SECTION II

II- STREET DESIGN STANDARDS

A. DEFINITIONS

	TABLE 1					
	Type	**Designation**	**R-O-W**	**Pavement (Face to Face)**	**Median (Face to Face)**	**Parkway Width**
THOROUGHFARES	Principle 6 Lane Divided	P6D	120'	2-36'	20'	13.5'
	Major 6 Lane Divided	M6D	120'	2-36'	20'	13.5'
	Principle 4 Lane Divided	P4D	100'	2-24'	20'	15.5'
	Major 4 Lane Divided	M4D	100'	2-24'	20'	15.5'
	Principle 4 Lane Undivided	P4U	70'	44'	None	12.5'
	Major 4 Lane Undivided	M4U	70'	44'	None	12.5'
	Regional 4 Lane Divided	R4D	110'	2-24'	20'	20'
	Regional 2 Lane Divided	R2D	90'	2-24'	20'	10'
RESIDENTIAL STREETS	Collector Street	C2U	65'	36'	None	14'
	Estate Street	E2U	60'	32'	None	14'
	Local Street	L2U	50'	26'	None	11.5'

Above defined by the City of Farmersville, Texas, Comprehensive Plan and most recent Major Thoroughfare Plan.

B. MINIMUM HORIZONTAL DESIGN RADIUS

Minimum Centerline Radius is defined by the design speed of the respective street. The design speed of each street in the City of Farmersville, as defined by the Thoroughfare Plan, can be determined from Table 2.

TABLE 2
DESIGN SPEED OF EACH TYPE OF STREET

Street Type	Design Speed
P6D, M6D	45 mph
P4D, M4D, R4D, R2D	40 mph
P4U, M4U, C2U	35 mph
E2U, L2U	30 mph

The minimum acceptable horizontal centerline radius, for each respective street's design speed, is shown in Table 3. The cross slope is assumed to be ¼" per foot from the inside toward the outside.

TABLE 3
MINIMUM HORIZONTAL CENTERLINE RADIUS

Y (mph)	f	e (ft/ft)	(e+f)	R (Calculated) (ft)	R (Rounded for Design) (ft)
25	0.170	-0.0208	0.1492	279.27	280
30	0.160	-0.0208	0.1392	431.03	440
35	0.150	-0.0208	0.1292	632.09	640
40	0.145	0.0208	0.1242	858.83	860
45	0.142	0.0208	0.1212	1,113.86	1,120
50	0.140	-0.0208	0.1192	1,398.21	1,400
55	0.130	-0.0208	0.1092	1,846.76	1,850
60	0.120	-0.0208	0.0992	2,419.35	2,420

Minimum centerline design radius for residential streets shall be 280-feet for curves with a length over 125 feet long. Other important considerations in the design of curves on city streets and thoroughfares include the location of intersecting streets, drives, bridges and other topographic features. In residential areas long, straight streets are discouraged. The maximum allowable tangent length on a street in a residential area is 600 feet, and a maximum allowable curve radius in 1,000 feet. Curvilinear streets in a residential area will be evaluated on a case by case basis. When reverse curves are designed into a roadway the stopping sight distance must be maintained throughout the section. Reverse horizontal curves must be separated by a minimum 100 foot tangent section, and the centerline offset from the initial tangent to the final tangent must be a minimum of 30 feet.

C. SUPERELEVATION

When super elevation is used on secondary and major thoroughfares as approved by the City

Engineer, use the following equation to calculate the rate of super elevation necessary for the design radius:

$$E = \frac{V^2}{15R} - f$$

Where:

E = rate of roadway super elevation, foot per foot
f = side friction factor (See Table 3)
V = vehicle design speed, mph
R = radius of curve in fee

The maximum allowable rate of super elevation for urban roadways in the City of Farmersville is 4%. When used, super elevation runoff must be designed consistent with TXDOT Roadway Design Manual and AASHTO Green Book.

D. MINIMUM VERTICAL ALIGNMENT

Vertical Alignment is a function of Stopping Sight Distance (SSD), which is given by:

$$SSD = 1.47PV + \frac{V^2}{30(f+g)}$$

Stopping Sight Distances are calculated for g – 0, rates of vertical curvature are derived from the AASHTO Green Book and used (K) to determine crest curve lengths per Table 4 & Table 5.

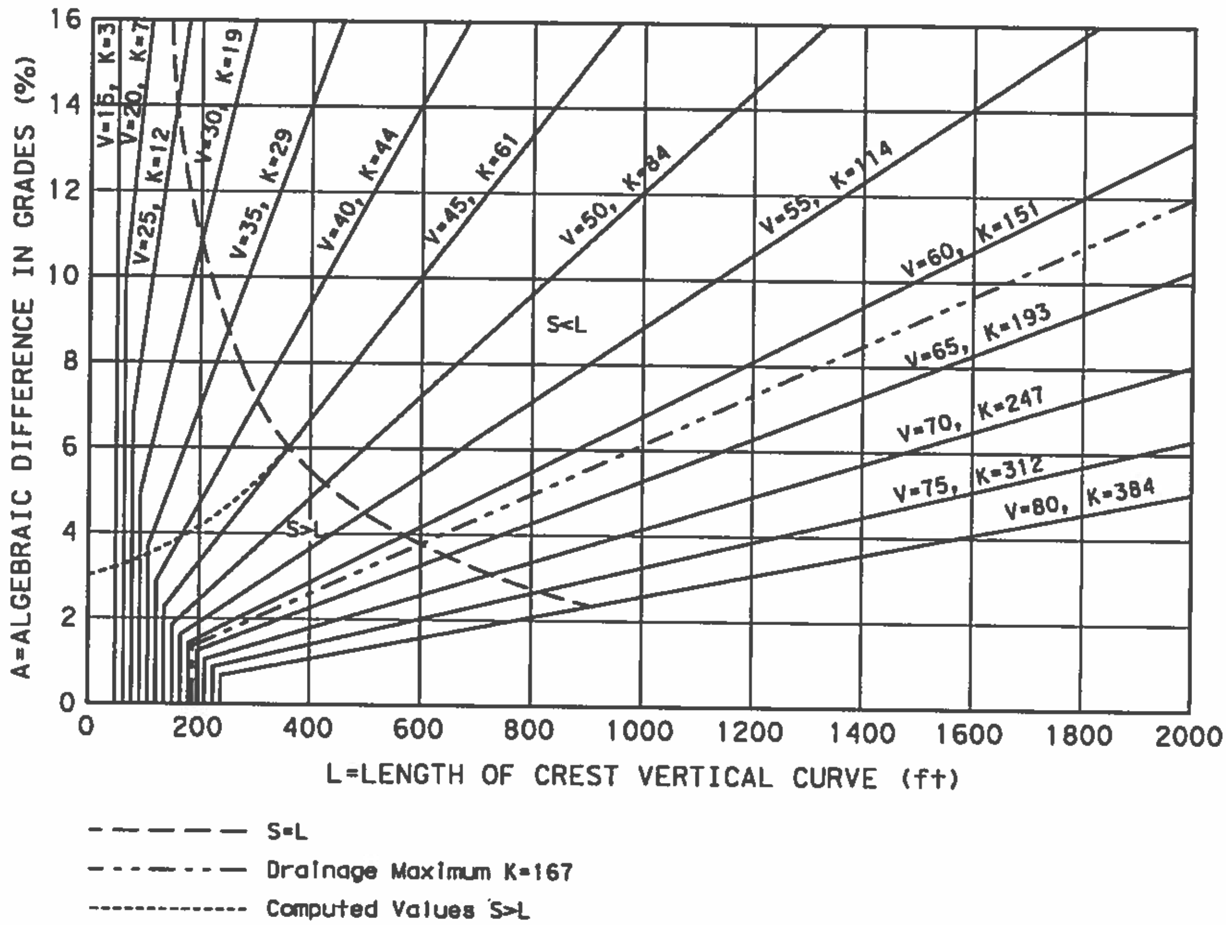
The maximum grade for residential streets is 10% unless otherwise approved by the City where natural topography is such as to require steeper grades. The maximum grade for all other streets shall be 7.50%. The minimum grade for all streets is 0.50%.

TABLE 4

MINIMUM ACCEPTABLE CREST CURVE GIVEN SPEED AND DIFFERENCE IN GRADE OF ROAD

V	S	K	L-KA									
MPH	FT	FT	A-1	A-2	A-3	A-4	A-5	A-6	A-7	A-8	A-9	A-10
30	200	19	100	100	100	100	100	120	140	150	170	190
35	250	29	120	120	120	120	150	180	200	230	260	290
40	325	44	140	140	140	180	220	270	310	350	400	440
45	400	61	150	150	180	250	300	370	430	490	550	610
50	475	84	160	170	250	340	420	500	590	670	760	840
55	550	114	170	230	340	460	570	690	800	920	1030	1140
60	650	151	180	300	450	600	760	910	1060	1210	1360	1510

CURVE 4.1



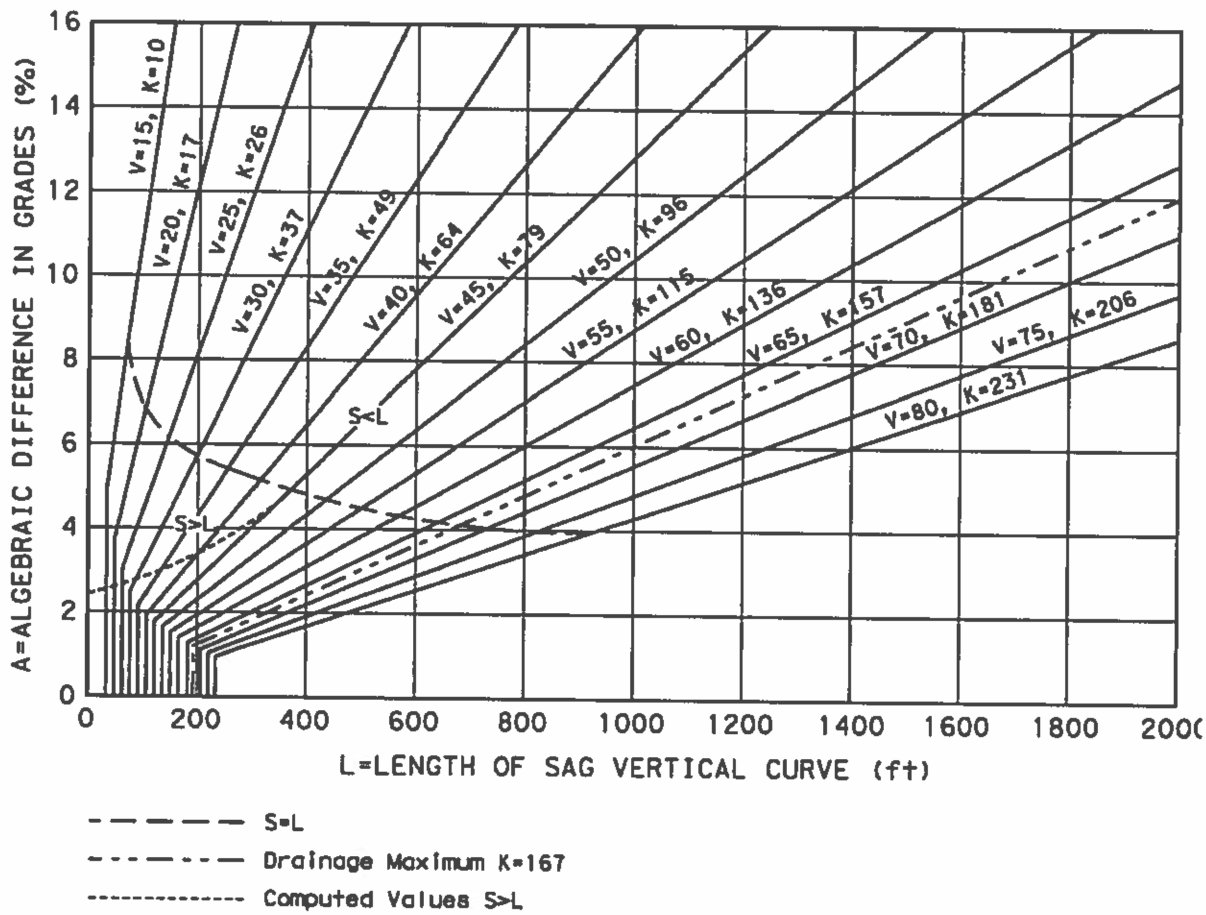
DESIGN CONTROLS FOR CREST VERTICAL CURVES (US CUSTOMARY)

TABLE 5

MINIMUM ACCEPTABLE SAG CREST CURVE GIVEN SPEED AND DIFFERENCE IN GRADE OF ROAD

V	S	K	L-KA									
MPH	FT	FT	A-1	A-2	A-3	A-4	A-5	A-6	A-7	A-8	A-9	A-10
30	200	37	100	100	120	160	200	240	280	320	360	400
35	250	49	100	100	150	200	250	300	350	400	450	500
40	325	64	100	130	190	260	320	390	450	520	580	640
45	400	79	110	160	240	320	400	480	560	640	720	790
50	475	96	120	200	290	390	480	580	670	770	870	960
55	550	115	140	230	350	460	580	690	810	920	1040	1150
60	650	136	160	280	410	550	680	820	960	1090	1230	1360

CURVE 5.1



DESIGN CONTROLS FOR SAG VERTICAL CURVES (US CUSTOMARY)

E. INTERSECTION CURB RADII

The radius shall be thirty (30) feet at the intersection of all intersecting streets unless otherwise approved by the City engineer or Authorized Representative. See Figure 1.

Note: At many intersections, the curb radius encroaches on the right-of-way so as to not provide sufficient room for sidewalks, utilities, etc. within the parkway. Therefore, right-of-way will be dedicated at the intersection of all streets such that a minimum of nine and one-half (9.5) feet of parkway shall be maintained from the back of the curb along the curb's radius.

F. RESIDENTAIL FRONTAGE

Residential houses shall not front a thoroughfare unless parallel access roads are provided. Minimum distances between adjacent curbs or the thoroughfare and the access road shall be twenty (20) feet.

G. STATE DESIGNATED ROADS

All such roads within the City of Farmersville will conform to State Design Standards unless otherwise directed by the City Engineer.

SECTION III

III - MEDIAN AND LEFT TURN LANE DESIGN STANDARDS

A. WIDTH OF MEDIAN

Median widths vary from a minimum of 4' (with left turn lanes) to a maximum of 20' (see Table 1).

B. REQUIRED MEDIAN OPENING AND LEFT-TURN LANE

Median openings on divided thoroughfares shall be provided at all dedicated street intersections and at private drives where they conform to the City's spacing requirements. A left turn lane for the proposed drive or street shall accompany the median opening.

C. COST OF MEDIAN OPENINGS AND LEFT-TURN LANES

Median openings and left-turn lanes constructed to serve private drives and new roads shall be paved to City standards, inspected by City Inspectors, and paid for by owners served by the median openings and left-turn lanes. The City shall be responsible for, and pay the costs of, the paving of median openings and left-turn lanes, constructed to serve existing dedicated streets, and those that exist for drives, when a part of the Capital Improvement widening program is undertaken by the City on an existing public street.

D. MINIMUM LEFT-TURN STORAGE, TRANSITION LENGTH, AND MEDIAN OPENING WIDTH, LOCATION, AND SPACING REQUIREMENTS

(1) Left Turn Storage
All left-turn storage areas shall be ten (10) feet wide with minimum storage requirements for left-turn lanes as in Table 6.

TABLE 6
MINIMUM LEFT-TURN STORAGE REQUIREMENTS

Intersecting Thoroughfares	Minimum Storage	
Principle with Principle	150	feet
Principle with Major	100	feet
Principle with Collection/Local	60	feet
Principle with Private Drive	60	feet
Major with Principle	100	feet
Major with Collection/Local	60	feet

Major with Private Drive..60 feet

(2) Transition Length

The transition curves used in left-turn lanes shall be two 250-foot radius reverse curves, which will require a total transition length of 100-feet.

(3) Median Openings

a) Median openings at Intersections shall be from right-of-way to right-of-way or the intersecting street.

b) the minimum width of mid-block median opening shall not be less than sixty (60) feet. See Figure 1.

(4) Medians Where No Left-Turn Pocket is Needed

a) If left-turn storage is provided in only one dir4ection, (i.e., a drive cannot be installed for the other direction), the minimum length of median must be the required left-turn storage and transition length, plus 30-feet of median length beyond the end of the transition.

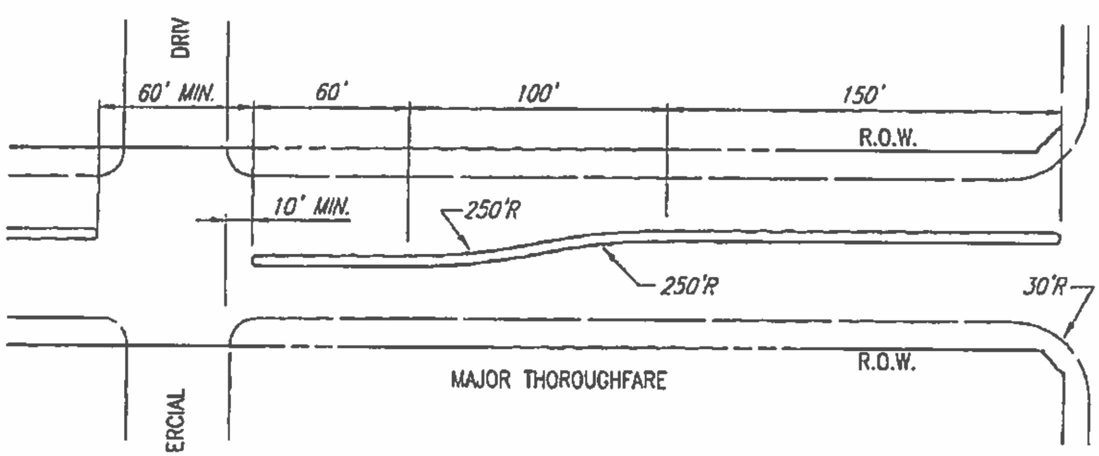
b) If the left turn storage is not required in either direction, but the median is simply a spacer between two median openings, the minimum length of the spacer must be 50-feet. See Figure 2.

(5) Medians into Developments on Public Streets

Medians installed on undivided streets at entrances to subdivisions for aesthetic or any other purpose will be a minimum of 4-feet wide and 100-feet long.

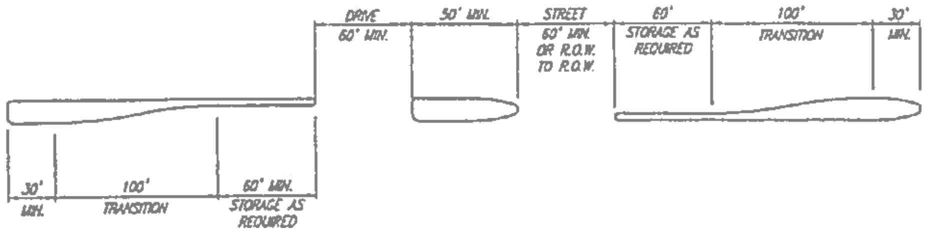
Note: Storage requirements listed herein are absolute minimums. Storage requirements may increase based upon actual and projected traffic demands.

FIGURE 1

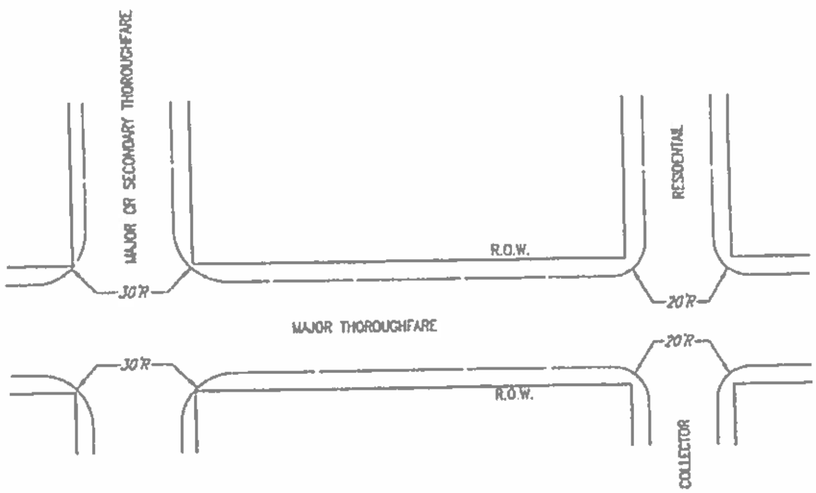


TYPICAL MEDIAN OPENING SPACING
MAJOR THOROUGHFARE

FIGURE 2



TYPICAL MEDIAN DIMENSIONS WITHOUT BACK TO BACK LEFT TURN POCKETS



CURB RADII AT INTERSECTION

SECTION IV

IV - ALLEY DESIGN STANDARDS

A. ALLEY REQUIREMENTS FOR DEVELOPMENTS

Alleys shall be constructed in accordance with City of Farmersville Subdivision Ordinance. Alleys shall be provided in all residential areas and shall be paved with concrete in accordance with the City's Standard Construction Details. The City Council may waive the residential alley requirement upon determination of the Council that such a waiver is in the best interest of the city. Alleys may be required in commercial and industrial development. The city may waive the commercial and industrial alley requirement upon detemination of the council, if in its opinion adequate provisions are made for service access such as off-street loading, unloading and parking consistent with the uses proposed.

B. ALLEY INTRSECTIONS

Alleys shall not intersect major or secondary thoroughfares with medians. Alleys which run parallel to and share a common right-of-way line with a major thoroughfare shall turn away from the major street not less than one subdivision lot width or a minimum of 50-feet (whichever is greater) from the cross-street intersection.

C. ALLEY WIDTHS

The minimum alley right-of-way width shall be twenty (20) feet with a minimum 12-foot paved width. Dead-end alleys shall not be permitted without special permission from the City Engineer or Authorized Representative. The geometry of alley construction shall conform to the Standard Construction Details.

D. ALLEY RADIUS

Alley radii at street intersections in residential developments shall not be less than 10-feet. Alley radii at street intersections in commercial and residential developments shall not be less than 30-feet unless approved by the City Engineer or Authorized Representative.

SECTION V

V - DRIVEWAY DESIGN STANDARDS

A. DEFINITION OF DRIVEWAY TYPES

For purposes of interpreting the provisions of these Rules and Regulations, the following definitions shall apply:

(1) A "residential" driveway provides access to a single-family residence, to a duplex, or to a multi-family building containing five or fewer dwelling units. These drives shall intersect residential and commercial roadways only. All access to residential property abutting all other thoroughfares shall be off the alley or a service road. All residential driveway approaches shall be concrete.

(2) A "commercial" driveway provides access to an office, retail or institutional building, or to a multiple-family building having more than five dwelling units. It is anticipated that such buildings will have incidental truck service. Commercial drives shall access to Major or Secondary Thoroughfares only. All commercial driveway approaches shall be concrete.

(3) An "industrial" driveway serves substantial numbers of truck movements to and from loading docks of an Industrial facility, warehouse, or truck terminal. A central retail development, such as a community or regional shopping center, may have one or more driveways specially designed, signed, and located to provide access for trucks and such driveways shall be considered industrial driveways. Industrial plant driveways whose principle function is to serve administrative or employee parking lots shall be considered commercial driveways. Industrial drives shall access to Major or Secondary Thoroughfares only. All industrial driveway approaches shall be concrete.

Note: Two-way driveways shall always be designed to intersect the street at a 90° angle. One-way driveways may be designed to intersect a street at a 45° angle.

B. DRIVEWAY WIDTH

As the term is used here, the width of a driveway refers to the width of pavement at the property line.

(1) Residential driveway onto streets shall have a minimum width of 12-feet and a maximum width of 24-feet. Joint access residential drives shall have no less that nine (9) feet on any property. See Figure 4.

(2) Commercial/Industrial. Two-way operation: See Figure 5.

   a) Commercial driveways shall have a minimum width of twenty-four (24) feet and a maximum width of thirty (30)-feet.

b) Industrial driveways shall have a minimum width of 30-feet and a maximum width of 40-feet. Joint access commercial/industrial drives shall have no less than ten (10) feet on any property, with the full drive width and access pavement to the property built for the development at the same time.

(3) Commercial/Industrial – One way operation:

a) 90-degree drives shall have a width of 18-feet for ingress and 22-feet for egress, with the separation median width being a minimum of 4-feet and a maximum of 10-feet. See Figure 6.

b) 45-degree drives shall have a width of 18-feet for ingress and 16-feet for egress, with the separation median width being a minimum of 4-feet and a maximum of 10-feet. Joint access commercial/industrial drives shall have no less than 10-feet on any property, with the full drive width and access pavement to the property built for the development at the same time. See Figure 7.

C. DRIVEWAY RADIUS

All driveways intersecting dedicated streets shall be built with a circular curb radius connecting the 6-inch raised curb of the roadway to the design width pavement of the driveway. All driveways shall provide for barrier free access. Driveway radii shall fall entirely within the subject property so as to begin at the street curb, at the extension of the property line.

(1) 90-degree Intersection (See Detail)

a) The curb radii for a residential drive shall be a minimum, of 5-feet and a maximum of 10-feet.

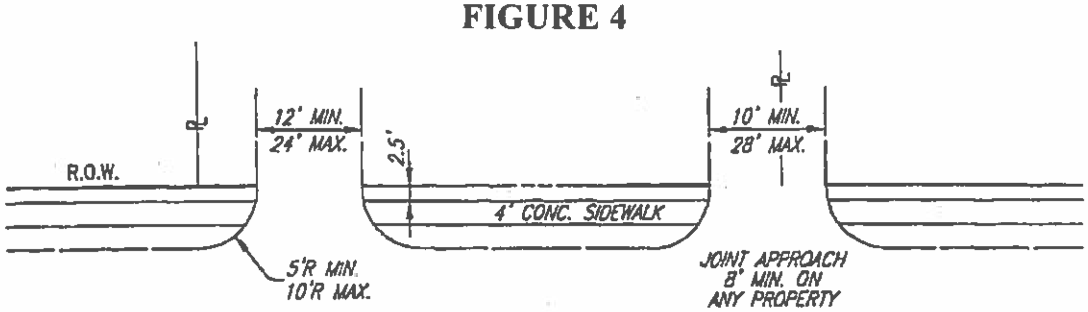
b) The curb radii for commercial and industrial drives shall be 30-feet unless otherwise approved by the City.

(2) 45-degree Intersection

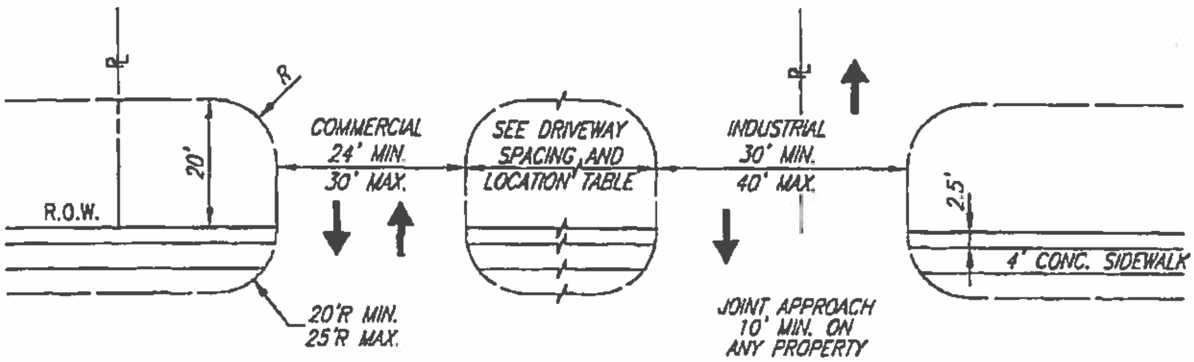
The curb radii shall be 5-feet for the outside of the drive and 2-feet for the median. See Detail.

In order that the definition of the location of the edge of pavement for the thoroughfare may be maintained, driveway radii shall always be designed to become tangent to the street curb line. All commercial and industrial drives will have an unbroken curb length of not less than 20-feet from the right-of-way, or 30-feet from the roadway curb extending into the site on each side of the drive.

FIGURE 4

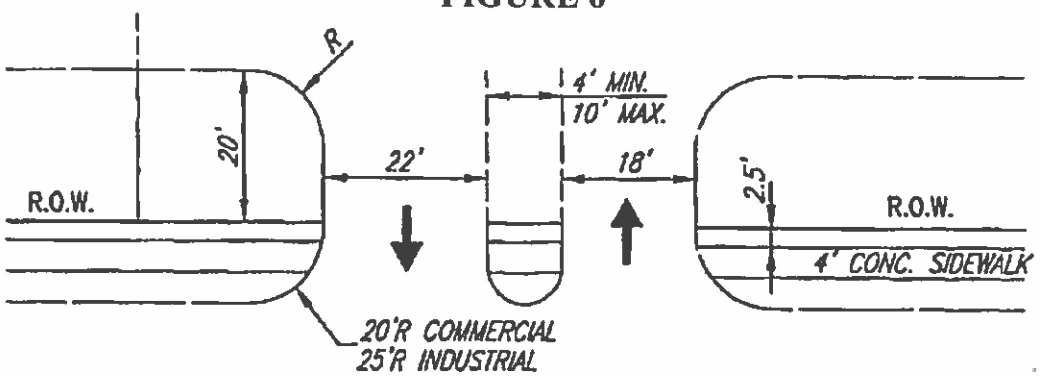


DRIVEWAYS WIDTH, RADIUS, SPACING
FIGURE 5



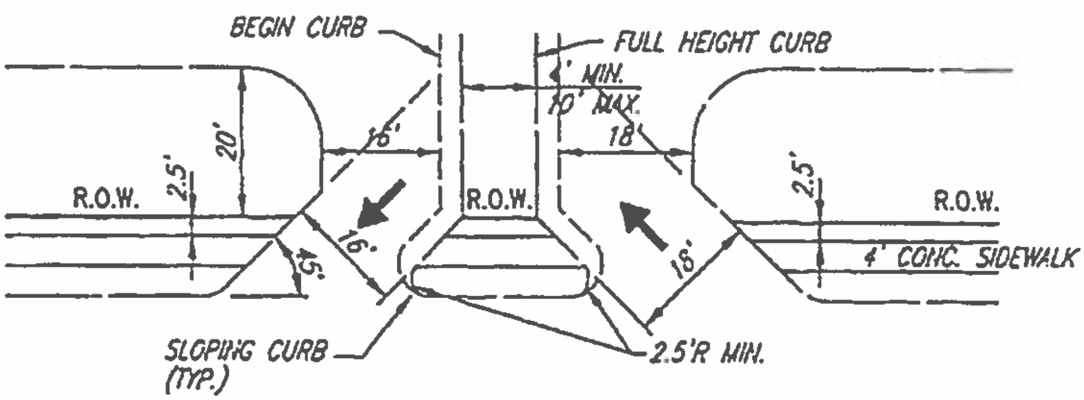
DRIVEWAYS WIDTH, RADIUS, SPACING

FIGURE 6



DRIVEWAYS WIDTH, RADIUS, SPACING

FIGURE 7



DRIVEWAYS WIDTH, RADIUS, SPACING

D. DRIVEWAY SPACING AND LOCATION IN RELATION TO OTHER DRIVES

(1) Residential
Driveway approaches on a tract of land devoted to one use shall not occupy more than 70% of the frontage abutting the roadway. No more than two driveway approaches shall be permitted on any parcel of property on each street.

(2) Commercial and Industrial
The spacing and location of driveways shall be related to both existing adjacent driveways and those shown on approved development plans. The spacing between driveways shall depend upon the design speed of the street as shown in Table 7. Driveways shall not be permitted in the transition area of a deceleration lane or a right turn lane.

TABLE 7
DRIVEWAY SPACING IN RELATION TO OTHER DRIVES GIVEN THE DESIGN SPEED OF THE STREET

Design Speed (MPH)	Driveway Spacing (Ft.)
25	65
30	90
35	100
40	120
45	150
50	200

The minimum spacing shall not be more than 10-feet less than shown above. Spacing between driveways will be measured along the property line from the edge of one driveway to the closest edge of the next driveway and not from centerline to centerline.

E. DRIVEWAY SPACING IN RELATION TO A CROSS STREET

(1) 90 Degree Intersection – Drive to Road

a) Driveways that intersect at 90 degrees to a residential or "secondary street" shall be located a minimum of the drive radius from a residential street's end of curb radius.
b) A driveway that intersects at 90 degrees to a residential or secondary street shall be located a minimum of thirty (30) feet from a secondary or major street's end of curb radius. See Figure 8.

c) A driveway that intersects at 90 degrees to a major street shall be located a minimum of 100-feet from any intersecting street's right-of-way or from the end of any intersecting street's curb radius as determined by the City Engineer. If the property length, along the street, is such that both the drive and the drive's curb radius cannot be totally within the proposed development, the drive will be situated so as to be a joint access drive. See Figure 9.

(2) 45 Degree Intersection – Drive to Road

a) If one-way angle drives are used, the radius for the driveway on a residential or secondary may not begin less than 35-feet from an intersecting street's end of curb radius.

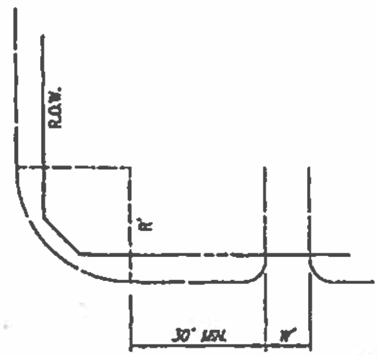
b) On a major street the drive shall be located a minimum of 100-feet from any intersecting street's right-of-way. If a property length, along the street, is such that both the drive and drive's curb radius cannot be totally within the proposed development, the drive will be situated so as to be a joint access drive. See Figure 10.

A summary of driveway widths, radii, and angle requirements are given in Table 8

TABLE 8
SUMMARY OF DRIVE REQUIREMENTS

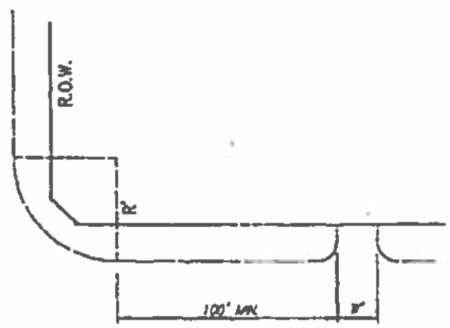
	Residential	Commercial	One-Way In	One-Way Out	Industrial
Width (ft)					
Minimum	12	20			30
One-way (only)					
90°			18	22	
45°			18	16	
Maximum	24	30			40
Curb Radius (ft)					
45° (one-way)	5	10	10	10	10
90°	5 - 10	30	Same	Same	30
Intersection					
Angles (deg.)	90° 45°	90° 45°	90° 45°	90° 45°	90° 45°

FIGURE 8



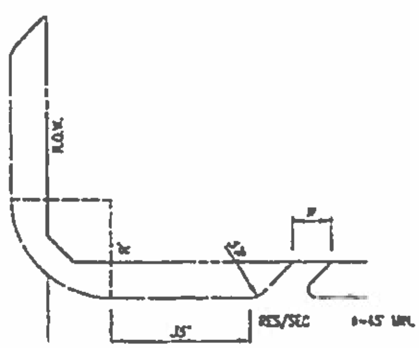
DRIVE INTERSECTION A RESIDENTIAL OR SECONDARY

FIGURE 9



90° DRIVE INTERSECTING A MAJOR

FIGURE 10



ANGLE DRIVE

SECTION VI

VI - SIDEWALK AND LOCATION DESIGN STANDARDS

A. DEFINITION OF SIDEWALK

A sidewalk is the paved area in a street ROW between the curb lines or the edge of pavement of the roadway and the adjacent property lines for the use of pedestrians. The City of Farmersville considers a sidewalk to be an "accessible route" as specified in Section 4.3 of the Texas Accessibility Standards (TAS) and considers a public sidewalk a "facility" under the TAS and the U.S. Department of Justice Americans with Disabilities Act (ADA) regulations at 28 C.F.R. Part 35. Sidewalks are subject to the requirements of Chapter 469 of the Texas Government Code as a City-funded public ROW project for Texas Department of Licensing and Registration (TDLR) inspection purposes (Per 16 Texas Administrative Code, Chapter 68) unless exempted by the City Engineer. Compliance with these regulations shall be the responsibility of the owner/developer. Sidewalks shall conform to all current TAS, ADA requirements, and in accordance with this section, and if there is a conflict among those standards, the more stringent shall govern. The maximum running grade (longitudinal slope) of the sidewalk shall not exceed 55 unless approved by the City Engineer. The maximum cross-fall (cross slope) of the sidewalk shall not exceed 2%.

These sidewalks shall conform to the following standards:

1) Zoning Classification Requiring Sidewalks: Concrete sidewalks designed and located according to City standards shall be constructed along all streets in all zoning classifications except agricultural zoning. The Owner shall build sidewalks at the time of site development. Should it be impractical to install the sidewalk at the time, funds for the sidewalk construction shall be placed in escrow with the City for use at the time when sidewalks are needed. Payment or escrow shall be made at the time of site plan or final plat approval.

2) Residential Areas (Single Family, Two Family and Multi-Family): Sidewalks shall be 5-feet in width and located 2-foot from the back of the curb line or the edge of pavement and the adjacent property line. Along thoroughfares with inadequate right-of-way the sidewalk width shall be 5-feet in width and constructed adjacent to the back of curb.

3) Non-residential Areas: Sidewalks shall be 6-feet in width and located at least 2-foot from the back of the curb line or the edge of pavement and the adjacent property line. Along

thoroughfares with inadequate right-of-way the sidewalk width shall be 6-feet in width and constructed adjacent to the back of curb.

4) Exceptions: In areas where mailboxes and other structures interfere with a clear width of 5-feet for the sidewalk, the specified width shall be wrapped around and along one side of the mailbox or other structure.

5) Waiver: The sidewalk required in non-residential areas may be waived by the City Council either temporarily or permanently at the time of site plan or final plat approval. Waiver may be granted based on site conditions and/or location of the tract.

6) Areas Without Screening Walls: In areas on major and secondary roadways where either screening is not required or a type of screening other than a wall is used, (e.g., a berm, foliage, etc.) a 5-foot sidewalk will be constructed not more than 2-feet from the back of the curb line or the edge of pavement and the adjacent property line as required by the Thoroughfare Plan.

7) Areas with Screening Walls: In areas where a screening wall is provided, a concrete sidewalk shall be constructed contiguous with the screening wall. The street side of the sidewalk shall run parallel to the street curb. The sidewalk shall be a minimum of 5-feet wide and the measurement shall be made from the street side of the sidewalk.

8) Sidewalk on Bridges: Bridges on thoroughfares shall have a sidewalk constructed on each side of the bridge. The sidewalk shall be a minimum of 6-feet wide with a parapet wall provided adjacent to the curb of the thoroughfare and with a standard pedestrian bridge rail protecting the sidewalk on the outside edge of the bridge.

9) Sidewalks Under Bridges: When new bridges are built as part of the construction of a roadway or the reconstruction of a roadway and a pedestrian crossing is needed a 10-foot sidewalk will be built as a part of the embankment design underneath the bridge structure. The 10-foot sidewalk shall be located generally along the toe of the embankment.

B. BARRIER-FREE RAMPS (Compliance shall be with the Americans with Disabilities Act)

Curbs and walks constructed at intersections or all streets and thoroughfares must comply with the provisions of the Americans with Disabilities Act and be constructed in a manner to be easily and safely negotiated by physically challenged persons.

SECTION VII

VII – PUBLIC RIGHT-OF-WAY VISIBILITY

A. STREET/DRIVE INTERSECTION VISIBILITY OBSTRUCTION TRIANGLES-FRONTAGE PLAN/PROFILE

A landscape plan showing the plan/profile of the street on both sides of each proposed drive/street to the proposed development with the grades, curb elevations, proposed street/drive locations, and all items (both natural and man-made within the visibility triangles as prescribed below shall be provided with all site plans, if they are not on engineering plans that are submitted at the same time. This profile shall show no horizontal or vertical restrictions (either existing or future) within the areas defined below.

(1) Obstruction/Interference Triangles-Defined
No fence, wall, screen, billboard, sign, structure, foliage, hedge, tree, bush, shrub, berm, or any other item, either man-made or natural shall be erected, planted, or maintained in a position, which will obstruct or interfere with the following minimum standards.

a) Vision at all intersections where streets intersect at or near right angles shall be clear at elevation between 2-feet and 9-feet above the average gutter elevation, except single trunk trees, within a triangular area formed by extending the two curb lines from their point of intersection, 45-feet, and connecting these points with an imaginary line, thereby making a triangle. (See Figure 11) If there are no curbs existing, the triangular area shall be formed by extending the property lines from their point of intersection 30-feet and connecting these points with an imaginary line, thereby making a triangle. (see Figure 12)

b) Definitions for desirable minimum sight distance requirements for non-residential streets, commercial driveways, and industrial driveways that intersect at or near right angles are presented in the following pages. The values presented are minimum sight distances which would permit the following:

• T-Upon turning left or right, an exiting vehicle could accelerate to the operating speed of the street.

FIGURE 11
HORIZONTAL CLEAR TRIANGLE FOR STREETS WITH CURB

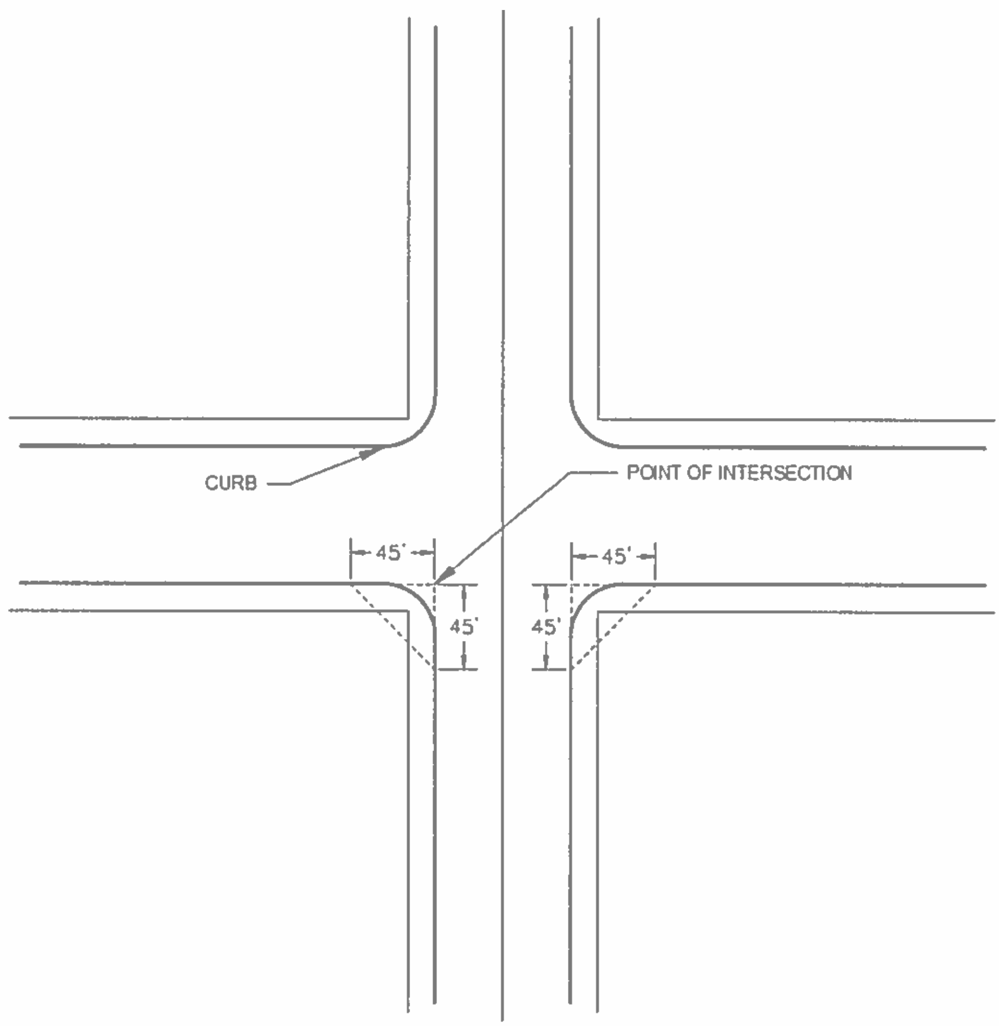
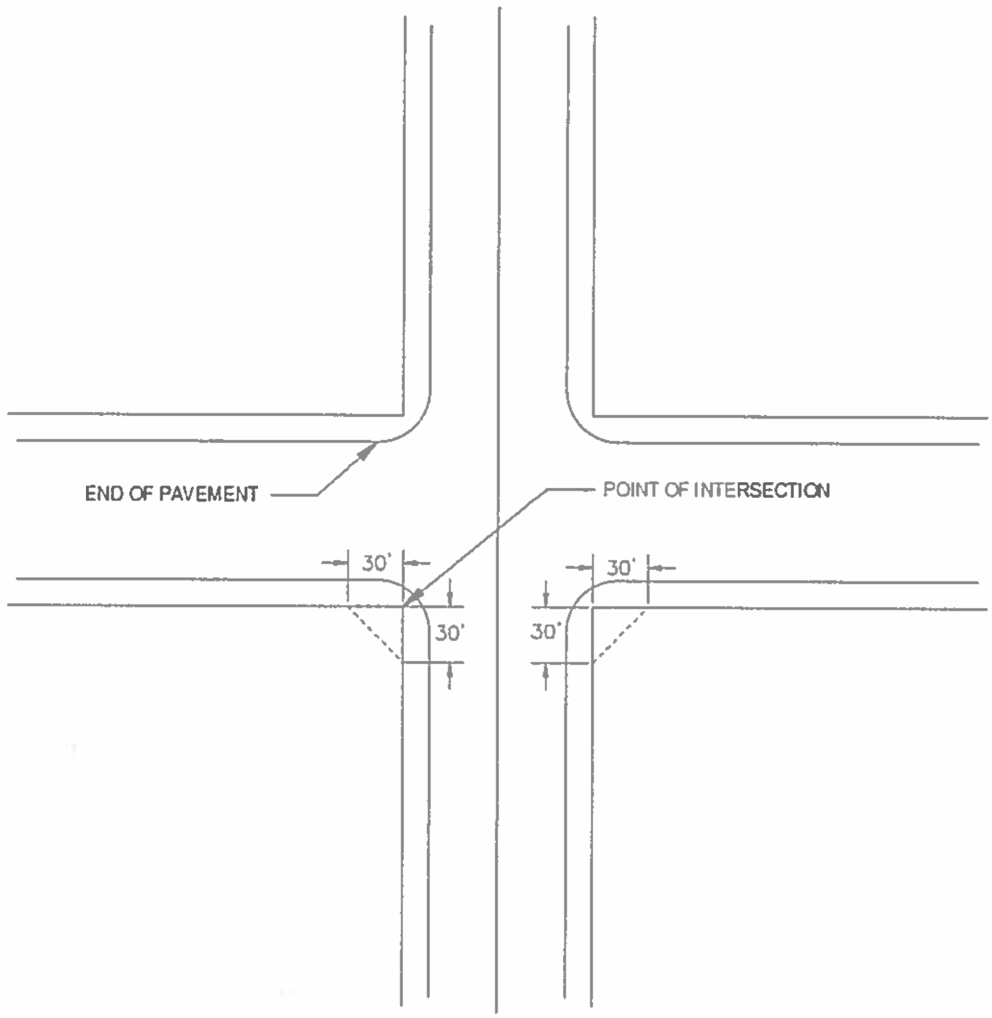


FIGURE 12
HORIZONTAL CLEAR TRIANGLE FOR STREETS WITHOUT CURB



The desirable minimum sight distances are based on the premise that the approaching driver can observe the intersecting vehicle 2.5 seconds before he must apply the brakes and travel the minimum stopping distance for his approach speed. They are, therefore, particularly applicable to arterial streets. Actual sight distances provided at Intersections should be much greater than these minimum values if practical. The minimum sight distance triangle shall also apply to visibility obstructions at intersections.

Conditions for Intersection Sight Triangle-Plan/Profile:

• In the plan view, the horizontal clear area at the Intersection of a proposed street/drive shall be defined as being within two triangular areas formed by:

(I) A line that is on the centerline of the proposed street/drive, beginning at the intersecting street's tangent curb and continuing for a distance of 15-feet back into the proposed street/drive to the end point.

(II) A line that is parallel to and 5-feet out from the intersecting street's curb, beginning at the centerline of the proposed street/drive and continuing for a distance "T_L" as prescribed in Table 9, to the end point.

(III) A line that is parallel to and 5-feet out from the intersecting street's centerline, beginning at the centerline of the proposed street/drive and continuing for a distance "T_R" as prescribed in Table 9, to the end point.

(IV) A straight line that connects the end point of the 15-foot centerline of the proposed street/drive described in (I) above to:

- the end point of the parallel line 5-feet out from the intersecting street's curb described in (II) above.
- the end point of the parallel line 5-feet out from the intersecting street's centerline described in (III) above.

See Figure 13

In the profile view, the clear window shall be defined as being within the horizontal clear area and clear between 2.5 feet and 9 feet above the average pavement elevation.

Note: Single trunk trees within the triangles and in the median shall be allowed and spaced so as to not cause a "picket fence" effect. Because of the large variation of ways in which trees can be planted, the spacing will be decided upon by the City Engineer and the developer at the time of review of the landscape plans. Any other item that obstructs these lines so as to interfere with the above requirements will not be allowed.

Figure 13

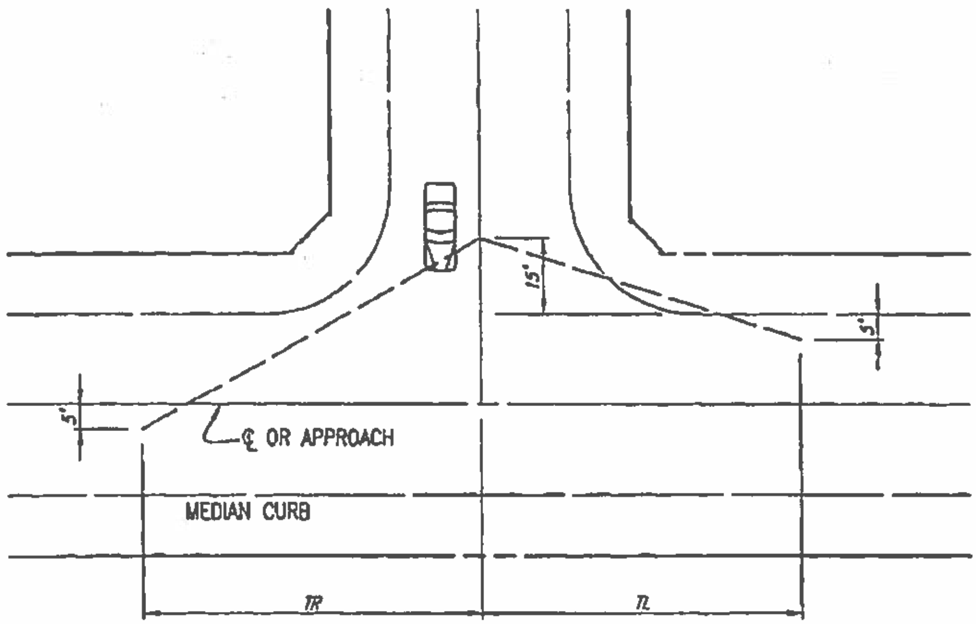


TABLE 9	
MINIMUM SIGHT DISTANCE FOR A CAR AT AN INTERSECTION (For Level-Two Lane Streets)	
MPH	**TL/TR**
30	110+200=300
35	130+250=380
40	130+325=475
45	165+400=565
50	190+475=665

AASHTO P138, Break Reaction Distance
+ Stopping Site Distance (In Feet)

The aforementioned restrictions also apply to streets that do not intersect at right angles, except that the triangle dimensions shall not necessarily be minimum requirements. In such cases the City Engineer shall have the authority to vary such requirements as he deems necessary to provide safety for both vehicular anti pedestrian traffic.

B. R.O.W. OBSTRUCTIONS OUTSIDE THE VISIBILITY TRIANGLES

1) Foliage of hedges, trees and shrubs in public right-of-ways which are not governed by Zoning Ordinance of the City, or the above triangles shall be maintained such that the minimum overhung above a sidewalk shall be 7-feet, the minimum overhang above a street shall be 14-feet.

2) All other areas within the street right-of-ways shall be clear at elevations between 2.5-feet and 9-feet above the average street grade,

3) Plants in the public right-of-way that will grow over 30-inches (when mature) above the adjacent street's curb will conform to all of the above requirements, where applicable. All landscape plans shall show the locations and type of such plants, and show each of the prescribed triangles.

4) Ground elevations, within both triangles, will be shown by contour lines.

Note: No plantings over 30-inches above the adjacent gutter elevation are allowed in the median for the length of the left turn stacking space unless specifically agreed upon by the City Engineer.

C. ALLEY VISIBILITY OBSTRUCTIONS

No fence, wall, screen, billboard, sign, structure, or foliage of hedges, trees, bushes, or shrubs shall be erected, planted or maintained in any alley right-of-way. Foliage or hedges, trees, bushes, and shrubs planted adjacent to the alleys right-of-way which are not governed by the above triangles or by Zoning Ordinance of the City, shall be maintained such that the minimum overhang or encroachment shall be 14-feet above the alley surface at the edge of the pavement.

D. EXCEPTIONS

The provisions of this manual shall not apply to, or otherwise interfere with, the following:

1) Placement and maintenance of traffic control devices under governmental authority and control.
2) Existing and future screening requirements Imposed by the City Council.
3) Existing and future City, State and Federal Regulations.

SECTION VIII

VIII - OFF STREET REQUIREMENTS

A. STACKING SPACE FOR DRIVE-UP WINDOWS

The minimum stacking space for the first vehicle stop for a commercial drive-through shall be 100-feet, and 40-feet thereafter, for any other stops.

B. PARKING - LOT LAYOUT

1) All parking lots shall be paved with concrete unless otherwise approved by City Council.

2) No parking area will be allowed to dead-end unless adequate turnaround space is proved.

3) Each standard off-street parking space shall contain not less than 162 square feet and measure not less than 9 feet by 18 feet, exclusive of access drives and aisles, and shall be of usable shape and condition.

4) The width for two-way aisles shall be a minimum of 24-feet and a maximum of 45-feet.

5) Handicapped parking spaces shall be a minimum 10-feet in width with a 5-foot minimum walkway. The walkway can be shared by two spaces. For parallel parking the space shall be a minimum of 24-feet by a minimum 13-feet with a 3-foot minimum walkway one end in addition to the minimum 24-foot dimension. (see Figure 14)

6) Parking Overhang: No parking stall shall be situated so as to allow vehicle overhand into public right-of-way. Curb or parking stops shall be installed so that the distance between the face of the curb or car stop is a minimum of 2-feet from the public right-of-way.

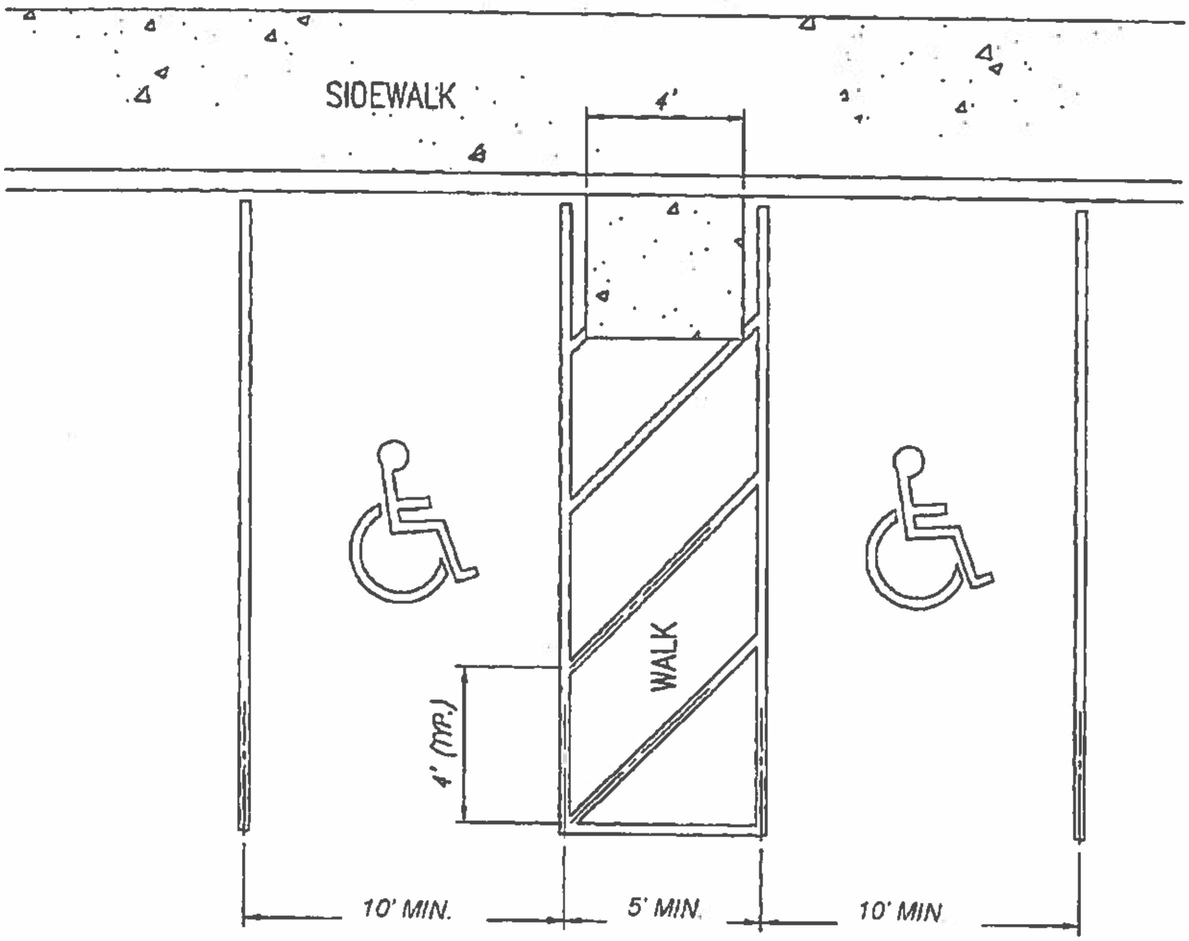
7) Movements in Public Right-of-Way: No parking stall shall be so designed as to allow any movement into or out of the stall, upon public right-of-way.

8) Parking lot illumination shall be designed and constructed to direct the light to the parking lot and away from any adjoining property or street.

9) Fire lanes shall be constructed as required by Fire Department rules and regulations.

FIGURE 14

HEAD-IN OR ANGLE PARKING DIMENSIONS



SECTION IX

IX - RURAL SUBDIVISION REQUIREMENTS

A. Construction and Improvement Specifications for Rural Subdivisions

1. Driveways in a rural subdivision shall be constructed with a minimum of six inches (6") of 3,600 p.s.i. concrete, reinforced with #4 rebar on twenty-four inch (24") centers.

2. Driveway approaches in a rural subdivision shall allow for a twelve foot (12') shoulder extension to the county road (see specifications to improvements to county roads), shall be a minimum of thirty feet (30') long, shall provide a minimum eighteen inch (18") reinforced concrete culvert, with 6 to 1 safety end treatments, and shall provide a minimum of six inches (6") flex base material for approach subgrade (flex base material type A grate 1 TXDOT standard).

3. If the existing county road has less than an asphalt surface, the developer shall improve the road as follows; minimum three inches (3") of asphalt base coarse Type B TXDOT with two inches (2") of surface coarse asphalt Type B TXDOT standard, allowing for future shoulder improvement.

4. A rural subdivision shall have a minimum county road frontage of one-hundred twenty-five feet (125').

5. Prior to development of the interior of a rural subdivision, the existing asphalt county road shall be improved as follows:

   a. A twelve foot (12') shoulder must be added, to the following specification; a six inch (6") stabilized sub base with a minimum 6% by dry weight of lime (27 pounds per square yard for 6" of depth); a minimum of twelve inches (12") flex base material Type A grade 1 TXDOT standard; minimum three inches (3") asphalt base coarse Type B TXDOT standard; and
   b. Overlay both sections, shoulder and existing road, with two inches (2") surface course asphalt Type B TXDOT standard.

6. A rural subdivision shall provide right-of-way as provided in the subdivision ordinance, according to street type.

CITY OF FARMERSVILLE

Farmersville

MANUAL

FOR THE DESIGN OF

STORM DRAINAGE SYSTEMS

Adopted________________

By Ordinance #________________

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TABLE OF CONTENTS

I - INTRODUCTION

1.1 GENERAL

1. 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.

2. All work and materials shall be in accordance with the latest editions of the City of Farmersville Design Manuals, Ordinances, Standard Construction Details, Standard Specifications, and the North Central Texas Council of Governments (NCTCOG) Public Works Construction Standards. Should a conflict be found between the two publications, the City of Farmersville Design Manuals, Ordinances, and Standards shall take precedence.

   In the event that an item is not covered by the City of Farmersville Design Manuals, Ordinances, or Standards; the NCTCOG Public Works Construction Standards shall apply. Notification in writing by the contractor shall be made to the engineer of record, City inspector and the City of the issue. The City of Farmersville shall make the final decision regarding all construction materials, methods, and procedures specified in construction plans. Reference to all documents contained in the project specifications shall refer to the latest edition of each document or the version adopted by the City Council.

3. Inspection of construction activities shall be conducted by staff of the City of Farmersville under direction of the City Engineer or authorized representative. The City inspector shall observe and check the construction in sufficient detail to satisfy

themselves that the work is proceeding in general conformance with the standards and specifications for the project, but they will not be a guarantor of the Contractor's performance. The City will not accept any development until City staff has approved all construction. The developer shall be responsible for any additional expense to the City for inspection that is necessary after normal business hours, or when the improvements will be privately owned. The City will establish the rate for compensation and other expenses.

The developer will be responsible for furnishing the original reproducible engineering drawings corrected to show any revised construction conditions to the City before any improvements will be accepted. All public works improvements must be accepted by the City before any City Building permits will be issued.

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. The time of concentration has been adopted from the Texas Department of Transportation Hydraulic Design Manual, revised March 2004.

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.

2.9 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. (Note: When accumulating times, base the time of concentration on the actual calculated time of concentration at the initial point of interception, even if it is less than the minimum time of concentration. For the design of the storm sewer use the minimum time of concentration as shown in TABLE 1 until such time that the actual time of concentration exceeds the minimum time.

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:

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

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

$$Q_p = q_p * A$$

$$S = I * 2$$

$$R = S_D - L_{is}$$

$$Q_u = R_T * Q_p$$

- "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)	<u>0.36 inch</u>
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 <u>FLOW IN GUTTERS AND INLET DESIGN</u>

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

<u>MAJOR THOROUGHFARE</u>: 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.

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

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

$$A = \frac{W_o\ C_o}{6} \qquad \text{(Equation 4)}$$

- "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:

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

- "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:

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

- "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_0, 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_0, 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:

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

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

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

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

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

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:

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

$$Q = 0.6A\,(2*g*h)^{1/2} \qquad \text{(Equation 13)}$$

- "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 considerably 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:

$$Q = A*V$$

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

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

- "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:

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

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

- "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.

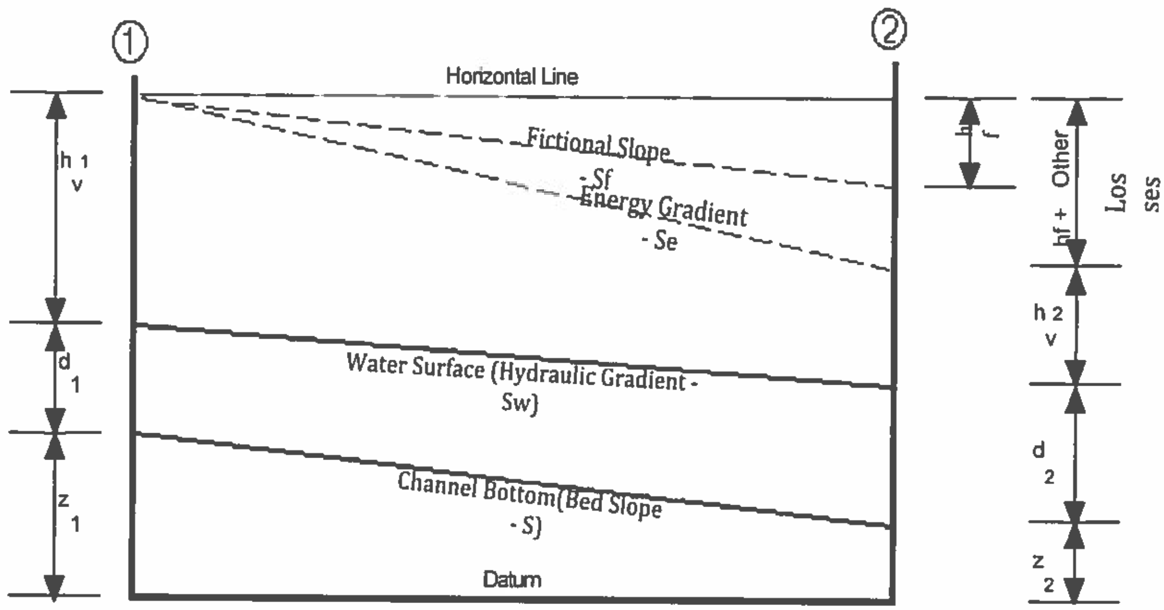
- "d_1" and "d_2" is depth of flow at upstream and downstream sections, respectively.

- "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:

$$Q = A * V$$

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

and Manning's Equation:

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

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:

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

thus giving the total friction head

$$h_f = L * \frac{S_{f1} + S_{f2}}{2}$$

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.,

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

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.

2.30 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.

2.31 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.

2.32 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:

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

and simplifying:

$$H = \left(1 + K_e + \frac{29.2_n^2 \; L}{R^{1.33}}\right) \frac{V^2}{2_g} \quad \text{for full flow}$$

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) dc + D/2 where dc = critical depth. The critical depth, d_c, for box culverts may be obtained from FIGURE 29 or may be calculated form the formula:

$$d_c = 0.315 * \frac{Q}{B}^{2/3}$$

- "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.

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

- "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.

2.35 MANHOLES

Place manholes for cleanout and inspection purposes on all storm sewer lines as shown in the table below.

Pipe Diameter (Inches)	Maximum Distance (feet)
18 - 24	500
24 - 36	500
39 - 54	500
=>60	900

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) MAJOR THOROUGHFARES: 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) ALLEYS: The 100-year Design Frequency flows shall not exceed the capacity of the alley sections shown in FIGURE 6.

e) EXCAVATED CHANNELS: 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) MINIMUM LOT AND FLOOR ELEVATIONS: 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) Easements.

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) EASEMENTS: Drainage and floodway easements shall be provided for all open channels. Drainage and floodway easements for storm sewer pipe shall 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 3.8, 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 3.6. 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 (I).

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 3.8. 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:

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

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 ofdesign.

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 ofthe 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 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 downstream 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)*A*R^(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.

3.37 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 $Q = (1.486/n)*A*R^{(½)}*S^{(½)}$ 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) Significant Rise: 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) Floodway Line: 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) Equal Conveyance Principle: 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) Criteria for Filling in the 100 Year Floodway Fringe

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 floodway, 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.

3.38 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 photogrammetric 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.

5.01 B 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, $1/\theta_0$.
θ_0	Crown slope of pavement in feet per foot.
Y	Conveyance factor calculated for unimproved channels.

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VI - LIST OF TABLES

Table No.	Content

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 unless approved by City or City Engineer.

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.**

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.