CITY OF FARMERSVILLE, TEXAS

MANUAL

FOR THE DESIGN OF

STORM DRAINAGE SYSTEMS

Subdivision Ordinance #2007-08
Exhibit A
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I - INTRODUCTION

1.1 GENERAL

Storm water runoff is that portion of the precipitation that flows over the ground surface during and for a period after a storm. The objective of designing storm sewer systems is to convey runoff in a functional and efficient way from places it is not wanted to the nearest acceptable discharge point. This transfer of runoff is done in sufficient time and methods to avoid damage and unacceptable amounts of inconvenience to the general public. Prior to the design of a storm drainage system, an overall drainage plan shall be submitted to the City for review. Upon written approval of the drainage plan by the City, the actual construction plans can be designed.

This manual provides guidelines for design of storm drainage facilities in the City of Farmersville. The procedures outlined herein shall be followed for all drainage design and review of plans submitted to the City.

1.2 SCOPE

The information included in this manual has been developed through a comprehensive review of basic design technology as published in various sources listed in the Bibliography and as developed through the experience of individual Engineers who have contributed to the content.

The manual concerns itself with storm drainage conditions that are generally relative to the City of Farmersville and the immediate geographical area. Accepted engineering principles are applied to these situations in detailed documented procedures. The documentation of the procedures is not intended to limit initiative but rather is included as a standardized procedure to aid in design and as a record source for the City.

1.3 ORGANIZATION OF MANUAL

This manual is divided into six basic sections. The first section is the INTRODUCTION, which is a general discussion of the intended use of the material and an explanation of its organization.

Section II: DRAINAGE DESIGN THEORY, explanation of the basic technical theory employed by the design procedures prescribed in this manual.

Section III: CRITERIA AND DESIGN PROCEDURES, lists recommended design criteria and outlines the design procedures followed by the City of Farmersville.

Section IV: CONSTRUCTION PLAN PREPARATION, describes construction plans for drainage facilities in the City of Farmersville.

Section V: APPENDIX, contains a definition of terms, definition of symbols and abbreviations and the Bibliography.

Section VI: TABLES, contains all the tables which are used in the design of drainage facilities.

Section VII: FIGURES, contains all of the basic graphs, nomographs and charts for use in design of drainage facilities.

Section VIII: FORMS, contains forms with detailed instructions for their use.

II - DRAINAGE DESIGN THEORY

2.1 GENERAL

This section covers the technical theory utilized in the design procedures outlined in the manual. It is intended as an application of basic hydraulic and hydrologic theory to specific storm drainage situations.

2.2 DRAINAGE AREA DETERMINATION AND SYSTEM DESIGNATION

The size and shape of each drainage area and sub-area must be determined for each storm drainage facility. This size and shape should be determined from topographic maps at scale of 1 inch = 200 feet.

Where the contour interval is insufficient or physical conditions may have changed from those shown on existing maps, it may be necessary to supplement the maps with field topographic surveys. The actual conditions should always be verified by a reconnaissance survey. In preparing the drainage area maps, careful attention must be given to the gutter configurations at intersections. The direction of flow in the gutters should be shown on the maps and on the construction plans. The performance of these surveys is the responsibility of the Engineer designing the drainage facility.

2.3 RAINFALL

FIGURE 1, which shows anticipated rainfall rates for storm durations from 5 minutes to 6 hours, has been prepared utilizing the information contained in the U. S. Department of Commerce, Weather Bureau, HYDRO-35 (National Technical Information Service Publication No. PB272-112, dated June, 1977). Interpolation of rainfall rates versus durations from the isopluvial maps contained in HYDRO-35 were used to prepare FIGURE 1 for durations less than 60 minutes. For durations beyond 60 minutes the information shown in FIGURE 1 was derived from Weather Bureau Technical Paper No. 40, dated May, 1961.

2.4 DESIGN STORM FREQUENCY

The individual curves shown on FIGURE 1 labeled "5 Yr.", "10 Yr.", "25 Yr.", "50 Yr.", and "100 Yr." are referred to as "Design Storm Frequency". The term "100-year storm" does not mean that a storm of that severity can be expected once in any 100-year period, but rather that a storm of that severity has a one in one hundred chance of occurring in any given calendar year.

Each storm drainage facility shall be designed to convey the runoff which results from the 100-year design storm as shown in Section III, CRITERIA AND DESIGN PROCEDURES.

2.5 DETERMINATION OF DESIGN DISCHARGE

Prior to hydraulic design of drainage facilities the amount of runoff from the particular drainage area must be determined. The Rational, the Unit Hydrograph, and the HEC-I Computer Program are the accepted methods, for computing volumes of storm water runoff. Data from the Flood Insurance Study shall be used in lieu of Rational Method, Unit Hydrograph or HEC-I for determination of drainage and floodway easement elevations and design discharge flows, if such data is available. However, all discharge values shall be based on full development of the drainage basin as outlined on the current zoning maps available from the City. In the event that the Flood Insurance Study is not based on current zoning, the study should be reanalyzed, revised and submitted to FEMA for acceptance. In the event that the revised study indicates a water surface is less than that shown on the Flood Insurance study the higher value shall be used if the study is not submitted to FEMA.

2.6 RATIONAL METHOD

The use of the Rational Method, introduced in 1889, is based on the following assumptions:

a) The peak rate of runoff at any point is a direct function of the average rainfall intensity during the time of concentration to that point.

b) The frequency of the peak discharge is the same as the frequency of the average rainfall intensity.

c) The time of concentration is the time required for the runoff to become established and flow from the most remote part of the drainage area to the design point.

The Rational Method is based on the direct relationship between rainfall and runoff expressed in the following equation:

$$Q = C\,I\,A, \text{ where}$$

- "Q" is the storm flow at a given point in cubic feet per second (c.f.s.).
- "C" is a coefficient of runoff representing the ratio of runoff to rainfall.
- "I" is the average intensity of rainfall in inches per hour for a period equal to the time of flow from the farthest point of the drainage area to the point of design and is obtained from FIGURE 1.
- "A" is the area in acres that is tributary to the point of design.

The determination of the factors, runoff coefficient and time of concentration shown in this manual have been developed through past experience in the City's system and by review of values recommended by others.

2.7 RUNOFF COEFFICIENT

The runoff coefficient "C" in the Rational Formula is dependent on the character of the soil and the degree and type of development in the drainage area. The nature and condition of the soil determine its ability to absorb precipitation. The absorption ability generally decreases as the duration of the rainfall increases until saturation occurs. Infiltration rates in the Farmersville area generally are low due to the cohesive soils.

As a drainage area develops the amount of runoff increases generally in proportion to the amount of impervious areas such as streets, parking areas and buildings.

2.8 TIME OF CONCENTRATION

The time of concentration is defined as the longest time that will be required for water to flow from the upper limit of a drainage area to the point of concentration, without interruption of flow by detention devices. This time is a combination of the inlet time, which is the time for water to flow over the surface of the ground from the upper limit of the drainage area to the first storm sewer inlet, and the flow time in the conduit or channel to the point of concentration. The flow time in the conduit or channel is computed by dividing the length of the conduit by the average velocity in the conduit.

Although the basic principles of the Rational Method are applicable to all sizes of drainage areas, natural retention of flow and other interruptions cause an attenuation of the runoff hydrograph resulting in over-estimation of rates of flow for larger areas. For this reason, in development of runoff rates in larger drainage areas, use of the Unit Hydrograph Method is recommended.

2.9 UNIT HYDROGRAPH METHOD

The Unit Hydrograph Method to be used in calculation of runoff shall be in accordance with Snyder's synthetic relationships.

The computation of runoff quantities utilizing the Unit Hydrograph Method is based on the following equations:

$$t_p = C_t \; (L \; L_{ca})^{0.3}$$

$$q_p = \frac{C_p^{\;640}}{t_p}$$

$$Qp = q_p \, A$$

$$S_D = I \;\; 2$$

$$R_T = S_D \; - \; L_{is}$$

$$Q_u = R_T \;\; Q_p$$

- "t_p" is the lag time, in hours, from the midpoint of the unit rainfall duration to the peak of the unit hydrograph.

- "C_t" and "C_p640" are coefficients related to drainage basin characteristics. Recommended values for these coefficients are found in TABLE 2.

- "L" is the measured stream distances in miles from the point of design to the upper limit of the drainage area.

- "L_{ca}" is the measured stream distance from the point of design to the centroid of the drainage area. This value may be obtained in the following manner:

Trace the outline of the drainage basin on a piece of cardboard and trim to shape. Suspend the cardboard before a plumb bob by means of a pin near the edge of the cardboard and draw a vertical line. In a similar manner, draw a second line at approximately a 90-degree angle to the first line. The intersection of the two lines is the centroid of gravity of the area.

- "q_p" is the peak rate of discharge of the unit hydrograph for unit rainfall duration in cubic feet per second per square mile.

- "Q_p" is the peak rate of discharge of the unit hydrograph in cubic feet per second.

- "A" is the area in square miles that is tributary to the point of design.

- "I" is the rainfall intensity at two hours in inches per hour for the appropriate design storm frequency.

- "S_D" is the design storm rainfall in inches for a two-hour period.

- "L_{is}" is the initial and subsequent losses, which have a recommended constant value of 1.11 inches.

- "R_T" is the total runoff in inches.

- "Q_u" is the design storm runoff in cubic feet per second.

2.10 UNIT HYDROGRAPH COEFFICIENTS

The U. S. Army Corps of Engineers published, in August 1952, a report, which contains observed unit hydrographs from records on several storms, which occurred during the period from May 1948 through May 1950 on the Turtle Creek drainage basin. Data developed in that report, which is entitled "Definite Project Report on Dallas Floodway, Volume I - General, Hydrologic and Economic Data, together with additional measurements made since that time, was used to establish the coefficients for the Farmersville area.

In Section III of the manual, certain values for factors involved in a unit hydrograph analysis are recommended. These values are not to be considered inflexible, but are intended as guidelines when more specific data is not available. Detailed review of the development of all these factors is not warranted, but several should be discussed where the documentation for the selected values might not be apparent.

The recommended rainfall intensity to be used is selected based on a duration of two hours. The two hours are representative of the time elapsed from the beginning of the rainfall to the peak rate of runoff. Where more definite relationships are known to exist on any particular stream, this time should be adjusted accordingly. When using a duration of two hours, multiply the rainfall rate (intensity) by two hours, subtract the losses, and the total runoff is obtained.

There are two losses to be considered when arriving at the total runoff. These are termed the "initial" and "subsequent" losses and are shown in Section III, CRITERIA AND DESIGN PROCEDURES, as having a constant value of 1.11 inches. This is arrived at by assigning a value of 0.75 inches as the total initial loss occurring during the first one-half hour of rainfall and a loss of 0.24-inch per hour for the remaining one and one-half hour rainfall period, calculated as follows:

Initial Loss	0.75 inch
Subsequent Loss (1.5 hrs x 0.24 inch/hr)	0.36 inch
Total Losses	**1.11 inches**

As in the case of other recommended specific values, where more definite information is available, it should be used.

2.11 FLOW IN GUTTERS AND INLET DESIGN

In the design of storm drainage facilities, the geometrics of specific types of streets are an integral part of drainage design. Throughout this manual references is made to certain types and widths of streets with specific crown characteristics. The following terms are defined for reference purposes:

MAJOR THOROUGHFARE: A street that moves traffic from one section of the city to another section.

COLLECTOR STREET: This is a street that has the dual purpose of traffic movement plus providing access to abutting properties.

RESIDENTIAL STREET: A street whose primary function is to provide local access to abutting properties.

WIDTH OF STREET: The horizontal distance between the faces of the curbs.

STRAIGHT CROWN: A constant slope from one gutter flow line across a street to the other gutter flow line.

PARABOLIC CROWN: A pavement surface shaped in a parabola from one gutter flow line to the other.

VERTICAL DISPLACEMENT BETWEEN GUTTER FLOW LINES: Due to topography, it will be necessary at times that the curbs on a street be placed at different elevations.

2.12 STRAIGHT CROWN STREETS

Storm water flow in a street having a straight crown slope may be expressed as follows:

$$Q = 0.56 \frac{Z}{n} S^{1/2} Y^{8/3} \quad \text{(Equation 1)}$$

- "Q" is quantity of gutter flow in cubic feet per second.
- "Z" is the reciprocal of the crown slope.
- "n" is the coefficient of roughness as used in Manning's Equation; a value of 0.0175 was used.
- "S" is the longitudinal slope of the street gutter in feet per foot.
- "Y" is the depth of flow in the gutter at the curb in feet.

This formula is an expression of Manning's Equation as referenced in Highway Research Board Proceedings, 1946, Page 150, Equation 14.

Based on this equation, FIGURE 3 was prepared and inlet design calculations, as explained elsewhere, were made.

2.13 PARABOLIC CROWN STREETS

FIGURES 4 and 5 show the capacity of gutters in streets with parabolic crowns. The following formulas can be used for determining the gutter capacity or refer to the figures for solution.

$$Q = \frac{1.486 \quad AR^{2/3} \quad S^{1/2}}{n} \qquad \text{(Equation 2)}$$

$$R = \frac{A}{P} \qquad \text{(Equation 3)}$$

$$A = \frac{W_o \; C_o}{2} - \frac{8\,C_o}{W_o^2} \int_{o}^{\frac{W_o}{2}} X^2 \, dx \qquad \text{(Equation 4)}$$

- "Q" is quantity of gutter flow in cubic feet per second.
- "n" is the coefficient of roughness; a value of 0.0175 was used.
- "A" is the cross section flow area in square feet.
- "R" is the hydraulic radius in feet.
- "S" is the longitudinal slope of the street gutter in feet per foot.
- "P" is the wetted perimeter in feet.
- "W_o" is the width of the street in feet.
- "C_o" is the crown height of the street in feet.

As discussed in Section III, CRITERIA AND DESIGN PROCEDURES, it may, at times, be necessary for one curb to be at a different elevation than the opposite curb due to the topography. Where parabolic crowns are involved, the gutter capacities will vary radically as one curb becomes higher or lower. The maximum vertical displacement values shown in FIGURES 4 and 5 were developed based on a minimum depth of flow in the high gutter of approximately two inches.

2.14 ALLEY CAPACITY

FIGURE 6, CAPACITY OF ALLEY SECTIONS, was prepared based on solution of Manning's Equation:

$$Q = \frac{(1.486)}{n} (AR^{2/3}) (S^{1/2}) \qquad \text{(Equation 2)}$$

- "Q" is the alley capacity, flowing full, in cubic feet per second.
- "n" is the coefficient of roughness; a value of 0.0175 was used.
- "A" is the cross section flow area in square feet.
- "R" is the hydraulic radius in feet.
- "S" is the longitudinal slope in feet per foot.

2.15 INLET CAPACITY CURVES

The primary objective in developing the curves shown in FIGURES 8 through 22 was to provide the Engineer with a direct method for sizing inlets that would yield answers within acceptable accuracy limits.

2.16 RECESSED AND STANDARD CURB OPENING INLETS ON GRADE

The basic curb opening inlet capacity curves, FIGURES 8 through 12, Recessed and Standard Curb Opening Inlets on Grade, were based upon solution of the following equation:

$$L = \frac{Q\ (H_1 - H_2)}{(H_1^{5/2} - H_2^{5/2})\ (.70)} \qquad \text{(Equation 6)}$$

- "L" is the length of inlet, in feet, required to intercept the gutter flow.
- "Q" is the gutter flow in cubic feet per second.
- "H_1" is the depth of flow, in feet, in the gutter approaching the inlet plus the inlet depression, in feet.
- "H_2" is the inlet depression, in feet.

This is an empirical equation from "Hydraulic Manual", Texas Highway Department, dated September 1970. The data from solution of this equation was used to plot the curves shown on FIGURES 8 through 12.

2.17 RECESSED AND STANDARD CURB OPENING INLETS AT LOW POINT

FIGURE 13, Recessed and Standard Curb Opening Inlets at Low Point, was plotted from the solution of the following equation:

$$Q = 3.087\,L \quad h^{3/2} \qquad \text{(Equation 7)}$$

- "Q" is the gutter flow in cubic feet per second.
- "L" is the length of inlet, in feet, required to intercept the gutter flow.
- "h" is the depth of flow, in feet, at the inlet opening. This is the sum of the depth of the flow in the gutter, y_O, plus the depth of the inlet depression.

This equation expresses the capacity of a rectangular weir and is referenced in "The Design of Storm Water Inlets," dated June 1956, The John Hopkins University.

The calculated inlet capacities were reduced by ten percent of the preparation of FIGURE 13 due to the tendency of inlets at low points to clog from the collection of debris at their entrance.

2.18 COMBINATION INLET ON GRADE

FIGURES 14 through 16, Combination Inlet on Grade, were prepared based on the length of grade in feet, L_O, required to capture the portion of the gutter flow which crosses the upstream side of the grade and on the length of grate in feet, L', required to capture the outer portion of gutter flow. The figures were prepared with the solution of Equation 1 and the following equations:

$$L_o = 4v_o \left(\frac{Y_o}{g}\right)^{1/2} \qquad \text{(Equation 8)}$$

$$L' = 1.2\ v_o \quad \tan\ 0o \left(\frac{y_o\ -\ \tan^{w} 0}{g}\right)^{1/2} \qquad \text{(Equation 9)}$$

$$q_2 = \frac{L'\ -\ L}{4}\ (g)^{1/2} \left(y_o\ -\ \frac{w}{\tan\ 0o}\right)^{3/2} \qquad \text{(Equation 10)}$$

$$q_3 = Q_o \left(1 - \frac{L^2}{Lo^2}\right)^2 \qquad \text{(Equation 11)}$$

$$Q = Q_o - \left[q_2 + q_3\right] \qquad \text{(Equation 12)}$$

L_o = Length of grate required to capture 100% of all flow over grate in feet.

v_o = Gutter velocity in feet per second.

y_o = Depth of gutter flow in feet.

g = Gravitational acceleration (32.2 feet per second per second).

L' = Length of grate required to capture the outer portion of the gutter flow in feet.

0_o = Crown slope of pavement.

w = Width of grate in feet.

q_2 = Carry-over flow in c.f.s. outside of the grate.

L = Length of grate in feet.

q_3 = Carry-over flow in c.f.s. over the grate.

Q_o = Gutter flow in c.f.s.

Q = Capacity of grate inlet in c.f.s.

These equations are from "The Design of Storm Water Inlets," The John Hopkins University, June 1956.

2.19 COMBINATION INLET AT LOW POINT

FIGURE 20, Combination Inlet at a Low Point, was prepared based on the inlet having a capacity equal to 90 percent of the quantity derived from solution of Equation 7 (Paragraph 2.17) and 70 percent of the quantity derived from solution of the following Equation 13:

$Q = 3.087\ Lh^{3/2}$ (Equation 7)

$Q = 0.6A \sqrt{2gh}$ (Equation 13)

- "Q" is the gutter flow in cubic feet per second.
- "A" is the net cross section area, in square free, of the grate opening.
- "g" is gravitational acceleration (32.2 feet per second per second).
- "h" is the head, in feet on the grate.

2.20 GRATE INLET ON GRADE

FIGURES 16 through 19, Grate Inlet on Grade, were prepared based on the solution of Equations 1, 8, 9, 10, 11, and 12 as described in Paragraph 2.18, and with the assumption that the inlet was located in a curbed gutter. Grate Inlet on Grade shall only be used with the approval of the City Engineer.

2.21 GRATE INLET AT LOW POINT

FIGURE 21, Grate Inlet at Low Point, was prepared on the inlet having a capacity of 50 percent of the quantity derived from solution of Equation 13 as shown above. While this particular inlet capacity may appear to be considerable less than would be expected, it has been calculated based on observed clogging effects, primarily due to paper. The velocity of the gutter flow across the same inlet on grade tends to clear the grate openings. Grate Inlet at Low Point shall only be used with the approval of the City Engineer.

2.22 DROP INLET AT LOW POINT

FIGURE 22, Drop Inlet at Low Point, was prepared based on solution of Equation 7 as previously referenced, using a ten percent reduction in capacity due to clogging.

2.23 HYDRAULIC DESIGN OF CLOSED CONDUITS

All closed conduits shall be hydraulically designed through the application of Manning's Equation expressed as follows:

$Q = A\ V$

$$Q = \frac{1.486}{n} A R^{2/3} S_f^{1/2}$$

$$R = \frac{A}{P}$$

- "Q" is the flow in cubic feet per second.
- "A" is the cross sectional area of the conduit in square feet.
- "V" is the velocity of flow in the conduit in feet per second.
- "n" is the roughness coefficient of the conduit.
- "R" is the hydraulic radius, which is the area ("A") of flow divided by the wetted perimeter ("P").
- "S_f" is the friction slope of the conduit in feet per foot.
- "P" is the wetted perimeter.

2.24 VELOCITY IN CLOSED CONDUITS

Storm sewers should operate within certain velocity limits to prevent excessive deposition of solids due to low velocities and to prevent invert erosion and undesirable outlet conditions due to excessively high velocity. A minimum velocity of 2.5 feet per second and a maximum velocity of 12 feet per second shall be observed.

2.25 ROUGHNESS COEFFICIENTS FOR CLOSED CONDUITS

Roughness coefficients are directly related to construction procedures. When alignment is poor and joints have not been properly assembled, extreme head losses will occur. Coefficients used in this manner are related to construction procedures and assume that the pipe will be manufactured with a consistently smooth surface.

2.26 MINOR HEAD LOSSES IN CLOSED CONDUITS

The basic equation for calculation of minor head losses at manholes and bends in closed conduits is as follows:

$$h_j = K_j \frac{V^2}{2g}$$

- "h_j" is head loss in feet.
- "K_j" is coefficient of loss
- "V" is velocity in feet per second in conduit immediately downstream of point of loss.
- "g" is gravitation acceleration (32.2 feet per second per second).

The basic equations for calculation of minor head losses at wye branches (lateral connections to main storm sewer line) and pipe size change are as follows:

$$h_j = \frac{V_2^2 - V_1^2}{2g} \qquad \text{Where } V_1 < V_2$$

$$h_j = \frac{V_2^2 - V_1^2}{4g} \qquad \text{where } V_2 < V_1$$

- "h_j" is head loss in feet
- "V_2" is the downstream velocity in feet per second
- "V_1" is the upstream velocity in feet per second
- "g" is gravitational acceleration (32.2 feet per second per second)

2.27 HYDRAULIC DESIGN OF OPEN CHANNELS

Channel design involves the determination of a channel cross section required to convey a given design flow. The two methods outlined in this manual may be used for analysis of an existing channel or for the design of a proposed channel.

2.28 ANALYSIS OF EXISTING CHANNELS

The analysis of the carrying capacity of an existing channel is an application of Bernoulli's energy equation, which is written:

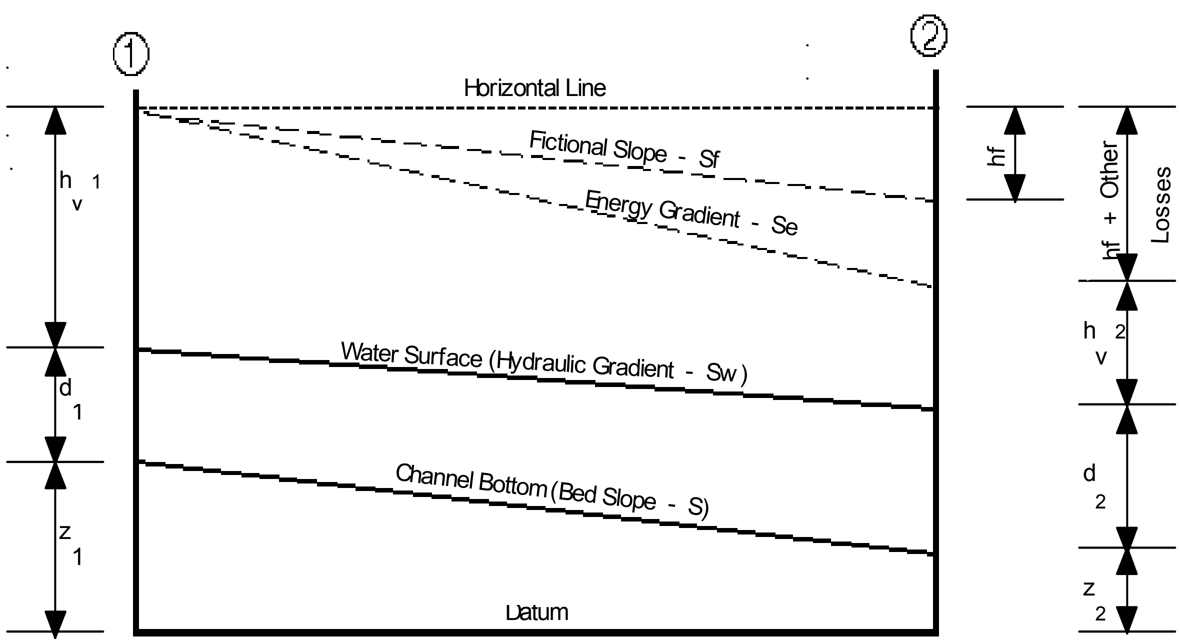
$$Z_1 + d_1 + h_{v1} = Z_2 + d_2 + h_{v2} + h_f + \text{other losses}$$

where

- "Z_1" and "Z_2" is the streambed elevation with respect to a given datum at upstream and downstream sections, respectively.
- "d_1" and "d_2" is depth of flow at upstream and downstream sections, respectively.
- "h_{v1}" and "h_{v2}" is velocity head of upstream and downstream sections, respectively.
- "h_f" is friction head loss.

Other losses such as eddy losses are estimated as 10 percent of the friction head loss where the quantity h_{v2} minus h_{v1} is positive and 50 percent thereof when it is negative. Bend losses are disregarded as an unnecessary refinement.

Bernoulli's energy equation is illustrated in graphic form as shown below.



The basic equations involved are:

$$Q = A\ V$$

$$h_v = \frac{V^2}{2_g}$$

and Manning's Equation:

$$Q = \frac{1.486}{n} AR^{2/3} \quad S^{1/2}$$

which is defined elsewhere in this chapter.

The friction head can be determined by using Manning's Equation in terms of the friction slope S_f, where:

$$S_f = \left(\frac{Q_n}{1.486\ AR^{2/3}} \right)^2$$

thus giving the total friction head

$$h_f = L \left(\frac{S_{f1} + S_{f2}}{2} \right)$$

using the respective properties of Sections 1 and 2 for the calculation of S_{f1} and S_{f2}.

The velocity head h_V is found by weighing the partial discharges for each subdivision of the cross section, i.e.,

$$h_v = \frac{V_s^2 \quad Q_s}{2_g \quad Q}$$

where

- "V_s" is velocity in subsection of the cross section.

- "A_s" is area of the subsection of the cross section.
- "Q_s" is discharge in the subsection of the cross section.
- "V_s" is $\frac{Q_s}{A_s}$

When severe constrictions occur the Momentum Equation may be required in determination of losses.

2.29 DESIGN OF IMPROVED CHANNELS

The hydraulic characteristics of improved channels are to be determined through the application of Manning's Equation as previously defined. In lieu of Manning's Equation a HEC-2 or HEC-RAS (Water Surface Profile) computer analysis can be utilized. The City, at its option, can require the use of a Computer Analysis in lieu of Manning's Equation. The HEC Computer Programs are available from the U.S. Army Corps of Engineers. The Hydrologic Engineering Center; 609 Second Street, Davis, California 95616, 916/440-2105 or can be downloaded from the Internet. User-friendly versions are available from a number of vendors.

CONCRETE BOX AND PIPE CULVERTS

The design theory outlined herein is a modification of the method used in the hydraulic design of concrete box and pipe culverts as discussed in Department of Commerce Hydraulic Engineering Circular No. 5 entitled "Hydraulic Charts for the Selection of Highway Culverts" dated December 1965.

The hydraulic capacity of culverts is computed using various factors and formulas. Laboratory tests and field observations indicate culvert flow may be controlled either at the inlet or outlet. Inlet control involves the culvert cross sectional area, the ponding of headwater at the entrance and the inlet geometry. Outlet control involves the tailwater elevation in the outlet channel, the slope of the culvert, the roughness of the surface and length of the culvert barrel.

2.30 CULVERTS FLOWING WITH INLET CONTROL

Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of the headwater and entrance geometry including the barrel shape and cross sectional area, and the type of inlet edge. Culverts flowing with inlet control can flow as shown on FORM "F", Case I (inlet not submerged) or as shown on FORM "F", Case II (inlet submerged).

Nomographs for determining culvert capacity for inlet control as shown on FIGURES 25 and 26. These nomographs were developed by the Division of Hydraulic Research, Bureau of Public Roads from analysis of laboratory research reported in National Bureau of Standards Report No. 4444, entitled "Hydraulic Characteristics of Commonly Used Pipe Entrances", by John L. French, and "Hydraulics of Conventional Highway Culverts", by H. G. Bossy. Experimental data for box culverts with headwalls and wingwalls were obtained from an unpublished report of the U. S. Geological Survey.

2.31 CULVERTS FLOWING WITH OUTLET CONTROL

Culverts flowing with outlet control can flow full as shown on FORM "F", Case III (outlet submerged), or part full for part of the barrel, as shown on FORM "F", Case IV (outlet not submerged).

The culvert is designed so that the depth of headwater, which is the vertical distance from the upstream culvert flow line to the elevation of the ponded water surface, does not encroach on the allowable freeboard during the design storm.

Headwater depth, HW, can be expressed by a common equation for all outlet control conditions:

$$HW = H + h_o - L\ (S_o)$$

- "HW" is headwater depth in feet.
- "H" is the head or energy required to pass a given discharge through a culvert.
- "h_o" is the vertical distance from the downstream culvert flow line to the elevation from which H is measured, in feet.
- "L" is length of culvert in feet.
- "S_o" is the culvert barrel slope in feet per foot.

The head, H, is made up of three parts including the velocity head, exit loss, H_V, an entrance loss, H_e, and a friction loss, H_f. This energy is obtained from ponding of water at the entrance and is expressed as:

$$H = H_v + H_e + H_f$$

- "H" is head or energy in feet of water.
- "H_v" is $\frac{V^2}{2_g}$ where V is average velocity in culvert or $\frac{Q}{A}$
- "H_e" is $K_e \frac{V^2}{2_g}$ where K_e is entrance loss coefficient
- "H_f" is energy required to overcome the friction of the culvert barrel and expressed as:

$$H_f = \frac{29.2_n^2 L}{R^{1.33}} \quad \frac{V^2}{2_g} \quad \text{where}$$

- "n" is the coefficient of roughness (See TABLE 5).
- "L" is length of culvert barrel in feet.
- "V" is average velocity in the culvert in feet per second.
- "g" is gravitational acceleration (32.2 feet per second per second).
- "R" is hydraulic radius in feet.

Substituting into previous equation:

$$H = \frac{V^2}{2_g} + K_e \frac{V^2}{2_g} + \frac{\left(29.2_n^2\ L\right)}{R^{1.33}} \quad \frac{V^2}{2_g}$$

and simplifying:

$$H \left(1 + K_e + \frac{29.2_n^2\ L}{\quad} \right) \quad \frac{V^2}{\quad} \quad \text{for full flow}$$

$$= \quad \frac{\quad}{R^{1.33}} \quad \frac{\quad}{2_g}$$

This equation for H may be solved using FIGURES 27 and 28.

For various conditions of outlet control flow, h_o is calculated differently. When the elevation of the water surface in the outlet channel is equal to or above the elevation of the top of the culvert opening at the outlet, h_o is equal to the tailwater depth or:

$$h_o = TW$$

If the tailwater elevation is below the top of the culvert opening at the outlet, h_o is the greater of two values: (1) Tailwater, TW, as defined above or (2) $d_c + D/2$ where d_c = critical depth. The critical depth, d_c, for box culverts may be obtained from FIGURE 29 or may be calculated form the formula:

$$d_c = 0.315 \left(\frac{Q}{B} \right)^{2/3}$$

- "d_c" is critical depth for box culvert in feet.
- "Q" is discharge in cubic feet per second.
- "B" is bottom width of box culvert in feet.

The critical depth for circular pipes may be obtained from FIGURE 30 or may be calculated by trial and error. Charts developed by the Bureau of Public Roads may be used for determining the critical depth. Try values of D, A and d_c which will satisfy the equation:

$$\frac{Q^2}{g} = \frac{A^3}{D}$$

- "d_c" is critical depth for pipe in feet.
- "Q" is discharge in cubic feet per second.

- "D" is diameter of pipe in feet.

- "g" is gravitational acceleration (32.2 feet per second per second).

- "A" is the cross sectional area of the trial critical depth of flow.

The equation is applicable also for trapezoidal or irregular channels, in which instances "D" becomes the channel top width in feet.

2.33 BRIDGES

Once a design discharge and the depth of flow have been established, the size of the bridge opening may be determined.

Specific effects of columns and piers may usually be neglected in the hydraulic calculations for determination of bridge openings. This is based on the assumption that all bents will be placed parallel to the direction of flow. Only in extenuating circumstances would it be desirable for bents to be placed at an oblique angle to the flow.

The basic hydraulic calculations involved in the hydraulic design involve solution of the following:

$$V = \frac{Q}{A}$$

- "V" is the average velocity through the bridge in feet per second.
- "Q" is the flow in cubic feet per second.
- "A" is the actual flow area through the bridge in square feet.

$$h_f = K_b \frac{V^2}{2_g}$$

- "h_f" is the head loss through the bridge in feet.

- "K_b" is a head loss coefficient.

- "V" is the average velocity through the bridge in feet per second.
- "g" is gravitational acceleration (32.2 feet per second per second).

As can be seen from the above, the loss of head through the bridge is a function of the velocity head. The section of a head loss coefficient as recommended in Section III, CRITERIA AND DESIGN PROCEDURES, will determine the exact hydraulic conditions.

2.34 DETENTION OF STORM WATER FLOW

As land changes from undeveloped to developed conditions, the peak rates of runoff and the total volume of runoff usually increase. This increase is due to an overall increase in impervious area as the watershed changes to a fully developed condition. Developments shall be required to provide adequate detention so that post-development peak flows do not exceed the peak flows calculated for the area using the rational method with the coefficient for runoff appropriate for the conditions prior to development.

The criteria for the design of detention facilities are based on the concept that post-development peak flows should not create an adverse condition when compared to pre-development peak flows. In applying such a concept, it is necessary to consider peak flows from a number of different design storms. By considering a range of design storms, it is possible to design an outlet system to limit the discharge from the detention facility and achieve zero or very little increase in flow for a range of storms. Such a design will allow the detention system to achieve maximum effectiveness since both the more frequent and more severe storm events can be controlled.

A form of the Rational Method should be used to calculate inflow volumes from areas less than 50 acres. A form of the inflow hydrographs or unit hydrograph method shall be used for areas of 50 acres or more. No reduction in the design storm frequency shall be considered when utilizing detention systems within the overall storm drainage design.

III - CRITERIA AND DESIGN PROCEDURES

3.1 GENERAL

This section contains storm drainage design criteria and demonstrates the design procedures to be employed on drainage projects in the City of Farmersville.

Applicable forms that can be used for the design of various storm drainage facilities are contained in Section VIII of this manual and shall be part of the drainage submittal to the City. These tables shall be reproduced in the plans.

3.2 RAINFALL

In determining the estimated runoff from a special drainage area, it is necessary to predict the amount of rain, which can be expected. FIGURE 1, RAINFALL INTENSITY AND DURATION, has been prepared to graphically illustrate anticipated rainfall intensity for storm duration from 5 minutes to six hours for selected return frequencies and shall be used for determining rainfall rates as required. Maximum time for design shall be 20-minutes.

3.3 DESIGN STORM FREQUENCY

Each storm drainage facility, including street capacities, shall be designed to convey the runoff, which results from the 100-year design storm.

Drainage design requirements for open and closed systems shall provide protection for property during a 100-year Design Frequency Storm, with this projected flow carried in the streets and closed drainage systems in accordance with the following:

a) RESIDENTIAL STREETS: Based on a transverse slope of a positive ¼" per foot behind the curb, the 100-year design storm frequency shall not exceed a depth of 1-inch over the top of curb. A maximum flow of 20 cfs will be allowed in each gutter or where gutter capacity is exceeded plus 1-inch. Bypass from upstream inlets shall not exceed 5 cfs through residential street intersections.

b) COLLECTOR STREETS: Based on parkway slopes of a positive ¼" per foot behind the curb, the 100-year Design Frequency flows shall not exceed a depth of ½" over the top of curb or where gutter capacity +½" is exceeded. A maximum flow of 20 cfs will be allowed in each gutter or where gutter capacity +½" is exceeded. Bypass from upstream inlets shall not exceed 5 cfs through collector street inlets.

c) <u>MAJOR THOROUGHFARES</u>: Based on a transverse slope of a positive ¼" per foot on the pavement, the 100-year Design Frequency flow shall not exceed the elevation of the lowest top of curb. A maximum of 35 cfs will be allowed in the street or where gutter capacity is exceeded. Bypass from upstream inlets shall be 0 cfs through major thoroughfare intersections.

d) <u>ALLEYS</u>: The 100-year Design Frequency flows shall not exceed the capacity of the alley sections shown in FIGURE 6.

e) <u>EXCAVATED CHANNELS</u>: Excavated channels shall have concrete pilot channels if deemed necessary by the City Engineer, for access or erosion control as outlined below. All excavated channels shall have a design water surface as outlined in 3.06 and be in accordance with FIGURE 24, Type II. Concrete lined channels shall be not less than Type III shown in FIGURE 24.

f) <u>MINIMUM LOT AND FLOOR ELEVATIONS</u>: Minimum lot and floor elevations shall be established as follows:

i) Lots abutting a natural or excavated channel shall have a minimum elevation for the buildable area of the lot at least at the highest elevation of the drainage floodway easement described in (g) <u>Easements</u>.

ii) Any habitable structure on property abutting a natural or excavated channel shall have a finished floor elevation at least 2-feet above the 100-year design storm or F.I.A. floodway elevation, whichever is greater.

iii) Where lots do not abut a natural or excavated channel, minimum floor elevations shall be a minimum of 1-foot above the street curb or edge of alley; whichever is lower, unless otherwise approved by the City Engineer.

g) <u>EASEMENTS</u>: Drainage and floodway easements shall be provided for all open channels. Drainage and floodway easements for storm sewer pipe shall not be the outside diameter of the conduit plus 10-feet with the minimum being 15 feet, and easement width for open or lined channels shall be at least 20 feet wider than the top of the channel, 15 feet of which shall be on one side to serve as an access for maintenance purposes.

h) POSITIVE OVERFLOW: The approved drainage system shall provide for positive overflow at all Low Points. The term "Positive Overflow" means that when the inlets do not function properly or when the design capacity of the conduit is exceeded the excess flow can be conveyed overland along a paved course. Normally, this would mean along a street or alley but can require the dedication of special drainage easements on private property. Reasonable judgment should be used to limit the easements on private property to a minimum. In specific cases where the chances of substantial flood damages could occur, the City of Farmersville may require special investigations and designs by the design engineer.

i) INLET DESIGN: Inlet spacing shall be in accordance with the design criteria contained in this manual, minimum 300 feet apart, or as required in Section 3.08, maximum length of inlets at one location shall not exceed 20 feet each side of street without prior approval from the City Engineer.

j) CULVERTS AND BRIDGES: All drainage structures shall be designed to carry the 100-year Design Frequency flow. Bridges and culverts shall be designed for a water surface elevation as outlined in 3.06. Two feet of freeboard is required for these structures.

k) MINIMUM STREET OR ALLEY ELEVATIONS: Streets or alleys adjacent to an open channel shall be designed with an elevation not lower than 1-foot above the drainage and floodway easements defined in (g) above or as directed by the City Engineer.

3.4 DETERMINATION OF DESIGN DISCHARGE

The Rational Method for computing storm water runoff is to be used for hydraulic design of facilities serving a drainage area of less than 600 acres. For drainage areas of more than 600 acres and less than 1200 acres, the runoff shall be calculated by both the Rational Method and the Unit Hydrograph Method with the larger of the two values being used for hydraulic design. For drainage areas larger than 1200 acres the runoff shall be calculated by the Unit Hydrograph Method, or as outlined in 3.06 (l).

In lieu of the Unit Hydrograph Method a HEC-1 (Flood Hydrograph) Computer Analysis can be utilized. The City at its option can require the use of HEC-1 Computer Analysis in lieu of the unit Hydrograph Method. The HEC-1 Computer Program is available from the U.S. Army Corps of Engineers, the Hydrologic Engineering Center, 609 Second Street, Davis, California 95616, 916/440-2105 or can be downloaded from their Internet site. User-friendly versions are available from a number of vendors.

3.5 RUNOFF COEFFICIENTS AND TIME OF CONCENTRATION

Runoff coefficients, as shown in TABLE 1, shall be used, based on total development under existing land zoning regulations. Where land uses other than those listed in TABLE 1 are planned, a coefficient shall be developed utilizing values comparable to those shown.

Times of concentration shall be computed based on the minimum inlet times shown in TABLE 1.

3.6 CRITERIA FOR CHANNELS, BRIDGES AND CULVERTS

Discharge flows and water surface elevations shall be based on the City's design criteria for the 100-year design storm frequency with 2-feet of freeboard. Where a unit hydrograph is used to determine the design flows, Coefficients for "Ct" and "C_p640" should be as shown in Table 2.

3.7 PROCEDURE FOR DETERMINATION OF DESIGN DISCHARGE

A standard form, STORM WATER RUNOFF CALCULATIONS, FORM A, is included in the Section VIII to record the data used in various drainage area calculations. In general, this form will be used in calculation of runoff for design of open channels, culverts and bridges. Explanation for use of this form is included in the Section VIII.

3.8 FLOW IN GUTTERS AND INLET DESIGN

Unless there are specific agreements to the contrary prior to beginning design of the particular storm drainage project, the City of Farmersville requires a storm drain conduit to begin, and consequently the first inlet to be located, at the point where the street gutter flows full based upon the appropriate design storm frequency. If, in the opinion of the City Engineer, the flow in the gutter would be excessive under these conditions, then direction will be given to extending the storm sewer to a point where the gutter flow can be intercepted by more reasonable inlet locations.

3.9 CAPACITY OF STRAIGHT CROWN STREETS

FIGURE 3, CAPACITY OF TRIANGULAR GUTTERS, applies to all width streets having a straight cross slope varying from 1/8-inch per foot to ½” per foot which are the minimum and maximum allowable slopes. Cross slopes other than ¼” per foot shall not be used without prior approval from the City Engineer.

3.10 CAPACITY OF PARABOLIC CROWN STREETS

FIGURES 4 and 5, CAPACITY OF PARABOLIC GUTTERS, apply to streets with parabolic crowns.

3.11 STREET INTERSECTION DRAINAGE

The use of surface drainage to convey storm water across a street intersection is subject to the following criteria:

a) A major thoroughfare shall not be crossed with surface drainage unless approved by the City Engineer.

b) Wherever possible, a collector street shall not be crossed with surface drainage.

c) Wherever possible, a residential street shall not be crossed with surface drainage in excess of 5 cfs.

d) At any intersection, only one street shall be crossed with surface drainage and this shall be the lower classified street.

3.12 ALLEY CAPACITIES

FIGURE 6 is a nomograph to allow determination for the storm drain capacity of various standard alley sections. In residential areas where the standard 10-foot wide alley section capacity is exceeded, a wider alley may be used to provide storm drain capacity.

As can be seen on FIGURE 6, the 20-foot wide alley section has the largest storm drain capacity. Curbs shall not be added to alleys to increase the capacity unless approved by the City Engineer. Where a particular width alley is required, such as a 12-foot width, a wider alley, such as a 16-foot width, may be required for greater capacity. Approximate increases in right-of-way widths will be necessary. Alley capacities are calculated to allow the entire alley right-of-way to carry the flow, 2½" above paving edge.

3.13 INLET DESIGN

FIGURE 7, STORM DRAIN INLETS, is a tabulation for the various types and sizes of inlets and their prescribed uses.

The information in FIGURE 7 and the general requirements of beginning the storm drain conduit where the street gutter capacity is reached will furnish the information necessary to establish inlet locations.

FIGURES 8 through 21 shall be used to determine the capacity of specific inlets under various conditions.

In using the graphs for selection of inlet sizes, care must be taken where the gutter flow exceeds the capacity of the largest available inlet size. This is illustrated with the following example.

Known: Major Street, Type C
Pavement Width = 24 Feet
Gutter Slope = 1.00%
Pavement Cross Slope = 1/4-inch/1 Foot
Gutter Flow = 11 cfs

Find: Length of Inlet Required (L_i)

Solution: Refer to FIGURE 8
Enter Graph at cfs
Intersect Slope = 1.00%
Read L_i = 16.9 Feet
DO NOT USE 14-FOOT INLET IN COMBINATION WITH 4-FOOT INLET
Enter Graph at L_i = 14 Feet
Intersect Slope = 1.00%

Read Q = 8.8 cfs
Enter Graph at L_i = 4
Intersect Slope = 1.00%
Read Q = 1.9 cfs

Therefore, the two inlets have a total capacity of 10.7 cfs, which is less than the gutter flow of 11 cfs.

USE TRIAL AND ERROR SOLUTION
Try 12-Foot Inlet plus 6-Foot Inlet
Enter Graph at L_i = 12 Feet
Intersect Slope = 1.00%
Read Q = 7.3 cfs
Enter Graph at L_i = 6 Feet
Intersect Slope = 1.00%
Read Q = 3.1 cfs

The two inlets have a capacity of 10.4 cfs, which is less than the gutter flow.

Try two 10-foot Inlets
Enter Graph at L_i = 10 Feet
Intersect Slope = 1.00%
Read Q = 5.7
x 5.7 = 11.4 cfs capacity which is equal to the gutter flow.

Use either two 10-foot inlets or other suitable combinations; whichever will best fit the physical conditions. Consideration should be given to alternate inlet locations or extension of the system to alleviate the problem of multiple inlets at a single location.

Inlets shall be sized to intercept all flow in the approaching gutter. In cases where the selection of particular size inlet would result in intercepting in excess of 90% of the gutter flow, consideration may be given to such an inlet on a minor or secondary street.

3.14 PROCEDURE FOR SIZING AND LOCATING INLETS

In order that the design procedure for determining inlet locations and sizes may be facilitated, a standard form, INLET DESIGN CALCULATIONS, FORM B, has been included in the Section VIII together with an explanation of how to use the form. Minimum distance between inlets on streets, especially major thoroughfares, shall be 300 feet or as required in Section 3.08. Remainder to be collected offsite before flowing into street.

3.15 HYDRAULIC GRADIENT OF CONDUITS

A storm drainage conduit must have sufficient capacity to discharge a design storm with a minimum of interruption and inconvenience to the public using streets and thoroughfares. The size of the conduit is determined by accumulating runoff from contributing inlets and calculating the slope of a hydraulic gradient from Manning's Equation:

$$S = \left(\frac{Qn}{1.486\ AR^{2/3}} \right)^2$$

Beginning at the upper most inlet on the system a tentative hydraulic gradient for the selected conduit size is plotted approximately 2 feet below the gutter between each contributing inlet to insure that the selected conduit will carry the design flow at an elevation below the gutter profile. As the conduit size is selected and the tentative hydraulic gradient is plotted between each inlet pickup point, a head loss due to a change in velocity and pipe size must be incorporated in the gradient profile. (See Table 6 for VELOCITY HEAD COEFFICIENTS FOR CLOSED CONDUITS.)

Also at each point where an inlet lateral enters the main conduit the gradient must be sufficiently low to allow the hydraulic gradient in the inlet to be below the gutter grade.

At the discharge end of the conduit (generally a creek or stream) <u>the hydraulic gradient of the creek for the design storm (100-year) must coincide with the gradient of the storm drainage conduit</u> and an adjustment is usually required in the tentative conduit gradient and, necessarily, the initial pipe selection could also change. The hydraulic gradient of the creek or stream for the design storm can be calculated by use of the HEC-2 or HEC-RAS Computer Program.

Concrete pipe conduit shall be used to carry the stormwater, a flow chart, Figure 23, based on Manning's Equation may be used to determine the various hydraulic elements including the pipe size, the hydraulic gradient and the velocity.

With the hydraulic gradient established, considerable latitude is available for establishment of the conduit flow line. The inside top of the conduit must be at or below the hydraulic gradient thus allowing the conduit to be lowered where necessary. The hydraulic gradient for the storm sewer line and associated laterals shall be plotted directly on the construction plan profile worksheet and adjusted as necessary.

There will be hydraulic conditions that cause the conduits to flow partially full and where this occurs, the hydraulic gradient should be shown at the inside crown (soffit) of the conduit. This procedure will provide a means for conservatively selecting a conduit size, which will carry the flood discharge.

3.16 VELOCITY IN CLOSED CONDUITS

TABLE 3 is a tabulation of minimum pipe grades, which will produce a velocity of not less than 2.5 fps when flowing full. Grades less than those shown will not be allowed. Only those pipe sizes shown in TABLE 3 should be used in designing pipe storm sewer systems.

TABLE 4 shows the maximum allowable velocities in closed conduits.

3.17 ROUGHNESS COEFFICIENTS FOR CONDUITS

Recommended values for the roughness coefficient "n" are tabulated in TABLE 5. Where engineering judgment indicates values other than those shown should be used, special note of this variance should be taken and the appropriate adjustments made in the calculations.

3.18 MINOR HEAD LOSSES

The values of K_j to be used are tabulated for various conditions in TABLE 6. In designing storm sewer systems, the head losses that occur at points of turbulence shall be computed and reflected in the profile of the hydraulic gradient.

3.19 PROCEDURE FOR HYDRAULIC DESIGN OF CLOSED CONDUITS

STORM SEWER CALCULATIONS, FORM C, has been included in the Section VIII, together with explanation for its use to facilitate the hydraulic design of a storm sewer.

3.20 OPEN CHANNELS

Open channels are to be used to convey storm waters where closed conduits are not justified. Consideration must be given to such factors as relative location to streets, schools, parks and other areas subject to frequent pedestrian use as well as basic economics.

Type II Channel Figure 24 is an improved section recommended for use where larger storm flows are to be conveyed or where the grade creates a velocity under 2-feet per second. The concrete flume in the channel bottom is to be used as a maintenance aid. The indicated width of the flumes is a minimum width and as the width of the channel increases, the required width of the flume may be increased.

Type III Channel, Figure 24, is a concrete lined section to be used for large flows in higher valued property areas and where exposure to pedestrian traffic is limited.

Where a recommended side slope and a maximum side slope are shown on a channel section, the Engineer shall use the recommended slope unless prior approval has been obtained from the City of Farmersville or soil conditions required a flatter slope.

The most efficient cross section of an open channel, from a hydraulic standpoint, is the one that, with a given slope, area and roughness coefficient, will have the maximum capacity. This cross section is the one having the smallest wetted perimeter. There are usually practical obstacles to using cross sections of the greatest hydraulic efficiency, but the dimensions of such sections should be considered and adhered to as closely as conditions will allow.

Landscaping is intended to protect the channel right-of-way from erosion, as well as present an aesthetically pleasing view. The Engineer shall include in his plans the type of grass and placement to be furnished. Full coverage of grass must be established prior to acceptance by the City.

Erosion and sediment control shall be included in the design and shown on the construction plans. These controls shall meet EPA requirements.

Design water surface shall be as shown on Figure 24 and as outlined in 3.06. Floodway easements shall be provided as shown in 3.03(g).

Special care must be taken at entrances to closed conduits, such as culverts, to provide for the headwater requirements. These calculations and the required explanations are included in Paragraph 3.32, PROCEDURE FOR HYDRAULIC DESIGN OF CULVERTS.

On all channels the water surface elevations, which may be assumed as coincident with the hydraulic gradient, shall be calculated and shown on the construction plans. One exception to the water surface coinciding with the hydraulic gradient would be in supercritical flow, which generally is not encountered in this geographical area. Designs utilizing supercritical flow should be discussed with the City of Farmersville in the preliminary stages of design.

Hydraulic calculations for Type I Channels Figure 24 shall be made as outlined on FORM "D". This procedure is applicable to a stream with an irregular channel and utilizes Bernoulli's Energy Equation to establish the water surface elevations at succeeding points along the channel.

Hydraulic calculations for Types II and III Channels shall be made as outlined on FORM "E".

In general, the use of existing channels in their natural condition or with a minimum of improvement and with reasonable safety factors is encouraged.

3.21 TYPES OF CHANNELS

FIGURE 24 illustrates the classifications and geometrics of various channel types, which are to be used wherever possible.

Type I Channel is to be used when the development of land will allow. It is intended to be left as nearly as possible in its natural state with improvements primarily limited to those which will allow the safe conveyance of storm waters, minimize public health hazards and make the flood plain usable for recreation purposes. In some instances it may be desirable to remove undergrowth.

3.22 QUANTITY OF FLOW

In the design of open channels it is usually necessary that quantities of flow be estimated for several points along the channel. These are locations where recognized discharge points enter the channel and the flows are calculated as previously outlined under "Determination of Design Discharge."

3.23 CHANNEL ALIGNMENT AND GRADE

While it is recognized that channel alignments must necessarily be controlled primarily by existing topography and right-of-way, changes in alignment should be as gradual as possible. Whenever practicable, changes in alignment should be made in sections with flatter grades.

Normally, the grade of channels will be established by existing conditions, such as an existing channel at one end and a storm sewer at the other end. There are times, however, when the grade is subject to modification, especially between controlled points.

Whenever possible the grades should be sufficient to prevent sedimentation and should not be overly steep to cause excessive erosion.

For any given discharge and cross-section of channel, there is always a slope just sufficient to maintain flow at critical depth. This is termed critical slope and a relatively large change in depth corresponds to relatively small changes in energy. Because of this instability, slopes at or near critical values should be avoided.

Maximum allowable velocities are shown in TABLE 7. When the normal available grade would cause velocities in excess of the maximums, plans shall include details for any special structures required to retard this flow.

3.24 ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

Roughness coefficients to be used in solving Manning's Equation are shown in TABLE 7, together with maximum allowable velocities.

3.25 PROCEDURE FOR CALCULATION OF WATER SURFACE PROFILE FOR UNIMPROVED CHANNELS

FORM "D" included in Section VIII, together with the explanation for its use, shall be used for calculating a profile of the water surface along an unimproved channel. The HEC-2 or HEC-RAS Computer Program is an alternate method to the use of Form "D" and may be required by the City.

3.26 PROCEDURE FOR HYDRAULIC DESIGN OF OPEN CHANNELS

FORM "E", included in Section VIII, together with the explanation for its use, shall be used in the design for open channels. The HEC-2 or HEC-RAS Computer Program is an alternate method to the use of Form "D" and may be required by the City.

3.27 HYDRAULIC DESIGN OF CULVERTS

The function of a culvert or bridge is to pass storm water from the upstream side of a roadway to the downstream side without submerging the roadway or causing excessive backwater that flows upstream property.

The Engineer shall keep head losses and velocities within reasonable limits while selecting the most economical structure. In general, this means selecting a structure that creates a headwater condition and has a maximum velocity of flow safely below the allowed maximums.

The vertical distance between the upstream design water surface and the roadway elevation should be maintained to provide a safety factor to protect against unusual clogging of the culvert and to provide a margin for future modifications in surrounding physical conditions. In general, a minimum of two feet shall be considered reasonable when the structure is designed to pass a design storm frequency of 100 years calculated by Farmersville's criteria. Unusual surrounding physical conditions may be cause for an increase in this requirement.

3.28 CULVERT HYDRAULICS

In the hydraulic design of culverts an investigation shall be made of four different operating conditions, all as shown on FORM "F". It is not necessary that the Engineer know prior to the actual calculations which condition of operation (Case I, II, III or IV) exists. The calculations will make this known.

Case I operation is a condition where the capacity of the culvert is controlled at the inlet with the upstream water level at or below the top of the culvert and the downstream water level below the top of the culvert.

Case II operation is also a condition where the capacity of the culvert is controlled at the inlet with the upstream water level above the top of the culvert with the downstream water level below the top of the culvert.

Case III operation is a condition where the capacity of the culvert is controlled at the outlet with the upstream and downstream water levels above the top of the culvert.

Case IV operation is a condition where the capacity of the culvert is controlled at the outlet with the upstream water level above the top of the culvert and the downstream water level equal to one of two levels to be calculated.

3.29 QUANTITY OF FLOW

The quantity of flow which the structure must convey shall be calculated in accordance with the Procedure for Determination of Design Discharge utilizing FORM "A".

3.30 HEADWALLS AND ENTRANCE CONDITIONS

Headwalls are used to protect the embankment from erosion and the culvert from displacement. The headwalls, with or without wingwalls and aprons, shall be constructed in accordance with the standard drawings as required by the physical conditions of the particular installation.

In general, straight headwalls should be used where the approach velocities in the channel are below 6 feet per second, where headwater pools are formed and where no downstream channel protection is required. Headwalls with wingwalls and aprons should be used where the approach velocities are from 6 to 12 feet per second and downstream channel protection is desirable.

Special headwalls and wingwalls may be required where approach velocities are in excess of 12 to 15 feet per second. This requirement varies according to the axis of the approach velocity with respect to the culvert entrance.

A table of culvert entrance data is shown on FORM "F". The values of the entrance coefficient, K_e, are a combination of the effects of entrance and approach conditions. It is recognized that all possible conditions may not be tabulated, but an interpolation of values should be possible from the information shown. Where the term "round" entrance edge is used, it means a 6-inch radius on the exposed edge of the entrance.

3.31 CULVERT DISCHARGE VELOCITIES

Velocities in culverts should be limited to no more than 15 feet per second, but downstream conditions very likely will impose more stringent controls. Consideration must be given to the effect of high velocities and turbulence on the channel, adjoining property and embankment. TABLE 8 is a tabulation of maximum allowable velocities based on downstream channel conditions.

3.32 PROCEDURE FOR HYDRAULIC DESIGN OF CULVERTS

FORM "F", included in the Section VIII, together with the explanation for its use, shall be used for the hydraulic design of culverts.

3.33 HYDRAULIC DESIGN OF BRIDGES

Wherever possible the proposed bridge should be designed to span a channel section equal to the approaching channel section. If a reduction in channel section is desired this should be accomplished upstream of the bridge and appropriate adjustments made in the hydraulic gradient.

Wherever possible bridges should be constructed to cross channels at a 90-degree angle, which normally will result in the most economical construction. Wherever the bridge structure is skewed the bents should be constructed parallel to the flow of water. Values of K_b, head loss coefficient, normally will vary from 0.2 to 0.5 with the exact value to be determined by an appraisal of the particular hydraulic conditions associated with the specific project. With a minimum of constriction and change in velocity, a clear span bridge would have a minimum coefficient. This would increase for a multispan bridge, skewed or with piers not placed perpendicular to the flow. The Bureau of Public Roads "Hydraulic of Bridge Waterways" should be used for determining the K coefficient.

In more complex bridge design such as long multiple spans and relief structures crossing an irregular channel section, the procedures outlined in the Texas Highway Department "Hydraulic Manual" or the Bureau of Public Roads "Hydraulics of Bridge Waterways", should be utilized.

A distance of 2 feet between the maximum design water surface and the lowest point of the bridge stringers shall be maintained.

3.34 QUANTITY OF FLOW

The quantity of flow which the structure must convey shall be calculated in accordance with the Procedure for Determination of Design Discharge utilizing FORM "A". The HEC-1 Computer Program is an alternate method to the use of Form "A" and may be required by the City.

3.35 PROCEDURE FOR HYDRAULIC DESIGN OF BRIDGES

FORM "G", included in the Section VIII, together with the explanation for its use, shall be used for the hydraulic design of bridges.

The Engineer should investigate several different bridge configurations on each project to determine the most economical that can be constructed within the velocity limitations and other criteria included in this manual.

3.36

3.37 PROCEDURE FOR FILLING IN A FLOOD PLAIN

Fill and development of floodplains, which is not unreasonably damaging to the environment is permitted where it will not create other flood problems. Following are the engineering criteria for fill requested:

a) Alterations of the flood plain shall result in no increase in water surface elevation on other properties. No alteration of the channel or adjacent flood plain will be permitted which could result in any degree of increased flooding to other properties, adjacent, upstream, or downstream. Increased flood elevation could cause inundation and damage to areas not presently inundated by the "design flood". The "design flood" for a creek is defined by either the 100-year flood -- the flood having a one percent chance of being equaled or exceeded at least once in any given year -- or the maximum recorded flood, whichever results in the highest peak flood discharges. Streams on the Federal Insurance Rate Maps must be designed using the FIRM 100-year design or the City design, whichever is greater.

b) Alterations of the flood plain shall not create an erosive water velocity on or off site. The mean velocity of stream flow at the down stream end of the site after fill shall be no greater than the mean velocity of the stream flow under existing conditions.

No alteration to the flood plain will be permitted which would increase velocities of flood waters to the extent that significant erosion of flood plain soils will occur either on the subject property or on other property up or downstream. Soil erosion results in loss of existing vegetation as well as augments destructive sedimentation downstream. Eventual public costs in channel improvements and maintenance (such as removal of debris and dredging of lakes) can be expected as a result. Staff's determination of what constitutes an "erosive" velocity will be based on analysis of the surface material and permissible velocities for specific cross-sections affected by the proposed alteration, using standard engineering tables as a general guide.

c) Alterations of the flood plain shall be permitted only to the extent permitted by equal conveyance on both sides of the natural channel. Staff's calculation of the impact of the proposed alteration will be based on the "equal conveyance" principle in order to insure equitable treatment for all property owners. Under equal conveyance, if the City allows a change in the flood carrying capacity (capacity to carry a particular volume of water per unit of time) on one side of the creek due to a proposed alteration of the flood plain, it must also allow an equal change to the owner on the other side. The combined change in flood carrying capacity, due to the proposed alteration plus a corresponding alteration to the other side of the creek, may not cause either an increase in flood elevation or an erosive velocity (Criteria 1 and 2) or violate the other criteria. Conveyance is mathematically expressed as KD = 1.486/n AR 2/3 where n = Manning's friction factor, A = cross sectional area, and R = hydraulic radius.

d) The toe of any fill slope shall parallel the natural channel to prevent an unbalancing of stream flow in the altered flood plain. If the alignment of the proposed fill slope departs from the contours of the natural flood plain, the flow characteristics of the floodwaters may be altered, causing possible damaging erosion and deposition in the altered flood plain. If the fill slope flows the natural channel, it will also tend to minimize the visual impact of the alteration.

e) To insure maximum accessibility to the flood plain for maintenance and other purposes and to lessen the probability of slope erosion during periods of high water, maximum slopes of filled area shall usually not exceed 4 to 1. Vertical walls, terracing and other slope treatments will be considered only as a part of a landscaping plan submission and if no unbalancing of stream flow results. The purposes of the slope restrictions are to maintain stability and prevent erosion of the slopes, to ease maintenance (e.g. mowing) on the slopes themselves, and to provide accessibility to the areas below the slopes. Being more frequently inundated and therefore subject to greater hazard of erosion, cut slopes must be shallower than fill slopes.

f) Landscaping plan submission shall include plans for erosion control of cut and fill slopes, restoration of excavated areas, and tree protection where possible in and below fill area. Landscaping should incorporate natural materials (earth, stone, wood) on cut or fill slopes wherever possible. Applicant should show in plan the general nature and extent of existing vegetation on the tract, and which areas will be preserved, altered, or removed as a result of the proposed alterations. Locations and construction details should be provided showing how trees will be preserved in areas which will be altered by filling or paving within the drip line of those trees. Applicant should also submit plans showing location, type, and size of new plant materials and other landscape features planned for altered flood plain areas.

Erosion control plans should demonstrate how the developer intends to minimize soil erosion and sedimentation from his site during and after the fill operation. Plans should include a timing schedule showing anticipated starting and completion dates for each step of the proposed operation. Area and time of exposed soils should be minimized, and existing vegetation should be retained and protected wherever feasible. Disturbed areas should be sodded or covered with mulch and/or temporary vegetation as quickly as possible. Structural measures (e.g. drop structures, sediment ponds, etc.) should be utilized where necessary for effective erosion control, but measures should also minimize structures and materials that detract from the natural appearance of the flood plain.

3.38 FILLING IN A 100 YEAR FLOODWAY FRINGE

a) Definitions

i) 100 Year Flood Plain Elevation (100 Year F.P.El.): That water surface elevation established by applying the Manning Equation (Q = 1.486/n AR 1/2 S 1/2) to the backwater analysis of a stream (river, creek or tributary) using the 100-year storm as the rate of flow (Q). The 100-Year F.P. Elevations are those based on the Corps of Engineer's analysis and form the basis of the Flood Insurance Rate Map (FIRM) as adopted by the Federal Insurance Administration, or subsequent amendments.

ii) Flood Plain: Area of land lying below the 100-year flood plain elevation.

iii) Floodway: That central portion of the flood plain which would remain clear of filling or other obstructions, unless modifications are made within or along the stream bed to offset the effect of additional filling or obstructions within the floodway.

iv) Floodway Fringe: Area between flood plain line and the floodway line that, if filled, would not produce a significant rise in the 100-year flood plain elevation.

v) <u>Significant Rise</u>: A rise in the 100-year water surface elevation greater than one (1) foot for fill on both sides of a stream or one-half (0.5) feet for fill on one side of a stream.

vi) <u>Floodway Line</u>: The inter-boundary of the floodway fringe determined by filling within a flood plain along the entire reach of a stream in such a manner that the total cumulative effect of the filling will not create a significant rise in the 100-year water surface elevation.

vii) <u>Equal Conveyance Principle</u>: An area of the cross section of a stream in its existing condition carrying a percentage of the stream flow, will continue to carry the same percentage of the stream flow after filling in the flood plain occurs without creating a significant rise in the 100-year flood plain elevation.

b) <u>Criteria for Filling in the 100 Year Floodway Fringe</u>

i) Applies only to creeks or portions of creeks with a drainage area of five (5) square miles, or less.

ii) Fill and development of the flood plains shall not create a "significant rise" in the 100-year flood plain elevation.

iii) For fill and/or other development within the <u>floodway</u>, supporting hydraulic analysis will be required prior to or at the time of submittal of the preliminary plat demonstrating that the proposed development will not create a "significant rise" in the "100-year flood plain elevation".

iv) In beginning a backwater analysis for development within a flood plain, the downstream water surface elevation will be determined as follows:

- For fill on one side only of a stream, add one-half (0.5) feet to the 100-year flood plain elevation at the downstream property line.

- For fill on both sides of a stream, add one (1.0) foot to the 100-year flood plain elevation at the downstream property line.

v) Alterations of the floodway shall not create velocities, which could produce maximum erosive velocities in excess of those set forth in Table 7.

vi) Floodway Line shall be established in accordance with the definition in (A) above.

vii) Equal Conveyance shall be required in accordance with the definition of Equal Conveyance Principle in (A) above.

viii) The requirements of 3.36, Paragraphs d, e and f shall apply.

ix) Final approval shall be by FEMA.

3.39 DETENTION PONDS

On-site detention shall be used to control post-development runoff. Developments shall be required to provide adequate detention so that post-development peak flows do not exceed the peak flows calculated for the area using the rational method with the coefficient for runoff appropriate for the conditions prior to development. Inflow volumes shall be calculated for the 5, 10, 25 and 100-year storm frequencies. For areas less than 50 acres a form of the Rational Method will be acceptable, while for areas 50 acres and larger an inflow hydrograph, unit hydrograph or HEC-1 computer model will be required. A hydraulic study that illustrates no adverse conditions are created downstream as a result of development may be accepted in lieu of storm water detention. City Council may waive storm water detention requirements upon determination by the Council that such waiver is in the best interest of the City.

The detention system shall be designed for the 100-year storm frequency, 24-hour design storm duration and a time to empty of 48 hours. Any type of pond design shall be designed with a freeboard of 30% the nominal depth of the pond, but not less than 2.0 feet. The maximum allowable headwater must be kept within the range of slope stability of the embankment construction. All design calculations shall be a part of the construction plans.

An outlet control structure such as an orifice and weir placed at the inlet end of the outfall pipe is to provide an integrated stage-discharge such that a wide range of storms can be effectively controlled. Perforated riser pipes, weirs and special outlet control boxes are acceptable. Pipe/culvert type outlet control will only be allowed with written approval from the City. All vertical structures shall have anti-vortex and trash rack devices. Emergency overflow structures and paved positive overflow channels shall be included with the design of detention systems.

Whenever possible, detention ponds shall fit in the natural contour of the land, be aesthetically pleasing and be free draining. A grading plan with 2-foot intervals shall be placed on the construction plans. Maintenance access shall be provided for each pond. The bottom slope shall be a minimum of 2% towards the outfall structure. Detention basins shall be designed with short and long term erosion control.

A detention system maintenance program shall be prepared and submitted to the City for approval before final acceptance of the construction plans.

IV - CONSTRUCTION PLANS PREPARATION

4.01 GENERAL

This section covers the preparation of drainage construction plans for the City of Farmersville.

4.02 PRELIMINARY DESIGN PHASE

The preliminary design phase shall be complete in sufficient detail to allow review by the City of Farmersville. To complete this phase, all topographic surveys should be furnished to allow establishment of alignment, grades and right-of-way requirements. These may be accomplished by on-the-ground field surveys, by aerial photogrametric methods, or by use of topographic maps.

Based upon the procedures and criteria outlined in SECTION III, CRITERIA AND DESIGN PROCEDURES, of this manual, the hydraulic design of the proposed facilities shall be accomplished. All calculations shall be made on the appropriate forms and submitted with the preliminary plans.

These plans shall show the alignment, drainage areas, size of facilities and grades.

a) Preliminary Plans

Preliminary storm drainage plans shall include a cover sheet, drainage area map, plan-profile sheets and channel cross sections if required. The proposed improvements shall be drawn on 22-inch by 34-inch sheets.

b) Drainage Area Map

The scale of the drainage area map should be determined by the method to be used in calculating the runoff as discussed in Section III. Generally, a map having a scale of 1" = 200' (showing the street right-of-way) is suitable unless dealing with a large drainage area. For large drainage areas a map having a scale of 1" = 2000' is usually sufficient. When calculating runoff the drainage area map shall show the boundary of the drainage area contributing runoff into the proposed system. This boundary can usually be determined from a map having a contour interval of 2 to 5 feet. The area shall be further divided into sub-areas to determine flow concentration points or inlet locations.

Direction of flow within streets, alleys, natural and manmade drainage ways and at all system intersections shall be clearly shown on the drainage area map. Existing and proposed drainage inlets, storm sewer pipe systems and drainage channels shall be clearly shown and differentiated on the drainage area map. Plan-profile storm sewer or drainage improvement sheet limits shall also be shown.

The Drainage Area Map should show enough topography to easily determine its location within the City.

All offsite drainage within the natural drainage basin shall be shown and delineated. Runoff calculations including inlet calculations, shall be a part of the drainage area map.

c) Plan-Profile Sheets

Inlets shall be given the same number designation as the area or sub-area contribution runoff to the inlet. The inlet number designation shall be shown opposite the inlet. Inlets shall be located at or immediately downstream of drainage concentration points. At intersections, where possible, the end of the inlet shall be ten feet from the curb radius and the inlet location shall also provide minimum interference with the use of adjacent property. Inlet locations directly above storm sewer lines shall be avoided.

Data opposite each inlet shall include paving or storm sewer stationing at centerline of inlet, size of inlet, type of inlet, number or designation, top of curb elevation and flow line of inlet as shown on the typical plans. Inlet laterals leading to storm sewers, where possible, shall enter the inlet at a 60 degree angle from the street side. Laterals shall be four and one-half feet from top of curb to flow line of inlet unless utilities or storm sewer depth requires otherwise. Laterals shall not enter the corners of inlets. Lateral profiles shall be drawn showing appropriate information including the Hydraulic Gradient.

In the plan view, the storm sewer designation, size of pipe, and length of each size pipe shall be shown adjacent to the storm sewer. The sewer plan shall be stationed at one hundred foot intervals and each sheet shall begin and end with even or fifty foot stationing. All storm sewer components shall be stationed.

The profile portion of the storm sewer plan-profile sheet shall show the existing ground profile along the centerline of proposed sewer, the hydraulic gradient of the sewer, the proposed storm sewer, and utilities which intersect the alignment of the proposed storm sewer. Also shown shall be the diameter of the proposed pipe in inches and the physical grade in percent. Hydraulic data for each length of storm sewer between interception points shall be shown on the profile. This data shall consist of pipe diameter in inches, discharge in cubic feet per second, slope of hydraulic gradient in percent, capacity of pipe in cubic feet per second and velocity in feet per second. Also, the head loss at each interception point shall be shown.

Elevations of the flow line of the proposed storm sewer shall be shown at one hundred foot intervals on the profile. Stationing and flow line elevations shall also be shown at all pipe grade changes, pipe size changes, lateral connections, manholes and wye connections.

4.03 FINAL DESIGN PHASE

During the final design phase the construction plans shall be placed in final form. All sheets shall be drawn in ink on 22-inch by 34-inch sheets and shall be clearly legible when sheets are reduced to half scale.

Review comments shall be considered, additional data incorporated and the final design and drafting of the plans completed. All grades, elevations, pipe sizes, utility locations, items and quantities should be checked and each plan-profile sheet shall have a bench mark shown.

V - APPENDIX

5.01A DEFINITION OF TERMS

Angle of Flare: Angle between direction of wingwall and centerline of culvert or storm drain outlet.

Backwater Curve: The surface curve of a stream of water.

Conduit: Any closed device for conveying flowing water.

Control: The hydraulic characteristic, which determined the stage-discharge relationship in a conduit.

Critical Flow: The state of flow for a given discharge at which the specific energy is a minimum with respect to the bottom of the conduit.

Entrance Head: The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head.

Entrance Loss: Head lost in eddies or friction at the inlet to a conduit, headwall or structure.

Flume: Any open conduit on a prepared grade, trestle or bridge.

Freeboard: The distance between the normal operating level and the top of the side of an open channel left to allow for wave action, floating debris, or any other condition or emergency without overflowing structure.

Headwater: Depth of water in the channel measured form the invert of the culvert inlet.

HEC-1: Computer Program to analyze a Flood Hydrograph. This program is available from the U. S. Army Corps of Engineers.

HEC-2/HEC-RAS: Computer Program to analyze a Water Surface Profile. This program is available from the U. S. Army Corps of Engineers.

Hydraulic Gradient: A line representing the pressure head available at any given point within the system.

Invert: The flow-line of conduit (pipe or box).

Manning's Equation: The uniform flow equation used to relate velocity, hydraulic radius and energy gradient slope.

Open Channel: A channel in which water flows with a free surface.

Rational Formula: The means of relating runoff with the area being drained and the intensity of the storm rainfall.

Soffit: The inside top of the conduit (pipe or box).

Steady Flow: Constant discharge.

Surcharge: Height of water surface above the crown of a closed conduit at the upstream end.

Tailwater: Total depth of flow in the downstream channel measured form the invert of the culvert outlet.

Time of Concentration: The estimated time in minutes required for runoff to flow from the most remote section of the drainage area to the point at which the flow is to be determined.

Total Head Line (Energy Line): A line representing the energy in flowing water. It is plotted a distance above the profiles of the flow line of the conduit equal to the normal depth plus the normal velocity head plus the pressure head for conduits flowing under pressure.

Uniform Channel: A channel with a constant cross section and roughness coefficient.

Uniform Flow: A condition of flow in which the discharge, or quantity of water flowing per unit of time, and the velocity are constant. Flows will be at normal depth and can be computed by the Manning Equation.

Watershed: The area drained by a stream or drainage system.

5.01B DETENTION SYSTEM DEFINITIONS

Detention Storage: Detention storage facilities are generally designed to control short, high-intensity local storms, as these are the major cause of flooding on small streams (1). Detention storage serves to attenuate the peak flow by reducing the peak outflow to a rate less than the peak inflow, which effectively lengthens the time base of the outflow hydrograph. The total volume of water discharged is the same; it is merely distributed over a long period of time (2). Discharge from detention storage facilities begins immediately at the start of the storm, and the facility is usually completely drained within a day after the storm event.

Retention Storage: Retention storage refers to those facilities where stormwater is collected and stored during the flood event. The stored water is released after the flood event by means of controlled outlet works. Alternatively, the water may be allowed to infiltrate into the ground or evaporate. For maximum effectiveness, the water contained in the retention storage facility must be released or lost before the next flood event occurs (2). In some cases, it may be desirable to maintain a permanent pool within the retention area. Such a facility is termed wet storage.

Conveyance Storage: As stormwater enters and flows in channels, floodplains, drains, and storm sewers, the flow is being stored in transient form and is termed conveyance storage. Conveyance storage is generally obtained by constructing low-velocity channels with large cross-sectional areas.

Upstream Storage: This storage occurs upstream of the design area to be protected. It is intended to contain runoff, which originates upstream and beyond the area to be protected.

Within-Area Storage: This storage occurs in the area to be protected. It is intended to store runoff originating in and around the area to be protected. It is common for such storage to be provided at the development sites.

Downstream Storage: This is storage located downstream from the area to be protected. The general purpose of downstream storage is to manage storm flows from the area to be protected and to control any detrimental downstream effects from development in the protected area.

Rainfall Storage: Rainfall storage refers to the storage of water in the vicinity of the rainfall occurrence or before storm water accumulates significantly (3). This storage classification is similar to "within-area storage" as described above.

Runoff Storage: Runoff storage refers to the storage of larger quantities of water, that have accumulated significantly and have begun to flow in the drainage system. This storage classification is closely related to "upstream storage" and "downstream storage" as described above.

Driveway Storage: This storage method involves the construction of depressed section in the driveway such that runoff from the lot and/or roof may be routed and stored there. A properly designed outlet system will regulate the discharge of this runoff into the drainage system (2).

Cistern/Infiltration: A cistern or tank can be located within the property area to collect runoff from the lot and roof. If local subsurface soil properties and geologic conditions permit, the water can be infiltrated after the storm subsides (2).

Cistern/Irrigation: This method is identical to the "cistern/infiltration" method except that the option is provided for the water in the cistern to be used for an irrigation water supply or to be discharged into the storm sewer system.

Rooftop Storage: This storage method is most applicable to industrial, commercial, and apartment buildings with large flat roofs. Rooftop storage is often an economical and effective alternative. Since it is common for buildings to be designed for snow loads, it is possible to accommodate an equivalent depth of water without significant structural changes. A six-inch depth of water is equivalent to 31.2 pounds per square foot, less than most snow load requirements in the northern United States and Canada (4).

Special roof drains with controlled outlet capacity are typically installed as an integral part of the rooftop storage method. With proper installation of such drains, peak runoff from roofs may be reduced by up to 90 percent (4).

An important consideration for the rooftop storage method would be to provide overflow mechanisms to ensure that the structural capacity of the roof is not exceeded. An additional consideration would be the watertightness of the rooftop.

Parking Lot Storage: Parking lots can be graded to route runoff to desired storage areas or areas of infiltration. If the flow is routed to a storage area, outlet works such as grated inlets or overflow weirs serve to regulate the design flow. Alternatively, the runoff may be routed to grassed or gravel filled areas for infiltration and percolation.

On-Site Ponds: On-site ponds provide for the collected stormwater to be released in a controlled manner by overflow weirs or orifices. When properly designed, on-site ponds can serve the hydraulic function while providing recreational and aesthetic benefits.

Slow-Flow Drainage Patterns: This storage method involves the design of conveyance systems with reduced grades to provide reduced flow velocities. The desired effect is to obtain temporary ponding and a form of transient storage. Slow flow drainage may be augmented by providing controls (e.g., weirs, checks) along channels to create a system of linear reservoirs (2). Use of such controls will provide temporary storage while allowing for a possible increase in infiltration.

Open Space Storage: Open spaces such as parks and recreation fields generally have a substantial area of grass covering and provide increased infiltration opportunities. Such open spaces produce only minimal quantities of runoff. Therefore, open spaces provide excellent opportunities for the temporary storage of storm runoff, provided the primary use of the open space is not altered. This is generally not a problem since recreation areas are seldom used during storm events.

Retention Reservoirs: Retention reservoirs located in a watershed catchment generally represent major storage facilities (2). They are most effective when located in valleys or recessed areas and should have the ability to regulate stream flow. Retention reservoirs maintain a permanent pool in the form of ponds or lakes. As such, they are well suited for water-oriented recreational features.

Detention Reservoirs: Detention reservoirs are generally located on streams and are frequently located above the reaches where there is a continuous flow (2). Since a permanent pool is not maintained, detention reservoirs do not provide opportunities for water-oriented recreation. However, they may be conveniently integrated into a park and open space plan.

Gravel Pits and Quarries: Gravel pits and quarries are located off-channel such that a side-channel spillway is necessary to intercept and direct the peak flow to the pit location. Outfall from such storage facilities must generally be pumped.

5.02 ABBREVIATION OF TERMS AND SYMBOLS

A	Drainage area in acres of tributary watershed. Cross-sectional area of gutter flow in square feet. Cross-sectional area of flow through conduit in square feet.
A_s	Sub-section area in square feet as used on unimproved channel calculations.
b	Bottom width of channel in feet.
c	Runoff Coefficient for use in Rational Formula representing the estimated ratio of runoff to rainfall which is dependent on the slope of the watershed, the land use and the character of soil.
C_o	Street crown height in feet.
C_t	A coefficient related to drainage basin characteristics and used in Unit

	Hydrograph calculations.
C_p640	Coefficient related to drainage basin characteristics and used in Hydrograph calculations.
c.f.s.	Cubic feet per second.
d	Depth of flow in feet.
d_n	Normal depth of flow in conduit feet.
d_c	Critical depth of flow in conduit feet.
FL	Flow line.
f.p.s.	Feet per second.
g	Gravitational acceleration (32.2 feet per second per second).
H	Depth of flow in feet required to pass a given discharge.
h	Depth of flow in feet.
HW	Headwater elevation or depth above invert at storm drain entrance in feet.
h_o	Vertical distance from downstream culvert flow line to the elevation from which H is measured, in feet.
h_f	Head loss due to friction in a length of conduit in feet.
h_j	Head loss at junction structures, inlets, manholes, etc., due to turbulence in feet.
h_v	Velocity head loss in feet.
I	Intensity, in inches per hour, for rainfall over an entire watershed.
K_b	Head loss coefficient at bridges.
K_e	Coefficient of entrance loss.
K_j	Coefficient for head loss at junctions, inlets and manholes.

L	Length of channel in miles measured along flow line.
L_{ca}	Length of stream in miles from design point to center of gravity of drainage area and used in Unit Hydrograph calculations.
L_i	Length of curb opening inlet in feet.
L_{is}	Initial and subsequent rainfall losses in inches and used in Unit Hydrograph calculations.
n	Coefficient of roughness for use in Manning's Equation.
P	Length in feet of contact between flowing water and the conduit measured on a cross section. (Wetted Perimeter)
Q	Storm water flow in c.f.s.
Q_R	Peak flow in c.f.s. as determined by Rational Method.
Q_u	Peak flow in c.f.s. as determined by Unit Hydrograph Method.
q_p	Peak rate of discharge of the Unit Hydrograph for unit rainfall duration of c.f.s. per square mile.
Q_p	Peak rate of discharge of the Unit Hydrograph in c.f.s.
R	Hydraulic Radius = $\dfrac{\text{Cross section area of flow in sq. ft. (A)}}{\text{Wetted perimeter in ft. (P)}}$
R_T	Total runoff in inches as used in Unit Hydrograph calculations.
S	Slope of street, gutter or hydraulic gradient in feet per foot or percent.
s_c	That particular slope in feet per foot of a given uniform conduit operating as an open channel at which normal depth and velocity equal critical depth and velocity for a given discharge.
S_D	Design storm runoff in inches for a two-hour period.
S_f	Friction slope in feet per foot in a conduit. This represents the rate of loss in the

conduit due to friction.

t_c	Time of Concentration in minutes.
t_p	Lag time in hours from the midpoint of the unit rainfall duration to the peak of the Unit Hydrograph.
TW	Tailwater elevation of depth above invert a culvert outlet.
V	Velocity of flow in feet per second.
v	Mean velocity of flow at upstream end of inlet opening in feet per second.
v_c	Critical velocity of flow in a conduit in feet per second.
$\frac{V^2}{2g}$	Velocity head. A measure, in feet, of the kinetic energy in flowing water.
V_1	Upstream Velocity
V_2	Downstream Velocity
W	Street width from face of curb in feet.
WP	Wetted perimeter in feet.
Z	Reciprocal of crown slope, $1/\theta_o$.
θ_o	Crown slope of pavement in feet per foot.
Y	Conveyance factor calculated for unimproved channels.

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VI - LIST OF TABLES

TABLE 1

COEFFICIENTS OF RUNOFF AND MINIMUM INLET TIMES

Land Use	Runoff Coefficient C	Minimum Inlet Time In Minutes
Residential	0.6	15
Commercial	0.9	10
Industrial	0.9	10
Multiple Unit Dwelling	0.8	10
Parks	0.4	15
Cemeteries	0.4	15
Pasture	0.4	15
Woods	0.3	15
Cultivated	0.6	20
Shopping Centers	0.9	10
Paved Areas	0.9	10
Schools	0.7	15
Patio Homes	0.6	15
Churches	0.8	10

TABLE 2

COEFFICIENTS "C_t" AND "C_p640"

Drainage Area Characteristics	Approximate Value of "C_t"	Approximate Value of "C_p640"
Sparsely Sewered Area		
Flat Basin Slope (less than 0.50%)	0.65	350
Moderate Basin Slope (0.50% to 0.80%)	0.60	370
Steep Basin Slope (greater than 0.80%)	0.55	390
Moderately Sewered Area		
Flat Basin Slope (less than 0.50%)	0.55	400
Moderate Basin Slope (0.50% to 0.80%)	0.50	420
Steep Basin Slope (greater than 0.80%)	0.45	440
Highly Sewered Area		
Flat Basin Slope (less than 0.50%)	0.45	450
Moderate Basin Slope (0.50% to 0.80%)	0.40	470
Steep Basin Slope (greater than 0.80%)	0.35	490

TABLE 3

MINIMUM SLOPES FOR PIPES
(n = 0.013)

Pipe Diameter (Inches)	Slope (Feet/100 Feet)
18	.180
21	.150
24	.120
27	.110
30	.090
33	.080
36	.070
39	.062
42	.056
45	.052
48	.048

Pipe Diameter (Inches)	Slope (Feet/100 Feet)
51	.045
54	.041
60	.036
66	.032
72	.028
78	.025
84	.023
90	.021
96	.019
102	.018
108	.016

NOTE: Minimum pipe diameter to be used in construction of storm sewers shall be 18-inches.

TABLE 4

MAXIMUM VELOCITIES IN CLOSED CONDUITS

Type of Conduit	Maximum Velocity
Culverts	15 f.p.s.
Inlet Laterals	30 f.p.s.
Storm Sewers	12 f.p.s.

Storm sewers that discharge into open channels shall be at a maximum velocity of 8-feet per second unless channel protection is provided for the reach from the point of discharge until velocity is less than 8-feet per second in the channel. This maximum velocity must be maintained in the last 200-feet of storm sewer conduit.

TABLE 5

ROUGHNESS COEFFICIENTS FOR CLOSED CONDUITS

Material of Construction	Recommended Roughness Coefficient "n"
New Monolithic Concrete Conduit	0.015
Concrete Pipe Storm Sewer	
Good Alignment, Smooth Joints	0.013
Fair Alignment, Ordinary Joints	0.015
Poor Alignment, Poor Joints	0.017
Concrete Pipe Culverts	0.012
Monolithic Concrete Culverts	0.012
Corrugated Metal Pipe	0.024
Corrugated Metal Arch Pipe	0.024
Corrugated Metal Pipe with Smooth Liner	0.015

NOTE: Reinforced concrete pipe is the accepted material for construction of storm sewers. The use of other materials for the construction of storm sewers shall have prior approval from the City Engineer. For design of all pipe material an “n” of 0.013 shall be used.

TABLE 6

VELOCITY HEAD LOSS COEFFICIENTS FOR CLOSED CONDUITS

MANHOLE AT CHANGE IN PIPE DIRECTION		
Description	**Angle**	**Head Loss Coefficient Kj**
	90	1.00
	60	0.80
	45	0.65
Angle	30	0.50

BEND IN PIPES		
Description	**Angle**	**Head Loss Coefficient Kj**
	*90^{O}	0.80
	*60^{O}	0.60
	**45^{O}	0.50
Angle	**30^{O}	0.45

ENLARGEMENTS IN PIPE SIZES WITH CONSTANT FLOW		
Description	**Ratio of Upstream Diameter to Downstream Diameter**	**Head Loss Coefficient Kj**
	0.81	1.00
	0.82	0.90
	0.84	0.80
	0.85	0.70
	0.86	0.60
	0.88	0.50
	0.90	0.40
	0.92	0.30

* Only as authorized by City Engineer.

** Horizontal curves are the accepted method of construction.

TABLE 7
ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

Channel Description	Roughness Coefficient			Maximum Velocity Ft/sec
	Minimum	Normal	Maximum	
MINOR NATURAL STREAMS – TYPE I CHANNEL				
Moderately Well Defined Channel				
• Grass and Weeds, Little Brush	0.025	0.030	0.033	8
• Dense Weeds, Little Brush	0.030	0.035	0.040	8
• Weeds, Light Brush on Banks	0.030	0.035	0.040	8
• Weeds, Heavy Brush on Banks	0.035	0.050	0.060	8
• Weeds, Dense Willows on Banks	0.040	0.060	0.080	8
Irregular Channel with Pools and Meanders				
• Grass and Weeds, Little Brush	0.030	0.036	0.042	8
• Dense Weeds, Little Brush	0.036	0.042	0.048	8
• Weeds, Light Brush on Banks	0.036	0.042	0.048	8
• Weeds, Heavy Brush on Banks	0.042	0.060	0.072	8
• Weeds, Dense Willows on Banks	0.048	0.072	0.096	8
Flood Plain, Pasture				
• Short Grass, No Brush	0.025	0.030	0.035	8
• Tall Grass, No Brush	0.030	0.035	0.050	8
Flood Plain, Cultivated				
• No Crops	0.025	0.030	0.035	8
• Mature Crops	0.030	0.040	0.050	8
Flood Plain, Uncleared				
• Heavy Weeds, Light Brush	0.035	0.050	0.070	8
• Medium to Dense Brush	0.070	0.100	0.160	8
• Trees with Flood Stage below Branches	0.080	0.100	0.120	8
MAJOR NATURAL STREAMS - TYPE I CHANNEL The roughness coefficient is less than that for minor stream of similar description because banks offer less effective resistance.				
• Moderately Well Defined Channel	0.025	---	0.060	8
• Irregular Channel	0.035	---	0.100	8
UNLINED VEGETATED CHANNELS - TYPE II CHANNEL				
• Mowed Grass, Clay Soil	0.025	0.030	0.035	8
• Mowed Grass, Sandy Soil	0.025	0.030	0.035	6

TABLE 8

CULVERT DISCHARGE VELOCITIES

Culvert Discharges On	Maximum Allowable Velocity (f.p.s.)
Earth (Sandy)	6
Earth (Clay)	8
Sodded Earth	8
Rock Gabions (Engineered)	12
Concrete	15
Shale (Lime Stone)	10

VII - LIST OF FIGURES

Insert Figures 1 through 30

VIII - LIST OF FORMS

Form

A. Storm Water Runoff Calculations

B. Inlet Design Calculations

C. Storm Sewer Calculations

D. Water Surface Profile Calculations

E. Open Channel Calculations

F. Hydraulic Design of Culverts

G. Bridge Design Calculations

NOTE: A copy of each applicable form must be submitted with the drainage plans to the City to review. Final plans must include these forms in the drainage plans.

STORM WATER RUNOFF CALCULATIONS - FORM "A"

Column 1	Location of the drainage structure for which the runoff calculation is being made or a design point on an open channel.
Columns 2 thru 6	Are to be used in calculating runoff by the Rational Method.
Column 2	Obtained from TABLE 1, or FIGURE 2
Column 3	Using the appropriate Design Storm Frequency, and the Time of Concentration in Column 2, the Intensity is obtained from FIGURE 1.
Column 4	Size of the drainage area tributary to the point of design shown in Column 1.
Column 5	Taken from TABLE 1 and is a weighted composite value if several different zoning districts fall within the drainage area.
Column 6	Column 3 multiplied by Columns 4 and 5.
Columns 7 thru 19	Are to be used in calculating runoff by the Unit Hydrograph Method.
Column 7	Taken from TABLE 2.
Column 8	Measured distance along the stream course from the upper-most limit of the drainage area to the point of design shown in Column 1.
Column 9	Measured distance along the stream course from the point of design shown in Column 1 to the measured center of gravity of the drainage area.
Column 10	A computed value using the values shown in Columns 7, 8 and 9.
Column 11	Taken from TABLE 2.

Column 12 Column 11 divided by Column 10.

Column 13 Size of the drainage area tributary to the plant of design shown in Column 1.

Column 14 Column 12 multiplied by Column 13.

Column 15 Using the appropriate Design Storm Frequency and a duration of two hours, this value is obtained from FIGURE 1.

Column 16 Obtained by multiplying the value in Column 15 times two.

Column 17 Constant value of 1.11 inches for the Ovilla geographic area.

Column 18 Result of subtracting Column 17 from Column 16.

Column 19 Column 14 multiplied by Column 18.

Column 20 The flow used for design depends on the size of the drainage area. If the size of the drainage area is less than 600 acres, Q_R should be entered. If the drainage area is larger than 600 acres and smaller than 1200 acres, the larger of the two flows (Q_R and Q_u) should be entered. If the drainage area is larger than 1200 acres, Q_u should be entered.

Form “A”
Storm Water Runoff Calculations

INLET DESIGN CALCULATIONS - FORM "B"

Column 1 Inlet number or designation. The first inlet shown is the most upstream.

Column 2 Construction plan station of the inlet.

Column 3 Design Storm Frequency is same as the Design Storm Frequency of the storm sewer.

Column 4 Time of concentration for each inlet is taken from TABLE 1, or FIGURE 2.

Column 5 Using the time of concentration and the Design Storm Frequency, rainfall intensity is taken from FIGURE 1.

Column 6 Runoff Coefficient is taken from TABLE 1 according to the zoning of the drainage area.

Column 7 Area drained by the specific inlet. Care should be taken to keep the drainage area flow separate into the appropriate street gutters.

Column 8 Product of Column 5 multiplied by Columns 6 and 7.

Column 9 If there is any flow that was not fully intercepted by an upstream inlet, it should be entered here.

Column 10 Sum of Columns 8 and 9.

Column 11 Capacity of the street gutter, in which the inlet is located, from either FIGURES 3, 4, 5 or 6. If the total gutter flow shown in Column 10 is in excess of the value in Column 11 the inlet should be moved upstream. If it is substantially less than the value in Column 11, an investigation should be made to see if the inlet could be moved downstream.

Column 12 Street gutter slope to be used in selecting the proper size inlet.

Column 13 Crown type of the street on which the inlet is located.

Column 14 Selected size of the inlet taken from FIGURES 8 through 22.

Column 15 Inlet type taken from FIGURE 7.

Column 16 If the selected inlet does not intercept all of the gutter flow, the difference between the two values should be entered here and in Column 9 of the inlet that will intercept the flow.

FORM "B"
INLET DESIGN CALCULATIONS

STORM SEWER CALCULATIONS - FORM "C"

Column 1	Upstream station of the section of conduit being designed. Normally, this would be the point of a change in quantity of flow, such as an inlet, or a change in grade.
Column 2	Downstream station of the section of conduit being designed.
Column 3	Distance in feet between the upstream and downstream stations.
Column 4	Drainage sub-area designation from which flow enters the conduit at the upstream station.
Column 5	Area in acres of the drainage sub-area entering the conduit.
Column 6	Runoff coefficient, obtained from TABLE 1, based on the characteristics of the subdrainage area.
Column 7	Column 5 multiplied by Column 6.
Column 8	Obtained by adding the value shown in Column 7 to the value shown immediately above in Column 8.
Column 9	This time in minutes is transposed from Column 19 on the previous line of calculations. The original time shall be equal to the time of concentration as shown on TABLE 1 or FIGURE 2, whichever value has been used.
Column 10	Design Storm Frequency.
Column 11	Using the time at the upstream station shown in Column 9 and the Design Storm Frequency shown in Column 10, this value is taken from FIGURE 1.

Column 12	Column 8 multiplied by Column 11.
Column 13	This slope should be computed from the profile of the ground surface. Normally, the hydraulic gradient will have a slope approximately the same as the proposed conduit and will be located above the inside crown of the conduit.
Column 14	Utilizing the values in Columns 12 and 13, a conduit size should be selected. In the case of concrete pipe, FIGURE 23 may be used.
Column 15	Velocity in the selected conduit based on the values in Columns 12, 13 and 14. Taken from FIGURE 23 for concrete pipe.
Column 16	Friction head loss is the product of Column 3 times Column 13.
Column 17	Calculation is made utilizing the values of Column 15 V_1 = Upstream Velocity V_2 = Downstream Velocity Head gains shall be taken to zero (0) in the storm sewer design.
Column 18	Calculation is based on the values of Columns 3 and 15.
Column 19	Sum of Columns 9 and 18.
Column 20	Special design comments may be entered here.

Form “C”
STORM SEWER CALCULATIONS

WATER SURFACE PROFILE CALCULATIONS - FORM "D"

Column 1	At each point where a water surface elevation is desired, a cross section must be obtained. The sections are numbered and subdivided according to the assigned roughness coefficient.
Column 2	Known or assumed water surface elevation at the particular section.
Column 3	Distance along the channel between sections.
Column 4	Area of sub-section calculated from plotted cross sections.
Column 5	Wetted perimeter of each sub-section exclusive of the water interfaces between adjacent sub-sections.
Column 6	Column 4 divided by Column 5. (Hydraulic Radius)
Column 7	Column 6 raised to 2/3 power.
Column 8	Roughness coefficient for Manning's formula from TABLE 7.
Column 9	Column 4 multiplied by 1.486 and the product divided by Column 8.
Column 10	Column 9 multiplied by Column 7.
Column 11	The total flow shown in the upper left of the calculation form divided by Column 10 and squared, which is the friction slope.
Column 12	Average friction slope between sections.
Column 13	Column 12 multiplied by Column 3.
Column 14	Flow in each individual sub-section. Varies directly with the conveyance

factor shown in Column 10. The sum of the values must equal the total flow.

Column 15 Column 14 divided by Column 4.

Column 16 Column 15 squared.

Column 17 Column 16 multiplied by Column 14.

Column 18 Sum of the values in Column 17 of a particular section divided by twice the acceleration of gravity and multiplied by the total flow.

Column 19 Algebraic difference in velocity heads between sections.

Column 20 Eddy losses are calculated as 10 percent of the value of Column 19 when such value is positive and 50 percent of the absolute value of Column 19 when such value is negative.

Column 21 Sum of Column 13, Column 19 and Column 20.

Column 22 The sum of the value shown in Column 2 for the previous section and the value in Column 21. If the elevations calculated for subsequent sections do not agree within a reasonable limit with the assumed elevations shown in Column 2 for that particular section, then the assumed elevations for such section must be revised and the section properties recomputed until the desired accuracy is obtained. An accuracy of + 0.3 feet is considered a reasonable limit.

Form "D"

OPEN CHANNEL CALCULATIONS - FORM "E"

Column 1	Downstream limit of the section of channel under consideration.
Column 2	Upstream limit of the section of channel under consideration.
Column 3	Type of channel as shown in FIGURE 24 is entered here.
Column 4	Flow in the section of channel under consideration.
Column 5	Roughness coefficient of the channel cross-section taken from TABLE 7.
Column 6	Slope of the channel that is most often parallel to slope of the hydraulic gradient.
Column 7	Square root of Column 6.
Column 8	Calculation is made using the values in Columns 4, 5 and 7.
Column 9	Assumed width of the bottom width of the channel.
Column 10	Assumed depth of flow.
Column 11	Assumed slope of the sides of the channel.
Column 12	Areas of flow that are calculated based on Columns 9, 10 and 11.
Column 13	Wetted perimeter calculated from Columns 9, 10 and 11.
Column 14	Value is calculated from Columns 12 and 13.
Column 15	Column 14 raised to 2/3 power.
Column 16	Product of Column 13 times Column 15.

When the value of Column 16 equals the value of Column 8 the channel has been adequately sized. When the value of Column 16 exceeds the value of Column 8 by more than five percent then the channel width or depth should be decreased and another trial section analyzed.

Column 17 Calculation is based on the values of Columns 4 and 12.

Column 18 Calculation is based on Column 17.

Column 19 Remarks concerning the channel section analyzed may be entered.

NOTE: Form "E" should be used only to size open channels. Form "D" should be used to calculate stream profile.

FORM “E”

HYDRAULIC DESIGN OF CULVERTS, FORM "F"

INFORMATION IN UPPER RIGHT OF SHEET:

Culvert Location:	This is a word description of the physical location.
Length:	The actual length of the culvert.
Total Discharge, QT:	This is the flow computed on FORM "A".
Design Storm Frequency:	Obtained from TABLE 1 and used on FORM "A".
Roughness Coefficient, n:	Obtained from TABLE 5.
Maximum Velocity:	Obtained from TABLE 4.
Tailwater:	This is the design depth of water in the downstream channel and is obtained in connection with the channel design performed on FORM "D" or FORM "E".
D. S. Channel Width:	This is the bottom width of the downstream channel obtained from the calculations on FORM "E". The culvert should be sized to approximate this width whenever possible.
Entrance Description:	This is a listing of the actual condition as shown in the "Culvert Entrance Data" shown on the calculation sheet.
Roadway Elevation:	The elevation of the top of curb at the upstream end of culvert.
U. S. Culvert F. L.	The flow line of the culvert at the upstream end.
Difference:	The difference in elevations of the roadway and the upstream flow line.
Required Freeboard:	The vertical distance required for safety between the upstream design water surface and the roadway elevation or such other requirements that may occur because of particular physical conditions.
Allowable Headwater:	This is obtained by subtracting the freeboard from the difference shown immediately above.
D.S. Culvert F.L.	The flow line elevation of the downstream end of the culvert.
Culvert Slope, So:	This is the physical slope of the structure calculated as indicated.

Columns 1 through 10 deal with selection of trial culvert size and are explained as follows:

Column 1 Total design discharge, Q, passing through the culvert divided by the allowable maximum velocity gives trial total area of culvert opening.

Column 2 Culvert width should be reasonably close to the channel bottom width, W, downstream of the culvert.

Column 3 Lower range for choosing culvert depth is trial area of culvert opening, Column 1, divided by channel width, Column 2.

Column 4 Allowable headwater obtained from upper right of sheet.

Column 5 Trial depth, D, of culvert corresponding to available standard sizes and between the numerical values of Columns 3 and 4.

Columns 6, 7 and 8 are solved simultaneously based on providing a total area equivalent to the trial area of opening in Column 1.

Column 6 Number of culvert openings.

Column 7 Inside width of one opening.

Column 8 Inside depth of one opening if culvert is box structure or diameter if culvert is pipe.

Column 9 Column 6 multiplied by Column 7 and Column 8.

Column 10 Total discharge divided by number of openings shown in Column 6.

Columns 11 through 15 (Inlet Control) and 16 through 27 (Outlet Control) deal with Headwater Calculations which verify hydraulics of trial culvert selected and are explained as follows:

Column 11 Obtained from upper right of sheet.

Column 12 When the allowable headwater is equal to or less than the value in Column 8, enter Case I. When the allowable headwater is more than the value in Column 8, enter Case II.

Column 13	Column 10 divided by Column 7.
Column 14	Obtained from FIGURE 25 for box culverts or FIGURE 26 for pipe culverts.
Column 15	Column 14 multiplied by Column 8.
Column 16	Obtained from upper part of sheet.
Column 17	Obtained from FIGURE 27 for box culverts and FIGURE 28 for pipe culverts.
Column 18	Tailwater depth from upper right of sheet.
Column 19	So, culvert slope, multiplied by culvert length, both obtained from upper right of sheet.
Column 20	Sum of Columns 17 and 18 minus Column 19.
Column 21	Obtained from FIGURE 27 for box culverts and FIGURE 28 for pipe culverts.
Column 22	Critical depth obtained from FIGURE 29 for box culverts and FIGURE 30 for pipe culverts.
Column 23	Sum of Columns 22 and 8 divided by two.
Column 24	Tailwater depth from upper right of sheet.
Column 25	Enter the larger of the two values shown in Column 23 or Column 24.
Column 26	Previously calculated in Column 19 and may be transposed.
Column 27	The sum of Columns 21 and 25 minus Column 26.
Column 28	Enter the larger of the values from Column 15, Column 20 or Column 27. This determines the controlling hydraulic conditions of the particular size culvert investigated.
Column 29	When the Engineer is satisfied with the hydraulic investigations of various culverts and has determined which would be the most economical selection, the description should be entered.

Form “F”

Form "Fb"

BRIDGE DESIGN CALCULATIONS - FORM "G"

Columns 1 & 2	Obtained from calculations on FORM "A".
Column 3	Assume an average velocity that is less than the maximum allowable velocity and more than 4 feet per second. Maximum velocities are equal to those specified for open channels.
Column 4	Total flow as shown on upper part of sheet divided by Column 3.
Column 5	Column 4 divided by Column 2.
Column 6	Selected bridge length utilizing standard span lengths.
Column 7	Calculated from bridge and channel geometrics.
Column 8	Total flow through bridge divided by Column 7.
Column 9	Selected head loss coefficient based upon specific conditions.
Column 10	Calculated utilizing values in Columns 8 and 9.

FORM “G”