.83 m

   b. Calculate S_X:

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

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

   For T = 1.83 m, Q = 0.029 m^3/s

36-9.0 INLETS

36-9.01 General

Inlets are drainage structures utilized to collect surface water through grate or curb openings and convey it to storm drains or a direct outlet to culverts. Grate inlets should be bicycle safe unless located on highways where bicycles are not permitted.

This section will discuss the various types of inlets in use and recommend guidelines on the use of each type.

36-9.02 Types

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

36-9.02(01) Grate Inlets

These inlets consist of an opening in the gutter covered by one or more grates. They are best suited for use on continuous grades. Grates are susceptible to clogging with debris and, thus, should be supplemented with curb boxes and additional grate capacity to allow for partial clogging at sag points. Flanking inlets are recommended at major sag points. Grates should be bicycle safe where bike traffic is anticipated and structurally designed to handle the appropriate loads when subject to traffic. See Section 36-10.0 for additional discussion.

36-9.02(02) Combination Inlets

Various types of combination inlets are in use. Curb box and grate combinations are common with the curb opening adjacent to the grate. Slotted inlets are also used in combination with grates, 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. Combination inlets may be desirable in sags because they can provide additional capacity in the event of plugging.

36-9.02(03) Slotted Drain Inlets

INDOT uses two types of slotted drain inlets˜ slotted drain pipe (used on mainline roadways) and slotted vane drain pipe (used on driveways). The slotted drain pipe is used to intercept sheet flow at roadway edges. It can also be installed in concrete gutters in conjunction with curb inlets at sag locations. Slotted vane drain pipe is used to intercept sheet flow on urban driveways. Slotted drain inlets are generally used as components of a storm drain 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. Both types function as weirs with flow entering from the side. They can be used to intercept sheet flow, collect gutter flow with or without curbs, modify existing systems to accommodate roadway widening or increased runoff, and reduce ponding depth and spread at grate inlets.

36-9.03 Inlet Locations

Inlets are required at locations needed to collect runoff within the design controls specified in the design criteria (Section 36-7.0). In addition, there are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity or runby. Examples of such locations are as follows:

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

Inlets should not be located in the path where pedestrians are likely to walk.

36-10.0 INLET SPACING

36-10.01 General

A number of inlets are required to collect runoff at locations with little regard for contributing drainage area as discussed in Section 36-9.0. These should be plotted on the plan first. Next, it is best to start locating inlets from the crest and working down grade to the sag points. The location of the first inlet from the crest can be found by determining the length of pavement and the area back of the curb sloping toward the roadway which will generate the design runoff. The design runoff can be computed as the maximum allowable flow in the curbed channel which will meet the design criteria as specified in 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:

$$L = \frac{10\ 000\ Q_t}{0.0028\ C\,i\,W} \qquad \text{(Equation 36-10.1)}$$

Where:
- L = distance from the crest, m
- Q_t = maximum allowable flow, m^3/s
- C = composite runoff coefficient for contributing drainage area
- W = width of contributing drainage area, m
- i = rainfall intensity for design frequency, mm/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 runby. The runby from the first inlet is added to the computed flow to the second inlet, the total of which must be less than the maximum allowable flow dictated by the criteria. Figure 36-10K is an inlet spacing computation sheet which can be utilized to record the spacing calculations.

36-10.02 Grate Inlets 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 both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, a portion of the frontal flow may tend to splash over the end of the grate for some grates. Figure 36-10A can be used to determine splash-over velocity for curved vane grates and reticuline grates. No data is available for other grate types used by INDOT. A good estimate of splash-over velocities for grates with rectangular openings, such as the alternative grates for castings 12, 13 and 14, is approximately 0.6 m/s less than the splash-over velocity for the reticuline grates.

INDOT recommends the curved vane grate for all curb and gutter applications. Section 36-17.0 presents a hydraulic capacity chart for the curved vane grate inlet typically used by INDOT. The chart is based on a roadway cross section typically used by the Department. For other inlets and roadway cross sections, the remainder of Section 36-10.0 presents the procedures for determining the hydraulic performance.

FHWA has developed computer software (HY12) which will analyze the flow in gutters and the interception capacity of grate inlets, curb-opening inlets, slotted drain inlets and combination inlets on continuous grades. Both uniform and composite cross-slopes can be analyzed. In addition, the program can analyze curb opening, slotted drain and grate inlets in a sag. Enhanced versions by private vendors have made the program more user friendly and improved its usefulness. Unfortunately, not all INDOT grate configurations have been included in the HEC 12 program. The curved vane and the reticuline grates used in the program are similar to INDOT grates and can be used by inputting the appropriate size. Other grates, such as INDOT’s castings Type 12, 13 and 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 is given by the following equation.

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

Where: Q = total gutter flow, m^3/s
Q_W = flow in width W, m^3/s
W = width of depressed gutter or grate, m
T = total spread of water in the gutter, m

Figure 36-8C provides a graphical solution of E_O for either straight cross slopes or depressed gutter sections.

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

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

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation.

$$R_f = 1 - 0.295\,(V - V_O) \qquad \text{(Equation 36-10.4)}$$

Where: V = velocity of flow in the gutter, m/s
V_O = gutter velocity where splash-over first occurs, m/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 splash-over 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.

$$V = \frac{0.752}{n} S^{0.5} S_X^{0.67} T^{0.67} \qquad \text{(Equation 36-10.5)}$$

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

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:

$$R_S = 1 / [1 + (0.0828V^{1.8}/S_X L^{2.3})] \qquad \text{(Equation 36-10.6)}$$

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

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.

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

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<u>Example Problem 36-10.1</u>

Given: Urban Non-freeway; 4-lane undivided with crown at centerline
Drainage Area: 61-m residential strip, C = 0.4, S = 0.005 m/m
3.6-m lane @ 0.02 cross slope and 0.61-m gutter at 0.025 cross slope
10-year design, IDF Curve for Indianapolis in Chapter Twenty-nine
Allowable spread T = 2.4 m, n = 0.016
$S_O = 0.01$, $S_X = 0.02$, $S_W = 0.025$
Use Curves & Equations
Use Standard INDOT Grate Type 10 and 11, 397 mm x 880 mm

Find: Maximum allowable flow, Q_T
Q_i intercepted by 397 mm □ 880 mm vane grate
Q_r runby
Location of first and second inlets from crest of hill

Solution:

1. <u>Step 1</u>. Solve for Q_S using Equation 36-8.1:

$$Q_S = \frac{0.377}{n} S_X^{1.67} S^{0.5} T^{2.67}$$

$$S_W / S_X = 0.025/0.02 = 1.25$$

$$Q_S = \frac{0.377}{0.016} (0.02)^{1.67} (0.01)^{0.5} \ (2.4 - 0.61)^{2.67}$$

$Q_S = 0.0162\ m^3/s$

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

3. Step 3. Find total Q_T (maximum allowable flow):

$W/T = 0.61/2.40 = 0.25 \qquad E_O = 0.55 = Q_W/Q$

$$Q_T = Q_S/(1 - E_O) = 0.0162/(1 - 0.55) = 0.36\ m^3/s$$

4. Step 4. From Equation 36-10.5:

$$V = \frac{0.75}{n} S^{0.5} S_X^{0.67} T^{0.67}$$

$$V = \frac{0.75}{0.016}(0.01)^{0.5}(0.02)^{0.67}(2.4)^{0.67} = 0.61\ m/s$$

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

5. Step 5. From Figure 36-10A, $R_f = 1.0$; from Figure 36-10C, $R_S = 0.35$

6. Step 6. Using Equation 36-10.8:

$$Q_i = 0.036[(1.0)(0.55) + 0.35(1 - 0.55)] = 0.025\ m^3/s$$

7. Step 7. $Q_r = Q_T - Q_i \quad Q_r = 0.036 - 0.025 = 0.011$

8. Step 8. Locate first inlet from crest:

Using Equation 36-10.1:

$$L = \frac{10\ 000\ Q_t}{0.00278 C\ i\ W}$$

To find i, first solve for t_C; from Figure 29-7D in a residential area:

30-m strip
C =0.4
S =0.5%, overland flow
t_C =15 min
Gutter flow estimated at V = 0.61 m/s in Step 4.

Try L = 100 m; t_C = 100/(0.61 x 60) - 2.7 min.
Total t_C = 15 + 2.7 = 17.7 min
From Figure 29-8C (IDF curve), i = 110 mm/h

Solve for weighted C value: $C = [(30)(0.4) + (7.8)(0.9)]/37.8 = 0.50$

Finally:

$L = 10\,000\,(0.036) / 0.00278\,(0.50)\,(110)\,(37.8) = 62.3$ m No Good

Try 60 m and rework:

$L = 10\,000\,(0.036) / 0.00278\,(0.50)\,(112)\,(37.8) = 61.2$ m OK

Therefore, place first inlet 60 m from crest.

9. Step 9. To locate second inlet:

$Q_T = 0.36\ m^3/s$, $Q_r = 0.011\ m^3/s$, $Q_{allowable} = 0.036 - 0.011 = 0.025\ m^3/s$

Assuming similar drainage area and t_C, i = 110 mm/hr

$L = 10\,000(0.025)/(0.00278)(0.50)(110)(37.8) = 43$ m

Therefore, place second inlet 43 m from first inlet.

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36-10.03 Grate Inlets In Sag

36-10.03(01) Standard Practice

Standard practice by INDOT is to install two curved vane grates (Types 10 and 11) on one frame casting at sag points. Each vane grate is positioned to receive water from each upstream direction. Curb boxes are combined with the grates to provide relief in the event the grate is plugged with debris. Curb boxes are ignored in the hydraulic capacity calculations.

A grate inlet in a sag operates as a weir up to a depth of about 0.12 m and as an orifice for depths greater than 0.43 m. 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, m
C = 1.66
d = depth of water at curb measured from the normal cross slope gutter flow line, m

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

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

Where: C = 0.67 orifice coefficient

A = clear opening area of the grate, m^2

g = 9.81 m/s^2

Figure 36-10D is a plot of Equations 36-10.9 and 36-10.10 for various grate sizes. The effects 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.

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Example Problem 36-10.2 (Sag Point)

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.

$Q = 0.03\ m^3/s$, design storm $Q_r = 0.01\ m^3/s$

$Q = 0.035\ m^3/s$, check storm $Q_r = 0.015\ m^3/s$

$S_X = 0.02$ m/m T = 2.4 m design

$d = TS_X = 0.048$ m n = 0.016

Use Grates 10 and 11 (397 mm x 880 mm)

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 standard Grate 10 and 11.

1. Design Storm.

a. P = 397 + 880 + 397 = 1674 mm = 1.67 m

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

$Q = 1.66\ P\ d^{1.5}$

$Q = 1.66\ (1.67)\ (0.048)^{1.5} = 0.029 \approx .03$ OK

In accordance with INDOT policy, two Type 10 and 11 Grates with curb boxes will be placed at the sag point. Curb boxes will not be analyzed hydraulically, but available in the event of plugging.

2. Check Storm.

a. P = 2(397) + 2(880) = 2554mm = 2.55m

b.

$$d^{1.5} = \frac{Q}{1.66\,P} = \frac{0.035}{(1.66)(2.55)} = 0.00827 \quad d = 0.041\,m \quad OK$$

INDOT practice is to provide a gradient of 0.3% within 15 m of the level point in a sag vertical curve. Check T at S = 0.003 for the design and check flow:

c. Use Equation 36-8.1:

$$Q = \frac{0.377}{n} S_x^{1.67} S^{0.5} T^{2.67}$$

d. Design :

$$T^{2.67} = \frac{0.01(0.016)}{0.377(0.02)^{1.67}(0.003)^{0.5}} = 5.33 \qquad T = 1.87\,m \qquad OK$$

e. Check:

$$T^{2.67} = \frac{(0.015)(0.016)}{0.377(0.02)^{1.67}(0.003)^{0.5}} = 7.99 \qquad T = 2.17\,m \qquad OK$$

Thus, a standard casting type 10 and 11 is adequate to intercept the design flow at a spread which does not exceed 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 major sag points where significant ponding may occur, such as underpasses or sag vertical curves in depressed sections, it is recommended practice to place a minimum of one flanking inlet on each side of the inlet at the sag point. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the sag inlet if it should become clogged. Figure 36-10E shows the spacing required for depths at curb criteria and vertical curve lengths defined by K = L/A, where L is the length of the vertical curve in meters and A is the algebraic difference in approach grades. The INDOT policy on geometrics specifies maximum K values for various design speeds and a maximum K of 51 considering drainage on curbed facilities.

* * * * * * * * * *

Example Problem 36-10.3

Given: Data from Example in 36-10.2:

Speed = 90 km/h K = 40 S_X = 0.02 m/m T = 2.4 m design

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

Solution: Allowable depth (d) at curb = 2.4 m @ 0.02 = 0.048 m

Spacing to flanking inlet = x = 19.3 m (from Figure 36-10E by interpolation).

* * * * * * * * * *

36-10.04 Slotted Inlets (Typical Practice)

36-10.04(01) Divided Facilities with CMB

Snow accumulation adjacent to a concrete barrier (inside or outside shoulder) can present a drainage problem. Therefore, Department practice is to use slotted drains in conjunction with inlets type H-5 or HA-5 as follows:

1. Tangent Sections. Use at every third inlet.

2. Low Side of Superelevated Curves. Use at all sites.

3. Sag Vertical Curves. Use three, centered on the low point.

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

36-10.04(02) High-Side Shoulder

The slotted drain pipe is typically used at sites which meet the following conditions.

1. located on high-side shoulder of superelevated sections;
2. the high-side shoulder slopes toward the traveled way;
3. located on high-volume freeways;
4. the roadway is either curbed or uncurbed.

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

36-10.05 Slotted Inlets on Grade

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

As noted in Section 36-9.02, the Department uses two types of slotted drain inlets – slotted drain pipe (used on mainline roadways) and slotted vane drain (used on driveways). See the INDOT *Standard Drawings* for details.

36-10.05(01) Longitudinal Placement

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

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:

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

Where: K = 0.817

L_T = slotted inlet length required to intercept 100% of gutter flow, m

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

The INDOT standard slotted drain pipe slot width is 40 mm and the length is 6 m. The efficiency of slotted inlets shorter than the length required for total interception is expressed as follows:

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

Where: L = slotted inlet length, m

Figure 36-10I 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 found by using an equivalent cross slope, S_e, in Equation 36-10.11.

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

Where: S_X = pavement cross slope, m/m
S_W = gutter cross slope, m/m
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)

Note that the same equations are used for both slotted inlets and curb-opening inlets. The following example illustrates the use of this procedure.

* * * * * * * * * *

Example Problem 36-10.4

Given: Longitudinal placement of slotted inlet adjacent to curb.

$S_O = 0.01$ m/m Allowable spread = 3.0 m $n = 0.016$ $W = 0.6$ m

(1) Uniform cross slope, $S_X = 0.02$
(2) Composite cross slope, $S_X = 0.02$, $S_W = 0.025$

Find: (1) Maximum allowable Q
Q_i for a 6.0-m slotted inlet on straight cross slope.
(2) Maximum allowable Q
Q_i for a 3.0-m slotted inlet on composite cross slope.

Solution: (1) $$\text{Max } Q = \frac{0.377}{n} S_X^{1.67} S^{0.5} T^{2.67}$$ (Equation 36-8.1)

$$\text{Max } Q = \frac{0.377}{0.016} (0.02)^{1.67} (0.01)^{0.5} (3.0)^{2.67} = 0.064 \text{ m}^3/\text{s}$$

$$L_T = 0.817\, Q^{0.42} S^{0.3} (1/n\, S_X)^{0.6}$$ (Equation 36-10.11)

$$L_T = 0.817\,(0.64)^{0.42} (0.01)^{.3} [(1/(0.016)(0.02)]^{0.6} = 8.08 \text{ m}$$

$$L/L_T = 6.0/8.08 = 0.74$$
From Figure 36-10I, E = 0.91.

$Q_i = EQ = 0.91\ (0.064) =$ <u>$0.058\ \text{m}^3/\text{s}$ intercepted</u>

(2) $$Q_S = \frac{0.377}{0.016} (0.02)^{1.67} (0.01)^{0.5} (3.0 - 0.6)^{2.67} = 0.035 \text{ m}^3/\text{s}$$

$W/T = 0.6/3 = 0.2$ $S_W/S_X = 0.025/0.02 = 1.25$

From Figure 36-8C, $E_O = 0.46$.

Max $Q = Q_S/(1 - E_O) = 0.035 / (1 - 0.46) = 0.065\ \text{m}^3/\text{s}$
$S'_W = S_W - S_X = 0.025 - 0.02 = 0.005$
$S_e = S_X + S'_W E_O = 0.02 + (0.005)\ (0.46) = 0.022$
$L_T = 0.817\ (0.065)^{0.42}\ (0.01)^{0.3}\ [(1/(0.016)\ (0.022)]^{0.6} = 7.69$ m
$L/L_T = 6/7.69 = 0.78$ E = 0.92 (Figure 36-10I)
Then, $Q_i = EQ = 0.92\ (0.065) =$ <u>$0.060\ \text{m}^3/\text{s}$ intercepted</u>.

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

At driveways where it is desirable to capture virtually all of the flow (e.g., in a driveway sloped toward the roadway), a slotted vane drain can be installed in conjunction with a grate inlet. Tests have indicated that, when the slotted vane drain is installed perpendicular to the flow, it will capture approximately 0.045 m^3/s per lineal meter of drain on longitudinal slopes of 0% to 6%. Capacity curves are available from the manufacturer. The ideal installation would utilize a grate inlet to capture the flow in the gutter and the slotted vane drain to collect the flow extending into the shoulder. Note that a slotted vane drain is shaped and rounded to increase inlet efficiency and should not be confused with a standard vertical riser type slotted inlet (i.e., a slotted drain pipe).

36-10.06 Slotted Inlets In Sag

Except adjacent to a concrete median barrier (Section 36-10.04), the use of slotted drain inlets in sag configurations is generally discouraged because of the propensity of such inlets to collect debris. However, there may be locations where it is desirable to supplement an existing low point inlet with the use of a slotted drain. Slotted inlets in sag locations perform as weirs to depths of about 0.06 m, dependent on slot width and length. At depths greater than about 0.12 m, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed by the following equation.

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

Where:
- W = width of slot, m
- L = length of slot, m
- d = depth of water at slot, m
- g = 9.81 m/s^2

For a slot width of 40 mm, the above equation becomes:

$$Q_i = 0.142 L d^{0.5} \qquad \text{(Equation 36-10.15)}$$

The interception capacity of slotted inlets at depths between 0.06 m and 0.12 m 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 depths between weir and orifice flow.

36-10.07 Inlet Spacing Computations

To design the location of the inlets for a given project, information such as a layout or plan sheet suitable for outlining drainage areas, road profiles, typical cross sections, grading cross sections, superelevation diagrams and contour maps is necessary. The inlet computation sheet (Figure 36-10K) should be used to document the computations. A step-by-step 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 even without considering any specific drainage area. See Section 36-9.03 for additional information.

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

4. Select a trial drainage area approximately 90 m to 150 m below the high point and outline the area including any area that may come over the curb. (Use drainage area maps.) Where practical, large areas of behind the curb drainage should be intercepted before it reaches the highway. See Section 36-6.05.

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

6. Col. 3. Compute the drainage area in hectares and enter in Column 3.

7. Col. 4. Select a C value from one of the tables in Chapter Twenty-nine and enter in Col. 4.

8. Col. 5. Compute a time of concentration for the first inlet. This will be the travel time from the hydraulically most remote point in the drainage area to the inlet. See additional discussion in Chapter Twenty-nine. The minimum time of concentration should be 5 minutes. Enter value in Col. 5.

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

10. Col. 7. Calculate Q by multiplying Col. 3 x Col. 4 x Col 6. Enter in Col. 7.

11. Col. 8. Determine the gutter slope at the inlet from the profile grade; check effect of superelevation. Enter in Col. 8.

12. Col. 9. Enter cross slope adjacent to inlet in Col. 9 and gutter width in Col. 13.
Col. 13. Sketch composite cross slope with dimensions.

13. Col. 11. For the first inlet in a series (high point to low point), enter Col. 7 in Col. 11 because no previous runby has occurred yet.

14. Col. 12. Using Equation 36-8.1 or the available computer model, determine the spread.
Col. 14. T and enter in Col. 14 and calculate the depth d at the curb by multiplying T times the cross slope(s) and enter in Col. 12. Compare with the allowable spread as determined by the design criteria in Section 36-7.0. If Col. 15 is less than the curb height and Col. 14 is near the allowable spread, continue on to Step 16. If not OK, expand or decrease the drainage area to meet the criteria and repeat Steps 5 through 14. Continue these repetitions until Col. 14 is near the allowable spread, then proceed to Step 15.

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

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

17. Col. 17. Calculate the Q intercepted (Q_i) by the inlet and enter in Col. 17. Use Equation 36-8.1 and Figure 36-8C or Figure 36-8D to define the flow in the gutter. Utilize 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. Col. 18. Calculate the runby by subtracting Col. 17 from Col. 11 and enter into Col. 18 and also into Col. 10 on the next line if an additional inlet exists downstream.

19. Cols. 1-4. Proceed to the next inlet down grade. Select an area approximately 90 m to 120 m below the first inlet as a first trial. Repeat Steps 5 through 7 considering only the area between the inlets.

20. Col. 5. Compute a time of concentration for the second inlet downgrade based on the area between the two inlets.

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

22. Col. 7. Determine the discharge from this area by multiplying Col. 3 x Col. 4 x Col. 6. Enter the discharge in Col. 7.

23. Col. 11. Determine total gutter flow by adding Col. 7 and Col. 10 and enter in Col. 11. Col. 10 is the same as Col. 18 from the previous line.

24. Col. 12. Determine "T" based on total gutter flow (Col. 11) by using Equation 36-8.1 or Figure 36-8D and enter in Col. 14.
Col. 14. If "T" in Col. 14 exceeds the allowable spread, reduce the area and repeat Steps 19-24. If "T" in Col. 14 is substantially less than the allowable spread, increase the area and repeat Steps 19-24.

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

26. Col. 17. Determine Q_i and enter in Col. 17. See instruction in Step 17.

27. Col. 18. Calculate the runby by subtracting Col. 17 from Col. 7 and enter in Col. 16. This completes the spacing design for this inlet.

28. Go back to Step 19 and repeat Steps 19-27 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

Manholes are utilized to provide entry to continuous underground storm drains for inspection and cleanout. Inlet boxes with grates may be used in lieu of manholes 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. Typical locations where manholes should be specified are as follows:

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

Manholes should not be located in traffic lanes; however, when it is impossible to avoid locating a manhole in a traffic lane, care should be taken to ensure it is not in the normal vehicle wheel path. Where practical, locate manholes off the roadway.

36-11.02 Spacing

The spacing of manholes should be a maximum of 120 m.

36-11.03 Types

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

36-11.04 Sizing

When determining the minimum round manhole size required for various trunk line pipe sizes and locations, two general criteria must be met:

1. The manhole or inlet structure must be large enough to accept the maximum pipe as shown in Figure 36-11B. *Note: 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 any two pipes, compute:

$$K = \frac{R_1 + T_1 + R_2 + T_2 + 355mm}{\Delta} \qquad \text{(Equation 36-11.1)}$$

Where: K is the mm/degree (see Figure 36-11B)
R_1 and T_1 are the interior radius and wall thickness of Pipe #1, mm
R_2 and T_2 are the interior radius and wall thickness of Pipe #2, mm
Δ = angle between the pipes, degrees

Example Problem 36-11.1

Given: Pipe # 1 = 1350 mm, Pipe # 2 = 1200 mm
Δ = 140 deg

Solution:

$$K = \frac{675\text{ mm} + 140\text{ mm} + 600\text{ mm} + 127\text{ mm} + 355\text{ mm}}{140°}$$

K = 13.55 mm/degree

The table indicates the minimum manhole barrel to be 1650 mm. For the 1650-mm MH barrel, the table indicates a maximum pipe size of 1200 mm. Because the maximum pipe size in the example is 1350 mm, an 1800-mm MH must be used.

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

36-12.0 STORM DRAINS

36-12.01 Introduction

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

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

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

For ordinary conditions, storm drains should be sized on the assumption that they will flow full or practically full under the design discharge but will not flow under pressure head. The Manning's formula is recommended for capacity calculations. In locations such as depressed sections and underpasses 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 and/or established critical elevations for the design storm.

36-12.02 Design Procedures

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

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

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

   a. Location of storm drains.

   b. Direction of flow.

   c. Location of manholes.

   d. Location of existing utilities such as water, gas, sanitary sewers or underground cables.

3. Determine drainage areas and runoff coefficients 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 by multiplying 0.00278 x A x C x I.

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

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

6. Calculate the new area (A) and multiply by the runoff coefficient (C), add to the previous (CA), multiply by 0.00278 and the new rainfall intensity 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 and/or nomographs to accomplish the design effort.

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

   a. Storm drain design computation can be made on forms as illustrated in 36-12F.

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

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 runby occurring at each inlet during a 50-year rainfall and accumulating it at the sag point. The inlet at the sag point as well as the storm drain pipe leading from the sag point must be sized to accommodate this additional runby within the criteria established. See Section 36-7.0. To design the pipe leading from the sag point, it may be helpful to convert the additional runby 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 runby by 0.00278 x I_{10} in the pipe at the sagpoint.

Some designers may want to design separate systems to prevent the above ground system from draining into the depressed area. This concept may be more costly but in some cases may be justified. Another method would be to design the upstream system for a 50-year design to minimize the runby to the sag point. Each case must be evaluated on its own merits and the impacts and risk of flooding a sag point assessed.

36-12.04 Hydraulic Capacity

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

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad \text{(Equation 36-12.1)}$$

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

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

$$Q = V A = \frac{1}{n} A \; R^{2/3} S^{1/2} \qquad \text{(Equation 36-12.2)}$$

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

For storm drains flowing full, the above equations become the following:

$$V = \frac{0.397}{n} D^{2/3} S^{1} \qquad Q = \frac{0.312}{n} D^{8/} \qquad \text{(Equation 36-12.3)}$$

Where: D = diameter of pipe, m

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

36-12.05 Minimum Grades

All storm drains should be designed such that velocities of flow will not be less than 0.8 m/s at design flow. For very flat grades, the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system should be checked to ensure there is sufficient velocity in all of the drains to deter settling of particles. Minimum slopes required for a velocity of 0.8 m/s can be calculated by the Manning's formula, or use values given in Figure 36-12E.

$$S = \frac{(nV)^2}{R^{4/3}} \qquad \text{(Equation 36-12.4)}$$

Desirably, the maximum velocity in the pipes will not exceed 2.5 m/s.

36-13.0 HYDRAULIC GRADE LINE

36-13.01 Introduction

The hydraulic grade line (HGL) is the last important feature to be established relating to the hydraulic design of storm drains. 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 when the system is operating from a flood of design frequency. INDOT policy is that the maximum HGL is at the top of the inlet and/or manhole for the Q_{50} flow.

In general, if the HGL is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the HGL is below the crown of the pipe, open channel flow calculations are appropriate. A special concern with storm drains designed to operate under pressure flow conditions is that inlet surcharging and possible manhole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made. Storm drain systems 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 this inplace system, then upstream through the proposed system to the upstream inlet. The same considerations apply to the outlet of a storm drain as to the outlet of a culvert. See Figure 31-5D for a sketch of a culvert outlet which depicts the difference between the HGL and the energy grade line (EGL). Usually it is helpful to compute the EGL first, then subtract the velocity head ($V^2/2g$) to obtain the HGL.

36-13.02 Tailwater

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

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

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

Note that when $V_d = 0$ as in a reservoir, the exit loss is one velocity head. At locations 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. The manufacturer should be consulted for information. For part full flow where the pipe outlets in 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:

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

Where: Δ = angle of curvature in degrees

36-13.05 Pipe Friction Losses

The friction slope is the energy gradient in m/m for that run. The friction loss is simply the energy gradient multiplied by the length of the run in meters. Energy losses from pipe friction may be determined by rewriting the Manning's equation with terms as previously defined.

$$S_f = [Qn/\ A\ R^{2/3}]^2 \qquad \text{(Equation 36-13.3)}$$

The head losses due to friction may be determined by the formula as follows:

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

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

$$H_f = 6.35\ n^2\ V^2\ L\ /\ D^{4/3} \qquad \text{(Equation 36-13.5)}$$

Where: H_f = total head loss due to friction, m

$$H_f = \left(\frac{19.62\ n^2\ L}{R^{4/3}}\right)\left(\frac{V^2}{2g}\right) \qquad \text{(Equation 36-13.6)}$$

n = Manning's roughness coefficient
D = diameter of pipe, m
L = length of pipe, m
V = mean velocity, m/s
R = hydraulic radius, m
g = 9.81 m/s^2
S_f = slope of hydraulic grade line, m/m

36-13.06 Manhole Losses

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

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

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.

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

Where: θ = the angle between the inflow and outflow pipes, degrees
b = manhole diameter, mm
D_o = outlet pipe diameter, mm

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

$$C_D = (D_0 / D_i)^3 \qquad \text{(Equation 36-13.9)}$$

Where: D_i = incoming pipe diameter, mm
D_o = outgoing pipe diameter, mm

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

$$C_d = 0.5(d / D_o)^{0.6} \qquad \text{(Equation 36-13.10)}$$

Where: d = water depth in manhole above outlet pipe, m
D_o = outlet pipe diameter, m

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 = the angle between the inflow and outflow pipes, degrees
Q_i = flow in the inflow pipe, m^3/s
Q_O = flow in the outlet pipe, m^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 cases to determine the impact of pipe 2 entering the access hole.

$C_{Q3\text{-}1} = (1 - 2 \sin 180)(1 - 0.09/0.12)^{0.75} + 1 = 1.35$

a. Case 1:

$Q_1 = 0.09$ m^3/s, $Q_2 = 0.03$ m^3/s,
$Q_3 = 0.12$ m^3/s, then $C_Q = 1.35$

b. Case 2:

$Q_1 = 0.03$ m^3/s, $Q_2 = 0.09$ m^3/s,
$Q_3 = 0.12$ m^3/s, then $C_Q = 1.81$

5. Plunging Flow. The correction factor for plunging flow, C_p, is calculated by the following:

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

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

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 when pipe No. 2 discharges plunging flow. The correction factor is only applied when $h > d$.

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

7. Summary. In 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 in this Chapter. The HYDRA computer program in the HYDRAIN system is recommended for design of storm drains and will include a HGL analysis and a pressure flow simulation. A step-by-step procedure is given to manually compute the HGL. Figure 36-13D can be used to document the procedure.

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

1. Enter in Col. 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 Col. 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 Col. 3 the diameter (D_O) of the outflow pipe.

4. Enter in Col. 4 the design discharge (Q_O) for the outflow pipe.

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

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

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

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

9. Enter in Col. 9 the friction slope (SF_O) in m/m of the outflow pipe. This can be determined by using Equation 36-13.3. Note: Assumes full flow conditions.

10. Enter in Col. 10 the friction loss (H_f) which is computed by multiplying the length (L_O) in Col. 5 by the friction slope (SF_O) in Col 9. On curved alignments, calculate curve losses by using the formula $H_b = 0.0033\ (\theta)(V_O^2/2g)$, where θ = angle of curvature in degrees, and add to the friction loss.

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

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

13. Enter in Col. 13 the correction factor for flow depth, C_d, as computed by Equation 36-13.10. Note this factor is only significant in cases where the d/D_O ratio is less than 3.2.

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

15. Enter in Col. 15 the correction factor for plunging flow, C_p, as computed by Equation 36-13.12. The correction factor is only applied when $h > d$.

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

17. Enter in Col. 17 the value of K as computed by Equation 36-13.7.

18. Enter in Col. 18 the value of the total manhole loss, $K\ V_O^2/2g$.

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

20. Enter in Col. 20 the sum of the friction head (Col. 10), the manhole losses (Col. 18), and the energy grade line (Col. 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 (Col. 21) throughout the system by subtracting the velocity head (Col. 7) from the EGL (Col. 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 pipes that are flowing full, as should be the condition for design of new systems. If a part full flow condition exists, the EGL is located one velocity head above the water surface.

Figure 36-13E presents 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 Chapter Fifty-two for INDOT's detailed 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. The next section presents an example problem using the HYDRA computer model.

If any other commercial package is used, it must be capable of computing the trunk line 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 the manufacturer's publications with capture rate information for sag and on-grade installations.

36-16.0 EXAMPLE PROBLEM

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. Drainage areas as indicated on Inlet Computation Sheet.

3.6-m travel lane @ 0.02; 0.6-m gutter @ 0.025
10-year design, IDF curve for Indianapolis
Allowable spread, T = 1.8 m   n = 0.016
$S_O = 0.012$ $S_X = 0.02$ $S_W = 0.025$
Curved vane grates (Type 10 and 11) (397 mm x 880 mm)

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

Solution:

1. Inlet Computations.

a. Hand calculations documented on Inlet Computation Sheet (Figure 36-16B).

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 on Storm Drain Computation Sheet (Figure 36-16C).

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 presents a hydraulic capacity chart for the curved vane grate, Grate and Frame Casting Type 10 and 11. See the INDOT *Standard Drawings*. The inlet capacity chart has been produced based on the following assumptions.

1. Grate Dimensions: 880 mm x 397 mm Curved Vane Grate
2. S_X = Roadway Slope = 0.02
3. S_W = Gutter Pan Slope = 0.025
4. W = Gutter Width = 0.6 m
5. Grate Length = 880 mm, Grate Width = 397 mm
6. n = 0.016
7. S = Longitudinal Slope = 0.5% - 7%
8. Q = Gutter Flow = 0.01 m^3/s to 0.24 m^3/s

The assumed roadway conditions for S_X, S_W and W are those that typically occur on curbed facilities in Indiana. 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 = 0.12 m^3/s

Figure 36-17A yields Q_i = 0.053 m^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. *Bridge Deck Drainage Guidelines*, FHWA Report No. RD-014, December 1986.

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

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

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

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

7. U.S. Department of Transportation, Federal Highway Administration, *Design of Urban Highway Drainage - The State of The Art*, FHWA-TS-79-225, 1979.