DRAINAGE CRITERIA MANUAL

THE TOWN OF

Addison!®

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Public Works Department
6801 Westgrove Road
Addison, TX 75001

Approved by City Council
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1 INTRODUCTION

1.1 Objectives and Purpose

The quality of life for the citizens of Addison depends in part upon adequate drainage facilities. As such, the Town of Addison is dedicated to the protection of local water resources. Land development and redevelopment projects with associated increases in impervious cover can increase storm water runoff rates and volumes, negatively affecting the water quality of receiving water bodies. This Manual establishes criteria, procedures and data for drainage evaluation to ensure the adequacy of new drainage facilities. This Manual also establishes post-construction runoff control objectives for new development and redevelopment projects to protect water quality and mitigate potential negative impacts caused by development.

The use of this Manual will provide a consistent approach to analyzing drainage and designing drainage improvements and structural controls within the Town of Addison. Because many engineering methods for analyzing drainage exist, these criteria for analyzing drainage are established to provide continuity of drainage design throughout the Town.

1.2 Manual Development

This Manual has been developed using the following guiding criteria and assumptions. These will assist the user in utilizing the Manual.

1. The user of this Manual is expected to be a registered professional civil engineer who is skilled in the appropriate disciplines necessary to evaluate drainage problems. Therefore, the terms used and the methods discussed in the Manual should be familiar to the user.
2. This Manual does not contain the solution for drainage problems. It establishes criteria and procedures to be used in designing drainage facilities in the Town of Addison.
3. The criteria set forth in this Manual are the minimum requirements. More stringent criteria may be required if the Director of Public Works determines it is necessary in the interest of the Town. Such determination will be made on a case-by-case basis.
4. The design engineer bears total responsibility for the adequacy of the design. Approval of plans or calculations by the Town does not relieve the engineer of this responsibility.
5. This Manual is not intended to limit creativity in developing solutions for drainage problems. Evaluation of the applicability of innovative solutions and new products is encouraged. The intent of this Manual is to provide continuity while encouraging

use of accepted procedures and the most current data and technology.

6. This Manual recommends procedures, but does not present the theory on which the procedures are based. The user should be familiar with the published sources that set forth this information.

7. Design aids published elsewhere and commonly available may be referenced, but are not reproduced in this Manual.

1.3 Warning and Disclaimer of Liability

The degree of flood protection established in this Manual is considered economically reasonable, and is based on scientific and engineering considerations. However, runoff amounts that exceed what is predicted by the design storm can occur, and flood heights greater than the design height can occur. This Manual does not imply that land outside right-of-ways or easements will be free from flooding or flood damages.

The criteria set forth in this Manual shall not create liability on the part of the Town of Addison, Texas, or any official, employee or consultant thereof, for any flood damages that result from reliance on this Manual or on any administrative decision lawfully made.

2 GENERAL POLICY

2.1 Drainage

The owner or developer of the property to be developed shall be responsible for the design and construction of all storm drainage facilities on and through the subject property. This responsibility includes all existing and proposed on-site drainage, the drainage directed to that property by prior development, and upstream, offsite drainage areas. The general storm drainage design policy for conveyance of upstream, offsite drainage areas for private land development projects and for public or capital improvement projects shall be as follows:

> Private Land Developments: Drainage systems shall be designed to convey existing flows from upstream offsite drainage areas.
>
> Public or Capital Improvement Projects: Drainage systems shall be designed to convey future, fully developed flows from upstream offsite drainage areas.

Provisions shall be made to allow for connection to the on-site storm drainage system from upstream, off-site drainage areas.

The post-construction storm water protection policies provided within this manual establish baseline criteria for the minimization of impacts to water quality. The owner or developer is responsible for mitigating water quality impacts from the development or redevelopment project.

A preliminary planning conference should be initiated by the owner's or developer's engineer with the Director of Public Works prior to the submittal of any development or redevelopment plan to determine their responsibility for the design of drainage improvements and any required permitting.

No increase or concentration of storm water may be conveyed off-site without easements and/or downstream drainage improvements. Increased storm water runoff attributable to new development shall not exceed the capacity of the downstream drainage system in accordance with the Ten Percent Rule, as defined below. If no downstream drainage system exists, increased storm water runoff shall not adversely affect adjoining property.

> Ten Percent Rule:
> Where proposed improvements will result in increased storm water discharge offsite, a downstream assessment will be required. The minimum downstream distance for assessment is defined by the point where the proposed project area constitutes no more than ten percent (10%) of the overall contributing watershed to that point.
>
> Example 1: A 10 acre development proposes to increase discharge offsite by 30 cfs. A downstream assessment reveals that the existing drainage

system has sufficient capacity to the discharge point into a creek that drains 150 acres (10 ÷ 150 = 6%). No detention is required.

Example 2: A 1 acre development proposes to increase discharge offsite by 8 cfs, into an earthen swale. A downstream assessment reveals that the swale has capacity but only drains 7 acres (1 ÷ 7 = 14%). Further downstream the receiving drop inlet drains 10 acres (10% rule) but does not meet current design criteria. Detention will be required.

In cases where the proposed runoff would exceed the capacity of downstream facilities, the developer will be required to either provide detention or downstream improvements. Multi-phase developments will be considered as a single entity in determining the requirement for detention.

In all new developments where storm water runoff has been collected or concentrated, discharge shall be conveyed off-site by creeks, channels or storm sewer systems. Easements shall be provided by the developer to the Town for all on-site and off-site drainage facilities. All flows shall be discharged in a non-erosive manner, and shall meet the established regulations governing storm water quality as described in Section 4.1, Land Development Post-Construction Runoff Controls.

The developer shall pay for the cost of all drainage improvements required, including any necessary off-site channels or storm sewers and acquisition of the required easements. The Town of Addison's Stormwater Master Drainage Study evaluates the degree of flooding that may occur during a major storm event for the Town's eight (8) major drainage basins. The Master Drainage Study should be reviewed by developers and engineers to determine flood prone areas and recommended drainage improvements. Land developments will be required to incorporate the recommendations of the Town's Stormwater Master Drainage Study. The map on Figure 1 in Appendix C delineates the limits of the Town of Addison's major drainage basins as follows:

Town of Addison Major Drainage Basins:

- Hall Branch Basin
- Hutton Branch Basin
- Keller Springs Branch Basin
- Rawhide Creek Basin
- Addison Circle Basin
- Farmers Branch Creek Basin
- South Addison Basin
- White Rock Creek Basin

2.2 Platting/Dedication of Easements for Drainage Facilities

Property developments containing drainage structures, detention facilities, floodways, open channels, creeks and streams shall be platted with Drainage Easements. The plat shall identify the maintenance responsibilities for the drainage facilities (private or public) within the development. All development plats with detention easements shall include the detention area easement statement provided in the Appendix A.

Placement of any fill or property development is prohibited in the 100-year floodplain (whether so designated by FEMA or as determined locally) except as allowed in accordance with FEMA regulations and the Town of Addison ordinances.

Easements for drainage facilities shall be designated as follows:

A. <u>Drainage Easements</u>
Drainage Easements shall be used for floodplains, natural drainage ways, improved open channels, man-made storm drain systems and drainage structures, including certain post-construction storm water runoff controls. Individual property owners are responsible for all necessary maintenance of the easement within their property

B. <u>Detention Area Easements</u>
Detention basins shall be maintained in Detention Area Easements. Detention area easements shall be maintained by the landowner(s) or neighborhood association. The detention area easement statement provided in Appendix A shall be placed on all plats.

2.3 Drainage Facility Easements

The owner or developer shall provide all necessary drainage facility easements dedicated to the Town as defined above and as required for drainage structures, including storm drains, channels and streams. Easements shall be required in all upstream and downstream off-site locations where construction of drainage improvements is proposed or required. All easements shall be identified on the Final Plat.

Minimum drainage easement widths for storm drain pipe shall be as follows:

Storm Drain Diameter	Min. Easement Width
≤ 48"	20'
> 48"	Outside Diameter + 15' (7.5' on either side, rounded up)

Storm drains deeper than ten (10) feet as measured from flowline to ground surface shall add an additional two (2) feet of easement width for each foot of depth greater than ten (10) feet. Box pipe shall have a minimum easement width equal to the width of the box plus fifteen (15) feet (7.5' on either side).

Drainage easement widths for natural drainage ways and improved open channels shall be a minimum of fifteen (15) feet wider than the top of the channel with a minimum of ten (10) feet being on one side to serve as access along the channel for maintenance purposes. Where steep natural channel banks are to remain, the easement shall be based on a predicted top of channel depicted by a 3:1 projected slope from the flowline to the natural ground, plus ten (10) feet to one side. Maintenance access ramps shall be provided on improved channels at logical locations, and contained within the easement.

Drainage easements shall extend a minimum of twenty-five (25) feet downstream of the outfall headwall or to the end of the energy dissipation structure if applicable (i.e. rock riprap, gabions, etc.), whichever is greater. Drainage easements shall be provided where grading is required to establish positive slope from storm drainage system discharges to natural grade.

Drainage easements shall be dedicated to encompass the limits of the 100-year, fully developed floodplain.

Any required offsite drainage easements shall be obtained and a filed copy provided to the Town of Addison prior to the approval of the development plans. The offsite drainage easements shall be included in the record drawings submittal and identified on the Final Plat, prior to acceptance of the subdivision.

Drainage easements are required to allow access for maintenance and repairs, and to prevent property owners from making modifications that would compromise the function of the system. However, individual property owners are responsible for all necessary maintenance of the easement within their property.

2.4 Development in Floodplains

It is the policy of the Town of Addison to regulate development and fill in the 100-year floodplain areas as designated by FEMA Flood Insurance Rate Maps and/or the Storm Drainage Master Study, whichever is more restrictive. Development or fill within a floodplain will require engineering analysis that shows the development or fill causes no rise in the 100-year water surface elevation upstream or downstream, except as allowed by current FEMA and Town of Addison regulations. A Floodplain Development Permit shall be obtained prior to beginning any construction activities within a designated floodplain. Refer to the Town of Addison's floodplain ordinance (Code of Ordinances, Chapter 42 – Floods) for further information.

Valley storage consists of the volume of water that can be contained by a creek during periods of flooding. The amount of valley storage affects the peak discharge in the creek as the flood wave moves downstream. Loss of valley storage due to an encroachment shall be offset by excavating an equal volume from the floodplain area, resulting in a net balance of valley storage.

3 DRAINAGE DESIGN STANDARDS

3.1 Design Storm Frequency

All drainage facilities shall be designed based on runoff from a 100-year storm event, assuming the entire contributing drainage area is fully developed. The Town of Addison Zoning Map shall be used to identify the anticipated makeup of full development.

All open and closed drainage systems shall be designed to provide positive overflow and protection of all public and private property during a storm event having a 100-year recurrence interval, regardless of the design storm frequency of a particular drainage facility.

The design of storm drainage facilities shall include sufficient calculations and analysis, as deemed necessary by the Director of Public Works, to prove the facility can convey the runoff from the following design storm frequencies.

DRAINAGE FACILITY	DESIGN STORM FREQUENCY
Closed Storm Sewer System	10-year within storm system, 100-year within right-of-way
Street	10-year below top of curb, 100-year within right-of-way
Roadside Ditch	10-year below pavement edge, 100-year within right-of-way
Closed System at Low Point or Sag	100-year within storm system with positive overflow
Culvert	100-year within culvert with 1 foot freeboard to pavement edge or gutter
Bridge	100-year with 2 foot freeboard to bridge low-chord
Channel	100-year with 1 foot freeboard to top of bank

A storm sewer system shall be designed to pick up flow from a street when the runoff from a ten (10) year frequency storm exceeds the capacity of the street to the top of curb; or the spread of water on a collector street does not leave one (1) traffic lane dry; or the spread of water on an arterial street does not leave at least two (2) traffic lanes dry, whichever is more restrictive. Residential streets and parking lots shall not direct flow into a collector or larger street in excess of the capacity stated above.

3.2 Runoff Calculations

Design flow of storm water runoff is to be calculated using the Rational Method for storm drainage systems serving a watershed area less than two hundred (200) acres. This method will primarily be used for the design of storm drainage systems and small channels. A unit hydrograph method shall be used to determine runoff from watershed areas larger than two hundred (200) acres and for more complex applications where the Director of Public Works deems the Rational Method is not appropriate. Refer to Figure 12 in Appendix C for the standard format for presentation of drainage area, inlet and storm drain computations.

3.2.1 Rational Method

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

$$Q = CIA$$

where:
- Q = the storm flow at a given point in cubic feet per second (cfs)
- C = a coefficient of runoff representing the ratio of rainfall to peak runoff.
- I = rainfall intensity in inches per hour for a period equal to the time of flow from the uppermost point of the drainage area to the point under consideration. Refer to Figure 2 in Appendix C for the Town of Addison rainfall intensity curves.
- A = the area contributing to the point of design, in acres.

Runoff coefficients for the Town of Addison are shown in Table 1. These coefficients shall be the minimum used and shall be based on full development. Smaller coefficients may be used if approved by the Director of Public Works in writing. For small drainage areas, roadways, land uses other than those listed in Table 1, and miscellaneous land uses like parks, schools, planned developments, etc., where the designated runoff coefficient is not representative, a composite runoff coefficient shall be calculated using the appropriate factor for undeveloped land given in the table and 1.00 for impervious areas.

The size and shape of the watershed and sub-areas shall be determined for each design point through the use of planimetric-topographic maps, and supplemented by field surveys in areas where topographic data has changed or where the contour interval is insufficient to adequately determine the direction of flow. Offsite drainage areas shall be delineated using a minimum of 2-foot topographic contours. NCTCOG topography is recommended for any offsite drainage area delineations. Drainage area maps shall identify the source of offsite topography. Drainage areas within a development area will be delineated based on field-surveyed topography. The outline of the drainage area contributing to the system being designed and an outline of the sub-drainage area contributing to each inlet point shall be determined and shown on the drainage area map.

TABLE 1		
RUNOFF COEFFICIENTS & MINIMUM INLET TIMES		
Zoning District Name*	Runoff Coefficient "C"	Minimum Inlet Time in Minutes**
Undeveloped Land/Open Space	--	--
▪ 0% - 3% Slope	0.25	--
▪ 3% - 5% Slope	0.30	--
▪ Greater than 5% Slope	0.35	--
Single Family Residential	0.60	15
Multi-Family Residential	0.90	10
Commercial	0.90	10
Industrial	0.85	10
Roadway Right-of-Way	0.90	10
Water Body	1.00	--

* For any land use not listed, the runoff coefficient shall be calculated based on a weighted percentage of pervious (C=given above) and impervious (C=1.00) surface.

** Inlet time may vary from Time of Concentration. See Section 3.2.1.1 for more information. Inlet time should be confirmed prior to using the minimum value.

Drainage areas shall conform to the natural topography of the watershed contributing to the proposed storm drainage facilities. Where discharge toward off-site adjacent property is relocated, such relocation shall be acknowledged by the downstream property owner or any other property owner who may be affected. No diversion of drainage from one watershed to another shall be permitted without the written approval of the Director of Public Works.

When calculating the quantity of storm water runoff, rainfall intensity will be determined from the rainfall intensity data provided on Figure 2 in Appendix C.

3.2.1.1 Time of Concentration

The time of concentration is defined as the longest time, without unreasonable delay, that will be required for a drop of water to flow from the upper limit of a drainage area to the design point under consideration. The time of concentration to any point in a storm drainage system is a combination of the inlet time and the time of flow in the storm drain system. The inlet time is the time for water to flow overland to the first storm drain inlet in the proposed drainage system.

The time attributable to overland flow may be calculated using the graphical method shown on Figure 3 in Appendix C. The maximum length allowed for overland flow shall not be more than 100 feet.

Shallow concentrated flow can occur on unpaved areas such as yards, parks, and open space, or on paved areas such as parking lots and street gutters. These three cases are represented on Figure 4 in Appendix C which can be used to determine the average flow velocity. This velocity can then be multiplied by the length of the flow path to determine the flow time. The maximum allowable length of shallow concentrated flow is one thousand (1,000) feet. Beyond this distance, the velocity of flow should be calculated for the specific pipe or channel that conveys the flow.

For purposes of calculating inlet capacity, inlet times may be calculated as described above, but in no case shall the inlet time used be less than the time shown in Table 1. Except in unusual cases, the inlet times shown in Table 1 should be used, as they will be shorter than the inlet time calculated as described above.

3.2.2 Unit Hydrograph Methods

Runoff from drainage areas larger than two hundred (200) acres will be determined using a Unit Hydrograph method. Various analysis methods can be implemented using computer software including the U.S. Army Corp's of Engineers' HEC-HMS computer program, XPSWMM, or InfoWorks. Other software programs may be used with prior approval by the Director of Public Works.

A 24-hour rainfall duration should be appropriate for most applications. An alternate duration may be utilized with written approval of the Director of Public Works. The duration shall be large enough to capture all excess rainfall as well as provide reasonable runoff volumes when performing storage analyses. Computation intervals should be tested for sensitivity to the hydrograph peak, and shall not be greater than 15 minutes.

The effects of urbanization should be reflected in the precipitation loss rates. The Soil Conservation Service (SCS) curve number method may be used in this way. Suitable curve numbers for various urban land uses have been published by the SCS in TR-55, Urban Hydrology for Small Watersheds.

Routing, when appropriate, shall use the Modified Puls methodology. Reach lengths shorter than the computation interval should not incorporate routing.

3.3 Street and Alley Capacities

Street and alley capacities shall be calculated as open channels using the continuity equation and Manning's equation.

$$\mathbf{Q = VA} \quad \text{where}$$

$$\mathbf{V = \frac{1.486\, R^{2/3}\, S^{1/2}}{n}} \quad \textbf{or}$$

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

where:
- Q = flow (cfs)
- A = cross sectional area of conduit or channel (sq ft)
- V = velocity of flow in conduit (fps)
- n = Mannings roughness coefficient of the conduit or channel (see Table 2)
- R = hydraulic radius (area of flow divided by wetted perimeter) (ft)
- S = friction slope (ft/ft)
- P = wetted perimeter (ft)

Figure 5 provides a graphical solution for the capacity of triangular gutters, while Figure 6 may be used to determine the capacity of a gutter in a street with a parabolic crown. Figure 5 and 6 are located in Appendix C. Streets and alleys shall contain runoff from the 100-year storm within the right-of-way or dedicated easement, or where no curbs exist, within the roadside ditches. Grate inlets or combination curb and grate inlets shall be placed at those locations where the alley right-of-way capacity is exceeded by runoff from the 100-year storm.

100-year discharge from alleys and driveways into streets shall not exceed four (4) cfs or the available street capacity at the next downstream inlet, whichever is less.

Special attention is required at turns and intersections of alleys to determine whether superelevation or curbs are necessary to contain the required flow. Superelevation calculations will be required at all turns and bends in alleys where the velocity in the alley exceeds 3 feet per second. The minimum radius for the invert at alley turns (at bends, intersections or tees) will be thirty-five feet (35'). Figure 7 in Appendix C may be used for determination of alley capacity and velocity. Superelevation shall be calculated using the following formula:

$$\Delta y = V^2W / gr$$

where:
- Δy = rise in water surface from the centerline to outside edge of curve (ft)
- V = mean velocity (fps)
- W = alley width (ft)
- g = acceleration due to gravity (32.2 ft/sec^2)
- r = radius of centerline curve (ft)

Alley turns and intersections shall be graded to provide no less than 0.2' of freeboard above superelevation as calculated above. Note that curbs may be necessary at the outside of some turns in order to direct water around the turn. In such cases, no driveways will be permitted where such curbs are required.

3.4 Valley Gutters

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

A. A major street (thoroughfare) shall not be crossed with a valley gutter.

B. A collector street shall not be crossed with a valley gutter unless approved in writing by the Director of Public Works.

3.5 Flow in Gutters and Inlet Locations

Storm drain conduit and inlets shall begin at the point where the depth of flow generated by the 10-year storm frequency reaches the height of the top of curb. Inlets are then to be located as necessary to maintain depth of flow below the top of curb. If, in the judgment of the Director of Public Works, the flow in the gutter would be excessive under these conditions, the storm sewer shall be extended to a point where the gutter flow can be intercepted farther upstream. Multiple inlets at a single location are permitted to a maximum of twenty (20) feet. Inlets should be placed upstream of intersections to prevent large amounts of water from running through the intersections.

Inlets should also be located on the lower traffic volume street at a street intersection and in alleys at the street intersection as necessary to prevent water from entering higher volume streets. In the case of parking lots, inlets shall intercept water before it enters the street gutter, subject to the limitations of Section 3.3. Inlets shall be placed at intersections such that the beginning of the inlet transition does not fall within the curb return radius or the lay down curb for barrier free ramps.

Recessed curb inlets shall be used on all thoroughfares. Recessed brick top curb inlets (per the Town's standard detail) should be used within the Urban Center District. Standard (non-recessed) curb inlets shall be used on collectors, residential streets and alleys.

Where water is conveyed from a street directly into an open channel, it shall be conveyed through an approved type of curb inlet or flume, and not through a curb cut.

Curb inlets and drop inlets can be used to collect runoff. Selection of the type of inlet depends on the location and conditions, and is subject to approval by the Director of Public Works.

Combination inlets (curb and grate) may be used in alleys, but may not be used in streets. The flowline of the alley shall transition to direct flow into the inlet. Combination inlets shall be provided at all alley low points.

Positive overflow shall be provided at all low points. Positive overflow refers to a means for safely conveying excess flow overland when underground storm drainage systems do not function properly or when their capacity is exceeded. Such overflow shall be in a public right-of-way or dedicated easement. Calculations will be required to demonstrate the capacity of the overflow route. Minimum finish floor elevations adjacent to such overflows shall be no less than 12" above the overland flow water surface elevation.

3.6 Inlet Sizing

Under normal conditions, the minimum curb inlet size shall be four (4) feet. Standard inlet sizes will be in increments of two (2) feet to a maximum of twenty (20) feet. The minimum criteria for the sizing and placement of on-grade inlets and sump inlets is provided below. In all cases, inlet calculations shall be provided by the design engineer to verify the inlet sizing is adequate to meet the design intent.

The capacity of a depressed curb inlet on-grade will be based on the following equation:

$$\mathbf{Q_1 = 0.7\ (H_1^{5/2} - a^{5/2})\ /\ y_0}$$

where: Q_1 = discharge into inlet per foot of inlet opening (cfs/ft)
H_1 = total depth of flow at inlet throat ($a + y_0$)
a = gutter depression (ft)
y_0 = depth of flow in approach gutter (ft)

Figure 8 in Appendix C may also be used to determine on-grade inlet capacity.

The capacity of low point or drop inlets will be determined based on the broad-crested weir formula where the depth of flow does not exceed the height of the inlet opening. The weir equation is as follows:

$$\mathbf{Q_1 = 3.0\ (H_1)^{3/2}}$$

Figure 9 in Appendix C may also be used to determine capacity of sump inlets based on the weir and orifice formulas. The orifice equation should be used to determine the capacity of an inlet where the depth exceeds the inlet opening height. The orifice equation used for curb inlet design is as follows:

$$\mathbf{Q_1 = 4.82\ (H_1)^{1/2} A_L}$$

where: A_L = area for 1 ft length of opening (sf)

Because of the tendency for clogging, combination inlets at alley low points shall be designed as curb inlets, neglecting any grate inlet capacity (100% clogging). Combination inlets on-grade shall be calculated using a clogging factor of 25%. Capacity of grate inlets on grade is a function of the number, size and orientation of the grate openings, in addition to the depth of flow in the gutter. Calculation of the capacity of combination inlets shall be based on these factors, and may be as provided by grate inlet manufacturers. Suitable grate inlet design calculations and efficiency data for various sizes and shapes of grates have been published by the FHWA in HEC-12, Drainage of Highway Pavements.

3.7 Hydraulic Design of Closed Conduits

After completing the computations of the quantity of storm runoff entering each inlet, the size and gradient of pipe required to carry the design storm are to be determined. All hydraulic gradient calculations are required to begin at the outfall of the system. Computer software can be used to aid in the design process, however, hydraulic gradient calculations shall be provided for review in the standard table format provided on Figure 12 located under Appendix C. The following shall apply for establishing the starting elevation of the hydraulic gradient:

A. The 100-year water surface elevation of the receiving creek, stream, or other open channel shall be used as the starting elevation (tailwater) for the hydraulic gradient in most cases.

B. When a storm sewer system is connected to larger creek or stream and the drainage area of the stream is substantially larger than the system area, the coincident storm frequencies shall be evaluated. The design engineer shall refer to the Frequencies of Coincidental Occurrence tables per the Texas Department of Transportation Hydraulic Design Manual for analysis of the coincident design storms. In such cases, the largest structure required to satisfy both frequency combinations shall be provided.

C. When a proposed storm sewer is connected to an existing storm sewer system, the hydraulic gradient for the proposed storm sewer should start at the elevation of the existing storm sewer's hydraulic gradient.

D. When a proposed storm sewer system is in full flow, the starting HGL shall begin at the inside top of pipe.

All closed conduits shall be hydraulically designed for full flow through the use of the continuity equation and Manning's equation (below or from Figure 10 in Appendix C):

$$\mathbf{Q = VA} \quad \text{where}$$

$$\mathbf{V = \frac{1.486\, R^{2/3}\, S^{1/2}}{n}} \quad \text{and}$$

$$\mathbf{Q = \frac{1.486\, AR^{2/3}\, S^{1/2}}{N}}$$

where: Q = flow (cfs)
A = cross sectional area of conduit or channel (sq ft)
V = velocity of flow in conduit (fps)
n = Mannings roughness coefficient of the conduit or channel (see Table 2)
R = hydraulic radius (area of flow divided by wetted perimeter) (ft)
S = friction slope (ft/ft)
P = wetted perimeter (ft)

TABLE 2 MANNING'S ROUGHNESS COEFFICIENTS "n" FOR STORM DRAINS	
Materials of Construction	Manning's Coefficient "n"
Monolithic Concrete Structure	0.015
Concrete Pipe	
Good alignment, smooth joints	0.013
Fair alignment, ordinary joints	0.015
Poor alignment, poor joints	0.017

The hydraulic grade line (HGL) shall be designed to be at least one (1) foot below the top of curb elevation. Once the HGL is set, the depth and slope of the pipe may be determined. The pipe shall be located so that the inside top of the pipe is at or below the HGL and at or above the minimum slope shown in Table 3. In some situations, generally at the upstream end of a pipe system, the inside top of the pipe may be above the HGL, which results in partial flow. In such cases, the pipe capacity and velocity shall be calculated at normal depth, neglecting minor losses. The HGL shall be shown in the profile on the plans for all storm drain lines, including inlet leads, except where in partial flow. In all cases the HGL elevation, including entrance headloss, shall be below the designed inlet throat elevation.

Any proposed drainage system conduit in public right-of-way or easement shall be reinforced concrete pipe (RCP) or reinforced concrete box sections, including detention pond piping. High density polyethylene pipe (HDPE) is not permitted in public right-of-way or easements. PVC pipe is only permitted for collection of roof drains, however PVC may be allowed in other unique situations if approved by the Director of Public Works. The minimum slope of a pipe or box section should be capable of producing a velocity of at least 2.5 feet per second when flowing full. Table 3 gives the minimum slopes for storm drain pipes based on this criterion.

From the time storm water first enters the storm drainage system at the upstream inlet until it discharges at the outlet, it will encounter a variety of structures such as inlets, manholes, junctions, bends, and enlargements that will cause minor head losses. In general, these minor losses can be expressed as a function of velocity head. Figure 11 in Appendix C shows the various cases and the method of computing the associated minor head loss. The minimum storm sewer pipe diameter shall be 18-inches. Pipe sizes shall not decrease in the downstream direction unless otherwise approved in writing by the Director of Public Works.

The junction of in-line pipes of different diameters shall be made such that the crowns (inside tops or soffits) are at the same elevation. When lateral pipes connect to trunk mains, they shall be connected with the center of the lateral matching the center of the trunk main.

TABLE 3 MINIMUM GRADES FOR STORM DRAINS	
Pipe Size (inches)	Concrete Pipe Slope (ft/ft) (n = 0.013)
18	0.00177
21	0.00144
24	0.00121
27	0.00103
30	0.00090
33	0.00079
36	0.00070
39	0.00063
42	0.00057
45	0.00052
48	0.00048
54	0.00041
60	0.00036
66	0.00031
72	0.00028
78	0.00025

Where a storm sewer system discharges into a pond or lake, the outside top of the pipe shall be set below the normal pool of the lake. The impact on tailwater from fluctuations in the pond or lake level should be considered in designing the pipe system. Where storm sewers discharge into channels or streams, adequate measures shall be taken to control erosion using concrete headwalls, rock riprap, gabions, and/or other means as necessary.

Storm drain junction boxes are needed for access to underground storm sewers for inspection and cleanout. Junction boxes should be located at junctions of large storm drain main lines, and at abrupt changes in alignment or grade. For pipes larger than 24-inches, junction boxes or other access points shall be located at intervals not to exceed eight hundred (800) feet. They shall be located at intervals not to exceed four hundred (400) feet for pipes 24-inches in diameter or smaller. Junction boxes shall be constructed in accordance with the Town's standard details.

The invert of the junction box should be rounded to match the inverts of the pipes entering the junction box in order to reduce eddying and resultant head losses. The junction box floor shall be provided with a minimum of 0.1' fall from upstream to downstream pipe flowlines. The invert should be rounded to a minimum depth equal to the design flow depth. At junctions with other storm drain main lines, the maximum interior angle of intersection of pipes in the junction box shall be ninety degrees (90°).

Storm sewers will typically be located in the center of the roadway. Storm sewers should be straight between junction boxes where possible. Where curves are necessary to conform to street layout, the radius of curvature should not be less than sixty-five (65) feet. The minimum radius of curvature should not be less than the pipe manufacturer's recommendation. Head losses shall be calculated for bends and curves as shown on Figure 11 in Appendix C.

A headwall shall be constructed at the outfall of any storm drainage system. If the outlet velocity exceeds the maximum permissible velocity for the channel (see Table 4), an apron of erosion protection or energy dissipation is required. Erosion protection shall consist of concrete, gabion mattress or grouted rock riprap between the storm sewer headwall and the channel. All gabion and grouted rock shall be constructed with a layer of filter fabric under the rock. The calculation for the length and width of the erosion protection outfall apron shall be per the "Gregory Method" as outlined in the Section 3.12. In unique situations, other methods for calculating erosion protection may be presented to the Director of Public Works for approval.

3.8 Open Channels

Concrete drainage channels will not be allowed within the Town of Addison unless written approval is received from the Director of Public Works. Open channels shall be designed to carry the 100-year frequency storm event with one (1) foot of freeboard to the top of bank. A 10-foot wide access ramp into the channel shall be included for maintenance purposes. If the outlet velocity of an improved channel discharging into an existing channel exceeds the maximum permissible velocity for the existing channel, erosion protection is required. This protection shall consist of a section of rock riprap or other energy dissipation measures at the intersection of the two channels. The dimensions and material of the erosion protection shall be specified as required to protect the channel from erosion. Erosion protection measures that promote sustainability and water quality are allowed and encouraged.

Freeboard is the height of the improved channel above the calculated water surface. Minimum freeboard shall be one (1) foot, and additional freeboard shall be provided to accommodate superelevation or other factors causing a rise in the water surface. Superelevation of the water surface shall be determined at horizontal curves in order to properly specify freeboard. The minimum radius of curvature shall be not less than three times the top width for improved channels in a sub-critical flow regime, unless detailed calculations indicate a smaller radius can be used without causing excessive velocities, shear stresses, or waves on the outside of the channel.

TABLE 4				
ROUGHNESS COEFFICIENTS AND MAXIMUM VELOCITIES FOR OPEN CHANNELS				
	Roughness Coefficient			
Channel Description	Minimum	Normal	Maximum	Maximum Velocity
MINOR NATURAL STREAMS				
Moderately Well-Defined Channel				
Grass and Weeds, Little Brush	0.025	0.030	0.033	8
Dense Weeds, Little Brush	0.030	0.035	0.040	8
Weeds, Light Brush on Banks	0.030	0.035	0.040	8
Weeds, Heavy Brush on Banks	0.035	0.050	0.060	8
Weeds, Dense Willows on Banks	0.040	0.060	0.060	8
Irregular Channel with Pools and Meanders				
Grass and Weeds, Little Brush	0.030	0.036	0.042	8
Dense Weeds, Little Brush	0.036	0.042	0.048	8
Weeds, Light Brush on Banks	0.036	0.042	0.042	8
Weeds, Heavy Brush on Banks	0.042	0.060	0.072	8
Weeds, Dense Willows on Banks	0.048	0.072	0.085	8
Flood Plain, Pasture				
Short Grass, No Brush	0.025	0.030	0.035	8
Tall Grass, No Brush	0.030	0.035	0.050	8
Flood Plain, Cultivated				
No Crops	0.025	0.030	0.035	8
Mature Crops	0.030	0.040	0.050	8
Flood Plain, Uncleared				
Heavy Weeds, Light brush	0.035	0.050	0.070	8
Medium to Dense Brush	0.070	0.100	0.160	8
Trees with Flood Stage below Branches	0.080	0.100	0.120	8
MAJOR NATURAL STREAMS				
The roughness coefficient is less than that for minor streams of similar description because banks offer less effective resistance.				
Moderately Well-Defined Channel	0.025	---	0.060	8
Irregular Channel	0.035	---	0.100	8
UNLINED VEGETATED CHANNELS				
Mowed Grass, Clay Soil	0.030	0.035	0.040	8
Mowed Grass, Sandy Soil	0.030	0.035	0.040	6
UNLINED NON-VEGETATED CHANNELS				
Clean Gravel Section	0.022	0.025	0.030	8
Shale	0.025	0.030	0.035	10
Smooth Rock	0.025	0.030	0.035	15
LINED CHANNELS				
Smooth Finished Concrete	0.015	0.0175	0.020	15
Rip-rap (Rubble), D_{50}=12"	0.030	0.040	0.050	10
Rip-rap (Grouted), D_{50}=12"	0.030	0.035	0.040	10
PAVEMENTS				
Concrete Pavement	0.015	0.0175	0.020	15
Asphalt Pavement	0.016	0.018	0.022	15

For waterways included in the Flood Insurance Study (FIS), flood elevations shall be determined by the same methodology as used in the FIS, unless other methods are approved by the Director of Public Works.

The composite roughness coefficient should account for the sediment, debris, and vegetation that can reasonably be expected in the channel environment. The roughness coefficients given in this Manual are minimum and should be increased at the discretion of the design engineer to account for expected conditions. If the possibility exists that high bed loads or debris can accumulate in the channel, the 'n' factor should be adjusted or other measures taken to ensure that flow shifts from super-critical to sub-critical will not cause flooding.

All channels shall be designed to operate in sub-critical conditions except at hydraulic jumps, such as at culverts, bridges and drop structures, and shall be designed for stable flow (Froude number less than 0.90). In instances of super-critical flow, or locations where flow passes through critical depth, channels shall be lined to prevent scour and erosion. Channels shall be designed to convey the 100-year storm, assuming fully developed watershed conditions, with one (1) foot of freeboard to the top of channel bank. Maximum allowable channel velocities shall be as shown in Table 4; however, good engineering judgment may indicate that lower velocities are necessary in specific situations. The values provided in Table 4 shall be used as a guide; actual conditions may require deviation from these values and shall be approved by the Director of Public Works. Drop structures, if needed to provide grade control and maintain sub-critical flow, shall be constructed of reinforced concrete lining or gabion structures.

3.9 Roadside Ditches

Roadside ditches, where permitted, shall be designed to convey runoff from a 100-year storm below the edge of pavement and contained within the right-of-way. All roadside ditches shall be protected with sod, back sloping, and/or other bank protection designed and constructed to control erosion. Any earthen slopes shall have proper vegetative cover and shall be no steeper than three horizontal to one vertical (3:1).

3.10 Culverts

A culvert is used to convey storm water runoff through roadway embankments. Culverts shall be designed to convey the 100-year frequency storm. A minimum of one (1) foot of freeboard is required between the 100-year headwater and either the top of curb or edge of pavement, whichever is applicable. The allowable headwater is the depth of water that can be ponded at the upstream end of the culvert. The headwater shall be based on the design storm, and shall not increase the flood hazard of adjacent property.

The culvert length shall be chosen to provide minimum 3:1 embankment slopes. The culvert flowline shall be aligned with the channel bottom and the skew angle of the stream.

The culvert skew shall not exceed forty-five degrees (45°) as measured from a line perpendicular to the roadway centerline without the approval of the Director of Public Works.

Depending on the type of hydraulic operation, a culvert may function either under inlet control or outlet control. Inlet control exists when the barrel capacity exceeds the culvert inlet capacity and the tailwater is not high enough to control culvert operation. Headwater depth and entrance conditions control the culvert capacity. Outlet control exists when the culvert inlet capacity exceeds the barrel capacity, or the tailwater elevation is high enough to create a backwater condition through the culvert. The tailwater elevation and the slope, length and roughness of the culvert determine the culvert capacity. Both types of operation shall be considered, and the culvert capacity will be based on the type of operation that yields the higher headwater of the two. Inlet control shall not be assumed without the prior approval of the Director of Public Works.

Nomographs and hydraulic calculation tables for the solution of various inlet and outlet control scenarios for pipe culverts are included as Figures 13 through 15 located in Appendix C, although a backwater analysis is encouraged. The Director of Public Works may require submittal of documentation on any such program used to analyze culverts. If the culvert is being analyzed as part of a stream or waterway, which is being modeled using a water surface profile program such as HEC-2 or HEC-RAS, then the water surface profile model may be used to analyze the culvert.

The tailwater shall be determined for the design discharge, based on the hydraulic conditions of the downstream channel. Open channel flow methods should be used for this analysis.

A headwall or wingwalls and apron shall be constructed at both ends of all culverts. For small culverts (30" in diameter or less), a sloped end section or a sloped headwall may be specified. The headwall and wingwall design depends largely on hydraulic characteristics of the flow, the site conditions and potential for erosion and scour. The most current TxDOT details for headwalls and wingwalls are acceptable for use in construction of end treatments.

If the outlet velocity exceeds the maximum permissible velocity for the channel it may be mitigated with either channel improvements for erosion protection or energy dissipation. Erosion protection may consist of an apron of rock riprap between the storm sewer headwall and the channel. The apron length, width, and median stone diameter shall be specified as required to protect the channel from erosion. Alternate erosion protection measures that promote sustainability and water quality are allowed and encouraged.

The minimum velocity in the culvert barrel shall be 2.5 feet per second in a 5-year storm. Maximum allowable velocity in the culvert is 15 feet per second in a 100-year storm. However, downstream conditions will generally impose more stringent limits.

The minimum culvert diameter shall be eighteen (18) inches. Culvert material shall be limited to reinforced concrete unless otherwise approved by the Director of Public Works for alternate materials.

3.11 Bridges

Bridges shall be designed to span the entire stream or channel without restricting flow. To the extent possible, bridges will span streams and channels at a ninety-degree (90°) angle.

Flow hydraulics through the bridge shall be modeled using HEC-2 or HEC-RAS. Bridges shall be designed so that the lowest point of the bridge, the low chord, will be a minimum of two (2) feet above the 100-year water surface elevation, assuming fully developed upstream conditions.

3.12 Energy Dissipation

Erosive forces at work in the natural drainage network are often increased by urban land development projects. When the drainage network is altered and runoff is captured and released in a more concentrated fashion the potential for erosion can increase. Excessive velocities and erosion potential shall be considered for all proposed storm drain outfalls, culverts, detention ponds, bridges, and open channels. Design of drainage systems and outfalls shall result in a calculated velocity below the maximum allowable velocity noted in Table 4. In addition, any location that has the potential for erosion per the Director of Public Works but is below the maximum velocity identified in Table 4 shall require permanent erosion control and/or energy dissipation structures.

Several resources are available for the design of energy dissipation structures. The iSWM™ Technical Design Manual contains design guidelines for rock riprap protection using the "Garry Gregory Method". Other techniques and design methods may be utilized, such as those documented in Engineering Manual 1110-2-1601 (USACE) and HEC-14, Hydraulic Design of Energy Dissipators for Culverts and Channels (FHWA).

3.13 Detention Facilities

Proposed developments shall provide detention or downstream drainage system improvements if the downstream drainage system does not have sufficient capacity. Detention facilities for proposed developments are to be maintained by the respective landowners within the development as detailed within the "Detention Area Easement" statement provided in the Appendix A. This detention statement shall be inserted on the Final Plat for any development project with detention facilities.

A downstream assessment shall be performed for detention facilities to a point that meets the Ten Percent Rule, or to a recognized water course (See Section 2.1 and Figure 1, Appendix C) to verify the adequacy of the downstream system. Increased storm water runoff shall not adversely affect adjoining property. Permanent impoundments of water shall be constructed in such a way to prevent negative effects on aesthetics, function, flooding, health, and safety.

Detention storage facilities serving drainage areas smaller than fifty (50) acres may use the Modified Rational Method for storage calculations. If the Modified Rational Method is used, computations shall be provided for review and approval in the format provided in Appendix B, and summaries of methods and results shall be provided in the construction plans.

All storage facilities serving drainage areas greater than fifty (50) acres, or where the Modified Rational Method is not applicable shall be analyzed using HEC-HMS or other approved software for reservoir routing of an inflow hydrograph. The software program or computational method shall be approved by the Director of Public Works. The analysis should consist of comparing the design flows at a point or points downstream of the proposed storage site with and without storage. Design calculations shall show the effects of the detention facility in each of the 2-, 10,-and 100-year storm events. This may require the use of multi-stage control structures. The detention facility shall be designed to provide the required detention for all of the above-listed frequencies.

The potential for the impact of sedimentation on the detention facility should be evaluated. A means of access for maintenance of the facility shall be provided.

The outlet control structures for detention storage facilities typically include a principal outlet and an emergency overflow. The principal outlet functions to restrict the outflow. The principal outlet shall be designed to accommodate the 2, 10, and 100-year frequency storms while maintaining a minimum freeboard of one (1) foot to the top of the pond embankment.

The outlet control structure may be drop inlets, pipes, culverts, weirs, or orifices. Checks should be made to determine if the outlet structure is controlled by weir or orifice flow. The tailwater on the structure could significantly affect its capacity. The engineer should carefully evaluate potential tailwater effects of the downstream system, whether it be a ditch, pipe, road, or creek.

If a riser pipe outlet is used, it shall be protected by a trash rack and anti-vortex plate. If an orifice plate is used, it shall be protected with a trash rack with at least 10 square feet of open surface area. In both cases, the rack shall be hinged or easily removable to allow for cleaning. The rack shall be adequately secured to prevent it from being removed or opened when maintenance is not occurring.

All ponds shall have an emergency overflow spillway or structure designed to convey the 100- year, 24-hour design storm for post-development site conditions, assuming the pond is full to the overflow spillway or structure crest. The overflow shall be designed to convey these extreme event peak flows around the berm structure for discharge into the downstream conveyance system. The overflow shall be designed and sited to protect the structural integrity of the berm.

The subgrade of the spillway shall be set at or above the 100-year overflow elevation of the control structure. The spillway shall be located to direct overflows safely into the downstream conveyance system. The emergency overflow spillway shall be armored with riprap or other flow-resistant material that will protect the embankment and minimize erosion. The overflow erosion protection shall be designed in accordance with the guidelines provided in Section 3.12, and shall extend to the toe of each face of the berm embankment. The emergency overflow spillway weir section shall be designed for the maximum design storm event for post-development conditions.

All TCEQ requirements for impoundment and dam safety shall apply. These requirements relate to both the size and the hazard classification of the embankment. Copies of all materials submitted to TCEQ for permitting, along with the TCEQ permits, shall be submitted to the Director of Public Works.

4 STORM WATER QUALITY DESIGN STANDARDS

The Town of Addison has been issued a storm water discharge permit from the Texas Commission of Environmental Quality (TCEQ) and is required under the Clean Water Act to implement a storm water management program with an overall goal of improving water quality of receiving water bodies. In compliance with the Town's TCEQ storm water permit and in an effort to improve the storm water quality of our local water resources, the Town of Addison has established the following criteria for land development projects and construction sites. It is the responsibility of the Developer, Owner and Contractor to ensure they are in compliance with the most current state or federal storm water regulations including the Clean Water Act, EPA National Pollutant Discharge Elimination System, and TCEQ Texas Pollutant Discharge Elimination System.

4.1 Land Development Post-Construction Runoff Controls

Post-construction runoff controls shall be used to minimize increases in storm water runoff rates and volumes, soil and stream channel erosion, sediment transport and the discharge of pollutants to the Town's municipal separate storm sewer system. The Town of Addison has adopted these standards for the purpose of protecting local water resources from degradation and to protect the general health, safety and welfare of the public residing in affected watersheds.

All new development and re-development projects, one acre or larger, or less than one acre if part of a larger common plan of development or sale, shall implement Best Management Practices (BMP's) to minimize water quality impacts from development after construction is complete (post-construction). Examples of various long-term post-construction BMPs are provided below.

- Floodplain and channel buffers
- Storm water treatment ponds and wetlands
- Grassed swales and channels
- Detention and retention facilities
- Energy dissipators and velocity control structures
- Infiltration trenches
- Rain gardens
- Green roofs
- Open spaces
- Native landscaping and
- Porous parking surfaces
- Storm water re-use and rainwater harvesting
- Tree preservation and tree planting
- Storm water treatment systems

It is the responsibility of the design engineer to verify the adequacy of the post-construction BMP for mitigating the runoff effects from the proposed project. Guidelines for the selection and design of post-construction BMPs may be found in the North Central Texas Council of Government's (NCTCOG) integrated Storm Water Management (iSWM) Technical Manual, Site Development Controls. These BMPs are acceptable for design of storm water controls within this section, however the design of detention facilities shall be as defined in the Drainage Criteria Manual.

It is recommended that the owner, developer and/or design engineer consult with the Director of Public Works prior to design of any post-construction BMPs to discuss and evaluate the intended BMP performance criteria (e.g. water quality, channel protection, runoff volume reduction, etc.) and related long term maintenance requirements. All post-construction BMP's shall be submitted to the Director of Public Works for review and approval prior to construction of the proposed project.

All post-construction BMPs shall have an enforceable long term operation and maintenance agreement to ensure the system functions as designed. This agreement will include any and all maintenance easements as required to access and inspect the storm water practices, and to perform routine maintenance as necessary to ensure proper functioning of the storm water practice. Unless otherwise agreed upon by the Town of Addison, the long term operation and maintenance of any post-construction BMPs shall be the responsibility of the property owner.

4.2 Construction Site Storm Water Pollution Prevention

All construction activity, regardless of size, shall have storm water pollution prevention BMP's installed prior to any land disturbing activities to prevent the discharge of pollutants to the Town's municipal separate storm sewer system. It is the responsibility of owners, developers and contractors to maintain compliance with the (TCEQ) storm water construction permit TXR150000. All storm water BMP's shall be provided, constructed and maintained per the Town of Addison's Erosion Control Details, or in the absence of an applicable Town detail, the standards set forth in the NCTCOG iSWM Technical Manual, Construction Controls.

The developer or owner is responsible for providing the Director of Public Works with a copy of the TCEQ Construction Site Notice for projects between 1 and 5 acres or the TCEQ Notice of Intent (NOI) for projects larger than 5 acres prior to any land disturbing activities. Reference the TCEQ general permit TXR150000 for further information. Figure 16 in Appendix C provides a TCEQ Construction Permit Flow Chart and may be used to help determine whether a construction site is regulated under the permitting program.

It is the intent of the Town of Addison that sediment from construction sites be contained on the site. Sediment ponds shall be provided for construction sites as required by the TCEQ TPDES Construction Permit. Accordingly, Storm Water Pollution

Prevention Plans (SWPPP) shall address two stages. Stage I refers to the initial grading and infrastructure construction phase of the development. Sediment ponds, boundary silt fence or other approved mass grading erosion and sediment controls shall be in installed before construction may commence. Inlet protection devices shall be installed until streets and alleys have been paved. Immediately after paving, an erosion control blanket (or other erosion control method as approved by the Director of Public Works) shall be installed adjacent to paved surfaces such as alleys, streets, flumes, etc., in addition to the placement of silt fence along the downslope boundaries of the site. No construction is permitted until Stage 1 erosion controls are installed and the Town has received a copy of the NOI or Construction Site Notice.

Stage 2 refers to the period of time after acceptance of the development by the Town and prior to completion of homes or other buildings. During this stage, an erosion control blanket (or other erosion control method as approved by the Director of Public Works) shall be placed adjacent to all streets and alleys and at the project perimeter to contain sediment within the block and prevent transport to the pavement. The maintenance of this perimeter erosion control system becomes the builder's responsibility once a building permit is issued. The developer is then responsible for the perimeter erosion controls until the development or phase is 95% built out.

5 CONSTRUCTION PLAN REQUIREMENTS

All construction plans prepared for public works or private development drainage facilities shall be prepared and sealed by a Professional Engineer who is licensed in the State of Texas and is experienced in civil engineering work with specific practical knowledge of storm drain design.

Plans shall be submitted on 22"x34" sheets to allow accurately scaled half-size (11"x17") reproduction. Plans shall include the following information.

5.1 Drainage Area Maps and Calculations

Drainage area maps and calculations shall be provided and shall include the following information at a minimum:

- ☐ Site map showing project limits with a scale of one (1) inch equals one hundred (100) feet. Large off-site drainage areas may use one (1) inch equals two hundred (200) feet. The Director of Public Works may require a larger scale if necessary to depict the necessary information in a readable format.
- ☐ The delineation of the overall watershed drainage area contributing runoff to the proposed system, including sub-drainage areas delineating contributing areas to the drainage design points/collection points.
- ☐ The limits of all floodplains and floodways (per the most recent FIS) within the project area.
- ☐ Provide inlet identifying numbers (design point numbers). These inlet numbers shall correlate to the inlet calculations table and shall also be identified within the construction plans.
- ☐ Show streets, property lines, right-of-way, building footprints and locations of creeks, streams or other adjacent waterbodies.
- ☐ Provide existing ground topography with one (1) foot or two (2) foot contours.
- ☐ Provide a hydrologic summary table showing drainage area calculations for both existing and proposed conditions for each design point basin.
- ☐ Provide calculations for the time of concentration (T_c) indicating how the total T_c was determined.
- ☐ Direction of flow (arrows) within streets, alleys, natural and man-made drainage ways, and at all system intersections including sags, crests and corners.
- ☐ All existing and proposed drainage facilities shall be clearly shown and differentiated on the drainage area map.
- ☐ The means provided for accommodating any increase in runoff due to the development shall be clearly depicted, along with the means for handling runoff that is conveyed to or through the site from upstream. This may be handled using general notes on the map.

- ☐ Provide drainage area, inlet, storm drain and culvert computation tables as required. Figures 12 and 15 located in Appendix C provide the format for these tables.
- ☐ Additional information may be required at the discretion of the Director of Public Works to adequately address the particular conditions of a given project.

5.2 Grading Plans

Grading plans shall be prepared for all proposed developments, and shall show the following information at a minimum:

- ☐ Identifying all proposed structures, buildings, streets, parking areas, flumes, storm drain systems, open channels, creeks, etc.
- ☐ Identify locations of right-of-way, property lines and easements.
- ☐ Provide existing and proposed contours at one (1) foot intervals for all commercial and industrial developments, along with proposed flow arrows.
- ☐ Grading plans for residential developments shall show existing contours at one (1) foot intervals, and shall depict proposed grading by the use of spot elevations and flow arrows.
- ☐ Spot elevations shall be shown at the top of the curb adjacent to each lot line, and adjacent to each building corner as well as the upper end of each swale.
- ☐ Clearly show swales, ditches and other means of conveying storm water runoff across the proposed site.
- ☐ Finish floor elevations shall be shown, and flow arrows used to indicate flow patterns.
- ☐ In residential developments, storm water may not cross more than one lot before being discharged to a street, alley, channel or other public storm drainage facility.
- ☐ In all other developments, concentrated storm water may not be discharged to an adjacent property (other than a recognized watercourse) except in a dedicated easement and an approved storm drainage system.
- ☐ Runoff from adjacent lots or properties shall be collected and conveyed in an easement rather than across lots.
- ☐ Positive overflow shall be provided at sump or low point inlets.
- ☐ Minimum finish floor elevations adjacent to such overflows shall be no less than 12" above the overland flow water surface elevation, with positive drainage provided away from the building.
- ☐ Minimum finish floor elevations shall be set at least two (2) feet above the 100-year base flood elevation of any adjacent stream for which base flood elevations have been set as depicted on the most current Flood Insurance Rate Map.

5.3 Storm Drainage Plans

Storm drain plans shall be provided for all storm drainage systems including culverts and shall include the following information at a minimum:

- ☐ Plan and profile sheets at a scale not greater than 1"=50' horizontally, and 1"=5' vertically. The storm drainage system shall be shown on separate sheets from the paving plans.
- ☐ All property lines, right-of-way lines, and easements shall be shown, and the storm drainage facilities tied to these as appropriate.
- ☐ Detailed geometry to facilitate construction, including stations at all junctions, structures, pipe size changes, inlets, and all changes in direction, including PC's, PT's, and PI's, along with complete curve data.
- ☐ Surface flow arrows shall be shown at all intersections and high points.
- ☐ Storm drain pipes shall generally be located in the center of the roadway.
- ☐ Junction boxes or in-line structures shall be spaced a maximum of 400' apart for pipe 24-inches or smaller in diameter, and a maximum of 800' apart for pipe 27-inches or greater in diameter.
- ☐ Where multiple inlet leads intersect the main at the same station, a junction box shall be constructed.
- ☐ All property lines, right-of-way lines, and easements shall be shown, and the storm drain facilities tied to these as appropriate.
- ☐ All existing and proposed utilities shall be shown in the plan view, and in the profile view where such information is available.
- ☐ Profiles shall include existing ground line at the center of the proposed storm drain, proposed ground line at the center of the proposed storm drain, 100-year hydraulic grade line (HGL) with HGL elevations at each junction, end of pipe, and pipe size change. Pipe flowline elevations shall be shown at fifty (50) foot intervals along the pipe. The profiles should identify the class of concrete pipe, size, length and slope of each pipe, along with the design flow (Q_{10} or Q_{100} depending on the design event), the full flow capacity of the pipe ($Q_{capacity}$) in partial flow situations, design velocity (V_{10} or V_{100}), the friction slope (S_f), and the velocity head (H_V).
- ☐ The hydraulic grade line shown shall include all minor losses at appurtenances.
- ☐ The starting (downstream) elevation of the hydraulic grade line shall be based on downstream conditions. Determination of this starting elevation shall be documented.
- ☐ In the plan view, indicate the size and type of inlet, top of curb elevation, hydraulic grade elevation, flowline elevation, and lead line length, size and slope for each inlet. All inlet laterals shall be shown in profile in the plans and shall show the HGL.
- ☐ All storm drain pipe connections shall match at the crowns of the pipe. Laterals shall connect to the main such that the center of the lateral matches the center of the main.
- ☐ Identify erosion protection measures for storm drain outfall structures, where required.
- ☐ Identify benchmarks, including any Town of Addison benchmarks if available.

- ☐ 60° inlet leads are preferred. Inlet leads shall connect to the main at a thirty-degree (30°) angle or greater (no connections greater than sixty degrees (60°) shall be permitted without a junction box. No connections greater than ninety degree (90°) will be permitted.
- ☐ All storm drain fittings including pipe bends and wyes shall be prefabricated, unless connecting to an existing system whereas the Town's standard connection detail shall be used.
- ☐ Applicable Town of Addison Standard Construction details shall be included in the construction plans. Modifications to these standard details shall not be made without prior approval of the Director of Public Works.
- ☐ Identify all proposed post-construction storm water quality BMPs. Include appropriate design calculations and other information in accordance with Town or NCTCOG iSWM design guidelines.

5.4 Channel Plans

- ☐ Plan and profile sheets at a scale not greater than 1"=50' horizontally, and 1"=5' vertically. Detailed geometry to facilitate construction, including stations at all junctions, structures, and all changes in direction, including PC's, PT's, and PI's, along with complete curve data.
- ☐ All property lines, right-of-way lines, and easements shall be shown, and the channel facilities tied to these as appropriate.
- ☐ All existing and proposed utilities shall be shown in the plan view, and in the profile view where such information is available.
- ☐ Profiles shall include existing ground line at the center of the proposed channel, proposed ground line at the center of the proposed channel, channel slope, proposed right and left top of bank, 100-year water surface elevation, and flowline elevations at each structure, grade change, etc., as well as at 50-foot intervals along the channel.
- ☐ The runoff to be conveyed (Q_{100}), the flow depth (D) in the channel, the capacity of the channel at full flow (Q provided), the Manning's "n" value used, and the velocity at Q_{100} shall be shown on the plans.
- ☐ Identify erosion protection measures for storm drain outfall structures.
- ☐ A typical section(s) shall be depicted on the plans, showing bottom width, side slopes, lining (if applicable), depth, etc.
- ☐ Earthen channels shall have side slopes no steeper than 3:1.
- ☐ Energy dissipation and erosion protection structures.
- ☐ Actual cross sections shall be shown at no less than 100-foot intervals.
- ☐ Specify compaction requirements where fill shall be placed.

5.5 Detention/Retention Facilities

- ☐ Show all hydrologic, routing, storage and outlet calculations. The Modified Rational method may be used for drainage areas less than fifty (50) acres for all systems where it applies. Refer to the Appendix B – MRM Detention Basin Design Example for methodology. For larger drainage areas and more complex smaller areas, a unit hydrograph method that employs reservoir routing calculations shall be used.

- ☐ Retention Ponds shall have a 5' (ft) wide safety shelf two feet below the normal pool elevation, then slope 6:1 (H:V) to the bottom of the pond. Minimum depth of a retention pond is 6' (ft).
- ☐ Provide detailed plans showing all aspects of the outlet structure(s), along with hydraulic calculations of the outlet(s).
- ☐ Pond routing summary, including return event, peak flow in, peak flow out, critical duration, max water surface, and max storage volume shall be included on the plans.
- ☐ Outfall rating curve showing the total pond discharge at all critical elevations from the flowline to the top of pond, in increments of 1' (ft) or less shall be included.
- ☐ Grading plans shall be provided for the facility. A concrete pilot channel with minimum slope of 0.50% shall be provided unless the pond bottom maintains a 2% slope to the outfall.
- ☐ Show downstream conditions and provide information that shows the effect of the discharge on downstream properties and/or structures.
- ☐ Show adequate erosion control measures at outlet structure(s).
- ☐ Show inundation areas for the 2-year, 10-year, and 100-year storm events, as well as any overflow facilities.
- ☐ Show existing and proposed contours to depict slopes of embankments. The maximum slope of embankments shall be 3:1, without the use of retaining walls
- ☐ Show emergency overflow provisions.

5.6 Erosion Control Plans

- ☐ Erosion Control Plans shall be prepared and sealed by a licensed professional engineer.
- ☐ Contours or other indication of flow direction shall be shown on the plan.
- ☐ Show the location of all structural sediment control measures, including sediment pond locations (when required).
- ☐ Sediment pond (when required) calculations shall be shown on the erosion control plan.
- ☐ Stabilization measures shall be identified.
- ☐ Required SWPPP maintenance and inspection procedures shall be outlined.
- ☐ Project sequencing and/or phasing shall be identified.
- ☐ Both construction (Stage 1) and post-construction (Stage 2) conditions shall be shown.

6 REGULATORY ISSUES

The Developer is responsible for obtaining approvals and maintaining compliance with any State or Federal regulations, permits, and programs including TCEQ, EPA, U.S. Army Corps of Engineers (USACE), FEMA or other agencies. The Developer is required to provide the Town with copies of all approved permits and associated submittals required by any State or Federal agency for developments within the Town of Addison.

6.1 U.S. Army Corps of Engineers 404 Permits

The U.S. Army Corps of Engineers (USACE) has been directed by Congress under Section 404 of the Clean Water Act (33 USC 1344) to regulate activities impacting all waters of the United States, including wetlands and other jurisdictional waters. The USACE has developed a permitting process to ensure compliance with the Clean Water Act. Developers will be expected to ensure that all requirements of the Clean Water Act are met. The Town of Addison assumes no responsibility for the compliance of the Developer with this or any other Federal regulations.

However, the Developer should be aware that under current regulations, most of the streams within the Town of Addison are likely to be considered jurisdictional waters by the USACE. As a result, permitting will likely be required for any projects impacting these streams by fill, excavation, utility crossings or roadway crossings. In many cases, these permits carry significant compensatory mitigation requirements to offset losses of jurisdictional waters and their associated habitat.

6.2 National Flood Insurance Program

The Town of Addison is a participant in the National Flood Insurance Program administered by the Federal Emergency Management Agency (FEMA). Copies of the Flood Insurance Rate Maps (FIRM) are available from the Town, for a fee, depicting the 100-year floodplain developed for insurance rating purposes. Floodplain management in the Town of Addison is under the direction of the Director of Public Works, who also functions as the Floodplain Administrator. Refer to the Town of Addison's floodplain ordinance (Code of Ordinances, Chapter 42 – Floods) for further information.

REFERENCES

Federal Emergency Management Agency. “Flood Insurance Study for Dallas County, Texas and Incorporated Areas”. FIS Study No. 48113CV001C. Revised June 16, 2005.

Federal Highway Administration. Drainage of Highway Pavements. Hydraulic Engineering Circular No. 12. March 1984.

Federal Highway Administration. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14, 3rd Edition. 2006.

Federal Highway Administration. Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5. September 2001.

Gregory, Garry H. “Stone Riprap Design Guidelines”. Unpublished paper. Fort Worth, Texas. June 1987

North Central Texas Council of Governments, iSWM™ (Integrated Storm Water Management) Criteria Manual for Site Development and Construction, 2010.

Texas Commission on Environmental Quality, Texas Pollutant Discharge Elimination System General Permit for Construction Sites, TXR150000. February 15, 2008.

Texas Department of Transportation, Design Division. Hydraulic Design Manual. 2009.

Town of Addison, Texas. Code of Ordinances. Flood Damage and Prevention Ordinance. 2001.

Town of Addison. Standard Construction Details for Paving, Storm Drainage, Erosion Control. Public Works Department. August 2010.

Town of Addison, Texas. Drainage Criteria Manual. March 1990.

U.S. Army Corps of Engineers. Hydrologic Engineering Center Hydrologic Modeling Software (HEC-HMS).

U.S. Army Corps of Engineers. Hydrologic Engineering Center River Analysis System (HEC-RAS).

U.S. Department of Agriculture, Soil Conservation Service, "Urban Hydrology for Small Watersheds," Technical Release 55, June 1986.

U.S. Army Corps of Engineers, Hydraulic Design of Flood Control Channels, Engineer Manual 1110-2-1601. 1991/1994

APPENDIX A

STANDARD EASEMENT DEDICATION STATEMENT

The following "Detention Area Easement" statement shall be placed on the Final Plat for all detention structures. Consult with the Town of Addison concerning easement statements that may be required for a Final Plat for long term operation and maintenance of other storm drainage structures or storm water quality structures.

DETENTION AREA EASEMENT STATEMENT

This plat is approved by the Town of Addison and accepted by the owner(s), subject to the following conditions which shall be binding upon the owner(s), his heirs, grantees and successors, and assigns:

The proposed detention area easement(s) within the limits of this addition, will remain as detention area(s) to the line and grade shown on the plans at all times and will be maintained by the individual owner(s) of the lot or lots that are traversed by or adjacent to the detention area(s). The Town of Addison will not be responsible for the maintenance and operation of said detention area(s) or any damage or injury to private property or person that results from the flow of water along, into or out of said detention area(s), or for the control of erosion.

No obstruction to the natural flow of storm water run-off shall be permitted by filling or construction of any type of dam, building, bridge, fence, walkway or any other structure within the designated detention area(s) unless approved by the Director of Public Works, provided; however, it is understood that in the event it becomes necessary for the Town of Addison to erect any type of drainage structure in order to improve the storm drainage that may be occasioned by the streets and alleys in or adjacent to the subdivisions, then, in such event, the Town of Addison shall have the right to enter upon the detention area(s) at any point, or points, to erect, construct and maintain any drainage facility deemed necessary for drainage purposes. Each property owner shall keep the detention area(s) traversing or adjacent to his property clean and free of debris, silt and any substance which would result in unsanitary conditions or blockage of the drainage. The Town of Addison shall have the right of ingress and egress for the purpose of inspection and supervision of maintenance work by the property owner(s), or to alleviate any undesirable conditions, which may occur.

The detention area(s) as in the case of all detention areas are subject to storm water overflow(s) to an extent which cannot be clearly defined. The Town of Addison shall not be held liable for any damages of any nature resulting from the occurrences of these natural phenomena, nor resulting from the failure of any structure or structures, within the detention area(s) or subdivision storm drainage system.

The detention area easement line identified on this plat shows the detention area(s) serving this addition.

APPENDIX B

MODIFIED RATIONAL METHOD DETENTION BASIN DESIGN EXAMPLE

GIVEN: A 25-acre site, currently open space, is to be developed for retail use. The entire site is the drainage area of the proposed detention basin.

DETERMINE: The maximum release rate and required detention storage, detention basin size and shape, and outlet structure configuration for the 2-, 10-, and 100-year events (sample calculations for the 100 year storm are provided below)

SOLUTION:

Step 1. Determine peak runoff rate prior to site development. This is the maximum release rate from the site after development.

Step 2. Determine inflow for storms of various durations in order to determine maximum volume required with release rate determined in Step 1.

NOTE: Incrementally increase durations to determine maximum required volume. The duration with a peak inflow less than maximum release rate, or where required storage is less than storage for the prior duration, is the last increment.

Present Conditions $Q = CIA$

C	=	0.30
Tc	=	20 min.
I_{100}	=	7.05 in./hr.
Q_{100}	=	0.30 X 7.05 X 25 = 52.9 cfs (Maximum release rate)

Future Conditions (Retail)

C	=	0.90
Tc	=	15 min. (calculated)
I_{100}	=	7.99 in./hr.
Q_{100}	=	0.90 X 7.78 X 25 = 179.8 cfs

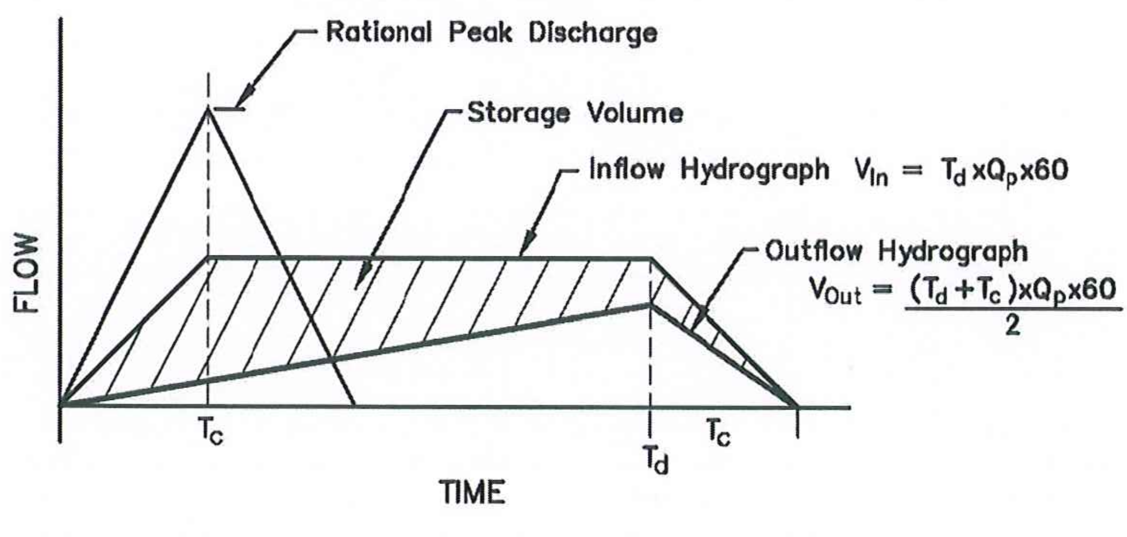
Check various duration storms: $Q_p = C \times I \times A$

15 min.	I = 7.99	Q_p = 0.90 x 7.99 x 25 = 179.8 cfs
20 min.	I = 7.05	Q_p = 0.90 x 7.05 x 25 = 158.6 cfs
30 min.	I = 5.77	Q_p = 0.90 x 5.77 x 25 = 129.8 cfs
40 min.	I = 4.92	Q_p = 0.90 x 4.92 x 25 = 110.7 cfs
50 min.	I = 4.32	Q_p = 0.90 x 4.32 x 25 = 97.2 cfs
60 min.	I = 3.86	Q_p = 0.90 x 3.86 x 25 = 85.9 cfs
70 min.	I = 3.50	Q_p = 0.90 x 3.50 x 25 = 78.8 cfs
80 min.	I = 3.21	Q_p = 0.90 x 3.21 x 25 = 72.2 cfs
90 min.	I = 2.97	Q_p = 0.90 x 2.97 x 25 = 66.8 cfs

Maximum Storage Volume is determined by deducting the volume of runoff released during the time of inflow from the total inflow volume.

Inflow = Storm duration x respective peak discharge x 60 sec./min.

Outflow = Half of inflow time ($T_d + T_c$) x control release discharge x 60 sec./min.



15 min. Storm	Inflow	15 x 179.8 x 60 sec./min	= 161,820 cf
	Outflow	0.5 x 30 x 52.9 x 60 sec./min	= 53,958 cf
	Storage		125,646 cf

20 min. Storm	Inflow	20 x 158.6 x 60 sec./min	= 190,320 cf
	Outflow	0.5 x 35 x 52.9 x 60 sec./min	= 55,545 cf
	Storage		134,775 cf
30 min. Storm	Inflow	30 x 129.8 x 60 sec./min	= 233,640 cf
	Outflow	0.5 x 45 x 52.9 x 60 sec./min	= 71,415 cf
	Storage		162,225 cf
40 min. Storm	Inflow	40 x 110.7 x 60 sec./min	= 265,680 cf
	Outflow	0.5 x 55 x 52.9 x 60 sec./min	= 87,285 cf
	Storage		178,395 cf
50 min. Storm	Inflow	50 x 97.2 x 60 sec./min	= 291,600 cf
	Outflow	0.5 x 65 x 52.9 x 60 sec./min	= 103,155 cf
	Storage		188,445 cf
60 min. Storm	Inflow	60 x 85.9 x 60 sec./min	= 309,240 cf
	Outflow	0.5 x 75 x 52.9 x 60 sec./min	= 119,025 cf
	Storage		190,215 cf
70 min. Storm	Inflow	70 x 78.8 x 60 sec./min	= 330,960 cf
	Outflow	0.5 x 85 x 52.5 x 60 sec./min	= 134,889 cf
	Storage		196,085 cf
80 min. Storm	Inflow	80 x 72.2 x 60 sec./min	= 346,560 cf
	Outflow	0.5 x 95 x 52.9 x 60 sec./min	= 150,765 cf
	Storage		195,795 cf
90 min. Storm	Inflow	90 x 66.8 x 60 sec./min	= 360,720 cf
	Outflow	0.5 x 105 x 52.9 x 60 sec./min	= 166,635 cf
	Storage		194,085 cf

Maximum volume required is 196,085 cf and the critical storm duration is 70 minutes for the 100-year storm.

Step 3.
Size the basin to contain the required volume for the 100-year storm while maintaining minimum slope and freeboard requirements.

Step 4.
Using the selected geometry of the basin as determined above and the storage volumes required for each storm event, determine the maximum depth in the basin for each

storm. To design the outlet structure for the required multiple frequencies, the calculations shown above may be repeated for each frequency in tabular form as shown below. For each storm event, the highest storage value calculated, along with the selected basin size and shape, determines the maximum depth for each event. This depth is the head used in outlet structure design.

Step 5.
A trial outlet structure is selected, and may be a weir, an orifice (pipe), a V-notch weir, or a combination of outlets.

The outlet structure is designed using the head calculated above, to provide a peak discharge that is no greater than 2% above existing (undeveloped) peak runoff for each of the 2-, 10-, and 100-year storm events.

Several iterations may be necessary to balance discharge from the outlet structure, pond geometry and head. If the discharge is significantly different (either higher or lower) than the discharge assumed in Step 2, the actual operation of the pond will not correspond to the calculations. Discharge for any given event may not exceed the allowable discharge determined in Step 1 by more than 2% nor may actual discharge be more than 10% below the allowable discharge.

APPENDIX C

FIGURES

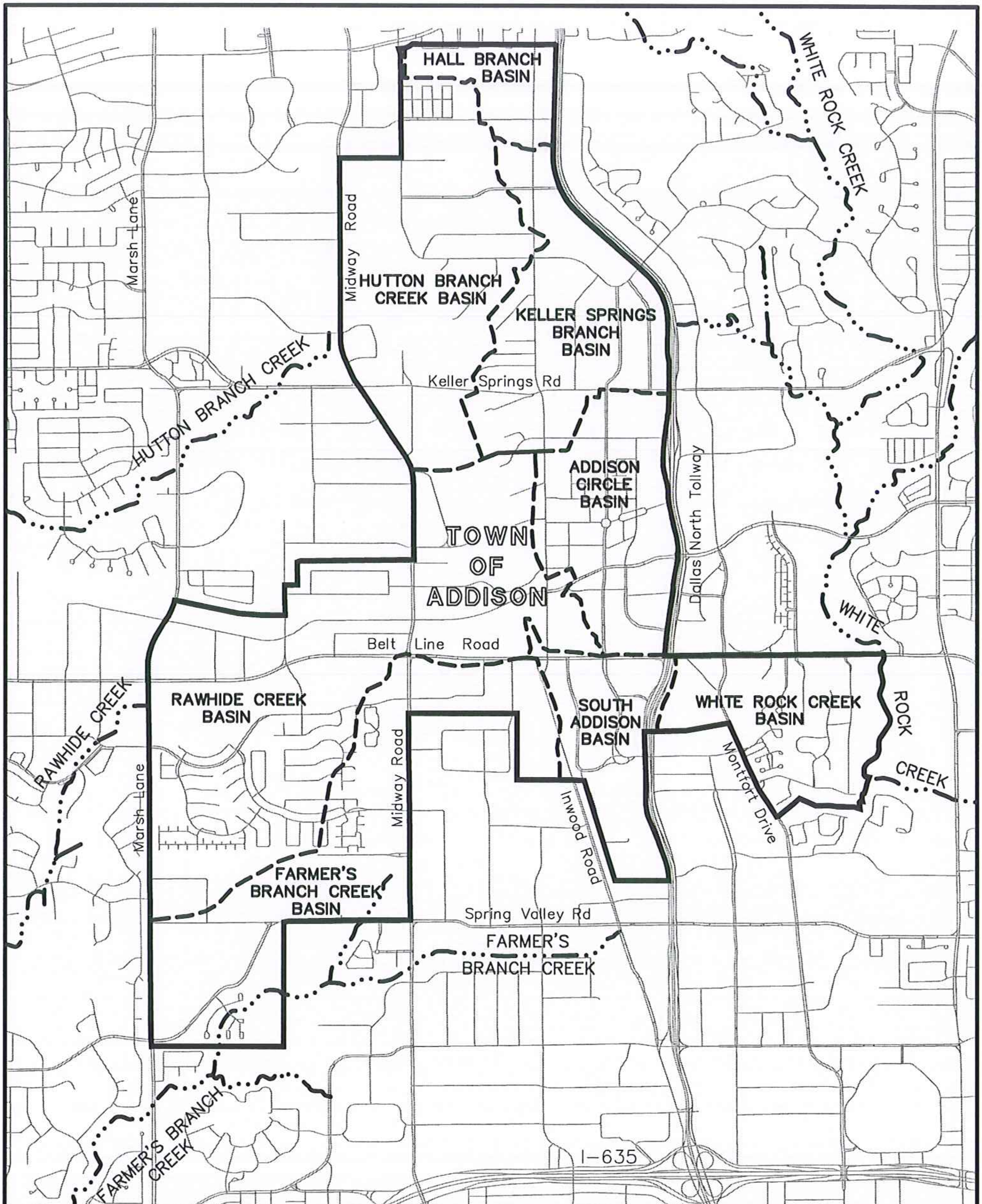


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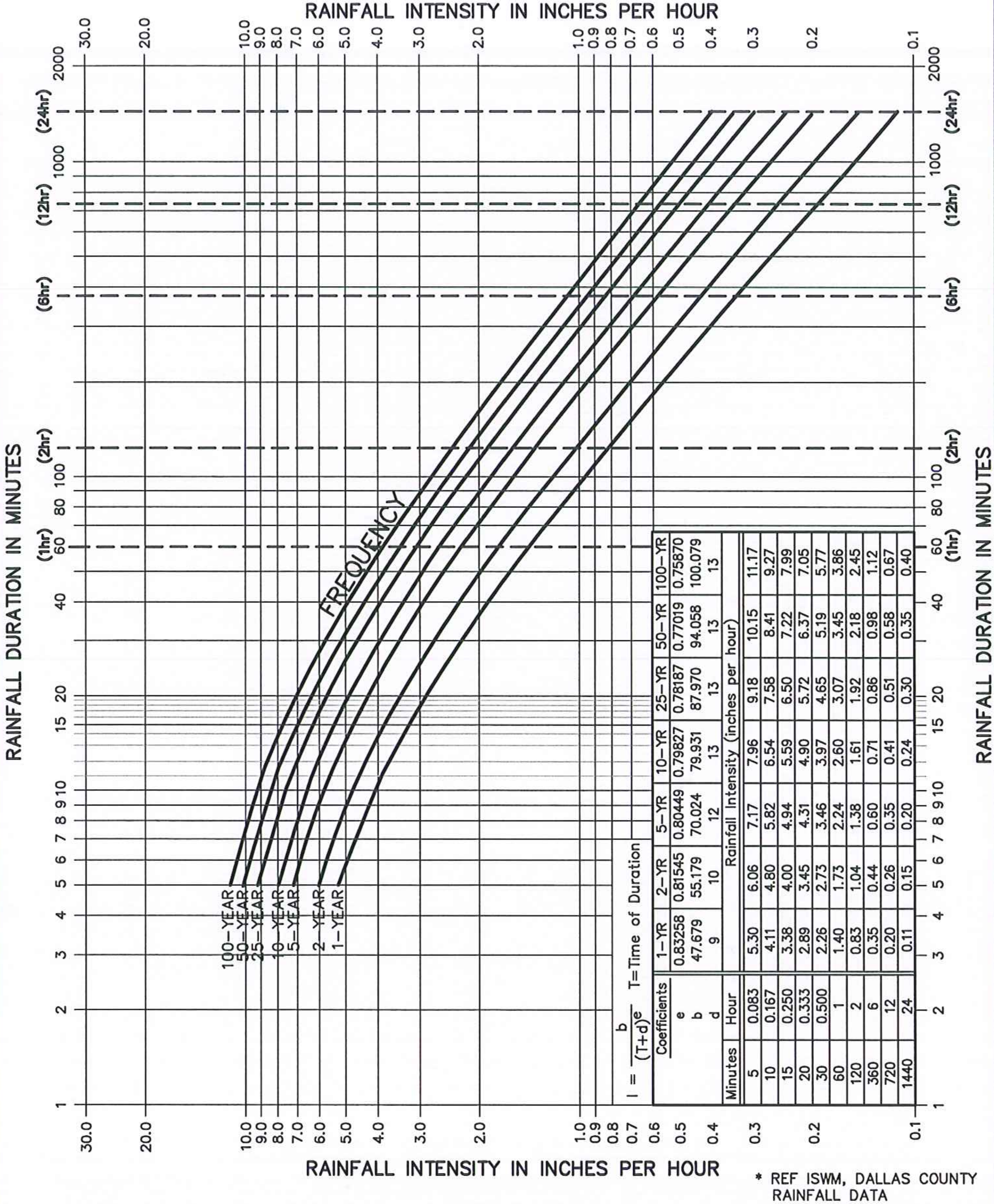
FIGURE 1

MAJOR DRAINAGE BASINS

Town of ADDISON
DRAINAGE CRITERIA MANUAL



$$I = \frac{b}{(T+d)^e} \quad T = \text{Time of Duration}$$

Coefficients		1-YR	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
e		0.83258	0.81545	0.80449	0.79827	0.78187	0.77019	0.75870
b		47.679	55.179	70.024	79.931	87.970	94.058	100.079
d		9	10	12	13	13	13	13
Minutes	Hour	Rainfall Intensity (inches per hour)						
5	0.083	5.30	6.06	7.17	7.96	9.18	10.15	11.17
10	0.167	4.11	4.80	5.82	6.54	7.58	8.41	9.27
15	0.250	3.38	4.00	4.94	5.59	6.50	7.22	7.99
20	0.333	2.89	3.45	4.31	4.90	5.72	6.37	7.05
30	0.500	2.26	2.73	3.46	3.97	4.65	5.19	5.77
60	1	1.40	1.73	2.24	2.60	3.07	3.45	3.86
120	2	0.83	1.04	1.38	1.61	1.92	2.18	2.45
360	6	0.35	0.44	0.60	0.71	0.86	0.98	1.12
720	12	0.20	0.26	0.35	0.41	0.51	0.58	0.67
1440	24	0.11	0.15	0.20	0.24	0.30	0.35	0.40

* REF ISWM, DALLAS COUNTY RAINFALL DATA

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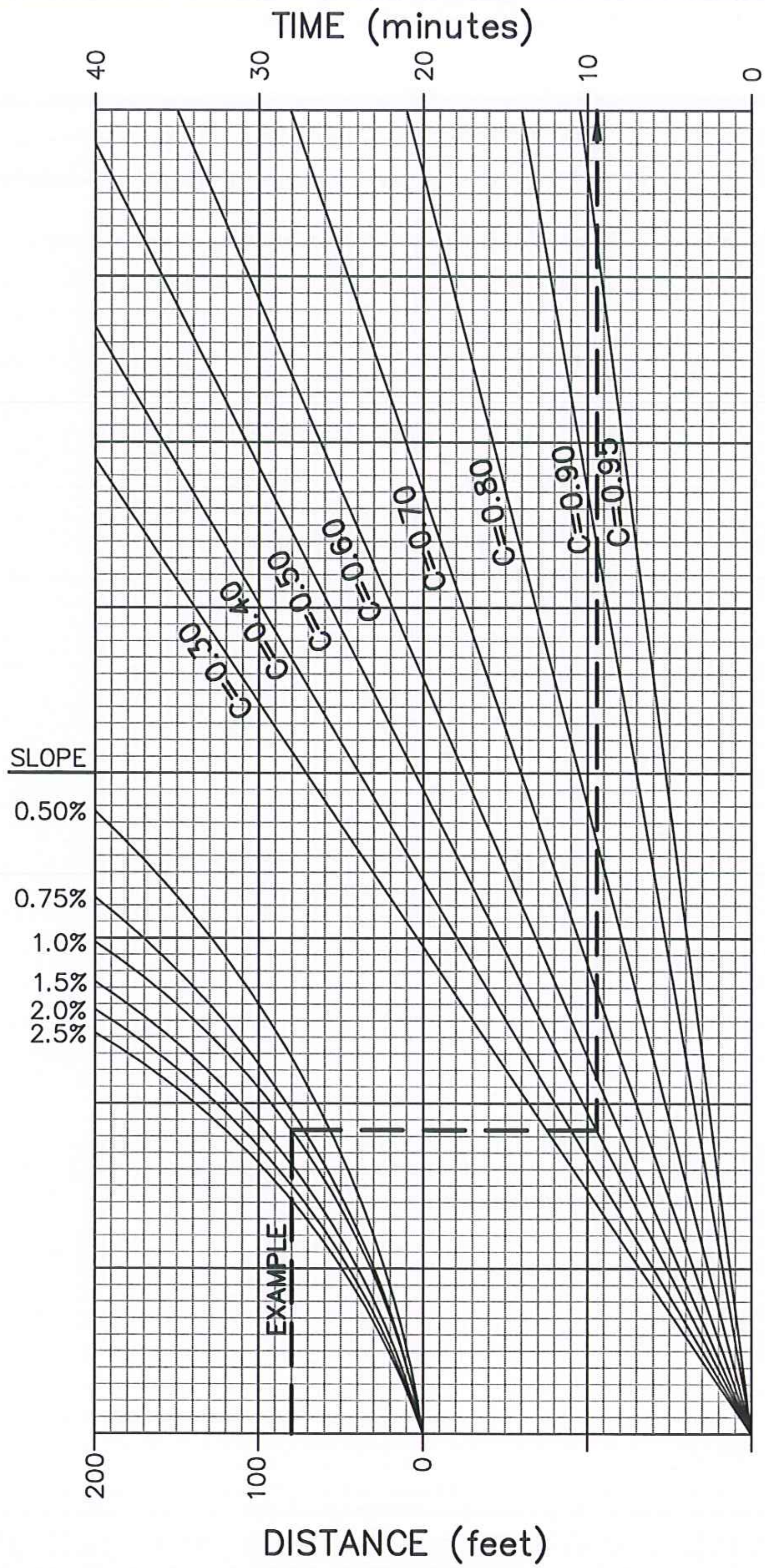
FIGURE 2

RAINFALL INTENSITY–DURATION–FREQUENCY CURVES

Town of ADDISON
DRAINAGE CRITERIA MANUAL



EXAMPLE: 80 feet at 1.0%, C = 0.50;
Time = 9.2 Minutes Overland Flow

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FIGURE 3

OVERLAND TIME OF FLOW

Town of ADDISON
DRAINAGE CRITERIA MANUAL

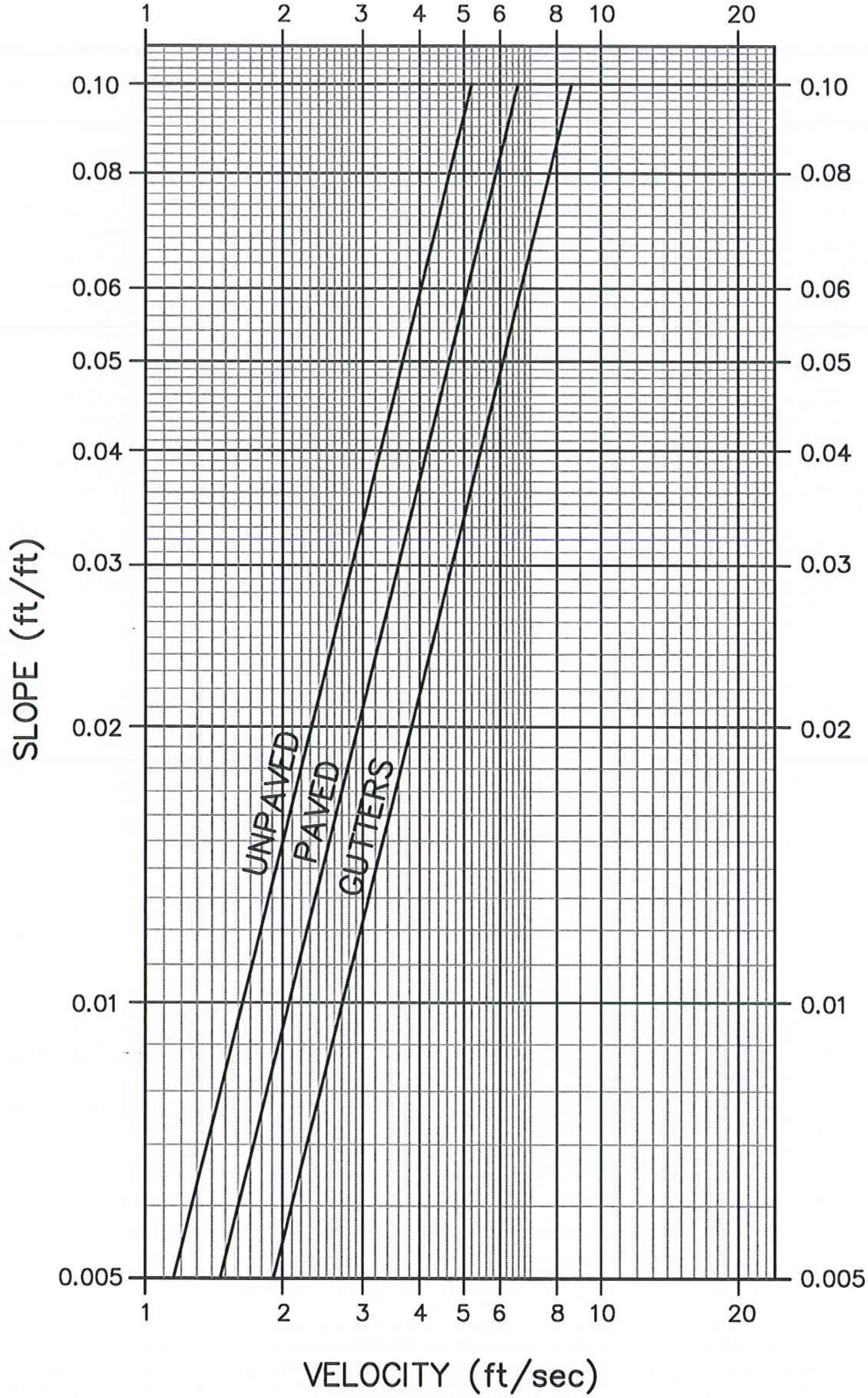

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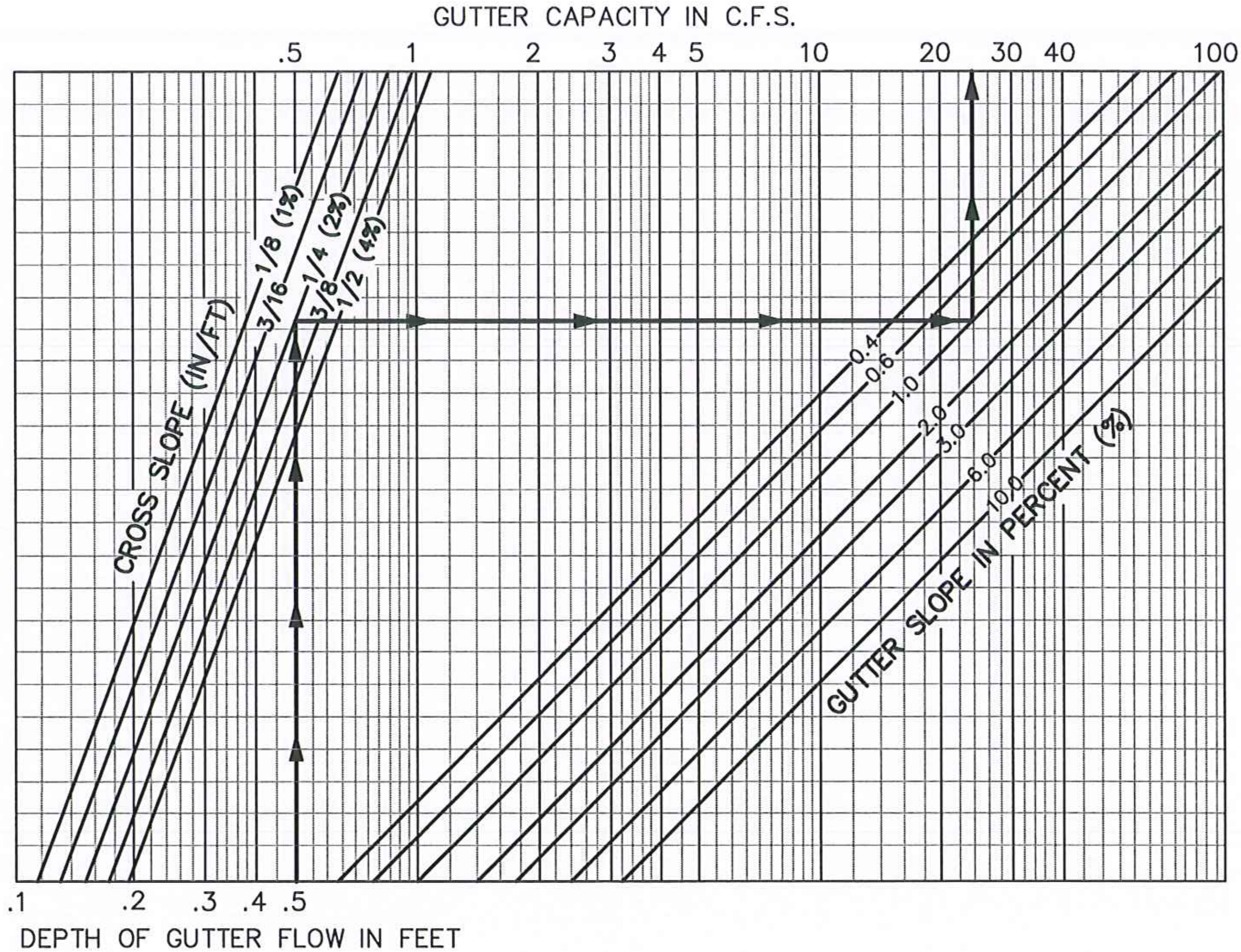
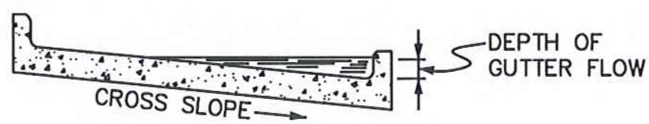
FIGURE 4

SHALLOW CONCENTRATED FLOW VELOCITY

Town of ADDISON

DRAINAGE CRITERIA MANUAL





(Roughness Coefficient "n" = .0175)

EXAMPLE

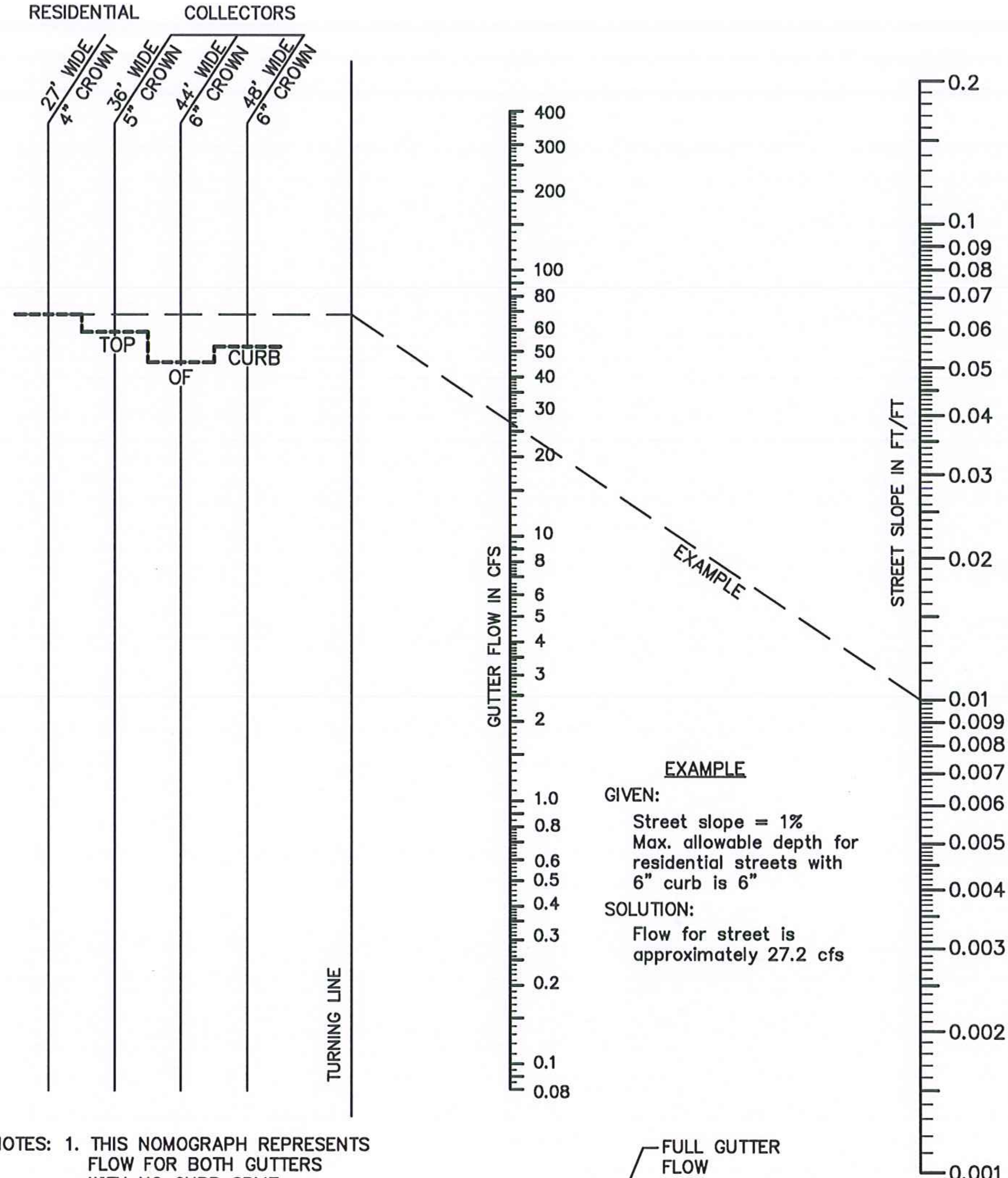
Known:
- Gutter Slope = 1.0%
- Pavement Cross Slope = 1/4"per ft.
- Depth of Gutter Flow = 0.5'

Find:
- Gutter Capacity

Solution:
- Enter Graph at 0.5'
- Intersect Cross Slope = 1/4"per ft.
- Intersect Gutter Slope = 1.0%
- Read Gutter Capacity = 24 c.f.s.

FIGURE 5

CAPACITY OF TRIANGULAR GUTTERS



NOTES:
1. THIS NOMOGRAPH REPRESENTS FLOW FOR BOTH GUTTERS WITH NO CURB SPLIT.
2. ROUGHNESS COEFFICIENT – "n"=0.0175.
3. CURB HEIGHT = 6".

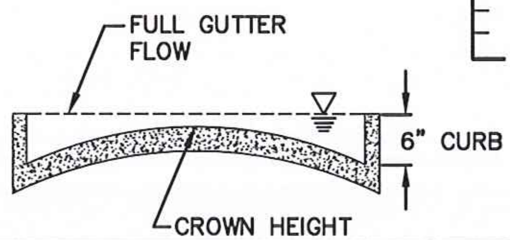


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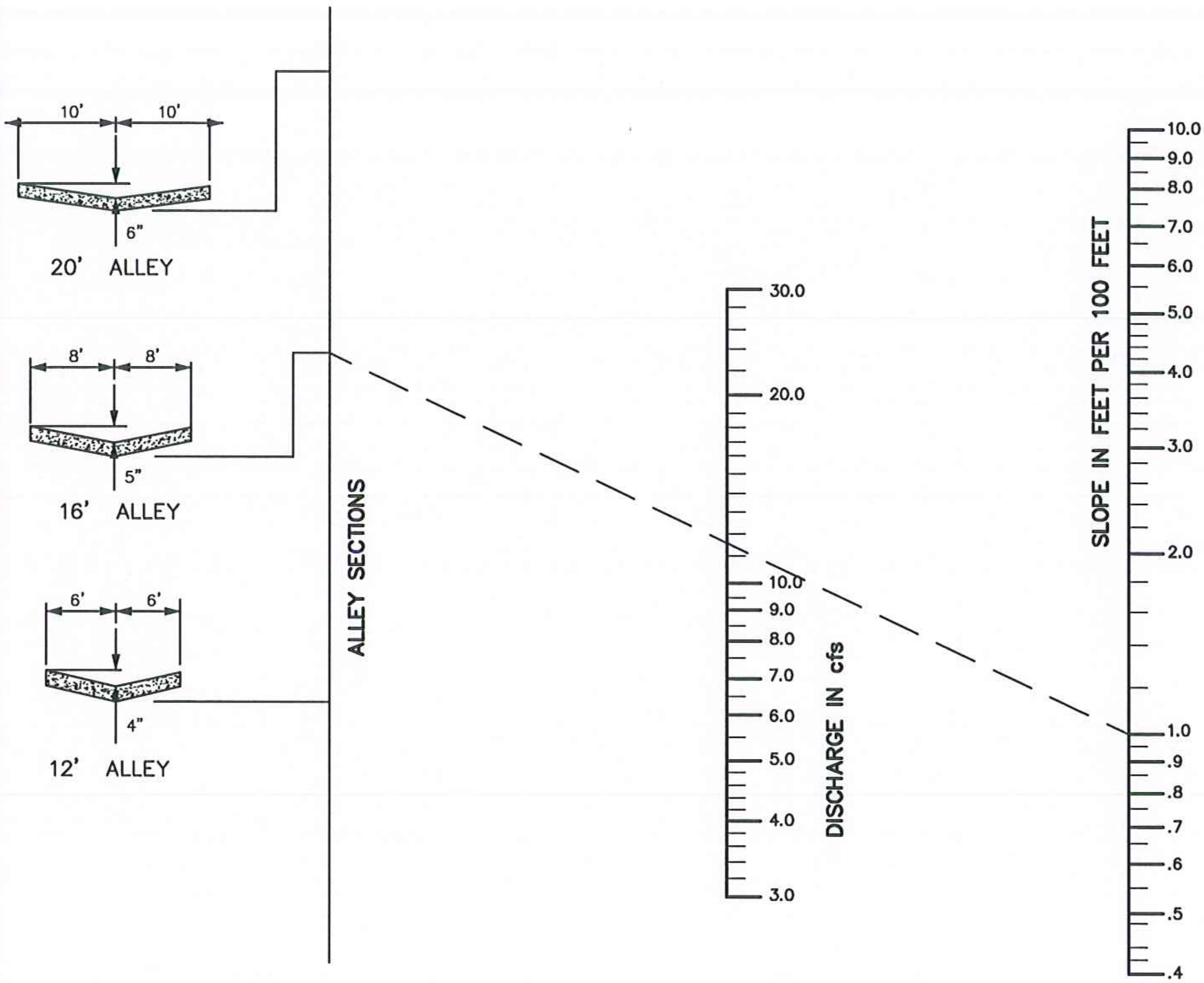
FIGURE 6

FULL GUTTER FLOW CAPACITY OF PARABOLIC CROWNS

Town of ADDISON
DRAINAGE CRITERIA MANUAL



NOTE:

- All alley capacities are to paving edge.
- The Capacities obtained from this Nomograph are based on a Straight Horizontal Alignment. Curved Alignments may result in Reduced Capacity.
- Capacities are for alleys without curbs.

EXAMPLE

KNOWN:
Alley width = 16'
Alley depression = 5"
Gutter slope = 1.0%
"n" = 0.015

FIND:
Gutter Flow (Q)

SOLUTION:
Connect the 16' alley section with slope = 1.0%. Read Q = 11.6 c.f.s.

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FIGURE 7

CAPACITY OF ALLEY SECTIONS

Town of ADDISON
DRAINAGE CRITERIA MANUAL

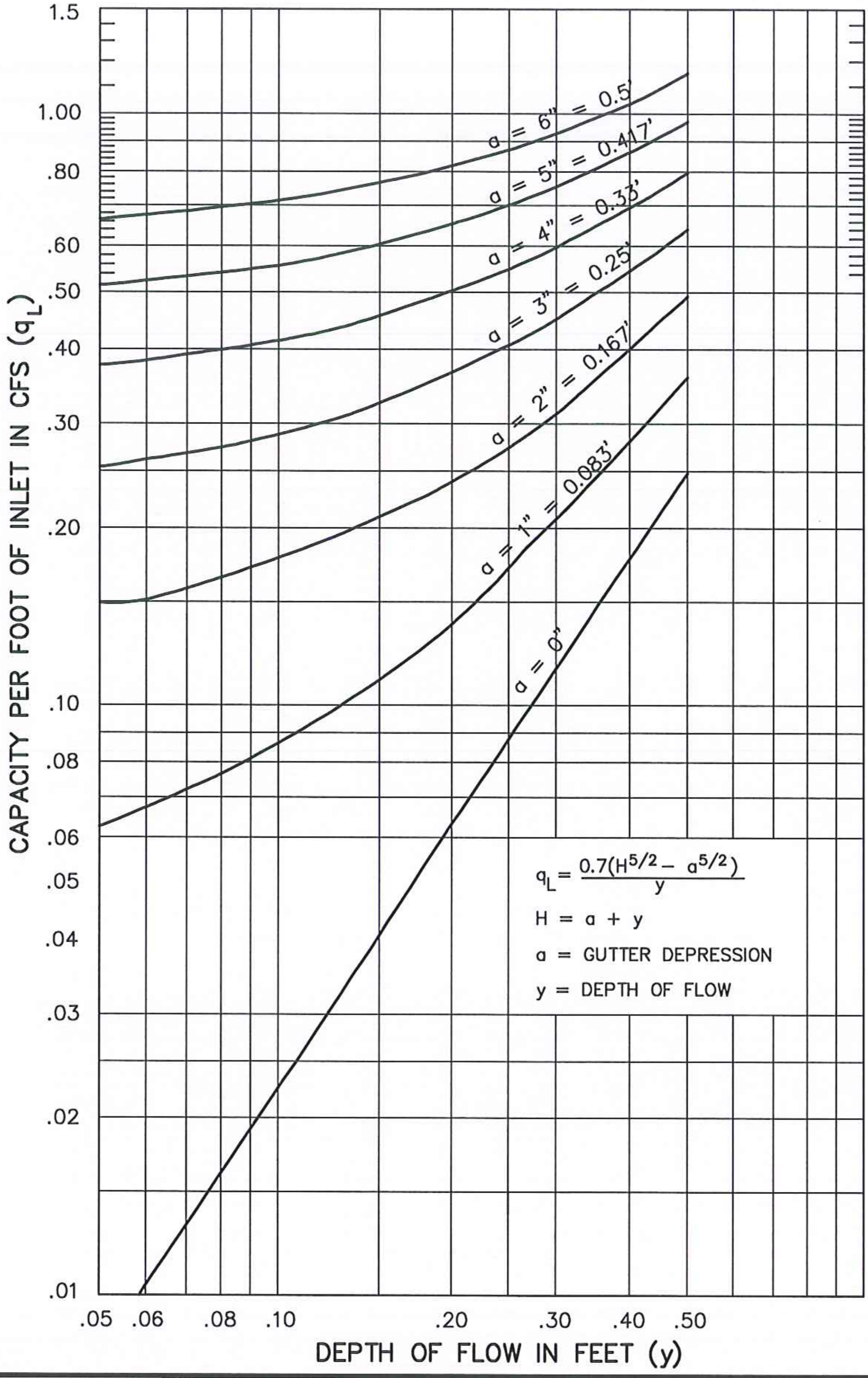

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FIGURE 8

CAPACITY OF CURB INLETS ON GRADE

Town of ADDISON
DRAINAGE CRITERIA MANUAL

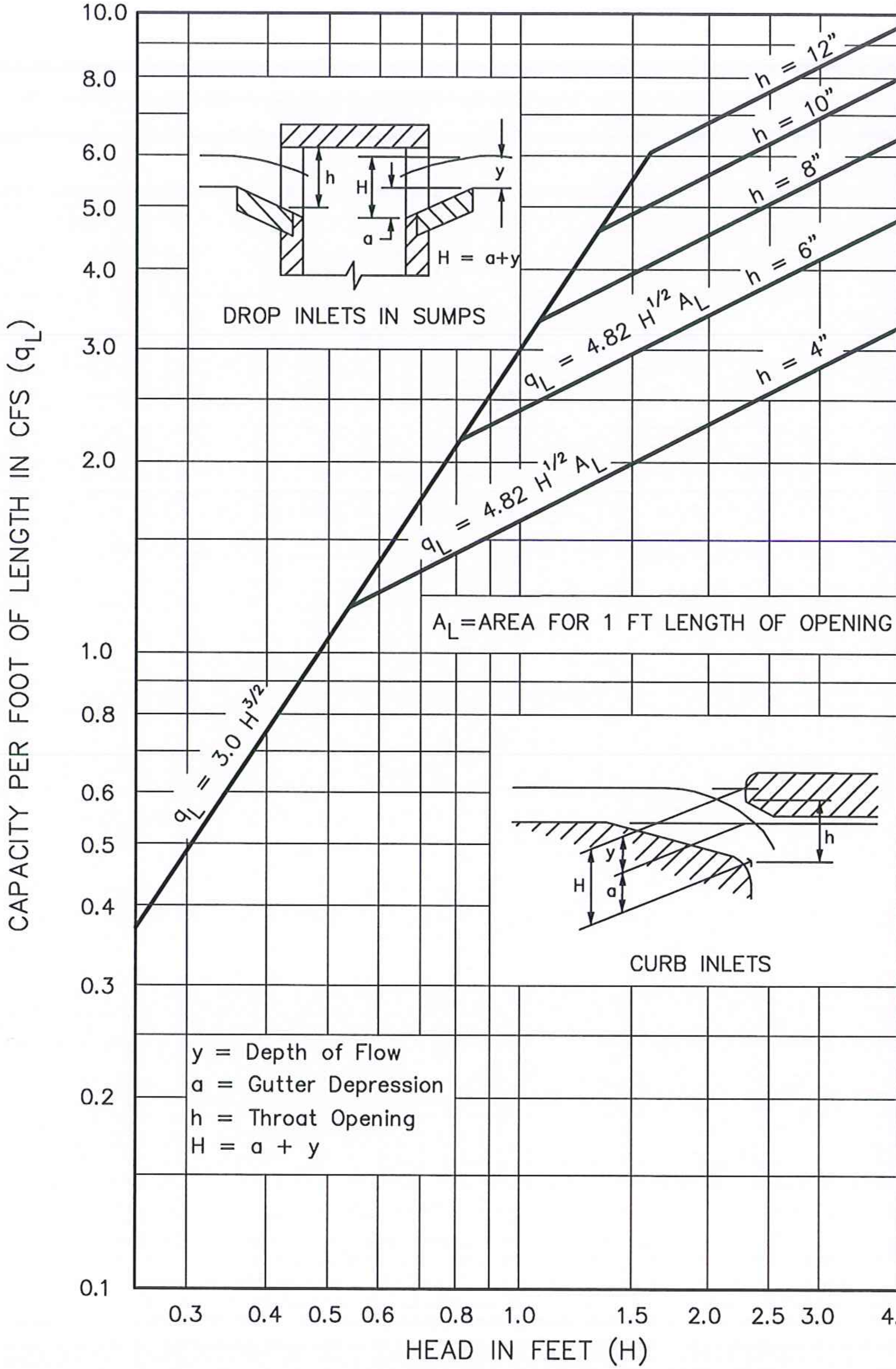

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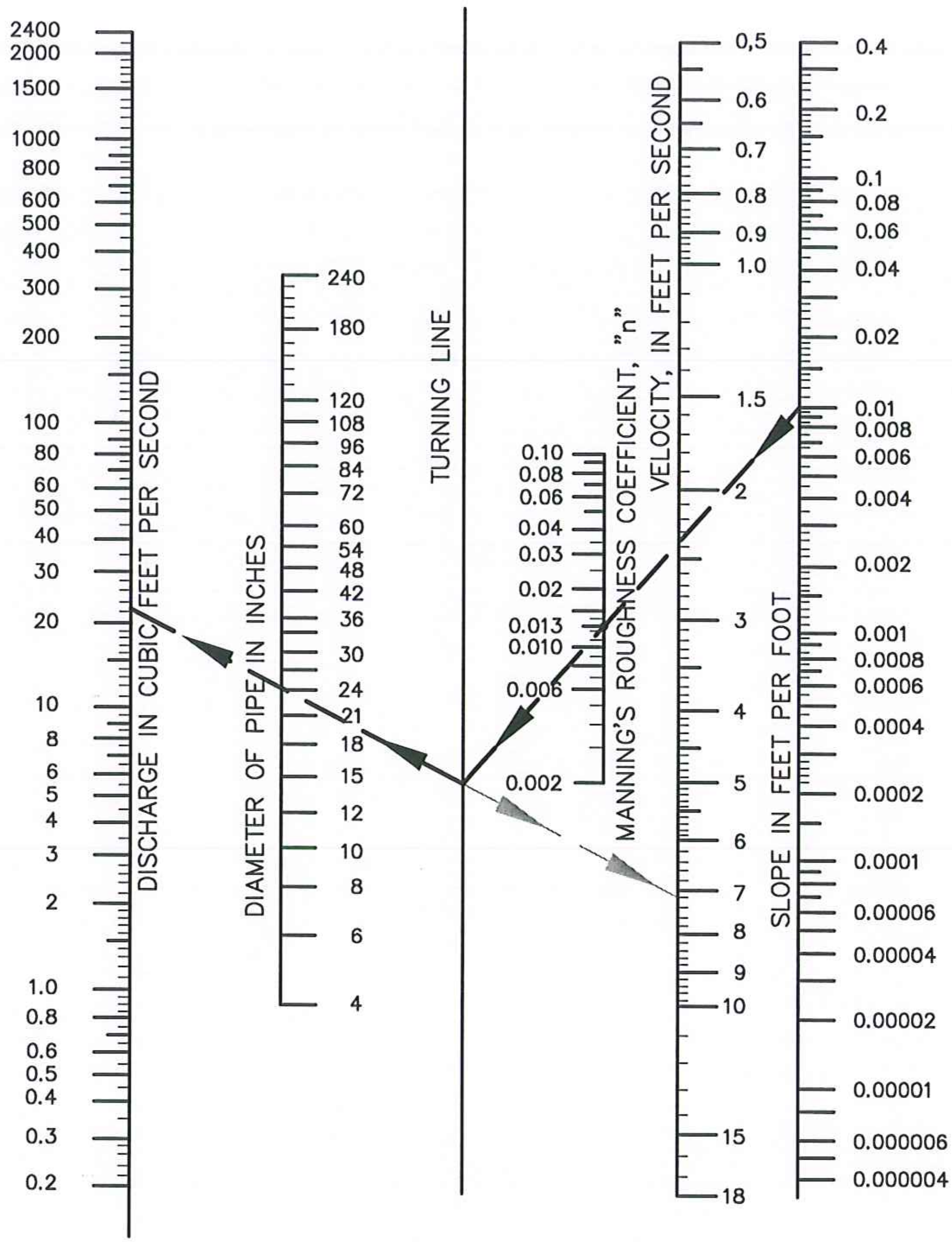
FIGURE 9

CAPACITY OF DROP INLETS AND CURB INLETS IN SUMPS

Town of ADDISON
DRAINAGE CRITERIA MANUAL



EXAMPLE

KNOWN:
"n" = 0.013
Slope(s) = 1.0%
Pipe Diameter = 24"

FIND:
Capacity (Q)
Velocity (V)

SOLUTION:
Connect Slope thru "n" Turning Line,
Connect to Pipe Size
Q=22.6 V=7.2

NOTE:
Solutions are for pipes flowing full depth.

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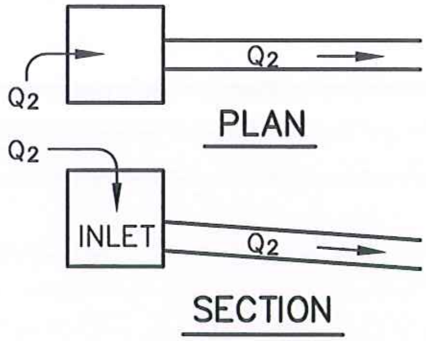
FIGURE 10

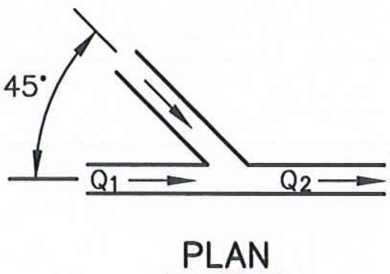
MANNING'S FORMULA FOR FLOW IN STORM SEWERS

Town of ADDISON
DRAINAGE CRITERIA MANUAL



$$H_j = 1.25 \frac{V_2^2}{2g}$$

INLET AT BEGINNING OF LINE

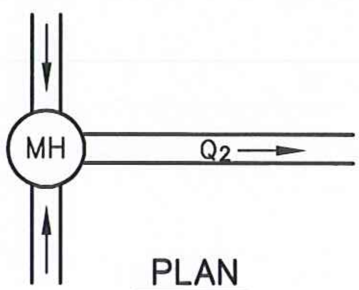


$$H_j = \frac{V_2^2}{2g} - \frac{K\,V_1^2}{2g}$$

$$H_j = \frac{V_2^2}{2g} - \frac{0.75\,V_1^2}{2g}$$

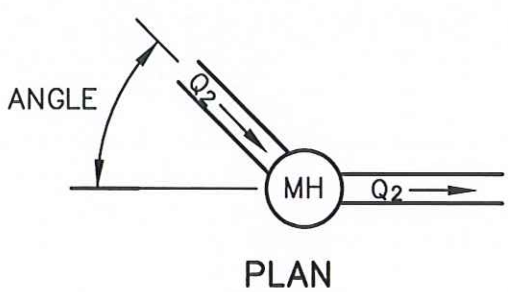
FOR 60°, USE K=0.43
(K=COEFFICIENT OF HEAD LOSS)

45° WYE CONNECTION



$$H_j = \frac{V_2^2}{2g}$$

INCOMING OPPOSING FLOWS AT MANHOLE



ANGLE	K
0	0.15
15	0.19
30	0.35
45	0.47
60	0.56
75	0.64
90	0.70

$$H_j = K \frac{V_2^2}{2g}$$

CHANGE IN DIRECTION AT MANHOLE

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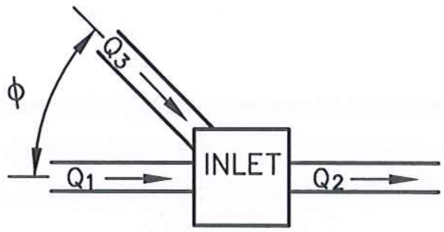
FIGURE 11

MINOR HEAD LOSSES
(1 OF 2)

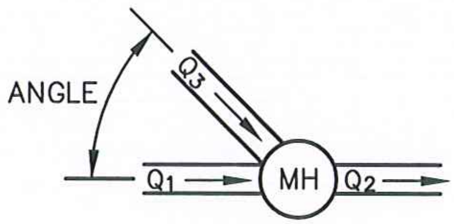
Town of ADDISON
DRAINAGE CRITERIA MANUAL



PLAN

$$H_j = \frac{Q_2 V_2^2 - Q_1 V_1^2 - \cos\phi \, Q_3 V_3^2}{2 g Q_2}$$

INLET WITH MULTIPLE ENTERING FLOWS

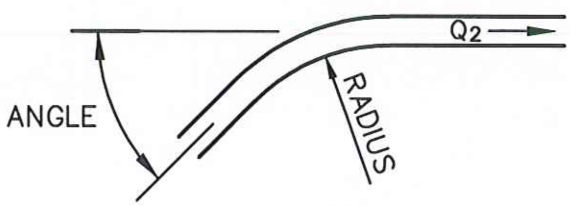


PLAN

ANGLE	K
15	0.81
30	0.65
45	0.53
60	0.44
75	0.36
90	0.30

$$H_j = \frac{V_2^2}{2g} - K\frac{V_1^2}{2g}$$

MANHOLE WITH LATERAL



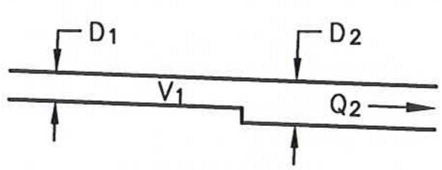
PLAN

K

ANGLE	RADIUS (# DIA's)		
	1	2–8	8–20
90	0.50	0.45	0.40
60	0.43	0.38	0.34
45	0.35	0.32	0.28
22½	0.20	0.18	0.16

$$H_j = K\frac{V_2^2}{2g}$$

CONDUIT ON CURVE



SECTION

$\frac{D_2}{D_1}$	K
1.2	0.10
1.4	0.23
1.5	0.29
1.6	0.35
1.8	0.44
2.0	0.52
2.5	0.65

$$H_j = K\frac{V_1^2}{2g}$$

PIPE ENLARGEMENT

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FIGURE 11

MINOR HEAD LOSSES
(2 OF 2)

Town of ADDISON
DRAINAGE CRITERIA MANUAL

DRAINAGE AREA COMPUTATIONS

AREA	ACRES	'C'	T_c	I_{10}	Q_{10}	I_{100}	Q_{100}	COMMENTS

INLET COMPUTATIONS

DESIGN POINT NUMBER	INLET LOCATION	CONTRIB DRNG AREA(S)	RUNOFF COEF. "C"	SIZE (AC.)	Tc (MIN)	RAIN INTENSITY (IN/HR)	Q (CFS)	UPSTREAM BYPASS	STREET SLOPE (%)	STREET WIDTH (FT)	INLET DEPRESSION (FT)	INLET CAPACITY (CFS/FT)	INLET LENGTH (FT)	INLET CAPACITY (CFS)

HYDRAULIC COMPUTATIONS FOR STORM DRAINS

RUNOFF COLLECTION POINT (Inlet or Manhole)		Distance Between Collection Points	INCREMENTAL DRAINAGE AREA				Accum-ulated "CA"	Time at Upstream Station	Intensity "I" (in./hr.)	Storm Water Runoff "Q"	Slope of Hydraulic Gradient "Sf"	Number of Boxes or Pipes	Width of Box (ft.) or Pipe Diameter	Height of Box (ft.)	Velocity In Sewer Between Collection Points	Velocity Head at Upstream Station	Flow Time in Sewer	Time at Downstream Station	Hydraulic Grade Line Elevation Downstream	Hydraulic Grade Line Elevation Upstream	REMARKS
UPSTREAM STATION	DOWNSTREAM STATION		Area No.	Drainage Area "A" (Acres)	Runoff Coeff. "C"	Incremental "CA"		(minutes)		(c.f.s.)	(ft./ft.)		(inches)		"V" (f.p.s.)	(feet)	(minutes)	(minutes)	(elev)	(elev)	

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FIGURE 12

HYDRAULIC COMPUTATIONS TABLES

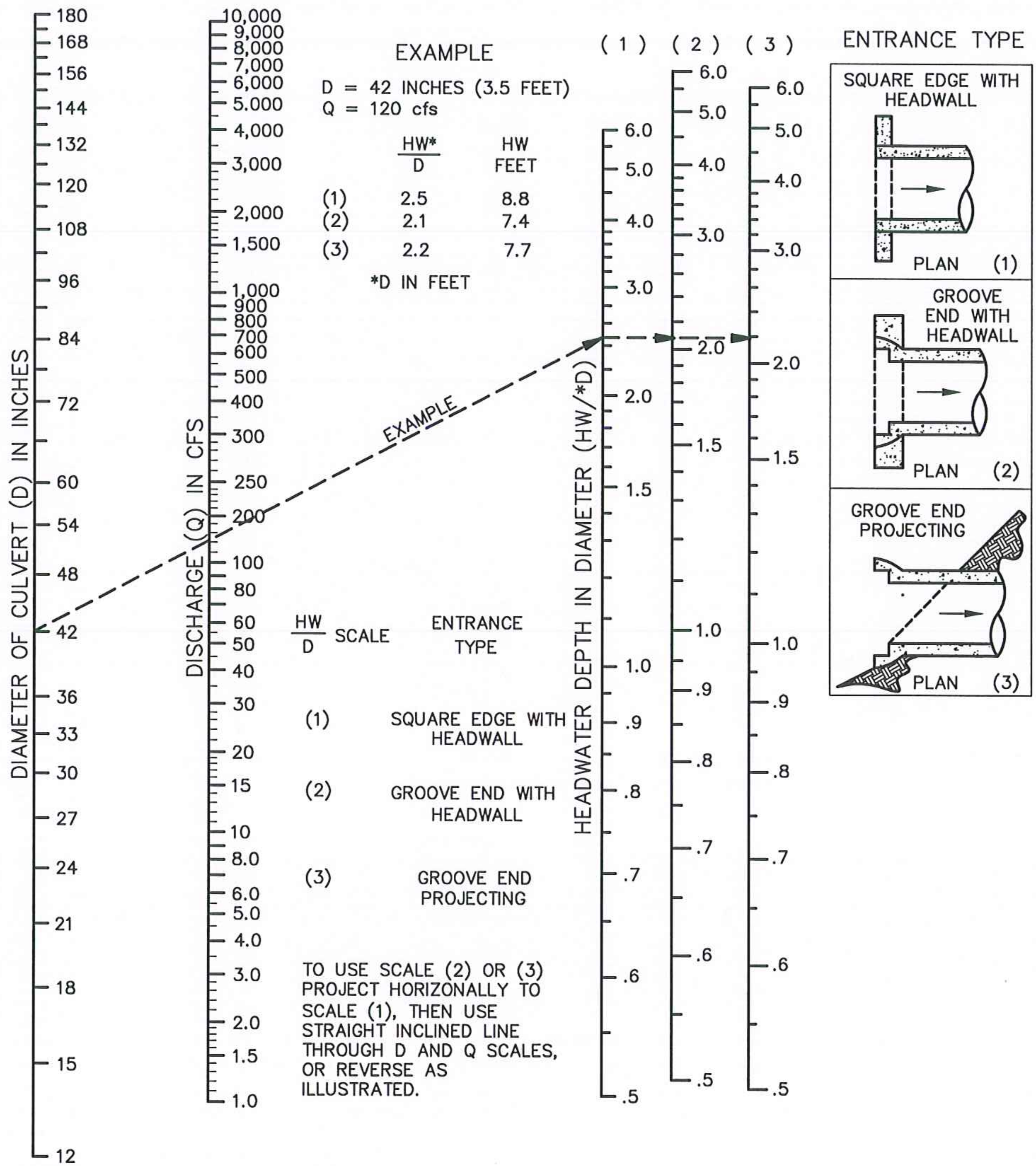

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FIGURE 13

HEADWATER DEPTH FOR PIPE CULVERTS WITH INLET CONTROL

Town of ADDISON
DRAINAGE CRITERIA MANUAL

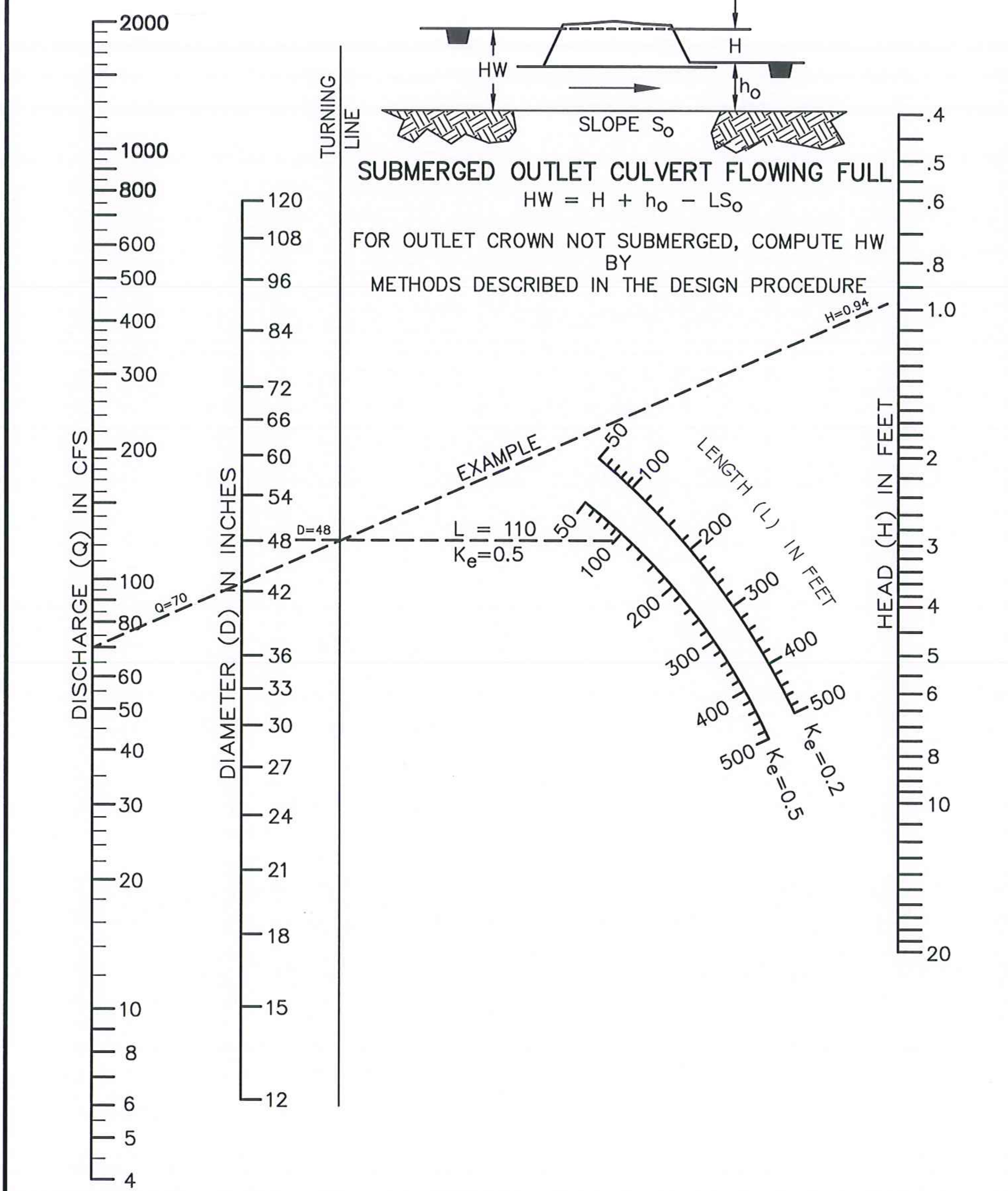

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FIGURE 14

HEADWATER DEPTH FOR PIPE CULVERTS FLOWING FULL WITH OUTLET CONTROL

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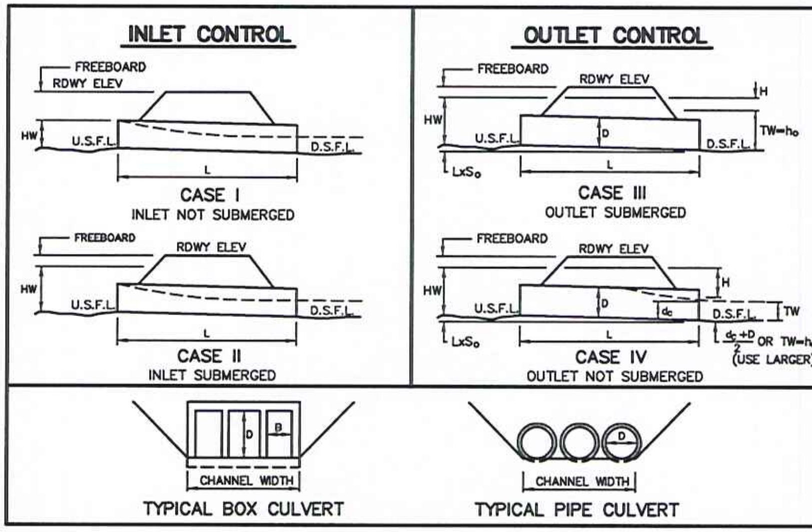

CULVERT ENTRANCE DATA

CONCRETE BOX CULVERT (WINGWALL FLARE ANGLE, ENTRANCE EDGE)

TYPE	FLARE ANGLE	ENTRANCE EDGE	K_e
1A	30° TO 75°	SQUARE	0.4
1B	30° TO 75°	ROUND	0.3
2A	15° TO 30° & 75° TO 90°	SQUARE	0.5
2B	15° TO 30° & 75° TO 90°	ROUND	0.3
3A	0° (EXTENSION OF SIDES)	SQUARE	0.7
3B	0° (EXTENSION OF SIDES)	ROUND	0.5

CONCRETE PIPE OR

TYPE	ENTRANCE DESCRIPTION	K_e
4	SPIGOT END WITH HEADWALL	0.5
5	BELL END WITH HEADWALL	0.2
6A	BELL END PROJECTING WITH NO HEADWALL	0.3
6B	SPIGOT END PROJECTING WITH NO HEADWALL	0.6

CULVERT DESIGN CALCULATIONS
TOWN OF ADDISON, TEXAS

BY ______________ DATE ______________

PROJECT ______________ FILE NO. ______________

CULVERT LOCATION ______________

______________ LENGTH, L ______________

TOTAL DISCHARGE, Q ______________ DESIGN STORM FREQ. ______________

ROUGHNESS COEFF., n ______________ MAX. VEL. ______________

TAILWATER ______________ D.S. CHANNEL WIDTH ______________

ENTRANCE DESCRIPTION ______________

RDWY. ELEV. ______________ U.S. CULV. F.L. ______________

U.S. CULV. F.L. ______________ D.S. CULV. F.L. ______________

DIFFERENCE ______________ DIFFERENCE ______________

REQ'D FREEBOARD ________ FT. CULV. SLOPE, $S_o = \frac{\text{DIFF. FT.}}{\text{LENGTH FT.}}$

ALLOW. HEADWATER ________ FT. S_o= ________

TRIAL CULVERT										HEADWATER CALCULATION																	THE GREATER CONTROLLING HEAD WATER (INLET OR OUTLET) (feet)	SELECTED CONDUIT SIZE
		DEPTH RANGE D.R.			POSSIBLE CULVERT SIZES					INLET CONTROL					OUTLET CONTROL													
																CASE III HW = H + TW − L x S_o (feet)				CASE IV HW = H + h_o − L x S_o (feet)	$h_o=\frac{d_c+D}{2}$ OR h_o=TW (USE LARGER)							
TRIAL AREA OF OPENING $A=\frac{Q}{V_{max}}$ (sq.ft.)	CHANNEL WIDTH "W" (feet)	$\frac{A}{W}$ (feet)	AHW (feet)	TRY DEPTH "D" (feet)	NO. OPENINGS	WIDTH OF BOX "B" (feet)	BOX DEPTH OR PIPE DIA. "D" (feet)	TOTAL CULVERT AREA "A" (sq.ft.)	"Q" EACH OPENING (c.f.s.)	ENTRANCE TYPE	CASE NO.	$\frac{Q}{B}$ (c.f.s.)	$\frac{HW}{D}$	HW	ENTRANCE COEFF. K_e	"H" (feet)	"TW" (feet)	L x S_o (feet)	"HW" (feet)	"H" (feet)	d_c (feet)	$\frac{d_c+D}{2}$ (feet)	TW (feet)	h_o (feet)	L x S_o (feet)	"HW" (feet)		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29

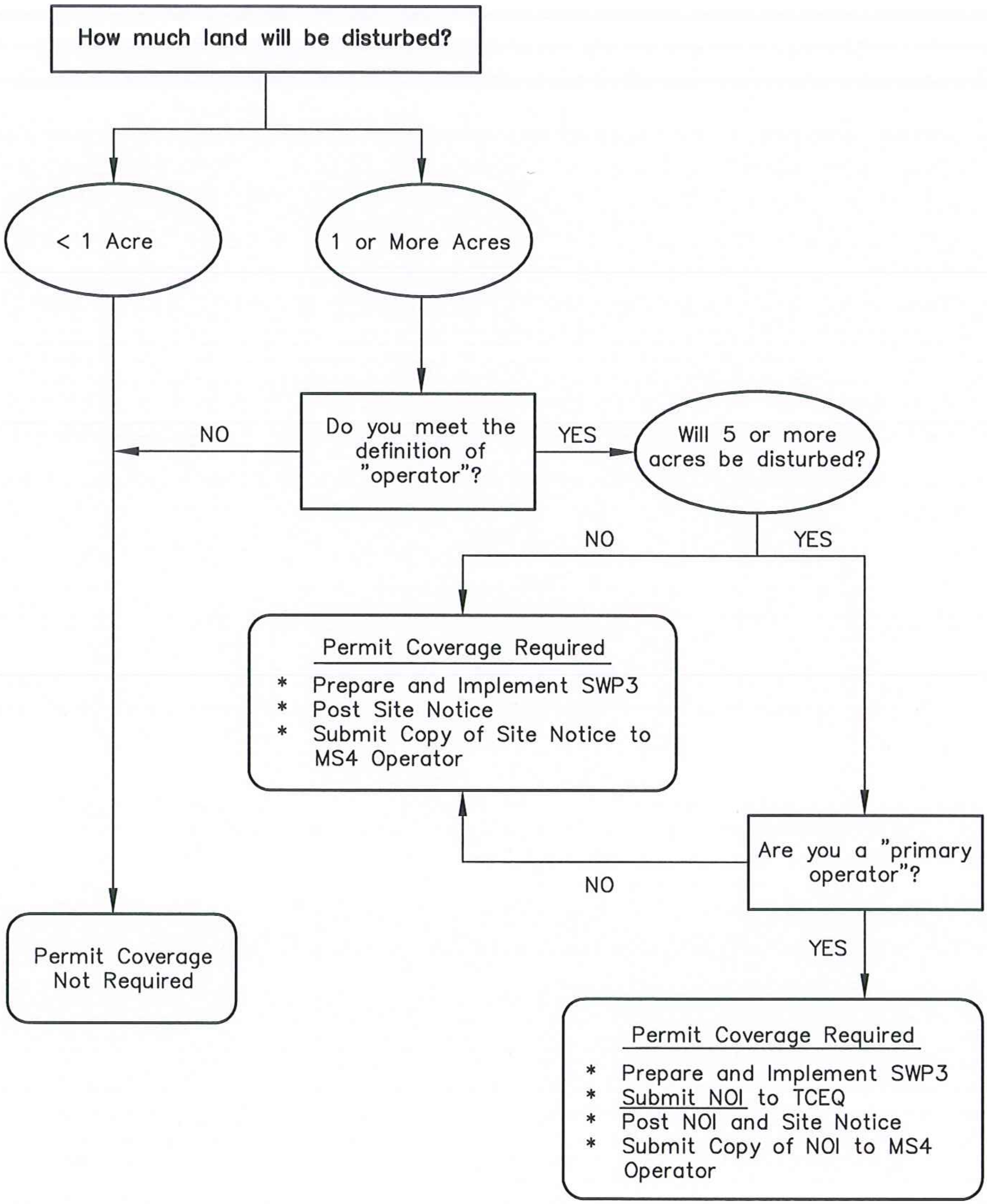

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FIGURE 16

TCEQ CONSTRUCTION PERMIT FLOW CHART

Town of ADDISON

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