CHAPTER 14

STORM DRAIN SYSTEMS

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1.1   INTRODUCTION

Storm drains are usually used as part of a minor storm drainage system when the other parts of the minor system, primarily curb, gutter, and roadside ditches, no longer have capacity for additional runoff and meet the established minor and major storm drainage design requirements (see Section 2, Chapter 3). A storm drain system may consist of a series of pipes, manholes, and inlets, and is frequently used to convey storm runoff from streets (gutter flow) to open channels or detention facilities. Inlets to the storm drain are sized to reduce the amount of street (gutter) flows to a level where the downstream street (gutter) flow limit is not exceeded before the location of the next inlet. Manholes in the drain system are provided to allow access to the storm drain for inspection and maintenance.

Storm drains may be designed as open channels or pressure conduits. Capacities of open channel storm drains should be computed using the Manning's equation. Storm drains with pressure flows should be designed to withstand the forces of such pressure in accordance with the appropriate standards. The size of the storm drain system is generally governed by the minor storm flows. This is a result of the incremental flow capacity between the allowable street flow during major and minor

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storms being generally greater than the incremental difference in the peak runoff from major and minor storms. In addition, the storm drain system will naturally carry some runoff in excess of the required minor storm capacity during major storms due to natural surcharging of the storm drain system. The placement of storm drain inlets should be determined by a thorough analysis of the drainage area and streets involved. These inlets should be located where sump (low-spot) conditions exist or where allowable street capacities are exceeded.

The size of the storm drain system is generally designed to convey the minor storm flows. There are conditions, however, when the storm drain system design will be governed by the major storm flows. A partial listing of some of the possible situations are as follows:

- Locations where street flow is collected in a sump with no allowable overflow capacity.
- Locations where the street cross-section is such that the allowable depth of flow in the street is limited to the curb height (i.e. elevated streets with negative slopes at the ROW line).

- Locations where the designed major storm flow direction is not reflected by the street flow direction during a major storm (i.e. flow splits at intersections).
- Locations where the subject storm drain system is accepting flow from an upstream storm drain system or branch, which is designed for major storm capacity.
- Regional storm drains.

The storm drain system designer should be aware that if a storm drain is to be designed to carry major storm flows, then the inlets to the storm drain must be designed accordingly.

The storm drain system design criteria presented in this section is provided to establish the recommended minimum design guidelines. The CWCB encourages that the local entities without established storm drain design guidelines adopt the standards provided herein, wholly or in part, depending on the needs of the adopting agency. Each entity adopting the contents of the Statewide Manual is responsible for enforcement of the Manual within its jurisdictional boundaries.

1.2   STORM DRAIN HYDRAULIC DESIGN

New storm drains should be designed to convey the minor storm flows without surcharging the drain system. To ensure that this objective is achieved, the design system hydraulic grade line should be calculated by accounting for pipe friction losses and pipe form losses. Total hydraulic losses should include friction, expansion, contraction, bend, and junction losses. The recommended methods for estimating these losses are presented in the following sections.

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1.2.1   ALLOWABLE STORM DRAIN CAPACITY

The storm drain system should be designed to convey a part or all of the minor or major storm (design storms) under open channel or surcharged (pressure flow) conditions. The storm drain should be considered surcharged when the depth of flow (hydraulic grade line - HGL) in the storm drain is greater than eighty percent of full flow depth. The maximum level of surcharging for the capacity analysis should be limited to maintaining the HGL to one foot below the final grade above the storm drain at all locations. Special site conditions that warrant additional surcharging will require locking type manhole covers or grated covers and should be reviewed on a case-by-case basis.

The energy grade line (EGL) and HGL should be calculated to include all hydraulic losses including friction, expansion, contraction, bend, and junction losses. The methods for estimating these losses and for calculating the EGL and HGL are presented in the following sections.

1.2.2 ALLOWABLE STORM DRAIN VELOCITY

The maximum allowable storm drain velocity is dependent on many factors, including the type of pipe, the acceptable water level during the pipe design life, proposed flow conditions (open channel versus pressure flows), and the type and quality of construction of joints, manholes, and junctions. In consideration of the above factors, the maximum velocity in all storm drains should be limited to 20 fps.

> *The maximum velocity in all storm drains should be limited to 20 ft/s. A generally accepted criteria is to maintain a minimum velocity of 3 fps at half or full conduit flow conditions.*

The need to maintain a self-cleaning storm drain system is recognized as a goal to minimize the costs for maintenance of storm drain facilities. Sediment deposits, once established, are generally difficult to remove without pressure cleaning equipment. However, the infrequency of storm runoff also possesses a problem in obtaining flows large enough to maintain the self-cleaning quality of the design. Thus, a balance must be drawn between obtaining a self-cleaning system and constructing a reasonably sized and sloped storm drain.

A generally accepted criteria is to maintain a minimum velocity of 3 fps at half or full conduit flow conditions. At half full, the storm drain will flow under open channel flow conditions and thus, the velocity in a given storm drain is governed by the pipe slope. However, storm drains generally cannot be constructed at slopes less than 0.25 percent and maintain a smooth even invert. Therefore, the minimum allowable storm drain slope should be 0.25 percent for pipe diameters of 18 inches or greater and 0.32 percent for 15-inch diameter pipes.

1.2.3 MANNING'S ROUGHNESS COEFFICIENT

All storm drain system hydraulic calculations should be performed using Manning's Formula (see Equation CH14-101). Manning's roughness factor or "n" value is determined based on the surface roughness of the storm drain pipe material. In addition, for a given pipe material, Manning's roughness coefficient theoretically varies based on depth of flow in the pipe. For the purposes of this Manual, Manning's roughness coefficient is assumed to be constant for all depths of pipe flow.

$$Q = \frac{1.49}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}} \qquad \text{(Eq CH14-101)}$$

Various pipe manufacturers have determined Manning's roughness coefficients for use with their specific product. However, for storm drain hydraulic design, Manning's roughness coefficient should also account for additional friction losses from pipe joints, potential debris and sediment in the storm runoff, and the pipe interior surface condition over the entire design life of the pipe. Therefore, presented on Table CH14-T101 are the Manning's

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roughness coefficients to be used for all storm drain designs and analysis prepared in accordance with this Manual.

1.2.4 PARTIAL FULL FLOW ANALYSIS

When a storm drain is not flowing full, the drain acts like an open channel and the hydraulic properties can be calculated using open channel techniques (refer to Chapter 13, Section 1). For convenience, charts for various culvert shapes have been developed by the pipe manufacturers for calculating the hydraulic properties associated with partial full flow (Figures CH14-F101, CH14-F102, and CH14-F103). The data presented assumes that the friction coefficient, Manning's roughness coefficient, does not vary throughout the depth.

For partial full flow analysis, the flow is assumed to be uniform, and the HGL and EGL are assumed to be parallel. The designer should check the available energy at all junctions and transitions to determine whether or not the flow in the storm drain will be pressurized due to backwater effects even if the design flow is less than the full flow capacity of the storm drain. In this case, a hydraulic jump will occur, and the pipe should be structurally designed to accommodate the jump. The storm drain upstream of the jump should then be analyzed as a pressure flow system.

1.2.5 PRESSURE FLOW ANALYSIS

New storm drain systems should be designed without surcharging the pipe, but frequently, situations are encountered where full pipe flow conditions occur. These situations might occur when tying into existing storm drains that are surcharged, outleting into detention basins, etc. When a storm drain is flowing under a pressure flow condition, the energy and hydraulic grade lines may be calculated using the pressure-momentum theory. The capacity calculations generally proceed from the storm drain outlet upstream accounting for all energy losses. These losses are added to the EGL and accumulate to the upstream end of the storm drain. The HGL is then determined by subtracting the velocity head, H_v, from the EGL at each change in the EGL slope.

1.2.6 ENERGY LOSS CALCULATIONS

Presented in this section are the energy loss equations and coefficients for use in the hydraulic analysis of storm drain systems. All storm drain system analyses should account for energy losses using the equations and coefficients provided in this section

1.2.6.1 PIPE FRICTION LOSSES

Pipe friction losses should be calculated using an equation for full flow conditions derived from Manning's equation as follows:

$$S_f = \frac{\Phi \, H_v}{R^{1.33}} \qquad \text{(Eq CH14-102)}$$

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Where S_f = Friction slope (feet/feet)
H_v = Velocity head (feet)
R = Hydraulic radius (feet)

The flow coefficient, Φ, is related to the Manning's "n" value for the pipe as follows:

$$\Phi = \frac{2gn^2}{2.21}$$ (Eq CH14-103)

Where n = Manning's roughness coefficient (Dimensionless)
g = Gravitational acceleration 32.2 ft/sec²

The total head loss due to friction in a length of pipe is then equal to the friction slope times the pipe length.

$$H_f = S_f \ x \ L$$ (Eq CH14-104)

Where L = Pipe Length, feet

1.2.6.2 PIPE FORM LOSSES

Generally, between the inlet and outlet, the flow encounters a variety of configurations in the flow passageway such as changes in pipe size, branches, bends, junctions, expansions, and contractions. These shape variations impose losses in addition to those resulting from pipe friction. Form losses are the result of fully developed turbulence and can be expressed as follows:

$$H_L = K\left(\frac{V^2}{2g}\right)$$ (Eq CH14-105)

where H_L = head loss (feet)
K = loss coefficient
$\frac{V^2}{2g}$ = velocity head (feet)
g = gravitational acceleration (32.2 ft/sec²)

The following is a discussion of a few of the common types of transition losses encountered in storm drain system design.

a) Expansion Losses

Expansion in a storm drain conduit will result in a shearing action between the incoming high velocity jet and the surrounding drain boundary. As a result, eddy currents and turbulence dissipate much of the kinetic energy. The loss of head can be expressed as:

Non-Pressure   **Under Pressure**

$$H_L = K_e\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \quad \text{or} \quad H_L = K_e\left(\frac{V_1^2}{2g}\right) \qquad \text{(Eq CH14-106)}$$

Where V is the average flow velocity, and K_e is the loss coefficient. Subscripts 1 and 2 denote the upstream and downstream sections, respectively. The value of K_e is about 1.0 for a sudden expansion, and about 0.2 for a well-designed expansion transition. Table CH14-T104 presents the expansion loss coefficients for various flow conditions.

b) Contraction Losses

The form loss due to contraction is expressed as:

Non-Pressure   **Under Pressure**

$$H_L = K_c\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \quad \text{or} \quad H_L = K_c\left(\frac{V_2^2}{2g}\right) \qquad \text{(Eq CH14-107)}$$

Where K_c is the contraction coefficient. K_c is equal to 0.5 for a sudden contraction and about 0.1 for a well-designed transition. Subscripts 1 and 2 denote the upstream and downstream sections, respectively. Table CH14-T104 presents the contraction loss coefficient for various flow conditions.

c) Bend Losses

The head losses for bends, in excess of that caused by an equivalent length of straight pipe, may be expressed as:

$$H_L = K_b\left(\frac{V^2}{2g}\right) \qquad \text{(Eq CH14-108)}$$

In which K_b is the bend coefficient. The bend coefficient has been found to be a function of, (a) the ratio of the radius of curvature of the bend to the width of the conduit, (b) deflection angle of the conduit, (c) geometry of the cross section of flow, and (d) the Reynolds number and relative roughness. A table showing the recommended bend loss coefficients is presented in Table CH14-T104.

d) Junction and Manhole Losses

A junction occurs where one or more branch drains enter the main drain, usually at manholes. The hydraulic design of a junction is in effect the design of two or more transitions, one for each flow path. Allowances should be made for head loss due to

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the impact at junctions. The head loss at a junction can be calculated from:

$$H_L = \frac{V_2^2}{2g} - K_j \frac{V_1^2}{2g}$$ (Eq CH14-109)

Where V_2 is the outfall flow velocity and V_1 is the inlet velocity. The loss coefficient, K_j, for various junctions is presented in Table CH14-T104.

For straight flow through manholes (single pipe with no inlet laterals), the head loss through the manhole is similar to a pipe bend. For this condition, the head loss at the manhole is expressed as:

$$H_L = K_m \left(\frac{V_2^2}{2g} \right)$$ (Eq CH14-110)

In which K_m is the manhole loss coefficient. Figure CH14-F104 presents values of K_m for various deflection angles.

1.2.6.3 STORM DRAIN OUTLET LOSSES

When a storm drain system discharges into a Major Drainageway System (usually an open channel), additional losses occur at the outlet in the form of expansion losses. For most storm drain outlets, the flow velocity in the storm drain is greater than the allowable or actual flow velocity in the downstream channel. Therefore, energy dissipation facilities are used to remove excess energy from the storm drain flow. In addition, the alignment of the storm drain at the outlet may not be the same as the downstream channel. Therefore, energy is lost in changing the flow direction between the storm drain to the downstream channel. For a headwall and no wingwalls, the loss coefficient K_o = 1.0, and for a flared-end section the loss coefficient is approximately 0.5 or less. The head loss at storm drain outlets is expressed as:

$$H_L = K_o \left(\frac{V_1^2}{2g} \right)$$ (Eq CH14-111)

Where K_o is the outlet loss coefficient.

The outlet loss should be added to the downstream EGL and compared to the critical depth EGL in the storm drain. The larger (higher) EGL should be used for starting the storm drain hydraulic calculations.

1.2.6.4 INLET LOSSES

When runoff enters a storm drain system from locations other than street inlets (i.e. open channels), an energy loss occurs at the

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entrance in the form of a contraction loss. The head loss at storm drain entrances is expressed as:

$$H_L = K_i \left(\frac{V_2^2}{2g} \right)$$ (Eq CH14-112)

In which K_i is the inlet (entrance) loss coefficient. The coefficient K_i is the same as the K_e coefficient used for the entrance loss calculation for culverts. A list of various K_i (K_e) coefficients is presented in Table CH14-T104.

1.3   STORM DRAIN PIPE

1.3.1   PIPE MATERIALS

A soils report listing the minimum resistivity, pH, sulfate content and chloride content of the soil and groundwater should be submitted to the local jurisdiction. The tests can either be performed at the same time as other routine soil tests for a development or along the pipe location once it is known.   Further tests may be recommended if the initial tests are inconclusive.   If there is a reason to believe the storm water flowing through the pipe is corrosive, an analysis of the storm water should also be submitted.

Only Reinforced Concrete Pipe (RCP) and Horizontal Elliptical Reinforced Concrete Pipe (HERCP) are acceptable beneath the street right-of-ways. In addition to RCP and HERCP, Polyvinyl Chloride Pipe (PVC), smooth wall High Density Polyethylene Pipe (HDPE) and Non-Reinforced Concrete Pipe (NRCP) are acceptable materials outside of the street right-of-way.

Only Reinforced Concrete Pipe (RCP) and Horizontal Elliptical Reinforced Concrete Pipe (HERCP) are acceptable beneath the street right-of-ways. In addition to RCP and HERCP, Polyvinyl Chloride Pipe (PVC), smooth wall High Density Polyethylene Pipe (HDPE) and Non-Reinforced Concrete Pipe (NRCP) are acceptable materials outside of the street right-of-way. The local jurisdiction will grant the final approval of the pipe material based on the results of the soil and water analysis. Corrugated Metal Pipe (CMP) and Aluminized Steel Pipe (ASP) are not permitted for storm drain construction. Table CH14-T101 summarizes a comparison of the various approved pipe materials.

1.3.2   PIPE SIZE AND STRENGTH

The minimum allowable pipe diameter should be 18 inches for main trunks, and 15" for laterals. The minimum inside dimension should be no less than 15 inches for elliptical and arch pipes. The conduit should be of sufficient structural strength to withstand the AASHTO HS-20-44 loading. Elliptical and arch pipe cannot be used for pressure flow.

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1.3.3 JOINT SEALANTS AND GASKETS

Pipe joints for concrete pipe are generally sealed with either joint sealants or gaskets. Joint sealants are generally mastics, which consist of bitumen and inert mineral fillers or joint mortar. The mastic is easily applied in the field but may not always provide a watertight joint. Joint gaskets are generally made of rubber and are either cemented to, recessed in, or rolled on the pipe joint. These gaskets generally provide watertight seal and can withstand some internal pressure. Since all storm drains should be generally designed for pressure flow conditions, rubber gasket joints should be used for all installations where the pressure head exceeds 5 feet for the design flow. The pressure head is computed as the difference between the hydraulic grade line and the inside top of pipe.

1.3.4 STORM DRAIN OUTLETS

Storm drain outlets should be constructed with outlet erosion protection for discharges to channels with unlined bottoms in accordance with the following:

Outlet Velocity (fps)	Required Outlet Protection
less than 5	Riprap Protection
between 5 and 15	Riprap Protection or Energy Dissipater
greater than 15	Energy Dissipater

For channels with lined bottoms, the outlet discharge velocity must not exceed the maximum allowable channel velocity without an energy dissipation structure. Refer to Chapter 13 of the Statewide Manual for detailed discussions on the design standards for the erosion protection measures.

1.3.5 GRATES FOR PIPES

Where a present danger exists such as with a siphon, a drop in elevation adjacent to a sidewalk or road, a long pipe with one or more manholes, or at pipes which are near playgrounds, parks and residential areas, a grate may be required. For most culverts through embankments and crossing streets, grates will not be required. The grate open area must be at least 4 times the open area of the pipe. The design engineer should coordinate with the local jurisdiction in deciding the needs for grates.

Grates should meet the following minimum requirements:

- Grates should be constructed of steel bars with a minimum diameter of 5/8-inch. Reinforcing bars should not be used.
- Welded connections should be ¼-inch minimum.
- Spacing between bars should normally be 5-inch unless site conditions are prohibitive.

- All exposed steel should be galvanized in accordance with AASHTO M 111.
- Welded joints should be galvanized with a rust preventive paint.
- Grates should be secured to the headwall or end section by removable devices such as bolts or hinges to allow maintenance access, prevent vandalism, and prohibit entrance by children.

1.4   STORM DRAIN VERTICAL ALIGNMENT

The storm drain grade should be such that a minimum cover is maintained to withstand AASHTO HS-20 loading on the pipe. The minimum cover depends upon the pipe size, type and class, and soil bedding condition, but should be not less than 1.5-foot at any point along the pipe. For instance, a storm drain passing perpendicular and beneath a road should have a minimum depth of cover of 1.5 feet measured from the lowest point on the street cross-section (usually the gutter flowline) to the outside of the bell of the pipe. The maximum cover is contingent upon the design pipe strength.

If alignment conflicts arise between storm drains and water mains, water mains are usually relocated. Whenever possible, storm drains should be installed 18 inches or more below water mains. When storm drains cross over or within 18 inches below water mains (but never less than 6 inches of clear separation), they should be constructed of structural drain pipe, a 20-foot section, centered over or under the water main and the connecting joints encased in concrete. If ductile iron pipe or other structural drainpipe is not used, the storm drainpipe should have all joints encased for the 20-foot section. If a storm drain is laid between 5 feet and 10 feet horizontally from a water main, these requirements should also apply. The minimum horizontal distance between a storm drain and a water main is 10 feet.

Encasements should consist of a reinforced concrete collar 6 inches thick and extended 12 inches on either side of the joint. The minimum reinforcement should be #4 bars, continuous, placed at each corner of the section tied with #3 bars at 3-foot centers.

The minimum clearance between storm drains and other utilities should be 18 inches unless one of the pipes is encased, which could reduce the clearance to 6 inches. The minimum horizontal distance between a storm drain and another utility should be 5 feet.

Ditch crossings should be constructed in accordance with the Ditch Crossing Detail Figure CH14-F105. Written approval from the irrigation company must be obtained prior to the local jurisdiction approval for crossing any irrigation ditch/canal.

In all cases, suitable backfill, compaction, and other protections, as deemed necessary by the local jurisdiction, should be provided to prohibit settling or failure of pipe system. If the storm drainpipe is below the ground water table, a clay barrier or concrete cutoff wall must be constructed every 200 feet to prevent ground water migration through the backfill material.

Table CH14-T105 is a summary of the utility crossing separation requirements.

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1.5 STORM DRAIN HORIZONTAL ALIGNMENT

Storm drain alignment between manholes should be straight. Manholes or bends should be used whenever storm drains require a change in direction. Storm drains should not be constructed with a curvilinear alignment.

1.6 STORM DRAIN MANHOLES

Manholes or maintenance access ports should be required whenever there is a change in size, direction, elevation, grade, or where there is a junction of two or more drains. The maximum spacing between manholes for various pipe sizes should be in accordance with Table CH14-T106. The required manhole size should be in accordance with Table CH14-T106.

Larger manhole diameters or a junction structure may be required when drain alignments are not straight through or more than one drain line goes through the manhole.

1.7 STORM DRAIN INLETS

Presented in this section is the criteria and methodology for design and evaluation of storm drain inlets. There are three types of inlets: curb opening, grated, and combination inlets. Inlets are further classified as being on a "continuous grade" or in a "sump". The term "continuous grade" refers to an inlet located so that the grade of the street has a continuous slope past the inlet and, therefore, ponding does not occur at the inlet. The "sump" condition exists whenever water ponds because the inlet is located at a low point. A sump condition can occur at a change in grade of the street from positive to negative, or at an intersection due to the crown of a cross street.

There are three types of inlets: curb opening, grated, and combination inlets. Inlets are further classified as being on a "continuous grade" or in a "sump".

The standard inlets and permitted use should be as follows:

- Curb Opening Inlet Type R (Standard Detail CH14-SD101). Permitted use in all street types with 6-inch vertical curb.
- Grated Inlet Type C (Standard Detail CH14-SD102). Permitted use in all street types with a roadside or median ditch.
- Grated Inlet Type 13 (Standard Detail CH14-SD103). Permitted use in alleys or drives with a valley gutter.
- Combination Inlet Type 13 (Standard Detail CH14-SD104). Permitted use in all street types with 6-inch vertical curb.

1.7.1 INLET HYDRAULICS

The procedures and basic data used to define the capacities of the standard inlets under various flow conditions were obtained from IZZARD, 1977 and LINSLEY 1964. The procedures consist of defining the amount and depth of flow in the gutter and determining the theoretical flow interception by the inlet. To account for effects that decrease the capacity of the various types of

inlets, such as debris plugging, pavement overlaying, and variations in design assumptions, the theoretical capacity calculated for the inlets is reduced to the allowed capacity by the factors presented in Table CH14-T107.

Allowable inlet capacities for the standard inlets have been developed and are presented in Figures CH14-F106 and CH14-F107 for "continuous grade" and Figure CH14-F108 for sump conditions. These figures include the reduction factors in Table CH14-T107. The allowable inlet capacity is dependent on the depth of flow (for continuous grade inlets) as determined from the street capacity calculations (refer to Section 2, Chapter 14) or on the depth of ponding (sump conditions) necessary to accept the desired flow rate. The values shown were calculated on the basis of the <u>maximum</u> flow allowed in the street gutter (or roadside ditch for Type C). For gutter flow amounts less than the maximum, the allowable inlet capacity should be proportionately reduced.

1.7.1.1 <u>CONTINUOUS GRADE INLET CONDITION</u>

For the "continuous grade" condition, the capacity of the inlet is dependent upon many factors including gutter slope, depth of flow in the gutter, height and length of the curb opening, street cross slope, and the amount of depression at the inlet. In addition, all of the gutter flow will not be intercepted, and some flow will continue past the inlet area ("inlet carryover"). The amount of carryover should be included in the drainage facility evaluation as well as in the design of the inlet.

1.7.1.2 <u>SUMP INLET CONDITION</u>

The capacity of an inlet in a sump condition is dependent on the depth of ponding above the inlet. Typically, the problem consists of determining the quantity or length of inlets required to reduce the depth of ponding to an acceptable level. The designer should be aware that several inlets or additional inlet length would generally be required when an inlet must be designed to accommodate major storm flows. Also, additional continuous grade inlets may be necessary upstream of the sump location to reduce the depth of ponding at the sump inlets to an acceptable level during major storm events. At all sump locations, the design should include provisions for emergency overflow if the sump inlets become completely plugged.

1.7.2 <u>INLET SPACING</u>

The optimum spacing of storm inlets is dependent upon several factors including traffic requirements, contributing land use, street slope, and distance to the nearest outfall system. The suggested sizing and spacing of the inlets is based upon the interception rate of 70% to 80%. This spacing has been found to be more efficient than a spacing using 100% interception rate. Using the suggested spacing only, the most downstream inlet in a development would be designed to intercept 100% of the flow. Also,

considerable improvement in over-all inlet system efficiency can be achieved if the inlets are located in the sumps created by street intersections.

Inlets shall be installed at low points of vertical curves, at street intersection sumps, and at sufficient intervals to intake the design peak flow so that said flows will not interfere with traffic or flood adjoining property.

1.8   STORM DRAIN SYSTEM DESIGN

Presented in this section are the recommended design procedures for a typical storm drain system. A typical drainage system within a development consists of flow in the storm drain and allowable flow in the gutter, which combined would carry both the minor and major storm flows. The design flow for the storm drain is generally governed by the amount of runoff in excess of the minor storm street capacity limit. In some cases,

> *A typical drainage system within a development consists of flow in the storm drain and allowable flow in the gutter, which combined would carry both the minor and major storm flows.*

however, the amount of runoff from the major storm in excess of the major storm street capacity limit may be larger than the excess from the minor storm. In this case, the storm drain and inlets would need to be designed to accommodate the excess major storm flows. To assist in this analysis, the allowable minor and major storm street capacity should be determined prior to sizing of the storm drain system (See Section 2, Chapter 14).

1.8.1   INITIAL STORM DRAIN SIZING

Preliminary street grades and cross-sections should be available to the storm drain designer so that the allowable carrying capacity for the streets can be computed. Beginning at the upper end of the basin in question, the designer should calculate the quantity of flow (minor and major storms) in the street until the point is reached at which the allowable carrying capacity of the street matches the design runoff. Initiation of the storm drain system would start at this point if there were no alternate method of removing runoff from the street surface. Removal of all the street flow by the storm drain system is not required except at sump areas. However, the sum of the flow in the drain plus the flow in the street must be less than or equal to the allowable capacity of the street and storm drain.

For preliminary sizing purposes, the diameter, type of pipe, and pipe slope may be determined assuming a full flow pipe capacity based on slope area calculations. If large energy losses are anticipated (i.e. large junctions, bends), then the preliminary pipe size may need to be upsized to assure that the final hydraulic calculations result in an acceptable HGL and EGL. In some instances, a profile may be required to check utility conflicts or to assure compatibility with the downstream drainage system.

The preliminary system should be reviewed to check that the system is hydraulically efficient as well as to locate segments that have potentially large energy losses. These segments should be examined carefully and options explored to minimize the energy loss. The designer should also check potential inlet locations to assure that the required inlet capacity is not larger then the allowable inlet capacities.

1.8.2 FINAL STORM DRAIN SIZING

Final design consists of the preparation of plan, profiles, and specifications for the storm drain system in sufficient detail for construction. The first step consists of the review and verification of the basic data, hydrologic analysis, and storm drain inlet sizing performed for the preliminary design. Plan and profile drawings are prepared containing the basic data. Drainage sub-basins are revised as necessary, and the design flood peaks recalculated. The storm drain and inlets are then sized taking into account actual street and storm drain grades, locations of existing and proposed utilities, and the design of the downstream drainage system. The calculations also include the determination of the hydraulic and energy grade lines. The manholes, junction structures, or other appurtenant structures should be evaluated for energy losses. If special transitions are required to reduce losses, the structural design of the facilities should include these requirements when detailing the structures.

1.9 EXAMPLE PROBLEM

The following example presents the hydraulic analysis of a storm drain system and demonstrates the use of the energy loss coefficients and the Hydraulic Calculations Figure CH14-F109.

The following procedure is based on full-flow pipe conditions. The storm drain empties into a detention basin, and the water surface elevation in the detention causes a backwater condition through the storm drain system. If the pipe is flowing substantially full (i.e., greater than 80 percent), the following procedures can be used with minimal loss of accuracy. However, the designer is responsible for checking the assumptions (i.e., check for full flow) to assure that the calculations are correct.

Problem: Compute the Energy Grade Line (EGL) and Hydraulic Grade Line (HGL) for Rose Subdivision shown in Figure CH14-F110. This example problem utilizes allowable street flow calculations performed in Section 2.9, Chapter 14 and runoff calculations performed in Section 5.6.1, Chapter 9. Assume the water surface elevation at the outlet in the detention basin (Point 7) is 4922.0 feet.

Solution:

Step 1:   Based on the allowable street flow calculations performed in the example problem in Section 2.9, Chapter 14, draw a plan view of the necessary storm drain system (See Figure CH14-F111).

Step 2:   Determine the location that the calculations will begin and the direction in which they will proceed. In this example, assume the normal depth at the storm drain outlet is greater than the critical depth ($d_n > d_c$), so the calculations will begin at Point 7 and proceed upstream.

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Step 3: Enter the known data into Standard Form 6 (See Figure CH14-F109). In this example, the assumed known data is input in columns 1, 2, 6, 10, and 27 and the first row of column 4.

Step 4: Assume, a storm drain type and diameter for the first reach of the storm drain system and fill in the first row of columns 3,8, 11, and 12.

Assume D_{7-4} = 1.5 feet
The storm drain velocity is:

$$V_{7-4} = \frac{Q}{A} = \frac{7.7}{\pi \times \left(\frac{1.5}{2}\right)^2} = 4.4 \text{fps}$$

and the velocity head is:

$$H_{v_{7-4}} = \frac{V^2}{2g} = \frac{4.4^2}{2 \times 32.2} = 0.3 \, feet$$

Step 5: Determine the starting HGL and EGL elevations.

As previously mentioned, the starting HGL is:

HGL_7 = 4922.0

The energy, or head, loss at the storm drain outlet is:

H_{LO} = K_o x $H_{v_{7-4}}$ = 1 x 0.3 = 0.3 feet

The initial EGL will be:

EGL_7 = HGL_7 + H_{LO} = 4922.0 + 0.3 = 4922.3 feet

Input the starting HGL (HGL_7) and EGL (EGL_7) above the first row of columns 24 and 25.

Step 6: Assume a value for the upstream invert elevation of the first storm drain reach, and fill the first row of columns 5 and 7.

Assume the storm drain invert elevation at Design Point 4 is 4619.0 feet. The slope in the first reach will be:

$$S_{7-4} = \frac{4919 - 4918}{100} = 0.01 \text{feet} / \text{feet}$$

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Step 7:   Calculate the friction slope for this reach.

The flow coefficient is:

$$\Phi = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

and the hydraulic radius is:

$$R_{7-4} = \frac{D_{7-4}}{4} = \frac{1.5}{4} = 0.375 \text{feet}$$

The friction slope is:

$$S_{f_{7-4}} = \frac{\Phi H_v}{R^{1.33}} = \frac{0.0049 \times 0.3}{0.387^{1.33}} = 0.005 \text{feet/feet}$$

Enter the flow coefficient, hydraulic radius, and the friction slope into the first row of columns 9, and 13, respectively.

Step 8:   Compute the average friction slope and input this value into column 14 of the first row.

The average friction slope is the average value of S_f for the current reach. When analyzing losses across long transitions, the average friction slope is the S_f for the upstream reach and the preceding reach averaged together.

$$\text{Ave } S_f = S_{f7-4} = 0.005 \text{feet/feet}$$

Step 9:   Calculate the energy loss due to pipe friction in the first reach.

$$H_{f_{7-4}} = (S_{f_{7-4}}) (L) = 0.005 \times 100 = 0.5 \text{ feet}$$

Enter $H_{f_{7-4}}$ in the first row of column 15.

Step 10:   Determine the EGL and HGL at the upstream station.

$$EGL_4 = EGL_7 + H_{f_{7-4}} = 4922.3 + 0.5 = 4922.8 \text{ feet}$$

$$EGL_4 = EGL_4 - H_v = 4922.8 - 0.3 = 4922.5 \text{ feet}$$

Enter EGL_4 in the first row of column 22 and HGL_4 in the first row of column 23.

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Step 11:   Check that full flow still exists (i.e., WSEL > 0.8D)

Flow Depth = HGL_4 - Invert elevation at Design Point 4

Flow Depth = 4922.5 - 4919.0 = 3.5 feet

0.8D = 0.8 x 1.5 = 1.2 feet

Since 3.5 > 1.2, pressure flow exists. Enter "yes" in the first row of column 26.

Step 12:   Assume a storm drain type and diameter from Design Point 4 to Design Point 6 (Reach 2) and a storm drain invert elevation at Design Point 6, and fill in the second row of columns 3, 4, 5, 7, 8, 9, 11, and 12.

Assume D_{4-6} = 1.5 feet

Upstream Invert Elevation = 4919.4

$$A_{4-6} = \Pi r^2 = \Pi(0.75)^2 = 1.77 ft^2$$

$$\varphi = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$$V_{4-6} = \frac{Q}{A} = \frac{2.5}{1.77} = 1.4 fps$$

$$H_{v_{4-6}} = \frac{V^2}{2g} = \frac{1.4^2}{2 * 32.2} = 0.1 feet$$

Step 13:   Check the controlling downstream flow condition for Reach 2. Compare the downstream flow condition for Reach 2 to the upstream flow condition for Reach 1. The highest value controls.

EGL_4 (Downstream of Reach 2) = D/S Invert $Elev_4$ +D+H_v

EGL_4 (Downstream of Reach 2) = 4919.0+1.5+0.1 = 4920.6 feet

EGL_4 (Upstream of Reach 1) = 4922.8 feet

Since EGL_4 (U/S of Reach 1) is greater than EGL_4 (D/S of Reach 2), the controlling downstream energy grade line elevation is 4922.8 feet. If the downstream EGL of Reach 2 had been greater, this value would be the controlling EGL (and HGL) and entered in columns 24

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and 25, respectively. Step 11 would be repeated in the next row down and the calculations would continue in this row with Step 13.

Step 14: Calculate the friction slope for this reach. The flow coefficient does not change.

The hydraulic radius is:

$$R_{4-6} = \frac{D_{4-6}}{4} = \frac{1.5}{4} = 0.375 \text{feet}$$

The friction slope is:

$$S_{f_{4-6}} = \frac{\Phi H_v}{R^{1.33}} = \frac{0.0049 \times 0.1}{(0.375)^{1.33}} = 0.002 \text{feet/feet}$$

Enter the hydraulic radius and the friction slope into the second row of column 14 and 13, respectively.

Step 15: Since the transition is relatively abrupt, the average friction slope is equal to the friction slope of the upstream reach. Input this value into the second row of column 14.

Step 16: Determine the head loss due to friction in the storm drain in Reach 2.

$$H_{f_{4-6}} = (\text{Ave } S_f)(L) = (0.002)(40) = 0.1 \text{ feet}$$

Enter this value in the second row of column 15.

Step 17: Calculate transition energy losses. In this case, there is a transition loss due to the junction.

Assume Reach 2 enters the junction at Design Point 4 at a 45° skew to the main storm drain alignment. From Table CH14-T104, the loss coefficient will be:

K_j=0.5

and the transition loss at the junction will be:

$$H_j = \frac{V_2^2}{2g} - K_j\left(\frac{V_1^2}{2g}\right) = 0.3 - (0.5)(0.1) = 0.3 \text{feet}$$

Enter this value in the second row of column 17.

For columns 17 through 20, enter the K values acquired from the appropriate tables and figures and the head values calculated from

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Equations 105 through 112. Separate the loss coefficient, K, and the head value, H, by a slash (/).

Step 18: Calculate the total energy loss and input this value into the second row of column 21.

$$H_{total} = H_{f_{4-6}} + H_j = 0.1 + 0.3 = 0.4 \text{ feet}$$

Step 19: Compute the EGL and the HGL at the upstream station (Design Point 6).

$$EGL_6 = EGL_4 + H_{total} = 4922.8 + 0.4 = 4923.2 \text{ feet}$$

$$HGL_6 = EGL_6 - H_v = 4923.2 - 0.1 = 4923.1 \text{ feet}$$

Enter EGL_6 and HGL_6 in the second row of columns 24 and 25, respectively.

Step 20: Check that full flow still exists.

$$\text{Flow Depth} = HGL_6 - \text{Invert Elevation at DP6}$$

$$\text{Flow Depth} = 4923.1 - 4919.4 = 3.7 \text{ feet}$$

Since the flow depth is greater than 0.8 x D (3.7 > 1.2), pressure flow exists, and "Yes" should be entered in the second row of column 27.

Step 21: Repeat Steps 11 through 19, as needed, to obtain the EGL and HGL for the entire storm drain system. Figure CH14-F109 supplies the results of this analysis, and the final EGL and HGL are plotted on Figure CH14-F110.

Note: The flow velocity in Reaches 2 and 3 under full flow conditions is less than 3 fps. Due to the small amount of flow needed to be carried in these reaches of storm drain, a low velocity is unavoidable. If the storm drain flow were not being controlled by the backwater conditions created by the detention basin, the flow velocities in Reaches 2 and 3 would be 4.9 fps, and 5.7 fps, respectively. These velocities should be sufficient to clean the storm drain of sediment and debris.

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SECTION 1
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HYDRAULIC PROPERTIES
CIRCULAR PIPE

Chart of Hydraulic Properties for Circular Pipe, plotting Depth of Flow (vertical axis, 0 to 1.0) against Proportion of Value for Full Flow (horizontal axis, 0 to 1.3), with curves for Area of Flow, Flow, Hydraulic Radius, and Velocity.

O:/2120/FIGURES/CH14-F101 – 1/6/06 – GPB

HYDRAULIC PROPERTIES
HORIZONTAL ELLIPTICAL PIPE
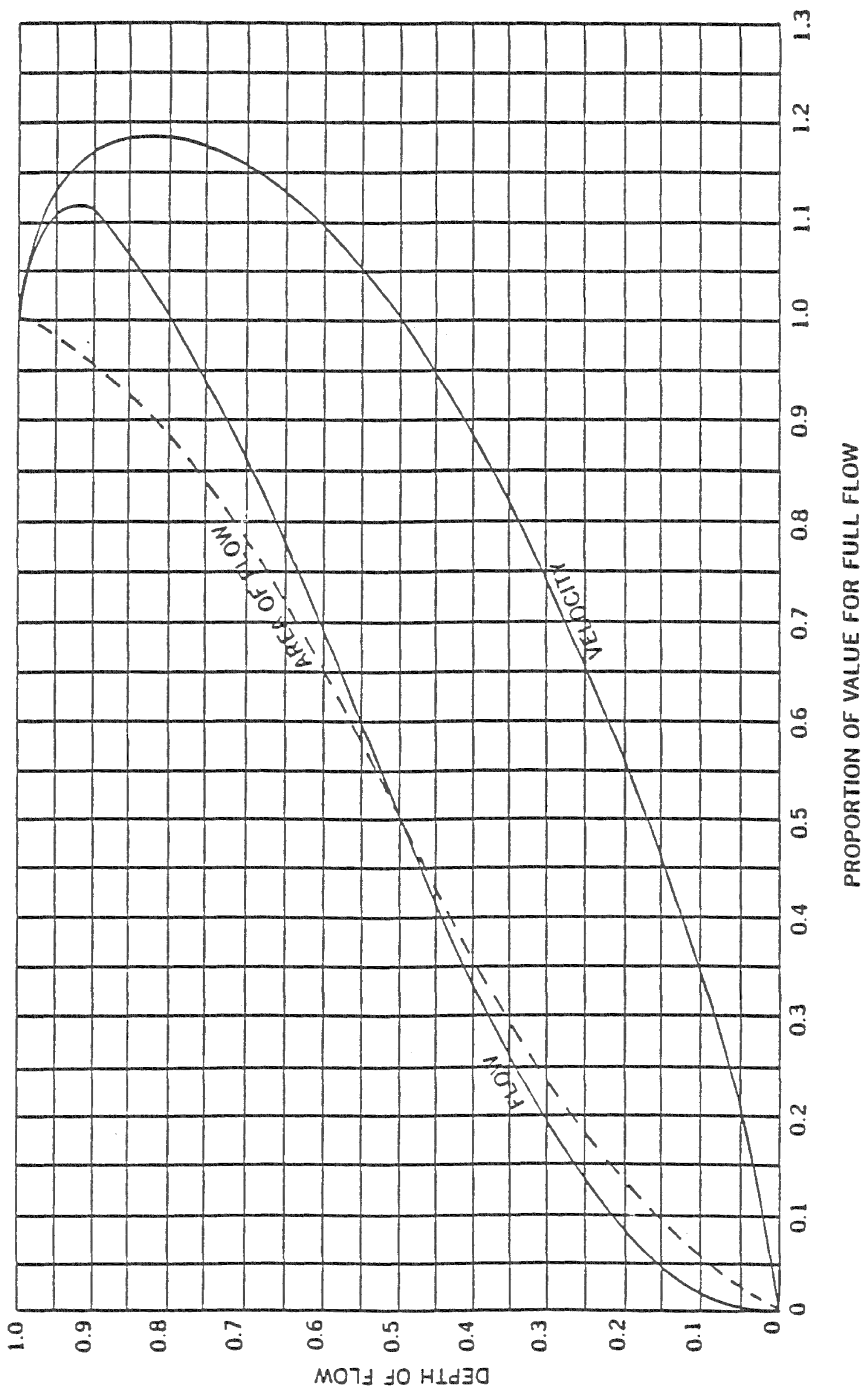
O:/2120/FIGURES/CH14-F102 – 1/6/06 – GPB

VERSION: JANUARY 2006

REFERENCE:
"CONCRETE PIPE DESIGN MANUAL,"
ACPA, 1970

FIGURE CH14-F102
HYDRAULIC PROPERTIES
HORIZONTAL ELLIPTICAL PIPE
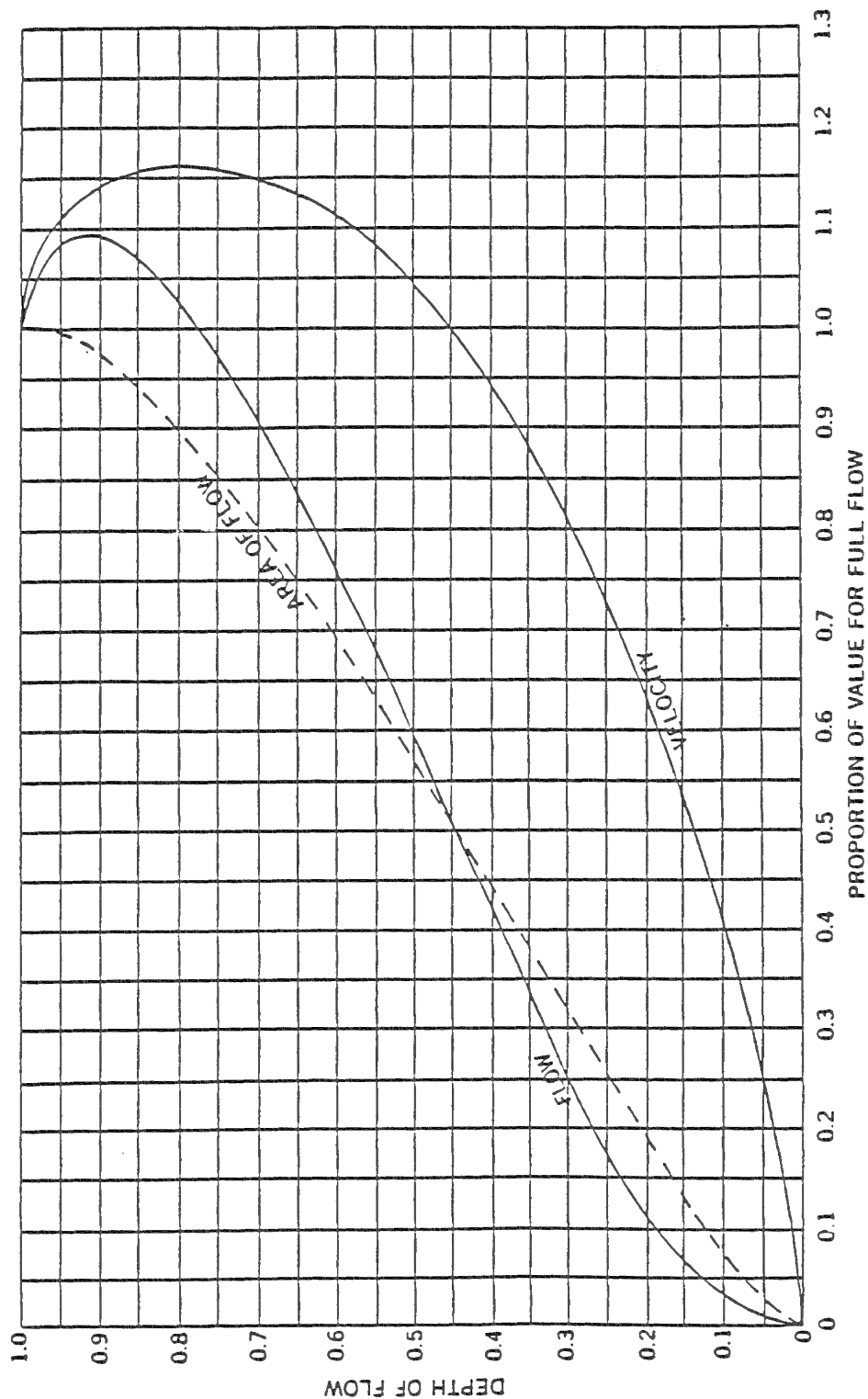
HYDRAULIC PROPERTIES
ARCH PIPE

VERSION: JANUARY 2006	REFERENCE: "CONCRETE PIPE DESIGN MANUAL," ACPA, 1970	FIGURE CH14-F103 HYDRAULIC PROPERTIES ARCH PIPE

ENERGY LOSS COEFFICIENT IN STRAIGHT THROUGH MANHOLE
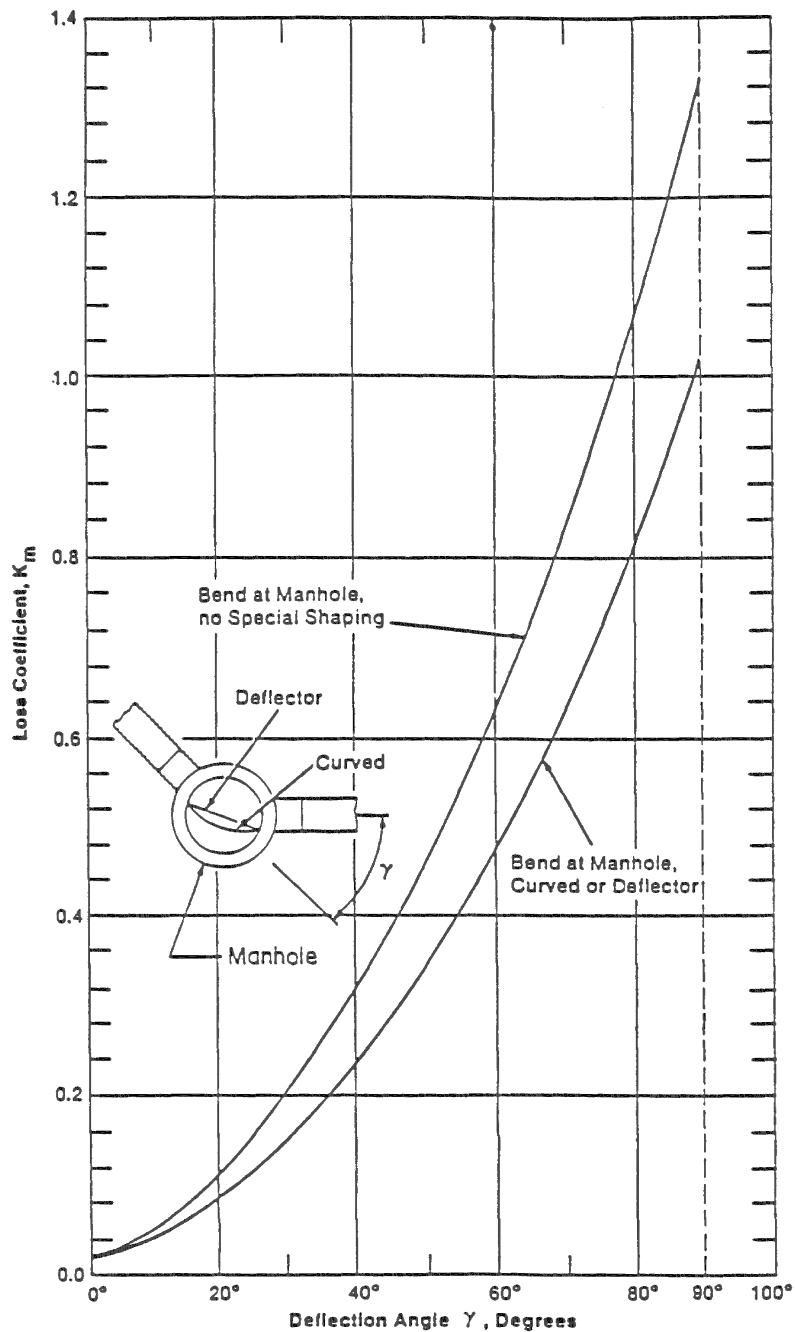
NOTE: Head loss applied at outlet of manhole.

VERSION: JANUARY 2006	REFERENCE: "MODERN SEWER DESIGN," AISI, WASHINGTON D.C., 1980	FIGURE CH14-F104 ENERGY LOSS COEFFICIENT IN STRAIGHT THROUGH MANHOLE

CONCRETE CUT-OFF WALL ON DOWN SLOPE SIDE

FLOW LINE OF DITCH

CONCRETE ENCASEMENT

2' MIN.

3' MIN.

12" MIN

6" MIN.

6" MIN.

FLOW

18" MIN.

6" MIN.

PLAN VIEW

TRENCH

2' MIN.

2' MIN.

6"

SECTION B-B

6" MIN.

6" MIN.

SECTION A-A
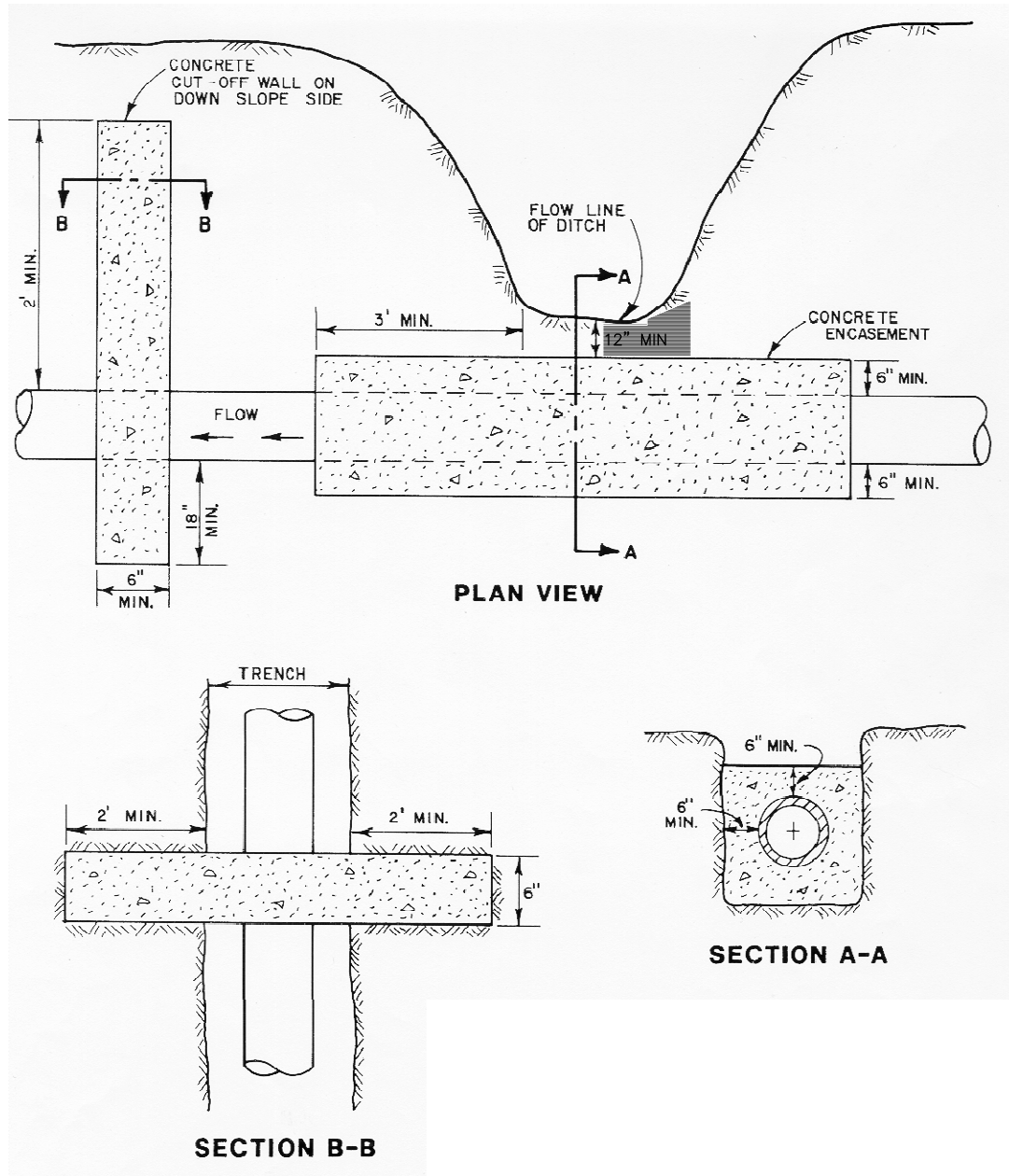
O:/2120/FIGURES/CH14-F105 – 1/6/06 – GPB

VERSION: JANUARY 2006

REFERENCE:
**CITY OF FORT COLLINS, COLORADO
ENGINEERING SERVICES UNIT
3/6/84**

FIGURE CH14-F105

DITCH CROSSING DETAILS

ALLOWABLE INLET CAPACITY
TYPE 13 COMBINATION ON A CONTINUOUS GRADE
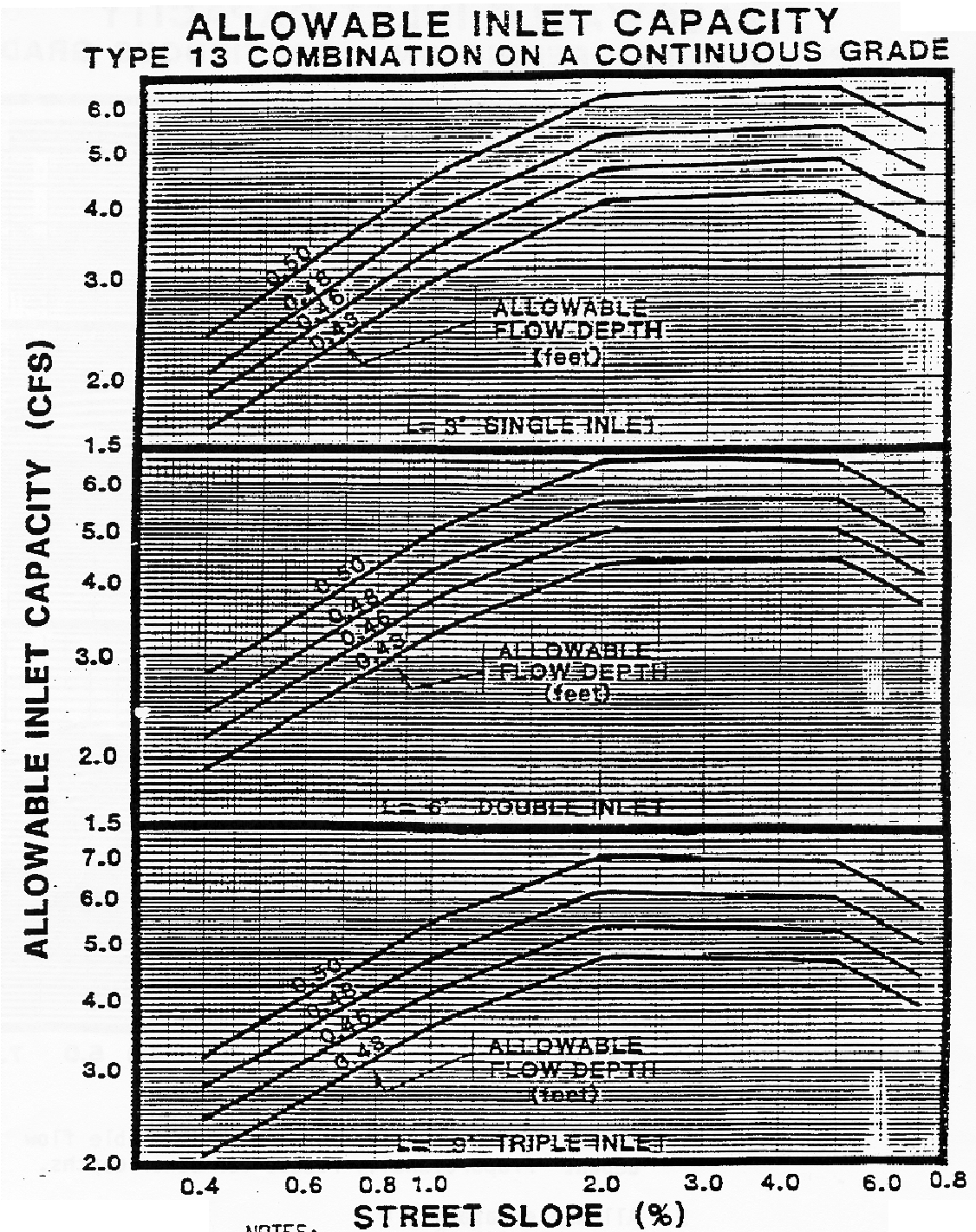
NOTES:
1. Allowable capacity = 66% theoretical capacity

2. Maximum inlet capacity at maximum allowable flow depth. Proportionally reduce for other depths.

VERSION: JANUARY 2006	REFERENCE:	**FIGURE CH14-F106**
	.	ALLOWABLE INLET CAPACITY TYPE 13 COMBINATION ON A CONTINUOUS GRADE

ALLOWABLE INLET CAPACITY
TYPE – R CURB OPENING ON A CONTINUOUS GRADE

STREET SLOPE (%)

ALLOWABLE INLET CAPACITY (CFS)
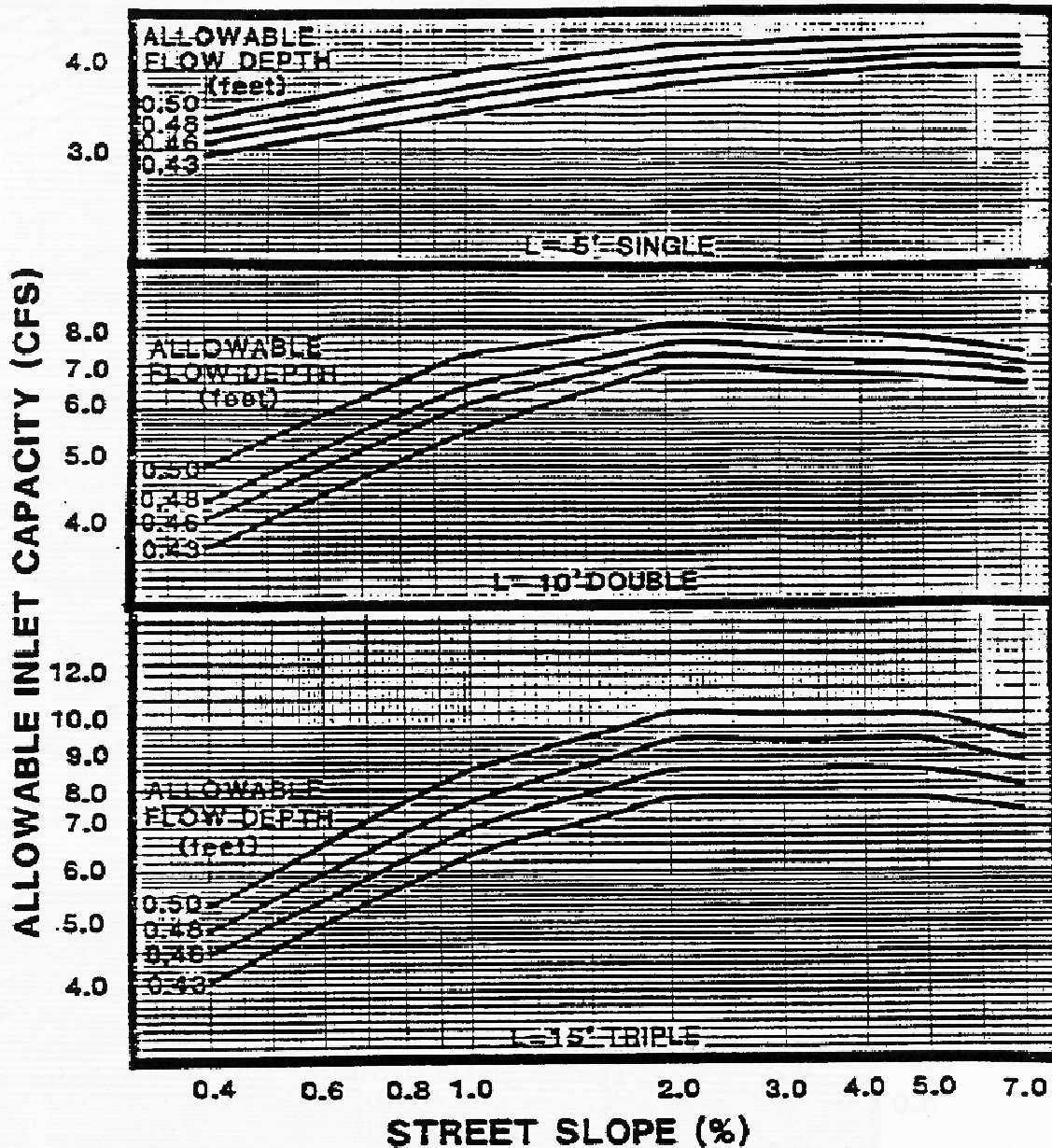
NOTES:
1. Maximum inlet capacity at maximum allowable flow depth. Proportionally reduce for other depths.

2. Allowable Capacity =
   88% (L = 5')
   92% (L = 10') } of Theoretical Capacity
   95% (L = 15')

3. Interpolate for other inlet lengths.

O:/2120/FIGURES/CH14-F107 – 1/6/06 – GPB

VERSION: JANUARY 2006	REFERENCE:	**FIGURE CH14-F107**
		ALLOWABLE INLET CAPACITY TYPE R CURB OPENING ON A CONTINUOUS GRADE

ALLOWABLE INLET CAPACITY
SUMP CONDITIONS - ALL INLETS

TYPE C INLET

TYPE-13 COMBINATION – – – –

CURB INLET TYPE-R ————

TYPE-13 GRATED —— – ——

O:/2120/FIGURES/CH14-F108 – 1/6/06 – GPB

VERSION: JANUARY 2006	REFERENCE:	**FIGURE CH14-F108**
		ALLOWABLE INLET CAPACITY **SUMP CONDITIONS - ALL INLETS**

STORM SEWER HYDRAULIC CALCULATIONS

STATION		CONDUIT DATA	INVERTS				FLOW DATA								ENERGY LOSS DATA						ENERGY AND HYDRAULIC DATA						COMMENTS	
FROM	TO	SIZE/TYPE	D/S ELEV.	U/S ELEV.	LENGTH (FT)	SLOPE (FT/FT)	A (FT²)	Φ	Q (CFS)	V (FPS)	$H_v = V^2/2g$ (FT)	S_f (FT/FT)	AVE. S_f (FT/FT)	PIPE H_f (FT)	H_t (FT)	TRANSITIONS H_m (FT)	H_j (FT)	TOTAL (FT)	PIPE U/S EGL ELEV.	U/S HGL ELEV.	TRANSITIONS U/S EGL ELEV.	U/S HGL ELEV.	PRESS. FLOW? (Y/N)	MINIMUM SURFACE ELEV.	OTHER
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28
																				Enter Starting EGL and HGL Here →							
7	4	1.5'/RCP	4918	4919	100	0.01	1.77	0.0049	7.7	4.4	0.30	0.0050	0.005	0.5					0.5	4922.8	4922.5	4922.3	4922.0	YES	4925.0	
4	6	1.5'/RCP	4919	4919.4	40	0.01	1.77	0.0049	2.5	1.4	0.10	0.0020	0.002	0.1	0.5 / 0.3				0.4			4923.2	4923.1	YES	4926.0	
4	3	1.5'/RCP	4919	4919.4	40	0.01	1.77	0.0049	4.4	2.5	0.10	0.0020	0.002	0.1	0.5 / 0.3				0.4			4923.2	4923.1	YES	4926.0	

Storm Sewer "n" Value = 0.013

EXAMPLE PROBLEM:
Storm Sewer Profile

VERSION: JANUARY 2006	REFERENCE:	FIGURE CH14-F110
	.	EXAMPLE PROBLEM: STORM SEWER PROFILE

EXAMPLE PROBLEM:
Schematic Drawing Storm Sewer System

REACH 3
Q = 4.4 cfs
L = 40'
S = 1.0%

DP3

22.5°

DP4

45°

REACH 2
Q = 2.5 cfs
L = 40'
S = 1.0%

DP6

REACH 1
Q = 7.7 cfs
L = 100'
S = 1.0%

DP7

VERSION: JANUARY 2006

REFERENCE:

.

FIGURE CH14-F111
EXAMPLE PROBLEM: SCHEMATIC
DRAWING STORM SEWER SYSTEM

TYPICAL PLAN VIEW

6'-4"
8"
L=5'-0"
o.c. 5½"
409
503
601
36"
o.c.8"
Face of Curb
18" TO STATION POINT
ONE 1¼" DIA. ROD IN 5' INLET
18"
30"
30"
TRANSITION GUTTER
3'

11'-4"
L=10'-0"
409
601
3 RODS IN 10' INLET
30"
30"

16'-4"
L=15'-0"
8"
601
409
5 RODS IN 15' INLET
30"
30"
3", EACH END
DIRECTION OF FLOW
ENDS OF 8 [BEAM 3" CLR.
FACE OF CURB
MEET SHAPE OF NORMAL BARRIER CURB AND GUTTER HERE.
TRANSITION GUTTER
3'

A A
B B

INLET PAY LENGTH

★ FOR LENGTH GREATER THEN 5 FT. PROVIDE ACCESS AT BOTH ENDS.

■ ADDITIONAL MANHOLE RING AND COVER REQUIRED WHEN L=10' OR MORE. CUT REINFORCEMENT BAR ACCORDINGLY.

TRANSITION CURB

BARRIER CURB FACE
INLET
★ 5' TRANSITION
MOUNTABLE CURB FACE
MOUNTABLE CURB AND GUTTER

★ WHEN A TYPE R INLET IS USED WITH MOUNTABLE CURB AND GUTTER, 5 FT. TRANSITION SHALL BE CONSTRUCTED. TRANSITION SHALL BE PAID FOR AS CURB AND GUTTER.

SECTION A-A REGULAR INLET

18" TO STA.PT.
L=5'
CUT OFF OR BEND BARS TO CLEAR MANHOLE.
409
601 MANHOLE 503 409
403
8" 403
401 601 2"clr. 3"clr. 401
3"clr.
11"o.c.
9"o.c. TYP. 9"o.c.
8"
407 403
407 401
403
12"
3" CLR. 405 406
6"o.c. x 6"o.c.
6'-8"
L=5'~All H
L=10'or15'~H<5'

L=10'
■ ■
409 409
403 403
601 601
H<5'
407
405 406
6"o.c. x 6"o.c.
11'-8"

L=15'
409 409
403 403
601 601
H<5'
407
403
405 406
6"o.c. x 6"o.c.
16'-8"
2" ALL AROUND TYPICAL

CURB FACE ASSEMBLY

SEE CHANNEL LAYOUT ON SHEET 2.
8"
1½" 4"
8 [8.5
NO.4 BAR 18" LONG
FLUSH WITH CURB FACE (TYPE 2-SEC. IIB)
1" LEG
THD. 5½"
3"
(¼)┤├
6"
(¼)
4:1
2:1
1½ PIPE SPACER AND 1¼ LOCK NUT
8"
2:1
1¼" DIA. x 24" GALV. STEEL ROD 2'-6"o.c.
4"
10" EMBEDMENT
¼
3"x3"x⅜" PLATE

▼ PLACE ENTIRE ASSEMBLY BEFORE POURING CONCRETE.

SECTION B-B TYPICAL END VIEW

▲ FOR A 1'-0" PAN SLOPE 2" PER FT.

2'-0"
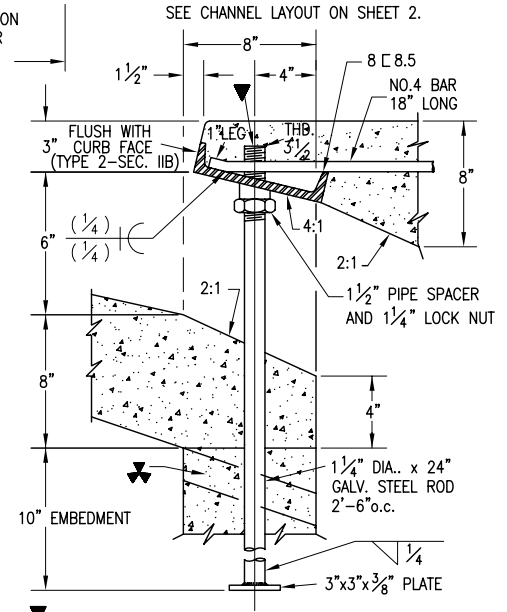4'-4"
1¼"
▲ SLOPE 1"/FT.
1¼" R
2" R
SLOPE 2% TO GUTTER
409 8"o.c.
601
4"
6"
501
5½"o.c.
408
12" o.c.
7"±
2"clr.
503
5½"o.c.
INLET STEPS REQ'D.FOR ALL H ≥ 3'-6"
16" MAX.
3" CLR. 3" CLR.
8" TYPICAL WALL
8" WING FL.
407
9"o.c.
402
11" o.c.
401
11"o.c.
401
11"o.c.
407
9"o.c.
403
9" o.c.
406 405
6" o.c. 6" o.c.
3" CLR.
2" ALL AROUND

▼ A 2" DIAMETER TEMPORARY HOLE FOR DRAINAGE SHALL BE PLACED AT SUBGRADE ELEVATION OR A MINIMUM THREE INCHES BELOW ROAD BASE. THE HOLE SHALL BE PLUGGED WITH CONCRETE BEFORE ACCEPTANCE OF THE INLET.

NOTE: MANHOLE RING & COVER, STATION POINT AND OUTFLOW PIPE SHALL BE LOCATED AT THE SAME END OF THE INLET.

SECTION A-A INLET WITH DROP BOX~H>5'

18"TO STA.PT.
601
8"
503 502 502
409 504 502
403
32" CONSTANT
413 409 403 413
403 403
5'-0" CONSTANT
3" CLR. MIN. 0.5% Slope
413
9"o.c.
401 407
8"
411 412
411 412
H > 5'
VARIABLE
403 410
2" CLR.
405
12"
406
C C D D C D

SECTIONS C-C & D-D

(DOTTED BARS ARE IN SECTION D-D)

501
409
502
403
408
412
413
413
3'-4"
3-411
11"o.c.

TABLE ONE ~ BAR LIST FOR CURB INLETS, TYPE "R"

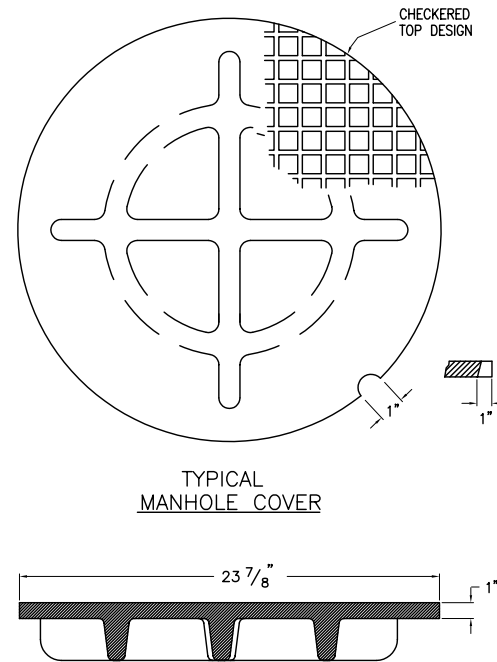| MARK	DIA IN.	O.C. SPACING	TYPE	ALL INLETS L=→5' NO.REQ'D.	LENGTH	INLETS. H ≲ 5' 10' NO.REQ'D.	LENGTH	15' NO.REQ'D.	LENGTH	INLETS, H 5' 10' NO.REQ'D.	LENGTH	15' NO.REQ'D.	LENGTH
401		11"	II	15	*	21	*	26	*	11	*	11	*
402		11"	II	7	*	13	*	18	*	7	*	7	*
403		9 "	II	*	4'-10"	*	4'-0"	*	4'-0"	*	4'-0"	*	4'-10"
405		6 "	VI	11	6'-10"	21	6'-10"	31	6'-10"	11	6'-10"	11	6'-10
406		6 "	VIII	7	8'-10"	7	13'-10"	7	18'-10"	7	8'-10"	7	8'-10"
407	1/2"	9 "	II	*	5'-10"	*	10'-10	*	15'-10"	*	5'-10"	*	5'-10"
408		12"	II	3	6'-10"	3	11'-0"	3	16'-0"	3	11'-0"	3	16'-0"
409		8 "	II	6	5'-10"	6	10'-10"	6	15'-10"	6	10'-10"	6	15'-10"
410		11"	VII							3	*	3	*
411		11"	II							3	5'-2"	3	10'-2"
412		11"	II							3	2'-9"	3	2'-9"
413		9 "	II							7	10'-10"	7	15'-10"
501		5 1/2"	IV	11	3'-4"	22	3'-4"	33	3'-4"	22	3'-4"	33	3'-4"
502	5/8"	5 1/2"	III			11	11'-5"	17	11'-5"
503		5 1/2"	II	5	3'-6"	16	3'-6"	27	3'-6"	6	3'-6"	6	3'-6"
504		5 1/2"	IX									5	8'-4"
601	3/4"	2 1/2"	V	2	8'-10"	2	8'-10"	2	8'-10"	2	8'-10"	4	8'-10"
Ø8[8.5				1	5'-10"	1	10'-10"	1	15'-10"	1	10'-10"	1	15'-10"
				2BARS,1ROD	—	4BARS,3RODS	—	8BARS,5RODS	—	4BARS,3RODS	—	8BARS,5RODS	—

REGULAR INLETS   DROP BOX INLETS

* VARIABLE, REFER TO TABLE TWO.
Ø INCLUDE 18" NO. 4 BARS (SEE CHANNEL LAYOUT DETAIL).
▼ SEE CURB FACE ASSEMBLY ON SHEET 1 AND CHANNEL LAYOUT DETAILS ON THIS SHEET.

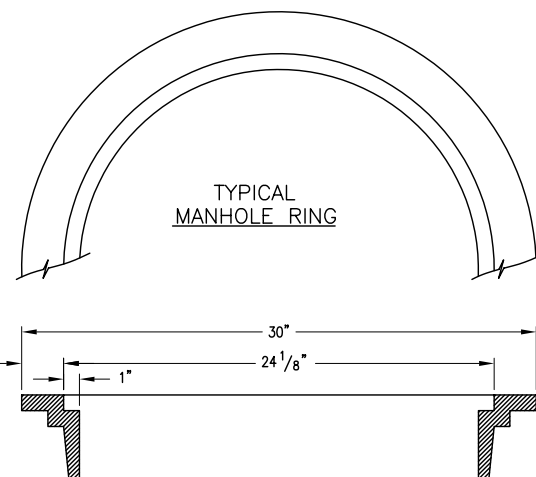## TABLE TWO ~ BARS AND QUANTITIES VARIABLE WITH "H"

'H'	LENGTH 401	402	410	NO. REQ'D. REGULAR 403	NO. REQ'D. DROP BOX 403	407	L=5' CU.YD.CONC.	LB.STEEL	L=10' CU.YD.CONC.	LB.STEEL	L=15' CU.YD.CONC.	LB.STEEL
3'-0"	2'-8"	1'-8"		10	7		3.2	285	5.3	497	7.4	706
3'-6"	3'-2"	2'-2"		10	7		3.4	305	5.7	528	7.9	747
4'-0"	3'-8"	2'-8"		12	9		3.7	326	6.0	559	8.4	786
4'-6"	4'-2"	3'-2"		12	9		3.9	334	6.4	571	8.8	803
5'-0"	4'-8"	3'-8"		14	11		4.1	354	6.7	602	9.3	844
5'-6"	5'-2"	4'-2"	3'-5"	16	13	15	6	4.4	375	6.0	607	7.4	850
6'-0"	5'-8"	4'-8"	3'-11"	16	13	16	6	4.6	382	6.2	616	7.6	860
6'-6"	6'-2"	5'-2"	4'-5"	18	15	18	8	4.8	402	6.4	637	7.8	880
7'-0"	6'-8"	5'-8"	4'-11"	20	17	19	10	5.0	423	6.6	654	8.0	897
7'-6"	7'-2"	6'-2"	5'-5"	20	17	20	10	5.3	430	6.9	664	8.3	907
8'-0"	7'-8"	6'-8"	5'-11"	22	19	22	12	5.5	451	7.1	684	8.5	927
8'-6"	8'-2"	7'-2"	6'-5"	24	21	23	14	5.7	471	7.3	702	8.7	944
9'-0"	8'-8"	7'-8"	6'-11"	24	21	24	14	6.0	479	7.6	711	9.0	954
9'-6"	9'-2"	8'-2"	7'-5"	26	23	26	16	6.2	499	7.8	732	9.2	974
10'-0"	9'-8"	8'-8"	7'-11"	28	25	27	18	6.4	520	8.0	749	9.4	992
10'-6"	10'-2"	9'-2"	8'-5"	28	25	28	18	6.7	527	8.3	759	9.7	1001
11'-0"	10'-8"	9'-8"	8'-11"	30	27	30	20	6.9	547	8.5	779	9.9	1022

NOTE: FOR L=5', L=10' AND L=15'
REGULAR INLETS: TOTAL QUANTITIES NEEDED ARE OUTSIDE OF THE HEAVY BLACK LINE.
DROP BOX INLETS: TOTAL QUANTITIES NEEDED ARE INSIDE OF THE HEAVY BLACK LINE.

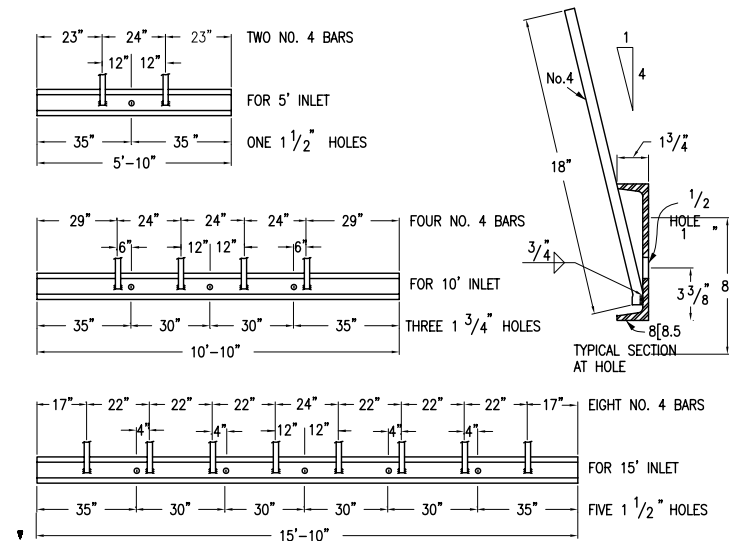STEEL WEIGHTS DO NOT INCLUDE STRUCTURAL STEEL.

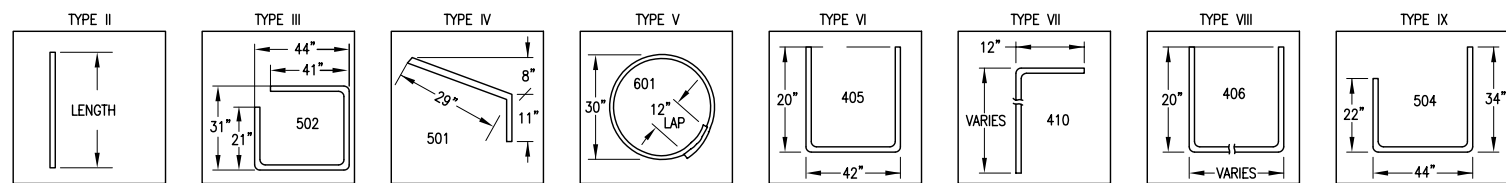### BAR BENDING DIAGRAMS ~ (Dimensions are Out-to-Out of bar)

TYPE II — LENGTH
TYPE III — 502 (44", 41", 31", 21")
TYPE IV — 501 (29", 8", 11")
TYPE V — 601, 12" LAP, 30"
TYPE VI — 405 (20", 42")
TYPE VII — 410 (12", VARIES)
TYPE VIII — 406 (20", VARIES)
TYPE IX — 504 (22", 34", 44")

TYPICAL MANHOLE COVER
CHECKERED TOP DESIGN
23 7/8"
1"

TYPICAL MANHOLE RING
2 15/16"
30"
24 1/8"
1"
4"

WEIGHTS: COVER = 125 LBS.
RING = 135 LBS.
TOTAL = 260LBS.

CHANNEL LAYOUT DETAILS

TWO NO. 4 BARS — FOR 5' INLET — 23", 24", 23", 12", 12", 35", 35", 5'-10", ONE 1 1/2" HOLES
FOUR NO. 4 BARS — FOR 10' INLET — 29", 24", 24", 29", 6", 12", 12", 6", 35", 30", 30", 35", 10'-10", THREE 1 3/4" HOLES
EIGHT NO. 4 BARS — FOR 15' INLET — 17", 22", 22", 22", 24", 22", 22", 22", 17", 4", 4", 12", 12", 4", 4", 35", 30", 30", 30", 30", 35", 15'-10", FIVE 1 1/2" HOLES

No.4 — 18" — 1 3/4" — 1/2 HOLE — 3/4 — 3 3/8" — 8" — 8[8.5
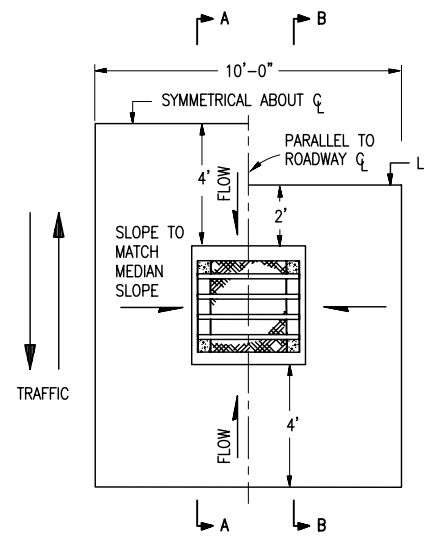TYPICAL SECTION AT HOLE

GENERAL NOTES

1. ALL CONCRETE SHALL BE CLASS B.
2. CONCRETE WALLS SHALL BE FORMED ON BOTH SIDES AND SHALL BE 8 IN. THICK.
3. INLET STEPS SHALL BE IN ACCORDANCE WITH AASHTO M 199.
4. CURB FACE ASSEMBLY SHALL BE GALVANIZED AFTER WELDING.
5. EXPOSED CONCRETE CORNERS SHALL BE CHAMFERED 3/4 IN. CURB AND GUTTER CORNERS SHALL BE FINISHED TO MATCH THE EXISTING CURB AND GUTTER BEYOND THE TRANSITION GUTTER.
6. REINFORCING BARS SHALL BE DEFORMED AND SHALL HAVE A 2 IN. MINIMUM CLEARANCE. ALL REINFORCING BARS SHALL BE EPOXY COATED.
7. DIMENSIONS AND WEIGHTS OF TYPICAL MANHOLE RING AND COVER ARE NOMINAL.
8. MATERIAL FOR MANHOLE RINGS AND COVERS SHALL BE GRAY OR DUCTILE CAST IRON CONFORMING TO 712.06.
9. SINCE PIPE ENTRIES INTO THE INLET ARE VARIABLE, THE DIMENSIONS SHOWN ARE TYPICAL. ACTUAL DIMENSIONS AND QUANTITIES FOR CONCRETE AND REINFORCEMENT SHALL BE AS REQUIRED IN THE WORK. QUANTITIES INCLUDE VOLUMES OCCUPIED BY PIPES.
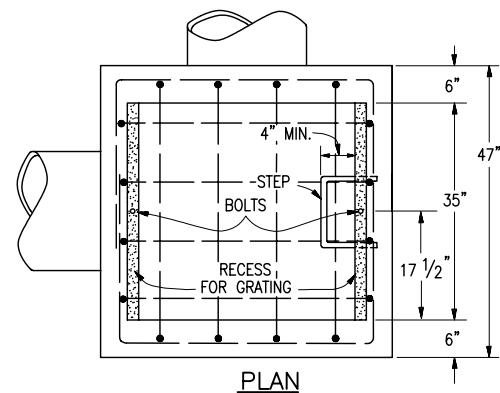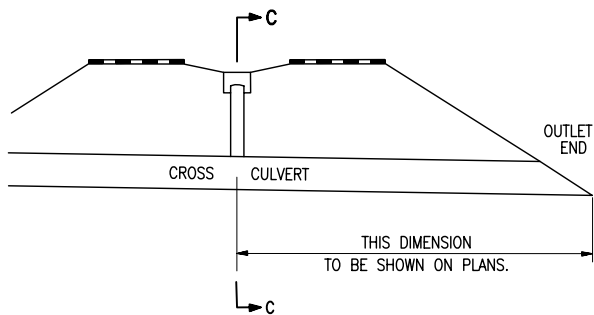10. STRUCTURAL STEEL SHALL BE GALVANIZED AND SHALL CONFORM TO THE REQUIREMENTS OF 712.06.

STEEL GRATE QUANTITIES

NO. PIECES	DESCRIPTION	LENGTH	LB. PER FT	WEIGHT (LBS.)
4	S4 x 7.7 BEAM	40"	7.70	103
2	3 1/2" x 1/4" FLAT	26 5/8"	2.98	13
2	3" x 1/4" FLAT	26 5/8"	2.55	12

TOTAL 128 LBS.

INLET WITH DITCH PAVING

SYMMETRICAL ABOUT CL
PARALLEL TO ROADWAY CL
SLOPE TO MATCH MEDIAN SLOPE
FLOW
TRAFFIC
10'-0"

CROSS CULVERT
OUTLET END
THIS DIMENSION TO BE SHOWN ON PLANS.

PLAN
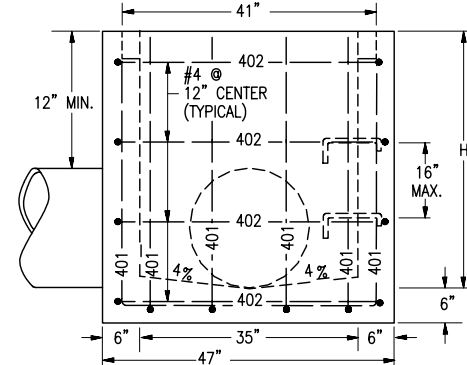
ELEVATION
CONCRETE INLET

#4 @ 12" CENTER (TYPICAL)
402
401
12" MIN.
16" MAX.
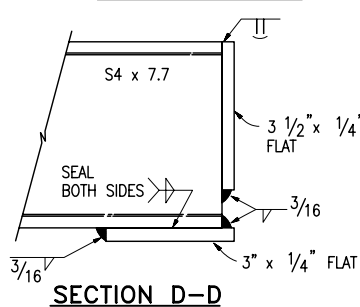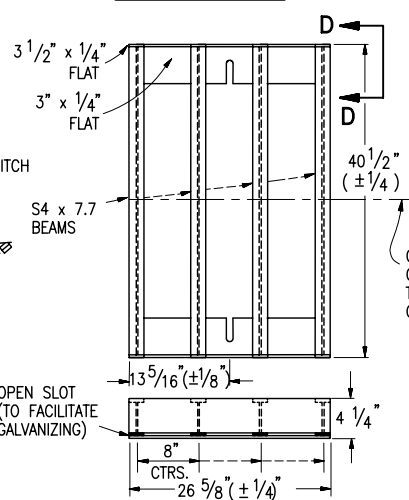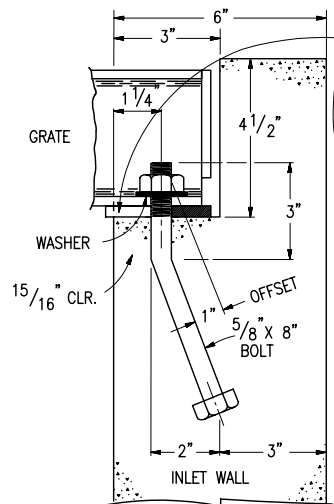
GRATE INSTALLATION DETAIL

SLOT DETAIL
1 1/2" 3/4" 3/8" R.
3" x 1/4" FLAT

ALTERNATE SLOT AND HOLD DOWN PLATE DETAIL
1 1/16" x 1" SLOTTED HOLE
3" x 1/4"

GRATE
WASHER
15/16" CLR.
OFFSET
5/8" X 1" BOLT
INLET WALL

QUANTITIES FOR ONE INLET

H	CONCRETE * (CU. YDS.)	REINF. STEEL (LBS.)	NO. STEPS REQ'D.
2'-6"	0.9	75	0
3'-0"	1.0	80	0
3'-6"	1.2	96	0
4'-0"	1.3	101	1
4'-6"	1.4	116	2
5'-0"	1.5	122	2
5'-6"	1.7	137	2
6'-0"	1.8	142	3
6'-6"	1.9	158	3
7'-0"	2.0	163	3
7'-6"	2.2	179	4
8'-0"	2.3	184	4
8'-6"	2.4	199	4
9'-0"	2.5	205	5
9'-6"	2.7	220	5
10'-0"	3.0	235	6
11'-6"	3.4	251	6

* INCLUDES VOLUME OCCUPIED BY PIPES.

BAR LIST FOR H=2'-6" AND BENDING DIAGRAM

MARK	NO. REQ'D.	HGT.	LENGTH
401	2	2'-3"	7'-11"
401	6	2'-7"	8'-7"
402	3	"U"	15'-0"

NO. 401 "U"
3'-5"
INCREASE DIMENSION 6" FOR EACH 6" INCREASE OF "H" ABOVE 2'-6".

NO. 402
3'-6" 6" 12"
ADD ONE BAR FOR EACH FOOT INCREASE OF "H" ABOVE 2'-6".

ALL BARS TO BE 1/2" DIA. CUT OR BEND AROUND PIPES AS REQUIRED.
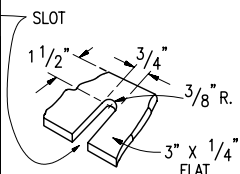
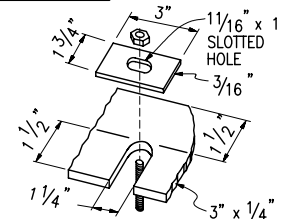402 BARS WILL BE EQUALLY SPACED FROM EACH OTHER.

GENERAL NOTES

1. CONCRETE SHALL BE CLASS B. INLET MAY BE CAST-IN-PLACE OR PRECAST.

2. REINFORCING BARS SHALL BE EPOXY COATED AND DEFORMED, AND SHALL HAVE A MINIMUM 2" CLEARANCE.

3. CONCRETE SLOPE AND DITCH PAVING SHALL CONFORM TO SECTION 507. REINFORCEMENT FOR CONCRETE SLOPE PAVING SHALL BE 6 X 6 – W1.4 X W1.4 OR 6 X 6 – W2.1 X W2.1.

4. STRUCTURAL STEEL FOR GRATES AND GRATE INSTALLATION HARDWARE SHALL BE GALVANIZED AND SHALL BE IN ACCORDANCE WITH 712.06.

5. THE STANDARD INLET GRATES SHALL BE USED ON ALL TYPE C INLETS UNLESS CLOSE MESH GRATES ARE SPECIFIED ON THE PLANS.

6. STEPS SHALL BE PROVIDED WHEN INLET DIMENSION "H" EXCEEDS 3'-6" AND SHALL BE IN ACCORDANCE WITH AASHTO M 199.

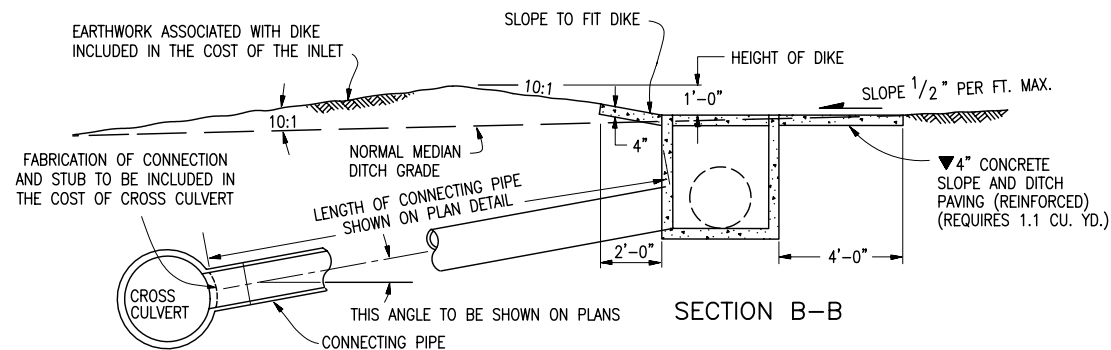7. SEE SHEET M-604-11, INLET, TYPE D, FOR REINFORCEMENT AROUND THE PIPE OPENING.

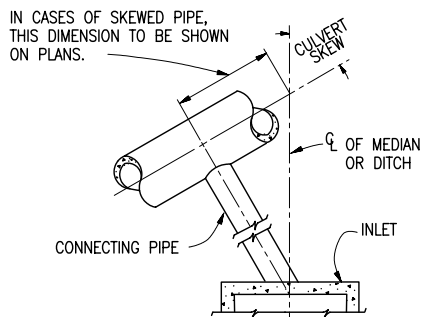▼ – CONCRETE SLOPE AND DITCH PAVING WILL BE REQUIRED WHEN SHOWN ON PLANS.

INLET CONNECTED TO A CROSS CULVERT

EARTHWORK ASSOCIATED WITH DIKE INCLUDED IN THE COST OF THE INLET
SLOPE TO FIT DIKE
HEIGHT OF DIKE
SLOPE 1/2" PER FT. MAX.
NORMAL MEDIAN DITCH GRADE
FABRICATION OF CONNECTION AND STUB TO BE INCLUDED IN THE COST OF CROSS CULVERT
LENGTH OF CONNECTING PIPE SHOWN ON PLAN DETAIL
CROSS CULVERT
THIS ANGLE TO BE SHOWN ON PLANS
CONNECTING PIPE
SECTION C-C
▼ 4" CONCRETE SLOPE AND DITCH PAVING (REINFORCED) (REQUIRES 1.1 CU. YD.)

SECTION B-B
S4 x 7.7
SEAL BOTH SIDES
3 1/2" x 1/4" FLAT
3" x 1/4" FLAT
3/16

INLET ON GRADE (FLOW FROM ONE DIRECTION)

SECTION D-D
3 1/2" x 1/4" FLAT
3" x 1/4" FLAT
S4 x 7.7 BEAMS
40 1/2" (± 1/4")
OPEN SLOT (TO FACILITATE GALVANIZING)
13 5/16" (± 1/8")
8" CTRS.
26 5/8" (± 1/4")
4 1/4"
CENTERLINE OF GRATE PARALLEL TO CENTERLINE OF ROADWAY

STANDARD INLET GRATE

SECTION E-E
4" x 3/8" BEARING BAR
3 1/2" x 1/4" FLAT
3" x 1/4" FLAT
3/16

INLET CONNECTED TO A SKEWED CROSS CULVERT

IN CASES OF SKEWED PIPE, THIS DIMENSION TO BE SHOWN ON PLANS.
CULVERT SKEW
CONNECTING PIPE
CL OF MEDIAN OR DITCH
INLET

SECTION A-A
INLET AT BOTTOM OF VERTICAL CURVE
SLOPE 1/2" PER FT. MAX.
MEDIAN DITCH GRADE
▼ 4" CONCRETE SLOPE AND DITCH PAVING (REINFORCED) (REQUIRES 1.3 CU. YD.)
4'-0"

CLOSE MESH GRATE
3/8" TYPICAL HEX. ROUND OR TWISTED CROSS BARS AT 8" CTRS., WELDED TO 4" X 3/8" BEARING BARS SPACED AT 2 3/8" CTRS.
SLOT DETAIL IN 3 1/2" x 1/4" FLATS SAME AS IN STANDARD INLET GRATE.
33 1/2"
31 1/2"
40 1/4"
4" x 3/8" BAR
3" x 1/4" FLAT
4 1/4"
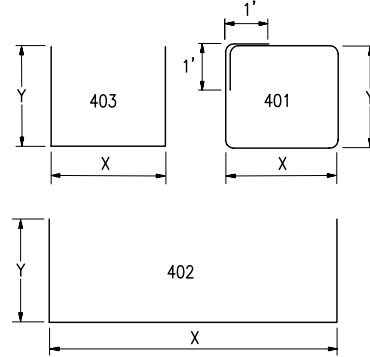HYDRAULICALLY INEFFICIENT, POOR TOLERANCE FOR DEBRIS AND ICE. USE FOR PEDESTRIAN AND BICYCLE AREAS.

PLAN OF TYPE 13 INLET FOR GUTTER TYPE 2

4'-3½"
2'-0"
4'-0"
2'-0"
B
A
A
B
℄ GUTTER TYPE 2

SECTION A-A

4'-3½"
8"
35½"
16" MAX.
401 @ EQUAL SPACING AT 9" ±
H
4%
8"
403 @ 8" O.C.

SECTION B-B

4'-0"
1"
8"
6"
6¼"
8"
19½"
8"
401 @ EQUAL SPACING AT 9" ±
8"
402 @ 8" O.C.

BENDING DIAGRAM

403
401
1'
1'
402
Y
X
Y
X
Y
X

ALL DIMENSIONS ARE OUT-TO-OUT OF BAR.

CORNERS RELIEVED TO PREVENT ROCKING
3/16"
3"
C
D
D
C
22½"
7/16" RADIUS TYP.
2"
1"
1"
3/4"
3½"

SECTION C-C

A = 1¾"
B = 11 11/16"
C = 7 15/16"

39¼"
C
C
C
A
A
1"
2"
1"
¼" R

SECTION D-D
NO. 13 GRATE

SECTION E-E
NO. 13 GRATING & FRAMES

F
F
E
E
39⅞"
23"
48⅛"
39⅞"
1⅛"
1"
5"
4 1/16"
40"
1 x 45°

APPROX. WEIGHT 590 LBS.

SECTION F-F

1"
23⅛"
31⅛"
23"
1⅛"
1"
4 1/16"
2"
1"

NOTE:

SEE PLAN DETAILS FOR LOCATION AND SIZE OF PIPE.

▲ WHEN BITUMINOUS MATERIAL IS TO EXTEND TO THE EDGE OF THE GRATING FRAME, CONCRETE MAY BE DEPRESSED.

GENERAL NOTES

1. CONCRETE SHALL BE CLASS B. INLET MAY BE CAST-IN-PLACE OR PRECAST.

2. CAST-IN-PLACE CONCRETE WALLS SHALL BE FORMED ON BOTH SIDES.

3. EXPOSED CONCRETE CORNERS SHALL BE CHAMFERED 3/4 IN.

4. REINFORCING BARS SHALL BE DEFORMED AND SHALL HAVE A 2 IN. MINIMUM CLEARANCE. ALL REINFORCING BARS SHALL BE EPOXY COATED.

5. STEPS SHALL BE PROVIDED WHEN INLET DIMENSION "H" EXCEEDS 3 FT.-6 IN. AND SHALL BE IN ACCORDANCE WITH AASHTO M 199.

6. ALL GRATES AND FRAMES SHALL BE GRAY OR DUCTILE CAST IRON CONFORMING TO 712.06. GRATES AND FRAMES SHALL BE DESIGNED TO WITHSTAND HS 20 LOADING.
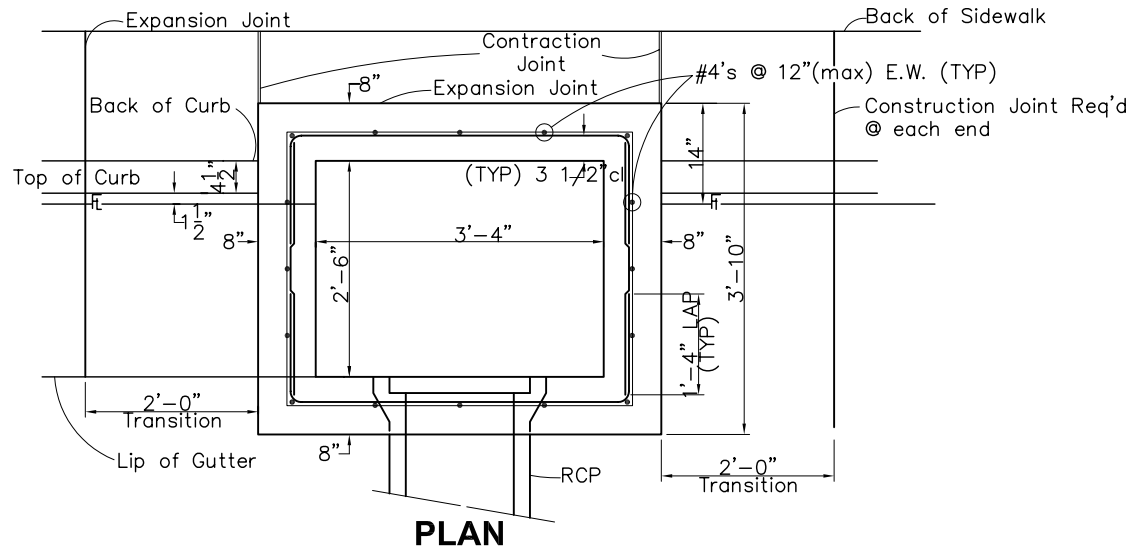
QUANTITIES

H	CONCRETE CU. YD.	REINFORCING STEEL θ LB.	NO. OF 401 BARS REQ'D.
3'-0"	1.3	72	4
3'-6"	1.5	76	4
4'-0"	1.6	90	5
4'-6"	1.8	104	6
5'-0"	1.9	109	6
5'-6"	2.1	122	7
6'-0"	2.2	136	8
6'-6"	2.4	141	8
7'-0"	2.5	154	9
7'-6"	2.7	168	10
8'-0"	2.8	173	10
8'-6"	3.0	187	11
9'-0"	3.1	200	12
9'-6"	3.3	205	12
10'-0"	3.4	219	13

θ INCLUDES 1% FOR OVERRUN.
NOTE: CONCRETE QUANTITIES INCLUDE VOLUME OCCUPIED BY PIPE.

BAR LIST FOR H=3'-0"

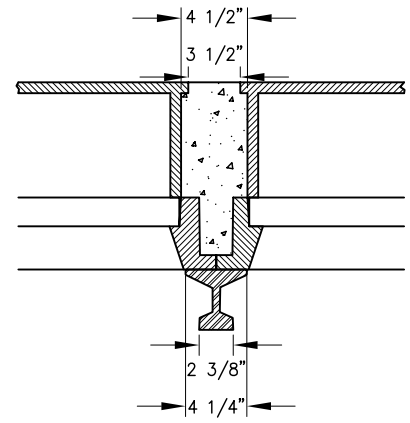| MARK	NO. REQ'D.	DIMENSIONS X	Y	LENGTH
401	4	3'-6"	2'-2"	13'-4"
402	2	3'-4½"	* 2'-6½"	8'-5½"
403	5	2'-½"	* 2'-7"	7'-2½"

* ADD 6 IN. TO THIS DIMENSION FOR EACH 6 IN. INCREASE OF "H" OVER 3 FT.-0 IN.

STANDARD DETAIL
CH14-SD103
CONCRETE INLET TYPE 13

O:/2120./FIGURES. CH14-SD103 - 1/6/06 - GPB

PLAN

Expansion Joint
Back of Sidewalk
Contraction Joint
Expansion Joint
8"
#4's @ 12"(max) E.W. (TYP)
Back of Curb
Construction Joint Req'd @ each end
Top of Curb
14"
(TYP) 3 1/2" cl
41"
42"
1 1/2"
8"
3'-4"
2'-6"
8"
3'-10"
1'-4" (TYP) AP
2'-0" Transition
Lip of Gutter
8"
RCP
2'-0" Transition
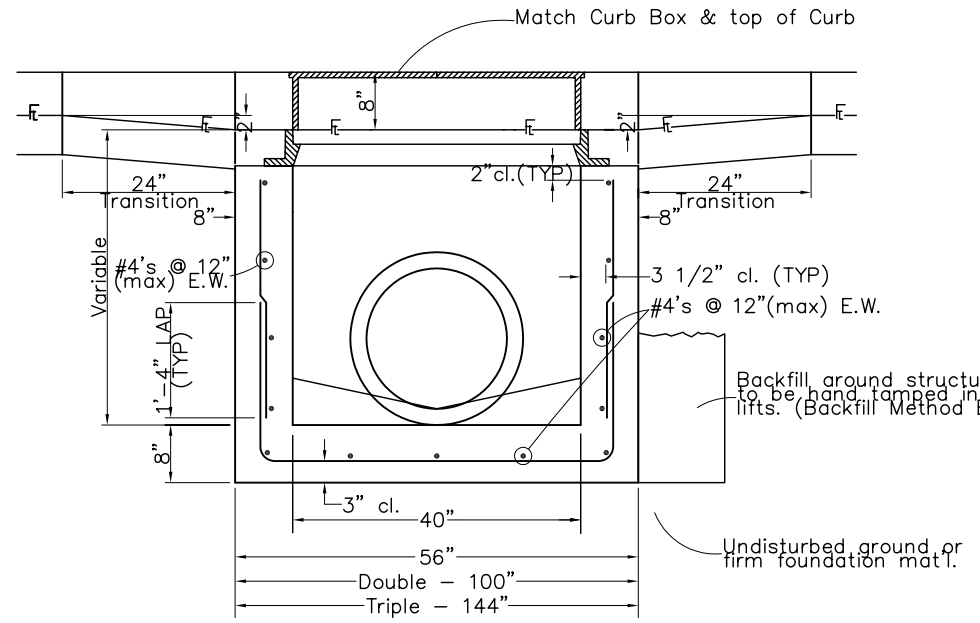
SECTION CENTER OF MULTIPLE INLETS

4 1/2"
3 1/2"
2 3/8"
4 1/4"

PLAN

48"
2 – 5/8"x3" BOLTS

SECTION A-A

41"
38 1/2"
3/4"
40"
1/2"
5"
1"
1 1/2"
4"
1"
1 1/2"

SECTION THRU LENGTH

Match Curb Box & top of Curb
8"
2"
24" Transition
8"
2" cl. (TYP)
24" Transition
8"
Variable
#4's @ 12" (max) E.W.
3 1/2" cl. (TYP)
#4's @ 12"(max) E.W.
1'-4" (TYP) AP
8"
Backfill around structure to be hand tamped in 6 lifts. (Backfill Method B)
3" cl.
40"
Undisturbed ground or firm foundation mat'l.
56"
Double – 100"
Triple – 144"

SECTION THRU WIDTH

See Standard Detail S-34-13-2b For Frame, Grate & Adjustable Curb Box Details.
Lip of Gutter
Sidewalk
Pavement
E Elevation
2" cl. (TYP)
1 7/8"
12"
Grout
2-#5's
No Flange
Shim
8"
30"
8"
#4's @ 12"(max)
(TYP) 3 1/2"cl.
Invert Out Elevation 1% (Typ)
8"
Lean Concrete Fill
3"cl.
3'-10"

SECTION B-B

1 1/2" RAD.
1"
3/4"
3/4"
3"
12 1/2"
1"
1"
5"
6 1/2"
4" 3"
1"
4"
6"

COMBINATION INLET TYPE 13

NOTES:

1) For payment purposes, inlet structure shall also include 2'-0" curb & gutter transition section at each end of inlet plus sidewalk sections where required behind Inlet structure and Transition sections.

2) Floor slope may be poured monolithic with base.

3) Outlet pipe(s) to be set flush with inside face of inlet.

4) Unless otherwise specified on the drawings or otherwise approved, all No. 13 inlets shall be constructed with an adjustable C.I. curb box.

5) Std inlet depths and pipe size ar noted in the following table. Deviations from minimum requirements shall be substantiated with appropriate analysis.

NOTES:

1) Casting Specifications: A.S.T.M. A-48 with a minumum tensile strength of 25 ksi (Class 25)

2) All castings to be dipped in asphalt base paint.

3) Minumum Curb Opening Area = 150 sq. in.

O:/2120/FIGURES, CH14-SD104 -1/6/06 – GPB

COLORADO WATER CONSERVATION BOARD

COLORADO
FLOODPLAIN AND STORMWATER CRITERIA MANUAL

STORM SEWER HYDRAULIC CALCULATIONS

CONDUIT DATA							FLOW DATA				ENERGY LOSS DATA					ENERGY AND HYDRAULIC DATA							COMMENTS

Storm Sewer "n" Value = ____

$$S_f = \frac{\bar{k}\, H_v}{R^{1.33}} \qquad \bar{k} = \frac{2g(n^2)}{2.21}$$

Enter Starting EGL and HGL Here →

VERSION: JANUARY 2006	REFERENCE:	**STANDARD FORM CH14-SF101**
	.	**STORM SEWER** **HYDRAULIC CALCULATIONS**

ITEM	Reinforced Concrete Pipe (RCP)	Horizontal Elliptical Reinforced Concrete Pipe (HERCP)	Non-Reinforced Concrete Pipe (NRCP)***	Polyvinyl Chloride Pipe (PVC)***	High Density Polyethylene Pipe (HDPE)***
Manning's "n" Value	0.013	0.013	0.013	0.011	0.012
Pipe Size (Inches)	15-144	14x23 to 58x91	15 – 36	15-54*	15-36*
Joints	Watertight Rubber Gasket	Watertight Rubber Gasket	Watertight Rubber Gasket	Watertight Rubber Gasket	Watertight Rubber Gasket
Typical Manufactured Length (Feet)	7.5 - 8	7.5 - 8	7.5 - 8	13	20
Minimum Stiffness (psi)	Rigid (Class III min.)	Rigid (Class HE-III min.)	Rigid (Class3 min.)	46	22(for 36") (varies with diameter)
Minimum Bury Depth (Feet)**	1.5	1.5	1.5	2.0 (ASTM 2321)	2.0 (ASTM 2321)
Maximum Bury Depth (Feet)**	See Table CH14-T102	Calculate Using Concrete Design Manual	See Table CH14-T102	See Table CH14-T103	10
Chemical Resistance	Moderate	Moderate	Moderate	High	High
Abrasion Resistance	Moderate	Moderate	Moderate	High	High
Corrosion Resistance	Moderate	Moderate	Moderate	High	High
Perform History (Denver Metro Area)	High	High	--	High	Moderate
Connections	Grouted or Insert Tee	Grouted or Insert Tee	Grouted or Insert Tee	Insert Tee	Insert Tee
Contractor Preference	High	High	--	Preferred	Acceptable

*Maximum diameter of current AASHTO/ASTM Standard. If AASHTO/ASTM approved specifications change in the future, the user may use larger accepted pipe diameter sizes.

**Maximum and minimum bury depths are general parameters and may be exceeded if a special design and application allows an acceptable usage. The distance is measured from the top of the pipe (i.e. top of the bell on RCP, etc.) to the bottom of the structural section or base course.

VERSION: JANUARY 2006	REFERENCE: .	**TABLE CH14-T101** COMPARITIVE EVALUATION SUMMARY OF PIPELINE MATERIALS

PIPE SIZE (Inches)	CLASS III (feet)	CLASS IV (feet)	CLASS V (feet)
15	11	16	25+
18	11	16	25+
21	11	16	25+
24	11	17	25+
30	11	17	25+
36	11	17	26+
42	11	17	27+
48	12	17	27+
54	12	17	27+
60	12	17	27+
66	12	17	27+
72	12	18	27+
78	12	18	28+
84	13	18	28+
90	13	18	28+
96	13	18	28+
102	13	18	28+
108	13	18	28+
114	13	18	28+
120	13	18	28+
144	13	18	28+

* For installation with greater fill heights, see Concrete Pipe Design Manual.
** Fill heights are valid only for Class B bedding installation

VERSION: JANUARY 2006	REFERENCE: .	TABLE CH14-T102
		MAXIMUM FILL HEIGHTS FOR RCP

PVC (46 psi Pipe Stiffness) Pipe

Embedment Class	Material Description	% of Proctor Density Range	Recommended Maximum Height of Fill (feet)
II	Sand and Gravel Soils – Clean	90-100	30+
		85	30+
		80	24
I, III, IV, and V		Not Allowed	

STORM SEWER ENERGY LOSS COEFFICIENTS

(A) EXPANSIONS
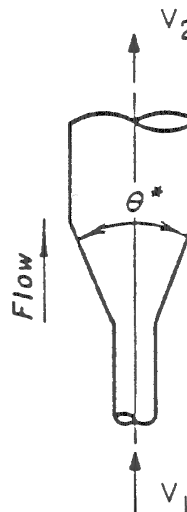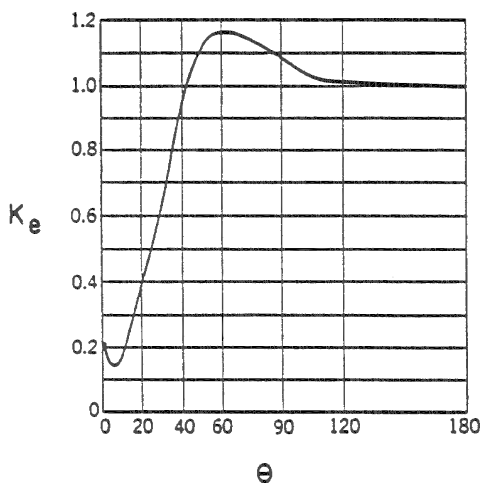
* The angle Θ is the angle in degrees between the sides of the tapering section.

(B) CONTRACTIONS

A_2/A_1	K_c
0.1	0.36
0.2	0.34
0.3	0.31
0.4	0.27
0.5	0.22
0.6	0.16
0.7	0.11
0.8	0.05
0.9	0.01
1.0	0.00
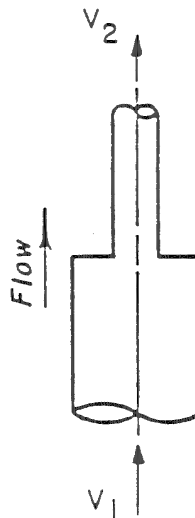
NOTE: Losses due to angular expansions and abrubt contractions will normally only occur in retro−fit storm sewer design.
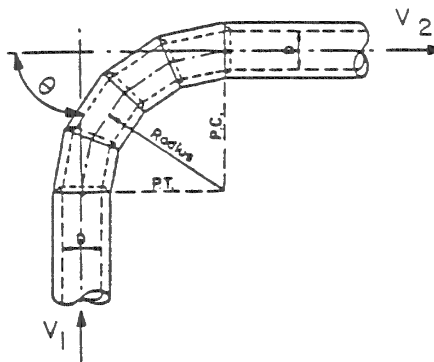
VERSION: JANUARY 2006	REFERENCE: (A) FLUID MECHANICS, DAUGHERTY AND FRANZIT, 1977 (B) FLUID MECHANICS, STREETER AND WYLIE, 1979	**TABLE CH14-T104A** STORM SEWER ENERGY LOSS COEFFICIENTS

STORM SEWER ENERGY LOSS COEFFICIENTS

(C) BENDS

I. Large Radius Bends
   (Pipe Diameter > Bend Radius)

$$K_b = 0.25 \ (\theta/90)^{0.5}$$

θ	K_b
90°	0.25
60°	0.20
45°	0.18
30°	0.14

Note: Head loss applied at P.C.

II. Sharp Radius Bends
    (Pipe Diameter = Bend Radius)

θ	K_b
90°	0.50
60°	0.43
45°	0.35
30°	0.25

Note: Head loss applied at entrance to bend.

VERSION: JANUARY 2006	REFERENCE:	TABLE CH14-T104B
	"URBAN STORMWATER MANAGEMENT," APWA SPECIAL REPORT NO. 49, 1981	STORM SEWER ENERGY LOSS COEFFICIENTS
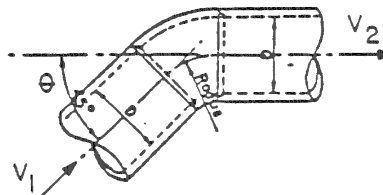
STORM SEWER ENERGY LOSS COEFFICIENTS

(D) JUNCTIONS
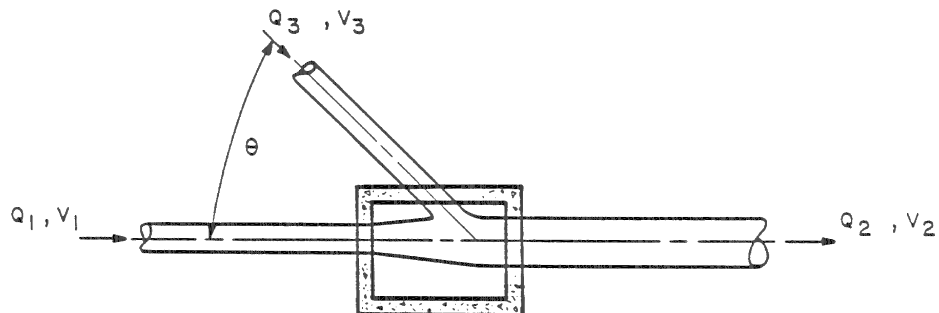
θ	K_j
90°	0.25
60°	0.35
45°	0.50
30°	0.65
15°	0.85

NOTE: Head loss applied at exit of junction

VERSION: JANUARY 2006	REFERENCE:	**TABLE CH14-T104C**
	"URBAN STORMWATER MANAGEMENT," APWA SPECIAL REPORT NO. 29, 1981	STORM SEWER ENERGY LOSS COEFFICIENTS

TYPE OF UTILITY CROSSING	HORIZONTAL SEPARATION (feet)	VERTICAL SEPARATION (inches)
Sanitary	10	18
Water	10	18
Electric	5 to 10	18
Cable – Phone, TV, etc.	5 to 10	18
Gas	5 to 10	18
Irrigation Ditch Laterals	10	18

STORM SEWER DESIGN AND ANALYSES PARAMETERS

A. MANHOLE SIZE

EQUIVALENT PIPE SIZE (INCHES)	MANHOLE DIAMETER
15 TO 18	4'
21 TO 42	5'
48 TO 54	6'
60 AND LARGER	4' WITH BOX BASE

B. MANHOLE SPACING

EQUIVALENT PIPE SIZE (INCHES)	MAXIMUM ALLOWABLE DISTANCE BETWEEN MANHOLES
15 TO 36	400'
42 AND LARGER	500'

C. MAXIMUM ALLOWED DEFLECTION FOR PULLED JOINT CONSTRUCTION

PIPE DIAMETER (SPAN) (INCHES)	ALLOWED DEFLECTION (PULL) PER JOINT (INCHES)
8 TO 33	1/2
36 TO 54	5/8
60 TO 78	3/4
84 TO 102	7/8
108 TO 144	1

VERSION: JANUARY 2006	REFERENCE: .	**TABLE CH14-T106**
		MANHOLE SIZE

ALLOWABLE INLET CAPACITY

CONDITION	INLET TYPE	PERCENTAGE OF THEORETICAL CAPACITY ALLOWED
SUMP OR CONTINUOUS GRADE	TYPE R (CH14−SD101) 5' LENGTH 10' LENGTH 15' LENGTH	88 92 95
SUMP OR CONTINUOUS GRADE	GRATED TYPE 13 (CH14−SD103)	50
CONTINUOUS GRADE	COMBINATION TYPE 13 (CH14−SD104)	66
SUMP	GRATED TYPE C (CH14−SD102)	50
SUMP	COMBINATION TYPE 13 (CH13−SD104)	65

CHAPTER 14

STORM DRAIN SYSTEMS

SECTION 2

STREETS

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STREETS

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CHAPTER 14

STORM DRAIN SYSTEMS

SECTION 2
STREETS

2.1   INTRODUCTION

The use of streets to convey storm runoff, although naturally occurring, interferes with the primary function of the street for transportation purposes. Streets are, however, an important component in the storm drainage system due to their large storm runoff carrying capacity obtained for little or no additional drainage related costs. In order to balance these two competing street uses, limits on the amount of flow conveyed in streets are required based on the street classifications (i.e., local, collector, arterial, etc.) related to emergency usage during flood events.

The recommended street flow conveyance and depth design limitations are provided in Section 2, Chapter 3 of this manual. The criteria presented in this section should be used in the evaluation of the allowable storm flow encroachment within public streets.   The review of all planning submittals (refer to Chapter 5, Section 3) will be based on the criteria provided herein.

> *The recommended street flow conveyance and depth design limitations are provided in Section 2, Chapter 3 of this manual.*

2.2   FUNCTION OF STREETS IN THE DRAINAGE SYSTEM

Urban and rural streets with curb and gutter facilities or roadside ditches are part of both minor and major storm drainage systems (Section 2, Chapter 3). When the storm runoff conveyed in the street exceeds the allowable conveyance limits, a storm drain system (Section 1, Chapter 14) or an open channel (Section 1, Chapter 13) is required to convey the excess flows.

The primary function of the urban streets is for traffic movement and, therefore, the drainage function is subservient and should not substantially interfere with the traffic function of the street. Design criteria for the collection and conveyance of runoff water on public streets are based on a reasonable frequency and magnitude of traffic interference. That is, depending on the character of the street, certain traffic lanes can be inundated during the major design storm return period. During less

intense storms, runoff will also inundate traffic lanes, but to a lesser degree. The primary function of the streets for the Minor Storm Drainage System is therefore to convey the nuisance flows to storm drain or open channel drainage facilities with minimal interference to traffic movements. For the Major Drainage System, the function of the streets is to provide an emergency passageway for the flood flows with minimal damage to the urban environment.

2.3   DRAINAGE IMPACT ON STREETS

Storm runoff can influence the traffic function of a street in the following ways:

- Sheet flow across the pavement resulting from precipitation runoff.
- Runoff in the gutter.
- Duration of the storm.
- Ponded water.
- Flow across traffic lanes.
- Deterioration of the street.

To minimize the impact of storm runoff on streets, each of the above factors should be understood and controlled to within acceptable limits. The effects of the above factors are discussed in the following sections.

2.3.1   SHEET FLOW

Rainfall on the paved surface of a street or road must flow overland in what is referred to as sheet flow until it reaches a channel. For streets with curb and gutter, the street acts as the channel, while on roads which have a drainage ditch, the ditch acts as the channel. In situations where the street is not inundated due to runoff originating from upgradient, the depth of sheet flow will be essentially zero at the crown of the street and will increase in the direction of the curb and gutter or drainage ditch.

Traffic interference due to sheet flow is by hydroplaning or by splash. Hydroplaning is the phenomenon of vehicle tires becoming supported by a film of water, which acts as a lubricant between the pavement and the vehicle. This phenomenon generally occurs at higher speeds associated with arterials and freeways and can result in loss of vehicle control. Drainage design can reduce the hydroplaning potential by increasing the street cross slope, which drains the runoff more quickly.

Splashing of the sheet flows interferes with traffic movement by reducing visibility. The increase in cross slope of the street crown also reduces the splash potential. The cross slope should be kept within acceptable limits to prevent the sideways slipping of traffic during icy conditions. In general, a 2 percent cross slope is a desirable practical slope.

2.3.2   GUTTER FLOW

Water which enters a street as sheet flow from the pavement surface or as overland flow from adjacent land area will flow in the gutter of the street until reaching some outlet, such as a storm drain inlet or a channel. As the flow progresses downhill and additional areas contribute to the runoff, the width of

flow will increase and progressively infringe upon the traffic lane. If vehicles are parked adjacent to the curb, the flow width will have little influence on traffic capacity until it exceeds the width of the vehicle by several feet. However, on streets where parking is not permitted, the flow width significantly effects traffic movement after exceeding the width of the gutter by a few feet, because the flow encroaches on a moving lane rather than a normal parking lane. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow. This creates a traffic hazard, which contributes to the rash of small accidents that occur during rainstorms.

As the flow width increases, the traffic must eventually move through the inundated lanes progressively reducing traffic movement as the depth of flow increases. Whereas some drainage effects on traffic movement are acceptable, emergency vehicles (i.e., fire equipment, ambulances, police vehicles) must be able to travel the streets, such as by moving along the street crown. Therefore, certain limitations on the depth of flow in the street are required.

2.3.3 PONDING

Storm runoff ponded on the street, due to grade changes or intersections, effects traffic movement by increasing flow depths and the duration of flow at greater depths. Ponding is also localized and vehicles may enter the ponded area at high speeds unaware of the problem until too late. Ponding will often bring traffic to a complete halt to negotiate the ponded area without stalling the vehicle, resulting in reduced traffic movement. Therefore, depths of ponding should be controlled similar to gutter flow and in some cases eliminated on high traffic volume streets.

2.3.4 CROSS FLOW

Whenever storm runoff, other than sheet flow, moves across a traffic lane, traffic flow is affected. The cross flow may be caused by super-elevation of a curve, by the intersection of two streets, by exceeding the capacity of the higher gutter on a street with cross fall, or simply poor street design. The problem associated with this type of flow is the same as for ponding in that it is localized in nature and vehicles may be traveling at a high speed when they reach the location. If the speed limits are slow and the traffic volume is light, then the influence of cross street flow may be within acceptable limits.

2.4 DRAINAGE IMPACT ON STREET MAINTENANCE

The use of the roadway system for drainage of runoff during and immediately after storm events also has an impact on the structural integrity of the pavement system and the roadway maintenance requirements. If water penetrates the road surface and saturates the sub-grade material, the sub-grade may fail and cause failure of the pavement.

Additionally, runoff from rural and urban areas can carry large amounts of debris and sediment, which may reduce the performance of hydraulic structures of become a safety hazard and should be remove.

2.4.1 BITUMINOUS PAVEMENTS

The efficient removal of storm runoff from pavement surfaces has a positive effect on street maintenance, and street maintenance procedures can in turn affect the efficiency of a street as a runoff carrier. Research has indicated that pavement deterioration is accelerated by the presence of storm runoff.

Pavement surfaces are subject to numerous types of distress such as weathering, raveling, long cracks, alligator cracks, chuck holes, bleeding, depression and edge breakup. Water is probably the greatest cause of distress in a pavement structure. Flow of water across a bituminous pavement surface has little effect on the pavement so long as the pavement retains its watertight condition. A number of types of pavement distress may cause the pavement to become permeable, allowing water to reach the sub-grade. Once the water reaches the sub-grade the problems multiply as the sub-base and sub-grade weakens and increases the cracks through the surface.

A common practice to reduce the problem of bituminous surface deterioration is to seal coat or overlay the surface. This reduces the problem of pavement deterioration, but indirectly creates a problem with the carrying capacity of the adjacent gutter. As the street section is resurfaced, the flow area of the section is decreased. Over a period of 20 to 30 years, a considerable portion of the runoff carrying capacity of the street may be lost.

A practical, inexpensive solution to the problem of overlaying streets has not been developed. Scarifying the surface to remove the upper layer of asphalt before applying the next overly minimizes the problem, but the method is expensive. In any case, the gutter capacity must be maintained or additional drainage facilities (i.e., inlets and storm sewers) added to the system.

2.4.2 CURB AND GUTTER

The break-up of pavement adjacent to the gutter is a problem recognized by both traffic and drainage engineers. The character of this problem varies from cracking and potholes to actual peeling of long sections of pavement during high flows. The damage is basically caused by intrusion of water into the sub-base through the interface between the pavement and the gutter. Poor bonding of the pavement to the gutter concrete, combined with shrinkage of the pavement, results in a crack. Even small amounts of water from the pavement surface are intercepted by the crack. During the high flow periods, larger quantities of curb flow and ponded water will pass through the crack into the sub-base. These factors result in almost continuous wetting of the sub-grade adjacent to the gutter face, failure of the sub-grade and deterioration of the pavement.

Several theories have been suggested to explain the peeling of pavement surfaces when subjected to storm runoff. These theories include consideration of tractive force, the washing out of fines from the supporting materials and the uplift forces resulting from conversion of velocity head to pressure head when flowing water is trapped under the pavement surface. Although all these factors contribute to pavement peeling, uplift is the most

significant. Taking measures to prevent water from getting under the pavement can minimize uplift forces. Sealing the space with crack filler may prevent the intrusion of water between the pavement and gutter face.

2.4.3 SEDIMENTATION AND DEBRIS

A common problem occurs when sediment and debris carried by high velocity flows settle out on the street as the velocity decreases. As sediment and debris build up, the flow carrying capacity of the street section is reduced, causing increased flow encroachment into the traffic lanes. The degree to which this occurs is dependent upon the amount of debris and settleable solids in the runoff as well as the magnitude of the flow velocity.

Additionally, sediment and other debris carried by runoff can impair the operation of hydraulic structures such as curb inlets and grated drop inlet structures. The sediment and debris can block a portion of the flow area into these facilities and cause artificially increased water surface elevations.

Immediately after a storm event, identified problem areas should be reviewed and street sweeping should be initiated to remove accumulated sediment and debris. By regularly scheduled sweeping of upstream areas, the source of some of the sediment can be eliminated. Also, runoff from construction sites may cause site-specific sedimentation problems and should be controlled as recommended in Chapter 15.

2.5 STREET CLASSIFICATION AND ALLOWABLE FLOW DEPTH

Public streets are generally classified as Local, Collector, or Arterial depending on the volume of traffic and right of way widths. The recommended allowable storm flow depth criteria for both minor and major storm events are provided in Section 2, Chapter 3 of this Manual.

> Public streets are generally classified as Local, Collector, or Arterial .

Calculation of the water surface elevation and velocity must be based on limiting the flow to the width of the street section (to the back of curb). This implies that, for calculation purposes only, an infinitely high vertical wall exists at the back of curb and any flow area outside of the curb is not considered in the analysis. This provides a conservative analysis for street capacity requirements. In addition, whenever flow depths are such that crown overtopping would occur, the one-half street calculations assume a vertical wall at the street crown with no associated wetted perimeter.

For street sag locations, provisions must be included to carry the 100-year runoff in a pipe or an overflow section and include an access and maintenance easement.

2.6   HYDRAULIC EVALUATION

The hydraulic analysis of flow in street sections is similar to open channel flow analysis for larger flood control channels (Section 1, Chapter 13). The basic governing equation, Manning's Equation, is as follows:

$$Q = \frac{1.49}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}}$$ (Eq. CH14-201)

Where Q = Flow rate (cubic feet per second, cfs)
       n = Roughness coefficient (0.016 for asphalt streets)
       A = Area (square feet, sf)
       P = Wetted perimeter (feet)
       R = A/P = Hydraulic radius (feet)
       S = Slope of the energy grade line (feet/feet)

Based upon the allowable storm flow depth criteria in Section 2, Chapter 3, the allowable storm capacity of the minor storm of each street section is calculated using the above Equation CH14-201. The calculation of depth of flow for the major storm event is also based on Equation CH14-201. For the calculation of flow depth and velocity, the area outside the back of curb should not be considered in the calculation of conveyance.

Streets with grades flatter than 0.5% must be given special consideration when calculating allowable flow depth. These streets are subject to ponding and are candidates for storm drains. Detailed discussions on storm drain systems are provided in Section 1, Chapter 14.

2.7   DESIGN CONSIDERATIONS

2.7.1   CROSSPANS

Where storm sewer systems are not justified, crosspans may be installed to transport runoff across local streets. Crosspans should be a minimum of six (6) feet wide. Larger widths may be required by the local jurisdictions. The minimum grade on crosspans should be 0.5% at the flowline of the pan and the maximum flow depth in the crosspan for the minor and major storm events should be 0.5 feet at the flowline. Mid-block crosspans are also allowed only across local streets and their design is governed by the same criteria mentioned above.

No crosspans are allowed across arterial or collector streets except in unusual cases when approved by the local jurisdiction. When used on arterial or collector streets, minimum width should be ten (10) feet. Crosspan approaches shall be designed in accordance with the local jurisdictions criteria standards. Covered crosspans, notched crosspans and bubblers should not be allowed.

At the intersection of two streets, any variation in grade should be governed by the characteristics of the drainage, the traffic patterns for that intersection and the design requirements of the local jurisdiction.

2.7.2 STREET SUMP LOCATIONS

For street sump locations, provisions should be included to carry the 100-year runoff in a pipe or an overflow section to an outfall so the water surface depth criteria provided in Section 2, Chapter 3 is not violated. Necessary access and maintenance easement should be provided, if the facility is not contained within the street ROW.

2.8 RURAL ROADS

Roadside ditch may be used in place of curb and gutter, when determined appropriate by the local jurisdiction for rural roads. When deciding to use roadside ditches, the following elements should be considered:
- Vehicle and human safety issues
- Sediment and debris deposition problems
- Private driveway crossings
- Scour and erosion problems
- Maintenance excess

2.7.1 ROADSIDE DITCHES

The criteria for the design of roadside ditches are similar to the criteria for grass-lined open channels (Chapter 13, section 1) with modifications for the special purpose of minor storm drainage conveyance. The recommended minimum criteria is as follows:

- Capacity
    - Roadside ditches should have adequate capacity to confine and convey the minor storm runoff peaks. For the minor design storm, the maximum flow depth in the ditch should not exceed 18-inches. The street flow depths criteria outlined in Section 2, Chapter 3 for major storm event should be used.
- Flow velocity
    - The maximum velocity for the minor storm flood peak should not exceed 5 feet per second (fps). The major storm flow velocity should not exceed 6 fps.
- Longitudinal slope
    - The allowable ditch velocity of the minor and major storm flood peaks should limit the ditch slope. Check or drop structures may be required where the ditch flow velocity exceeds the set limitation.
- Freeboard
    - No freeboard is required.
- Curvature
    - The minimum radius of curvature should be 25 feet.
- Roughness Coefficient
    - Manning n values presented in Figure CH13-T102 should be used for the capacity computation of roadside ditches.

- Grass Lining
  - ○ The grass lining should be in accordance with Chapter 13, Section 1.
- Driveway Culverts
  - ○ Driveway culverts should be sized to pass the minor storm ditch flow capacity without overtopping the driveway. The minimum size culvert should be a 22" x 13" CMPA (18" equivalent round pipe) with flared end sections. More than one culvert may be required.

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