CITY
OF LONGVIEW
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DRAINAGE CRITERIA AND EROSION CONTROL MANUAL

Prepared By:
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Public Works Engineer

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Utilities Engineer

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1.0 INTRODUCTION

1.1 PURPOSE

The purpose of this drainage manual is to supplement the City of Longview Master Drainage Plan with established drainage design procedures and standards for drainage areas of less than 100 acres. The Master Drainage Plan addresses the complexities and maintenance of areas draining more than 100 acres and should be referred to where applicable. In areas with contributing drainage basins greater than 100 acres and with a FEMA designated flood zone, all construction activity is subject to the City of Longview Flood Hazard Ordinance.

Methods of design other than those indicated herein may be considered in cases where experience clearly indicates they are preferable. However, there shall be no variations from the practices established herein without the approval of the Office of the City Engineer.

2.0 GENERAL DRAINAGE POLICY

The following policies apply to all drainage design and construction carried out within the jurisdiction of the City of Longview:

1. Acceptance of the design (e.g., size, type, and location) of all storm drainage facilities in the City of Longview is subject to the approval of the Office of the City Engineer.

2. The criteria/guidelines presented in this manual are minimum requirements.

3. The developer and his engineer shall bear responsibility for the adequacy of design. City of Longview approval of a given drainage facility in no way relieves the developer and his engineer of their responsibility.

4. Prior to the design of any channel improvement, storm sewer, stormwater detention facility, or other drainage feature, the Office of the City Engineer shall be consulted regarding preferred flood control strategies for the watershed(s) within which the alterations are to occur.

5. All drainage structures or improvements in the City of Longview shall be designed to properly accommodate the runoff from a storm event with a mean recurrence interval of 100-years. Watershed development conditions used for drainage design purposes shall reflect ultimate (future) conditions except that the minimum development condition will reflect SF-4 Zoning Conditions (5 units per acre) that are equivalent to a ration method C-factor of 0.70.

6. An approved site development permit and plan are required prior to clearing for the purpose of construction on any lot or development.

7. Plans for sedimentation and erosion control shall be submitted as specified in Section 3.3.

8. If downstream drainage improvements as specified in the Longview Master Drainage Plan have been built or approved and funded, stormwater detention will be necessary when proposed development conditions require a rational method C-factor in excess of 0.70. Detention facilities must be designed so as to reduce the 5-year, 25-year, and 100-year peak flow rates to the level of watershed development equivalent to a C-factor of 0.70. If downstream drainage improvements as specified in the Longview Master Drainage Plan have not been built or approved and funded, detention must be provided to assure that the proposed conditions peak flow rates will not exceed the <u>existing</u> conditions peak flow rates for the 5-year, 25-year, and 100-year storm events.

2.1 CRITERIA FOR CONSTRUCTION IN A FLOODPLAIN

An Area of Special Flood Hazard is defined as the land in the floodplain within a community subject to a one percent or greater chance of flooding in any given year. This area has been identified by the Federal Emergency Management Agency (FEMA) on its Flood Hazard Boundary Map, Community No. 480264, dated January 17, 1990 (revised). However, most of the floodplain information contained in this FEMA report is considerably dated (circa 1970) and shall be used with caution. Flood elevation determinations have been made in conjunction with the City of Longview Master Drainage Study (1990) and can be obtained from the Office of the City Engineer.

All construction or construction-related activity which is proposed to take place in a special flood hazard area shall be subject to the following specifications, and shall also conform to all requirements of the City of Longview Flood Hazard Ordinances (Nos. 1882 and 1902):

1. There shall be no increase in the elevation of flow on any property upstream, downstream, or on the opposite bank from the proposed site caused by construction activity in the floodplain. The floodplain is defined as the special flood hazard area as defined above. The property owner/developer shall be required to provide a technically acceptable analysis (such as a backwater analysis) that this restriction has not been or will not be violated.

2. Any increase in mean stream flow velocity shall be limited so as not to exceed the open channel velocity limitations delineated in Table 5-1 of this manual. In addition, there shall be no increase in erosional potential or activity on any property upstream, downstream, or on the opposite bank from the proposed site caused by construction activity in the floodplain. The owner/developer shall be required to provide a technically acceptable analysis (such as a backwater and soils analysis) that this restriction has not been violated.

2.2 REQUIRED TECHNICAL INFORMATION TO BE SUBMITTED FOR CITY REVIEW

All plans will be prepared under the supervision of a registered professional engineer licensed to practice in the State of Texas. The engineer shall affix his seal and signature to each plan sheet and any reports or calculations submitted to support the plans.

Prior to any land disturbance activity associated with proposed changes to the nature or character of a parcel of land, the engineer/developer shall be required to submit for City review appropriate hydraulic and hydrologic design calculations and technical information. This includes at a minimum:

- **HYDROLOGIC DESIGN**

1. A topographic drainage area map with all drainage areas and flow rate calculation points delineated.

2. A completed copy of Table 4-3, "Design Flow Calculations Summary Table," for all flow rate calculation points described in Item 2.

- **OPEN CHANNEL DESIGN**

1. A topographic map of the subject drainage basin showing proposed drainage areas and proposed channel locations.

2. Design flow rate calculations carried out as described in Section 4.0 of this manual. See Table 4-2.

3. For design using the Manning Equation, a listing of the following parameters:
   - flow depth (ft)
   - channel slope (ft/ft)
   - channel flow (cfs)
   - Manning's "n" value
   - channel sideslopes (H:V)
   - channel bottom width (ft)

4. For design using HEC-2:
   - output listing of computer run

5. Existing and proposed typical channel cross-sections with their locations delineated on the drainage map.

- **CULVERT DESIGN**

1. Calculations used to determine the design discharge for the culvert(s).

2. A summary table delineating design flow rate, culvert length, culvert slope, allowable headwater depth, tailwater depth, selected culvert type and size, flow velocity at culvert outlet, and designation of inlet or outlet control. See standard reporting sheet - Figure 7-1.

- **STORM SEWER DESIGN**

1. A contour and drainage area map showing the proposed development, including storm sewer locations, inlet locations and drainage areas contributing to each inlet.

2. A table summarizing drainage area, time of concentration, assumed C value, design discharge (inlet intake), and size and number of inlets for each inlet(s) location and its contributing drainage area.

3. If storm sewer hydraulic design is carried out by hand, calculations of HGL profile including pipe friction and junction losses. If storm sewer design is carried out with a computer pipe model, a listing of the model output. Storm sewer plans submitted to the City Engineer for review shall have the HGL plotted on the profile.

- **STORMWATER STORAGE FACILITY DESIGN**

1. Rational method calculations for the 5-, 25- and 100-year pre-development and post-development condition flows.

2. The HEC-1 output utilized for final design.

3. Calculations used to determine outflow structure rating curves.

- **LAND DISTURBANCE ACTIVITY**

1. A sequence of development showing which phases of construction will be done at which time, and what specific controls are required during each phase of the development. In all cases, erosion controls must be in place prior to the start of any land disturbance activity.

2. A schematic representation of each control strategy required in the engineer's plan, with adequate dimensions and references to the specific detail in the standards contained in the City of Longview Standard Specifications Manual so that feature can be built as intended.

3. Approved areas for construction traffic, parking, vehicular maintenance, and, if appropriate, vehicle washing.

4. Temporary spoils storage areas, including size, time of use, and ultimate restoration schedules.

5. Permanent spoils disposal areas, including size, depth of fill, and restoration procedures.

6. Contour maps showing lightly dashed lines for existing contours and solid lines for proposed contours, with each having a contour interval of two feet.

7. Restoration plans for all disturbed areas on the site that will include, as a minimum:

   (a) seed type and rate of application,
   (b) mulch type and rate of application,
   (c) application technique,
   (d) sod type,
   (e) maintenance requirements for each specific area,
   (f) contingency plan, if delays in construction upset the proposed timetable,
   (g) whether the restoration is of a permanent or temporary nature,
   (h) landscaping plan using a mixture of grasses, forbes and woody plants.
   (i) method for stockpiling and reuse of topsoil excavated from site.

8. Specific locations where special slope stabilization techniques are to be utilized and the extent of the slope stabilization to take place.

9. The identify of the person or firm who will be responsible for the maintenance of each sedimentation/erosion control scheme being used on the project.

10. A clear definition of criteria to be utilized in determining when acceptable restoration has taken place.

11. The location of any detention ponds on the construction site.

12. The location of any sedimentation or filtration ponds to be constructed on the site.

13. The name and phone number of the designated representative for the owner/engineer who will have the authority to make appropriate changes to the sedimentation/erosion plan, it if is discovered to be inadequate.

14. The length of time between start of rough-cutting and complete restoration may not exceed 12 months. If an applicant does not meet this deadline, the Office of the City Engineer will notify the applicant in writing that the City may complete the stabilization of disturbed areas at the applicant's expense, unless the applicant does so within 30 days of the day of notice, or unless the applicant provides acceptable erosion/sedimentation controls and the continuing maintenance thereof, acceptable to the Office of the City Engineer.

15. The owner/engineer shall conduct ongoing inspections of all erosion/sedimentation control methodologies and direct the person or firm responsible for the maintenance to make any repairs or modifications necessary to assure continued effective operation of each methodology.

16. No clearing or rough-cutting shall be permitted prior to final approval of the plan.

17. No clearing or rough-cutting for purposes other than those set out above, or for construction of temporary erosion and sedimentation controls, as per approved plan, shall be permitted until those controls are in place.

- **CONSTRUCTION IN A SPECIAL FLOOD HAZARD ZONE**

1. Hydraulic calculations and/or computer runs proving no increase of flood elevation greater than 1.0 foot resulting from construction activity in the "floodplain" and no increase of flood elevation whatsoever resulting from construction activity in the "floodway".

2. Landscaping plan specifying plans for erosion control on cut and fill slopes and restoration of excavated areas.

The Office of the City Engineer may require additional technical backup information.

2.3 TOOLS AVAILABLE FOR COMPLIANCE WITH CITY OF LONGVIEW DRAINAGE DESIGN REQUIREMENTS

Provided with this manual is a floppy disk containing several simple hydraulic routines. These easy-to-use, interactive programs will serve to greatly facilitate the engineering design process. Each is referenced in the appropriate sections of this manual and the design engineer is encouraged to utilize them.

The hydraulics library can be invoked by typing HYDROLIB. The user will then see a menu from which to select the appropriate routine. The menu will appear as follows:

1. Slope of hydraulic grade line
2. Junction and bend losses
3. Normal depth in non-circular channel
4. Normal depth in circular channel

5. Detention facility design
6. Exit

Routine #1, "Slope of Hydraulic Grade Line", will calculate the friction slope of the HGL assuming pressure flow conditions in a pipe conduit (storm sewer). Its technical basis and appropriate application are referenced and explained in Section 6.0, "Inlets and Storm Sewers".

Routine #2, "Junction and Bend Losses", utilizes an equation developed by the City of Los Angeles, Bureau of Engineering, Storm Drain Design Division (1968) to calculate head losses at junctions and bends in a storm sewer system. The technical basis for this methodology and its appropriate application are discussed in Section 6.6 (Step 5) of this manual.

Routine #3, "Normal Depth in Non-Circular Channel", carries out an iterative solution of Manning's Equation for uniform flow to determine depth, velocity, Froude Number, velocity head and specific energy in a trapezoidal channel flowing at normal depth (uniform flow). This routine is referenced and recommended for use in several sections of this manual.

Routine #4, "Normal Depth in Circular Channel", carries out an iterative solution of Manning's Equation for uniform flow as described above, except that the channel is a circular pipe conduit. This routine is referenced and recommended for use in several sections of this manual.

Routine #5, "Detention Facility Design", incorporates the U.S. Corps of Engineers hydrologic analysis program, HEC-1, to perform a simple hydrograph determination in a small subarea and to route the hydrograph through a detention facility of known configuration. This easy-to-use, interactive application of HEC-1 will insure proper design of stormwater detention facilities.

3.0 SEDIMENTATION AND EROSION

One of the most significant impacts associated with urbanization of undeveloped areas is an unhealthy acceleration of the natural processes of erosion and sedimentation. This impact is experienced primarily for two reasons. First, increased impervious cover and drainage efficiency in developed areas yield increased peak runoff flow rates and more frequent erosive velocities in natural channels. Second, earth disturbance associated with construction activity typically exposes unprotected soils to washoff and deposition in receiving waters and impoundments.

This section will present City of Longview sedimentation and erosion protection requirements for both temporary and permanent erosion control applications.

3.1 TEMPORARY EROSION PROTECTION - LAND DISTURBANCE/ CONSTRUCTION SITES

Erosion and sedimentation prevention at excavation and/or embankment sites calls for temporary measures to be implemented during the project duration and until the disturbed area is adequately stabilized. There are a great many acceptable approaches available for providing adequate protection. This subsection will describe those methods recommended by the City of Longview. Use of other strategies is acceptable pending approval from the Office of the City Engineer. Design criteria and standard specifications for implementation of erosion and sedimentation controls is presented in the City of Longview publication, "Standard Specifications Manual." **In all cases, erosion control measures must be in place prior to the start of any land or structure disturbance activity.**

It is suggested the engineer consult SCS publication, "Erosion and Sediment Control Guidelines for Developing Areas in Texas," (SCS, 1976) for guidance with respect to proper procedures.

3.1.1 General Design Considerations for Land Disturbance Sites

Adherence to the following general design considerations will serve to minimize erosion and sedimentation impacts at land disturbance/construction sites.

- Limit the size of disturbed areas to the greatest extent possible
- Stabilize disturbed areas as soon as possible
- Minimize runoff velocities
- Protect steep areas and disturbed areas from upstream runoff
- Capture sediment at the project site
- Promote sheet flow as opposed to channelized or gulley flow to the extent possible
- Preserve and protect existing vegetation to the extent possible

3.1.2 Listing of Acceptable Strategies

3.1.2.1 Diversion Dike

A diversion dike is a temporary ridge of compacted soil immediately above cut or fill slopes and constructed with sufficient grade to provide drainage. The purpose of a diversion dike is to intercept storm runoff from small upland areas and divert it from exposed slopes to an acceptable outlet. The diversion dike is used for the period of construction at the top of newly constructed slopes to prevent excessive erosion until permanent drainage features are installed and/or slopes are stabilized.

3.1.2.2 Interceptor Dike

An interceptor dike is a temporary ridge of compacted soil, located across disturbed areas or rights-of-way. The purpose of an interceptor dike is to shorten the length of exposed slopes, thereby reducing the potential for erosion, by intercepting storm runoff and diverting it to a stabilized outlet or sediment trapping device.

Interceptor dikes are constructed across disturbed rights-of-way such as for utility lines and streets or disturbed areas such as graded parking lots or landfills. The dikes should remain in place until the disturbed areas are permanently stabilized.

3.1.2.3 Perimeter Dike

A perimeter dike is a temporary ridge of compacted soil located along the perimeter of the site or disturbed areas. The purpose of a perimeter dike is to prevent offsite storm runoff from entering the disturbed area and to prevent sediment-laden storm runoff from leaving the construction site or disturbed area.

The perimeter dike is used for the period of construction at the perimeter of the disturbed area to transport sediment laden water to a sediment trapping device such as a sediment trap or sediment basin. This dike shall remain in place until the disturbed area is permanently stabilized. The storm runoff prevented from entering the disturbed area by the perimeter dike shall be adequately handled to prevent damage due to flooding or erosion to adjacent property.

3.1.2.4 Hay Bale Dike

A hay bale dike is a temporary barrier constructed with hay bales with a life expectancy of 3 months or less, installed across or at the toe of a slope. The purpose of a hay bale dike is to intercept and detain small amounts of sediment from unprotected areas of limited extent.

The hay bale dike is used where:

1. No other practice is feasible, and
2. There is no concentration of water in a channel or other drainageway above the barrier, and
3. Erosion would occur in the form of sheet and rill erosion, and
4. Contributing drainage area is less than one-half acre and the length of slope above the dike is less than 100 feet. The practice may also be used for a lone,

single-family lot if the slope is less than 15 percent. The contributing drainage area in this instance shall be less than 1 acre and the length of slope above the dike shall be less than 200 feet.

3.1.2.5 Rock Berm

A rock berm is a temporary berm constructed of open graded rock installed at the toe of a slope, the perimeter of a developing area, or across a small drainage channel. The purpose of a rock berm is to intercept sediment-laden water from unprotected areas, detain the sediment and release the water (in sheet flow for perimeter applications).

The rock berm is used where:

1. There is an adequate source of rock on or near the site, and
2. The contributing drainage area is less than 5 acres.

3.1.2.6 Silt Fence

A silt fence is a temporary barrier fence made of burlap or polypropylene material which is water permeable but will trap water-born sediment. The purpose of a silt fence is to intercept and detain water-borne sediment from unprotected areas of limited extent.

Silt fence is used during the period of construction near the perimeter of a disturbed area to intercept sediment while allowing water to percolate through. This fence shall remain in place until the disturbed area is permanently stabilized. Silt fence should not be used where there is a concentration of water in a channel or other drainageway.

3.1.2.7 Interceptor Swale

An interceptor swale is a temporary excavated drainageway located across disturbed areas or rights-of-way. The purpose of an interceptor swale is to shorten the length of exposed slopes, thereby reducing the potential for erosion, by intercepting storm runoff and diverting it to a stabilized outlet or sediment trapping device.

Interceptor swales are constructed across disturbed rights-of-way, such as for pipe lines and streets or disturbed areas such as graded parking lots or land fills. The swale shall remain in place until the disturbed areas are permanently stabilized.

3.1.2.8 Perimeter Swale

A perimeter swale is a temporary excavated drainageway located along the perimeter of the site or disturbed areas. The purpose of a perimeter swale is to prevent offsite storm runoff from entering the disturbed area and to prevent sediment-laden storm runoff from leaving the construction site or disturbed area.

The perimeter swale is used for the period of construction at the perimeter of the disturbed area to transport sediment-laden water to a sediment trapping device such as a sediment trap or sediment basin. This swale shall remain in place until the disturbed area is permanently stabilized. The perimeter swale also is used to prevent storm runoff from entering the disturbed area. This runoff shall be adequately handled to prevent damage due to flooding or erosion to adjacent property.

3.1.2.9 Sediment Basin

A sediment basin is a temporary barrier or dam constructed across a waterway or at other suitable locations to intercept sediment-laden runoff and to trap and retain the sediment. This standard applies to the installation of temporary sediment basins on sites where: (1) failure of the structure would not result in loss of life, damage to homes or buildings, or interruption of use or service of public roads or utilities; (2) the drainage area does not exceed 100 acres, and (3) the basin is to be removed within 36 months after the beginning of construction of the basin.

The purpose of a sediment basin is to intercept sediment-laden runoff and reduce the amount of sediment leaving the disturbed area in order to protect drainage ways, properties, and rights-of-way below the sediment basin from siltation.

A sediment basin applies where physical site conditions or land ownership restrictions preclude the installation of erosion control measures to adequately control runoff, erosion, and sedimentation. It may be used below construction operations which expose critical areas to soil erosion. It remains in effect until the disturbed area is protected against erosion by permanent stabilization.

3.1.2.10 Sediment Trap

A sediment trap is a small temporary basin formed by excavation and/or an embankment to intercept sediment-laden runoff and to trap and retain the sediment. The purpose of a sediment trap is to protect drainageways, properties and rights-of-way below the sediment trap from sedimentation.

A sediment trap is usually installed in a drainageway, at a storm drain inlet, or at other points of discharge from a disturbed area.

3.2 PERMANENT EROSION PROTECTION - CHANNELS AND OTHER DRAINAGEWAYS

In areas of potential or existing erosion problems along drainageways, acceptable erosion control methods must be employed. This subsection will outline permanent erosion control methodologies recommended for use in the City of Longview, including design and construction specifications.

3.2.1 Grass/Vegetation

Grassed channels are preferred in areas and drainageways with sufficient drainage right-of-way, moderate flow velocities and suitable soils for the establishment of vegetative cover. Maximum sideslopes shall usually be 3:1 unless a formal geotechnical investigation determines that unreinforced slopes may be steeper. Channel slopes must be revegetated immediately after construction to minimize erosion. Vegetation should be durable and require minimal maintenance. Revegetated ground cover in channels must be accepted by the City and the City may require topsoil placement if native soils are unsuitable. Acceptance shall not be granted until two years after channel construction completion.

If appropriate, a concrete or cellular grid paver pilot channel may be incorporated in a grassed channel. Pilot channels serve to limit erosion of the grassed channel by accommodating low to moderate flows.

Unless otherwise approved by the office of the City Engineer, Manning's "n" values for grassed channels shall not be less than 0.04 (0.035 in well-maintained condition).

3.2.2 Vegetation Stabilizers

Vegetation stabilizers refers to any of several blanket-type products used to protect newly grass-vegetated surfaces during the time necessary for the vegetation to become fully established. This includes jute, Enkamat, and Excellsior Mat, among others. These materials must be adequately staked to the channel sideslope.

3.2.3 Concrete Slope Paving

Concrete slope paving refers to the use of reinforced concrete along channel sideslopes, channel bottoms, at pipe/culvert outfalls and areas of high turbulence, to protect proposed grades against excessive erosion. Concrete-lined channels are often preferred in areas where available drainage right-of-way is limited.

Unless otherwise approved by the Office of the City Engineer, Manning's "n" values for concrete-lined channels shall not be less than 0.013.

3.2.4 Prepackaged Concrete Bagwall (R-Rap)

The term "R-Rap" refers to a patented prepackaged concrete riprap provided in biodegradable paper bags and stacked along channel sideslopes. R-RAP is typically reinforced on the sideslope with 6-inch, No. 3 rebar inserted vertically between adjacent bags and at the toe of the slope with a 4-foot, No. 6 rebar vertically traversing the bottom three levels of stacked bags. Once placed, individual bags are either left to be moistened via precipitation, or are hosed down to speed the curing process. The City does not promote the use of any one particular brand-name of product. The use of R-RAP by name is an example of something that has been used successfully in Longview. There are many other products that will be considered for use on an individual basis.

Unless otherwise approved by the Office fo the City Engineer, Manning's "n" values for concrete bagwall-lined channels shall not be less than 0.035.

3.2.5 Concrete Slope Pavers

The term "Concrete Slope Pavers" refers to cellular concrete revetment grids generally placed along a channel slope to prevent erosion. They are typically perforated to allow vegetative growth, infiltration and exfiltration of water, and some deflection against localized stresses.

Unless otherwise approved by the Office fo the City Engineer, Manning's "n" values for cellular grid-lined channels shall not be less than 0.04.

3.2.6 Stone Riprap

Stone riprap refers to a layer of loose rock or aggregate placed over an erodible soil surface. The purpose of riprap is to protect the soil surface from the erosive forces of water.

Riprap may be used, as appropriate, at such places as storm drain outlets, channel banks and/or bottoms, roadside ditches, drop structures and shorelines.

Unless otherwise approved by the Office fo the City Engineer, Manning's "n" values for stone riprap-lined channels shall not be less than 0.04.

3.2.7 Gabions

The term "gabions" refers to rectangular heavily-galvanized wire mesh baskets filled with graded rock. Gabion structures are generally built to be a homogeneous monolithic structure capable of providing erosion protection while absorbing unexpected or localized stresses without lessening the structure's integrity.

Unless otherwise approved by the Office fo the City Engineer, Manning's "n" values for gabion-lined channels shall not be less than 0.04.

3.3 MATERIALS TO BE PROVIDED FOR CITY REVIEW

As a minimum, the following items will be included on the construction plans prior to their approval by the Office of the City Engineer.

1. A sequence of development showing which phases of construction will be done at which time, and what specific controls are required during each phase of the development. In all cases, erosion controls must be in place prior to the start of any land disturbance activity.

2. A schematic representation of each control strategy required in the engineer's plan, with adequate dimensions and references to the specific detail in the standards contained in the City of Longview Standard Specifications Manual so that feature can be built as intended.

3. Approved areas for construction traffic, parking, vehicular maintenance, and, if appropriate, vehicle washing.

4. Temporary spoils storage areas, including size, time of use, and ultimate restoration schedules.

5. Permanent spoils disposal areas, including size, depth of fill, and restoration procedures.

6. Contour maps showing lightly dashed lines for existing contours and solid lines for proposed contours, with each having a contour interval of two feet.

7. Restoration plans for all disturbed areas on the site that will include, as a minimum:

   (a) seed type and rate of application,
   (b) mulch type and rate of application,
   (c) application technique,
   (d) sod type,
   (e) maintenance requirements for each specific area,
   (f) contingency plan, if delays in construction upset the proposed timetable,
   (g) whether the restoration is of a permanent or temporary nature,
   (h) landscaping plan using a mixture of grasses, forbes and woody plants.
   (i) method for stockpiling and reuse of topsoil excavated from site.

8. Specific locations where special slope stabilization techniques are to be utilized and the extent of the slope stabilization to take place.

9. The identify of the person or firm who will be responsible for the maintenance of each sedimentation/erosion control scheme being used on the project.

10. A clear definition of criteria to be utilized in determining when acceptable restoration has taken place.

11. The location of any detention ponds on the construction site.

12. The location of any sedimentation or filtration ponds to be constructed on the site.

13. The name and phone number of the designated representative for the owner/engineer who will have the authority to make appropriate changes to the sedimentation/erosion plan, it if is discovered to be inadequate.

14. The length of time between start of rough-cutting and complete restoration may not exceed 12 months. If an applicant does not meet this deadline, the Office of the City Engineer will notify the applicant in writing that the City may complete the stabilization of disturbed areas at the applicant's expense, unless the applicant does so within 30 days of the day of notice, or unless the applicant provides acceptable erosion/sedimentation controls and the continuing maintenance thereof, acceptable to the Office of the City Engineer.

15. The owner/engineer shall conduct ongoing inspections of all erosion/sedimentation control methodologies and direct the person or firm responsible for the maintenance to make any repairs or modifications necessary to assure continued effective operation of each methodology.

16. No clearing or rough-cutting shall be permitted prior to final approval of the plan.

17. No clearing or rough-cutting for purposes other than those set out above, or for construction of temporary erosion and sedimentation controls, as per approved plan, shall be permitted until those controls are in place.

4.0 HYDROLOGY

This section presents required City of Longview methodologies for the determination of design flow rates in developing areas of 100 acres or less. Drainage design for areas greater than 100 acres has been previously accounted for as a component of the City of Longview Master Drainage Plan and does not require hydrologic analysis.

For channels draining greater than 100 acres, the Office of the City Engineer will provide required design discharge rates or will inform the engineer of required hydrologic/hydraulic methodologies, calculations, and conditions.

4.1 EFFECT OF URBANIZATION

It is generally accepted that urban development has a pronounced effect on the rate and volume of runoff from a given rainfall. Urbanization generally alters the hydrology of a watershed by improving its drainage efficiency, reducing its surface infiltration and reducing its storage capacity. The hydrographs below illustrate the effect of urbanizing a watershed by presenting runoff rate versus time for the same storm with three different levels of watershed development.

The City of Longview requires stormwater detention in areas where development intensity exceeds a C-factor corresponding to a development density of SF-4 (5 units per acre); thus, the purpose of detention (when it is required) is to reduce flood peaks to levels which would be experienced for SF-4 residential development. This is true regardless of the C value present under pre-developed conditions. In other words, development activity which does not result in a comparable C-factor greater than that produced by SF-4 development, will not require storm water detention.

Changes to the hydrologic character of a watershed generally results from the elimination of porous surfaces and ponding areas by grading and paving building sites, streets, and parking lots, and by constructing buildings and other facilities characteristic of urban development. These effects are often especially evident in the development of smaller drainage areas where paving and building construction can cause dramatic changes in the hydrologic character of the basin.

4.2 REQUIRED HYDROLOGIC METHODOLOGY - THE RATIONAL METHOD

The Rational Method represents an accepted method for determining peak storm runoff rates for small watersheds (less than 100 acres) that have a drainage system unaffected by complex hydrologic situations such as ponding areas, storage basins and watershed transfers (overflows) of storm runoff. This widely used method provides satisfactory results if understood and applied correctly.

4.2.1 Definition of Rational Formula

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

$$Q = CiA \qquad (4\text{-}1)$$

where:

- Q is defined as the peak rate of runoff in cubic feet per second. Actually, Q is in units of inches per hour per acre. Since this rate of in/hr/ac differs from cubic feet/second by less than one percent, the more common cfs is used.

- C is the dimensionless coefficient of runoff representing the ratio of the runoff rate to the rainfall rate.

- i is the average intensity of rainfall in inches per hour for a period of time equal to the critical time of flow concentration for the drainage area to the point under consideration.

- A is the area in acres contributing runoff to the point of design during the critical time of concentration.

4.2.2 Assumptions of Rational Method

Basic assumptions associated with the Rational Method are:

1. The frequency or recurrence interval of the peak discharge is equal to the frequency of the average rainfall intensity associated with the critical time of concentration (duration).

2. The time of concentration is the critical time of concentration and is discussed in Section 4.2.5.

3. The ratio of runoff to rainfall, C, is uniform during the storm duration.

4. Rainfall intensity is uniform during the critical storm duration.

In the City of Longview, drainage calculations and improvements implemented under the City's Master Drainage Plan are based upon the assumption of basin-wide development to a minimum zoning density of SF-4 (5 units per acre), corresponding to a 100-year C-factor of 0.70. Thus, all calculations for drainage design in the city will be required to utilize a C-value no less than 0.70 regardless of actual land use type. In areas where development intensities exceed 0.70, stormwater detention will likely be required.

4.2.4 Rainfall Intensity

Rainfall intensity (i) is the average rainfall rate in inches per hour which is considered for a particular basin or sub-basin and is selected on the basis of design rainfall duration and design frequency of occurrence. The design duration is equal to the critical time of concentration at the design point on the drainageway. The design frequency of occurrence is a statistical variable which is established by design standards or chosen by the engineer as a design parameter. In Longview the design storm frequency for drainage design is 100 years in all cases. In other words, drainage structures must be sized to accommodate the runoff from a storm event which has a one percent chance of being equaled or exceeded in any given year (the so-called "100-year storm").

4.2.5 Time of Concentration

Time of concentration is the time required for all portions of an upstream drainage area to be contributing to the flow at a point of interest. The time of concentration used in the rational equation is the critical time of concentration for the point of interest. The critical time of concentration is the time from the beginning of a storm event to the time of occurrence of the peak flow rate at the point of interest assuming that during this specified time, rainfall is uniformly intense over the area contributing to the flow. Runoff from a watershed usually reaches a peak at the time when the entire drainage area is contributing, in which case, the critical time of concentration is the time for water to flow from the most remote point in the watershed to the point of interest. However, the runoff rate may reach a peak prior to the time the entire upstream drainage area is contributing. In this instance, only the portions of the drainage area able to contribute flow to the design point during the critical time of concentration should be used in determining the peak discharge.

The time of concentration to any point in a storm drainage system is a combination of the "overland flow" and the "time of channelized flow."

The overland flow time is the time for water to flow over the surface, generally via sheet flow, to a storm sewer inlet or area of channelized flow. Overland flow time decreases as the slope and the imperviousness of the surface increases, and it increases as the distance over which the water has to travel increases and as retention and/or infiltration by the contact surfaces increases. Average velocities for estimating travel time for overland flow can be calculated using Figure 4-1.

The overland flow time shall be determined by direct computation using the following formula:

$$T_o = D_f /60V \tag{4-2}$$

where:

T_o = overland flow time (minutes).
D_f = flow distance (feet).
V = average velocity of runoff flow (ft/sec).

Overland flow distances will rarely exceed 300' in developed areas. If the overland flow time is calculated to be in excess of 20 minutes, the designer should verify that the time is reasonable considering the projected ultimate development of the area.

The time of channelized flow is the quotient of the length of the conduit (storm sewer, channel, or gulley) and the velocity of flow as computed using the hydraulic characteristics of the conduit:

$$T_c = D_f /60V \tag{4-3}$$

where:

T_c = Time of channelized flow

4.2.6 Drainage Area (A)

The size and shape of the drainage area must be determined. The area may be calculated through the use of topographic maps supplemented by field surveys where topographic data has changed, or where the contour interval is too great to distinguish the direction of flow. A drainage area map shall be provided for each project (Section 4.4). The drainage area contributing to the system being designed and the drainage subarea contributing to each inlet point shall be identified. The drainage divide lines are determined by the natural slopes, pavement slopes, locations of downspouts, paved and unpaved yards, grading of lawns and many other features that are introduced by the urbanization process.

4.3 APPLICATION OF THE RATIONAL METHOD

Application of the Rational Method will generally proceed as follows (see Table 4-2).

Step 1 - Determine contributing drainage area. Obtain an appropriate topographic map of the subject drainage basin. Delineate the area draining to the point of interest. Determine the size of the drainage area in acres.

Note: **Drainage design methodologies traditionally utilized in the City of Longview require calculation of design flow rates for multiple smaller subbasins. Flow rates at points downstream are then typically calculated by summing the design flows from the upstream basins. This methodology generally results in unreasonably high design flow rates in the downstream areas and consequent excessive expenditures for construction of drainage facilities. The utilization of time of concentration in the rational method allows for design flow reductions in areas with larger contributing drainage basins and consequent decreases in drainage facility construction costs.**

Step 2 - Determine overland flow time. From the most remote point with respect to flow time in the upper region of the critical drainage area, calculate the time for flows to reach a point of channelized flow (such as a gully if the area is undeveloped or the storm sewer inlets or street if developed).

Overland Flow Time (minutes) = Flow Distance (ft)/[(60) Average Velocity (ft/sec)]

Average velocities for determining overland flow time can be estimated from Figure 4-1. Overland flow distances exceeding 300 feet are extremely unusual. Overland flow times should never exceed about 20 minutes in developed areas and about 30 minutes in undeveloped areas.

Step 3 - Determine the time of channelized flow. The time of channelized flow represents the time from when a particle of water begins to flow in a gully, channel or storm sewer to when it reaches the point of interest. The time of channelized flow is calculated utilizing the following formula:

Flow Time (minutes) = Flow Distance in Conduit or Drainageway (ft)/[(60) Average Velocity (ft/sec)]

Average velocity in the drainage path (usually a gulley, a street, a storm sewer or a channel) can be estimated using the hydraulic computer routines, "Normal Depth in Non-Circular Channel" and/or "Normal Depth in Circular Channel." The user inputs estimated slope, estimated flow rate (cfs), estimated channel roughness and channel configuration to obtain a reasonable estimate of flow velocity.

Step 4 - Determine Time of Concentration (Tc). The time of concentration from the hydraulically most remote point in a watershed to the point of interest is calculated to be the sum of "overland flow time" and "channelized flow time."

Step 5 - Determine design rainfall intensity (i). The design rainfall intensity is determined by consulting the frequency/intensity/duration curves for Longview presented as Figure 4-2. For a given storm duration and frequency (the 100-year storm in Longview), the appropriate uniform rainfall intensity is obtained. The required storm duration is equivalent to the time of concentration determined in Step 4 above.

Step 6 - Estimate C factor. Table 4-1 presents appropriate C values for a range of land use types and development densities. Table 4-1 is provided for calculation of composite C values in areas where development density may exceed SF-4 (C = 0.70). **In all circumstances design flows should be calculated utilizing the C value for ultimate watershed conditions, or 0.70, whichever is greater.**

Step 7 - Determine Design Flow Rate. The design flow rate is then calculated by applying the rational formula

$$Q = CiA \text{ where}$$

Q = design flow rate for the point of interest (cfs)
C = runoff coefficient
i = design rainfall intensity (in/hr)
A = drainage area (acres)

4.4 MATERIAL TO BE PROVIDED FOR CITY REVIEW

The following materials related to design flow rate calculations using the rational method must be provided for City review.

1. A topographic drainage area map with all drainage areas and flow rate calculation points delineated.

2. A completed copy of Table 4-3 outlining design flow calculations shall be placed on the topographic map or a related storm drainage sheet contained in the construction plans.

5.0 OPEN CHANNEL FLOW

This section summarizes the practical considerations, technical principles, and criteria necessary for proper design of open channels. The analysis of open channel flow also aids in determining other flow-related concerns, such as culvert tailwater depths, time of concentration calculations (travel times), and flood elevations.

5.1 OPEN CHANNEL HYDRAULICS - AN OVERVIEW

Flow conditions in an open channel are characterized as steady or unsteady, uniform or varied, subcritical or supercritical.

5.1.1 Steady or Unsteady Flow

Steady flow occurs when the flow rate Q (usually expressed in cubic feet per second) is constant with respect to time. For unsteady flow, the flow rate at a particular point is not constant with respect to time. For the purposes of design, all flows may be assumed to be steady.

5.1.2 Uniform or Varied Flow

Uniform flow occurs when the depth of flow is not changing with respect to distance along the channel. A true state of uniform flow is difficult to obtain under most natural conditions. Nevertheless, when a channel is sufficiently long and sufficiently unchanging such that the flow depth is not significantly changing, the flow may be assumed to be uniform for design purposes.

Varied flow occurs when the physical configuration, slope, or surface roughness of a channel changes, or when a disturbance such as a weir or bridge embankment is introduced in the channel. Under these conditions, the depth and velocity of the flow will vary along the channel in the vicinity of the disturbance.

5.1.3 Subcritical or Supercritical Flow

The speed of a small wave in a shallow channel is given by the term $(gy)^{1/2}$ where y is the depth (ft) and g is the acceleration of gravity (32.2 ft/sec^2). When the velocity of flow in a channel exceeds this value, the flow is supercritical. When it is less than this value, the flow is subcritical. Supercritical flow is generally characterized by high velocities and shallow depths, while subcritical flow is characterized by slower velocities and greater depths.

The most important distinction between these two states of flow is that the effect of a disturbance in the channel, such as a bridge constriction or a weir, does not influence upstream flow conditions in supercritical flow as it does in the case of subcritical flow. Therefore, subcritical flow is controlled by downstream channel conditions while supercritical flow is controlled by upstream channel conditions.

The value of the Froude Number indicates the state of flow in an open channel. A Froude Number greater than 1.0 indicates supercritical flow. A Froude Number less than 1.0 indicates subcritical flow. A Froude Number equal to 1.0 indicates flow at critical depth.

5.1.4 Critical Depth

When the velocity of flow in a channel is equivalent to the velocity of a gravity wave $(gy)^{1/2}$, critical flow at critical depth exists. Flow at or around critical is characterized by instability and should be avoided in channel design except at specific flow transition points such as weirs and sluice gates. Near critical flow, small changes in hydraulic conditions will cause exaggerated changes in depth and velocity.

5.2 CHANNEL DESIGN

5.2.1 General Design Considerations

The path taken by an existing, naturally-carved channel often represents the most logical general pathway of flow. For runoff rates associated with undeveloped conditions, the natural channel is largely stable against erosion and is topographically efficient in draining adjacent land. In light of this, it is logical that the engineer should consider taking advantage of naturally-carved drainageways when locating and designing open channels.

Although there are numerous channel designs available to the engineer, a judicious design must conform to certain hydraulic, aesthetic, and safety-related standards. In situations where the use of a natural drainage course is unfeasible, the engineer must choose to design either an earthen channel or a lined channel.

Grassed channels generally produce lower flow velocities and greater channel storage. They are, in most cases, aesthetically and economically superior to concrete or riprap-lined waterways. However, grass-lined channels require more right-of-way, are vulnerable to erosion, and must be continually maintained. They can also have problems with sideslope stability, local ponding of water, and/or sediment deposition.

In areas where land values are extremely high, or right-of-way is limited, concrete or riprap-lined channels may be the design of choice. However, concrete channels can be significantly more expensive. In addition, they tend to move water faster and store less water possibly resulting in higher peak discharges downstream.

5.3 OPEN CHANNEL DESIGN REQUIREMENTS

The following general criteria should be utilized in the design of open channels in Longview.

5.3.1 Design Frequency

Open channels in the City of Longview shall be designed to contain the runoff from the 100-year frequency storm within the right-of-way while providing a minimum of one foot of freeboard. In those cases where channel modifications are necessary to control increased flows from proposed development, proposed water surface profiles are restricted such that the proposed 100-year flood shall not exceed the existing 100-year flood profile. In addition, the channel must be designed to have sufficient freeboard to provide for adequate drainage of lateral storm sewers, during the 25-year storm.

5.3.2 Velocity

Excessive velocities can cause erosion and may pose a threat to safety. Velocities which are too low may allow sediment deposition and subsequent channel clogging. Table 5-1 provides maximum allowable velocities. Minimum velocities are those produced by a channel invert slope of 0.1 percent.

5.3.3 Flow Depth

Deep channels are generally difficult to maintain and can be hazardous. Therefore, design depths should be as shallow as practical.

5.3.4 Freeboard

Since there is no universally accepted rule governing the amount of freeboard required for a channel, selection of a safe amount should be based on confidence in the design discharge estimates, stability of the flow profile and the expected damage from water overflowing channel banks. A minimum value of one foot is required. The necessity for additional freeboard should be anticipated on the outside channel edge along curves where flooding of adjacent properties will cause excessive damages or possible injury or loss of life, the owner/engineer should consider utilization of extra freeboard above the one foot requirement.

5.3.5 Channel Alignment

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

5.3.6 Invert Slope

The slope of the channel invert is generally governed by topography and the energy head required for flow. Since invert slope directly affects channel velocities, channels should have sufficient grade to prevent significant siltation but grades should not be so steep as to create erosion problems. The minimum channel invert slope shall be 0.1 percent. The maximum channel invert shall be limited by maximum flow velocities as given in Table 5-1. Appropriate channel drop structures may be used to limit channel invert slope in steep areas.

5.3.7 Sideslope

In grass-lined channels, normal maximum sideslope is 3 (horizontal):1 (vertical), which is also the practical limit for mowing equipment. In some areas, sideslopes flatter than 3:1 may be necessary due to local soil conditions. Sideslopes steeper than 3:1 are acceptable if adequate geotechnical back-up is provided to the Office of the City Engineer.

5.3.8 Curvature

In general, centerline curves should be as gradual as possible and not have a radius of less than twice the design flow top width. The maximum curvature for any man-made channel should be 90°.

5.3.9 Manning's "n" Value

The following values of the Manning's roughness coefficient should be used in man-made channels unless permission for alternative values is received from the Office of the City Engineer.

Channel Cover	"n" value
grass-lined (maintained)	0.035
grass-lined (not maintained)	0.04 minimum
concrete-lined	0.013
"R-RAP"-lined	0.035
concrete paver-lined	0.04
stone riprap	0.04
gabions	0.04

Tables 5-2 and 5-3 present Manning's "n" values for a wide range of flow surfaces. These tables can be used in determining "n" values for a wide range of flow surface conditions other than those specified above.

5.3.10 Confluences

The angle of intersection between the tributary and main channels should be between 15° and 45°. Angles in excess of 45° are permissible but are discouraged. Angles in excess of 90° are not permitted. See Figure 5-1.

5.3.11 Transitions

Expansions and contractions should be designed to create minimal flow disturbance and thus change in water surface elevation. Transition angles should be less than 12° from the longitudinal channel alignment.

5.3.12 Right-of-Way

The amount of right-of-way required for open channels is dependent on channel top width and channel type (earthen or lined). Adequate area must be set aside for both the channel itself and the adjacent berm required for channel maintenance. Minimum maintenance berm width shall be approved by the City Engineer.

5.4 EROSION PROTECTION

Erosion protection is necessary to insure that channels maintain their capacity and stability and to avoid excessive transport and deposition of eroded material. The three main parameters which affect erosion are vegetation, soil type and the magnitude of flow velocity and turbulence. In general, silty and sandy soils are the most vulnerable to erosion. Section 3.2 of this manual presents recommended channel erosion protection strategies.

5.4.1 Typical Locations Requiring Erosion Protection

The necessity for erosion protection should be anticipated in the following settings:

A. Along grassed channel sideslopes during the time necessary to fully establish the vegetative cover.

B. Areas of channel curvature, especially where the radius of the curve is less than three times the design flow top width.

C. Around bridges where channel transitions create increased flow velocities.

D. When the channel invert is steep enough to cause excessive flow velocities.

E. Along grassed channel sideslopes where significant sheet flow enters the channel laterally.

F. At channel confluences.

G. In areas where the soil is particularly prone to erosion.

H. In areas where offsite deposition of eroded material will occur.

Sound engineering judgment and experience should be used in locating areas requiring erosion protection. It is often prudent to analyze potential erosion sites following a significant flow event to pinpoint areas of concern.

5.4.2 Minimum Erosion Protection Requirements

Minimum erosion protection requirements for the City of Longview are as follows:

5.4.2.1 Confluences

Figure 5-1 presents the minimum requirements for determining when and where erosion protection or channel lining are necessary given the angle of the confluence.

5.4.2.2 Bends

When required, erosion protection must extend along the outside bank of the bend and at least 20 feet downstream of it. Additional protection on the channel bottom and inside bank, or beyond 20 feet downstream, will be required if maximum allowable velocities are exceeded. See Table 5-1.

5.4.2.3 Culverts

In areas where outlet flow exceeds allowable velocities in the channel, channel lining and/or an energy dissipation structure will be required.

5.4.2.4 Outfalls

Erosion protection will be necessary in areas of high turbulence or velocity typically found at the outfall of storm sewers and roadside ditches into the main channel.

5.4.2.5 In Grassed Channels During Establishment of Vegetative Cover

Following planting of sideslope vegetation in grassed channels and if design flow velocities exceed 3.5 ft per second, the waterway sideslopes should be stabilized with sod, with seeding protected by jute, Excellsior Mat, Enkamat or with other comparable blanket products. If possible flows in the channel should be temporarily diverted until the vegetation is established.

5.4.3 Types of Erosion Controls

When flow velocities exceed those allowed in Table 5-1 or when soils are deemed excessively erosive by a geotechnical engineer, acceptable structural erosion control shall be provided. The slope protection must extend up to the top of the channel bank. Section 3.2 presents recommended channel erosion protection strategies and specifications for their use.

5.5 GRASS-LINED SWALES

Grass-lined swales may be used parallel to a roadway under special conditions, with approval from the City Engineer, and are useful in lot-to-lot drainage. However, a dangerous potential for flooding exists when flow in roadside ditches exceeds capacity. Consequently, provisions must be made to assure that the level of water flowing beside a roadway does not reach damaging levels. Section 5.6 presents approved methodologies for determining design water surface elevations in grassed swales. Figures 5-6 and 5-7 present details for grass-lined swales with and without a stone-lined trickle channel.

5.5.1 Approval

Approval for the use of roadside ditch systems must be obtained from the Office of the City Engineer prior to the submittal of contour and drainage area maps, and hydrologic and hydraulic calculations.

5.5.2 Design Criteria

The following requirements must be met in the design of roadside ditch systems:

1. Minimum acceptable ditch section shall have a sideslope no steeper than 3 horizontal to 1 vertical.

2. The minimum bottom width for roadside ditches shall be two feet.

3. The "n" coefficient for the ditch calculations shall be a minimum of 0.035. All values must be justified.

4. The minimum grade or slope of the ditches shall be 0.01 ft/ft.

5. Roadside ditches shall be designed to accommodate a 100-year flow. The computed design water surface of the ditches shall be a minimum of 0.5 feet below roadway surface elevation at edge of pavement.

6. Erosion control methods shall be utilized in ditch designs where velocities of flow are calculated to be greater than 5.0 feet per second or where soil conditions dictate their need.

7. The minimum depth of the ditches shall be 18 inches and the maximum depth shall be 4 feet unless approval to do otherwise is obtained from the Office of the City Engineer.

8. Ditches are to be designed so as not to erode.

10. Driveway culverts shall have a minimum diameter of 18".

5.6 DETERMINATION OF WATER SURFACE ELEVATIONS

The state of flow in an open channel is at all times either uniform or varied. A different method for determining water surface profiles is applicable to each of these conditions of flow.

5.6.1 Uniform Flow

When a section of channel is sufficiently long and unchanging such that the flow depth is not changing, then the flow profile can be analyzed assuming uniform flow. Under these circumstances, the depth, known as normal depth, is constant and can be determined with Manning's equation:

$$Q = (1.49/n)\ AR^{2/3}\ S^{1/2} \tag{5-1}$$

or

$$V = (1.49/n)\ R^{2/3}\ S^{1/2} \tag{5-2}$$

where

Q	=	flow rate (cfs)
V	=	flow velocity (fps)
n	=	Manning's roughness coefficient representing the magnitude of friction force exerted on the flow by channel sideslopes and bottom (see Section 5.3.9)
A	=	cross-sectional area of flow (ft^2)
R	=	hydraulic radius (cross-sectional area divided by the wetted perimeter of the cross-sectional area (ft))
S	=	channel slope (ft/ft)

To solve for the depth (y), the area and hydraulic radius are written in terms of y, the only unknown. A trial and error solution is usually necessary.

To determine depth with Manning's Equation, it is preferable to utilize the hydraulic computer routine "Normal Depth in a Non-Circular Channel" which will perform the trial and error solution and yield depth and velocity in a uniform trapezoidal channel given flow rate, Manning's "n", and the channel configuration.

Another means to determine depth via Mannings Equation is with Figure 5-3 which presents a nomograph for determining depth for uniform flow in a trapezoidal channel.

Determination of erosion potential in a natural or grassed channel is a function of flow velocity. Figure 5-2 presents a nomograph for determining flow velocity using the Manning Equation under uniform flow conditions. As mentioned above, flow velocity can also be determined utilizing the "Normal Depth in a Non-Circular Channel" computer routine.

Figure 5-8, "Ditch Design Form" presents a tabularized means for recording design parameters and calculations.

Determination of flow capacity, depth, and velocity in pipe conduits can be carried out utilizing the "Normal Depth in a Circular Channel" computer routine. Figure 5-4 presents a nomograph to determine flow characteristics in a pipe conduit. Figure 5-5 presents a nomograph to determine velocity in a pipe conduit.

5.6.2 Varied Flow

In the majority of channel flow situations, the state of flow is gradually varied. In other words, the depth is gradually changing with longitudinal distance along the channel due to the presence of nonuniformities in the channel configuration, such as channel constrictions or changes in channel slope.

The recommended means for determining flow profiles under these conditions is with a backwater model. The recommended backwater model for calculating water surface profiles for nonuniform flow in Longview is the U.S. Army Corps of Engineers' program HEC-2, Water Surface Profiles.

The HEC-2 model can readily accommodate modifications in channel design and water surface increases at bridges, culverts, drop structures, and transitions. The program begins computation at a cross-section of known or estimated water surface elevation and proceeds upstream for subcritical flow, and downstream for supercritical flow, to determine the channel flow profile.

The use of HEC-2 will generally be adequate for all channel design situations encountered in Longview. However, the use of alternative flow profile computer models or methodologies is acceptable but must be approved by the Office of the City Engineer.

5.7 MATERIALS TO BE PROVIDED FOR CITY REVIEW

a. A topographic map of the subject drainage basin showing proposed drainage areas and proposed channel location.

b. Design flow rate calculations carried out as described in Section 4.0 of this manual. See Table 4-2.

c. For design using the Manning Equation, a listing of the following parameters:
   - flow depth (ft)
   - channel slope (ft/ft)
   - channel flow (cfs)
   - Manning's "n" value
   - channel sideslopes (H:V)
   - channel bottom width (ft)
   - freeboard allowance

d. For design using HEC-2:
   - input and output listing of computer run

e. Existing and proposed typical channel cross-sections with their locations delineated on the drainage map.

6.0 INLETS AND STORM SEWERS

This section will present City of Longview specifications and required procedures for the design of inlets and storm sewer systems.

6.1 INLETS AND STREETS

Design and layout of streets and inlets is essentially a trial and error procedure requiring sound engineering judgement and experience to minimize construction costs while assuring proper hydraulic performance.

6.2 INLET AND STREET DESIGN REQUIREMENTS

The following minimum design requirements will serve to limit nuisance flooding of surface drainage features during the design rainfall event.

6.2.1 Inlet and Street Capacity Determination

Street drainage capacities shall be determined utilizing Figures 6-1(A-H). Inlet capacities shall be determined utilizing the weir equation as described in Section 6.3 (Step 5).

6.2.2 Street and Inlet Design Standards

All newly-constructed streets must conform to cross-sectional shape requirements presented in Figure 6-2. All newly-constructed inlets must conform to requirements presented in Figure 6-3. The use of alternate designs is acceptable subject to the approval of the Office of the City Engineer.

6.2.3 Cumulative Inlet Capacity

The cumulative capacity of the inlets at a particular point on a storm sewer line must exceed the required capacity of the storm sewer at that point. The required capacity of the storm sewer shall be based on drainage area size and not on inlet capacity.

6.2.4 Inlet Spacing

Inlets shall be spaced at a maximum interval of 500 feet on both sides of the road along a given storm sewer line.

6.2.4 Inlet Location at Intersections

Wherever possible, inlets shall be placed on the upstream side of an intersection to eliminate street flow across the intersection. Major streets shall not be crossed with surface drainage unless approved by the Office of the City Engineer. Whenever possible, secondary streets shall not be crossed either. At any intersection, only one street may be crossed by surface drainage.

6.2.5 Inlet Clogging

Inlets shall be overdesigned by 10 percent to account for possible clogging. In other words, the calculated design flow should be increased by 10 percent prior to design of inlets.

6.3 INLET AND STREET DRAINAGE DESIGN PROCEDURE

The following general procedure should be utilized for the design of inlet and street drainage systems in the City of Longview.

Step 1. Create a Drainage Area Map - On a topographic map of the area to be developed, plot ROW lines and lot lines for the proposed development. Indicate the proposed land use (e.g., single family, commercial, natural area, etc.). Draw flow arrows on the map to indicate the direction of overland flow. Label high points and low points of streets as determined by street profile design. Street profiles must be determined prior to the preparation of the drainage area map in order to accurately calculate street drainage capacity.

Step 2. Determine Street Drainage Capacities - Beginning at a high point, determine street drainage capacity for the given street slope downstream of the high point using the nomographs presented in Figures 6-1 (A-H).

Step 3. Determine Points Where Street Is At Full Capacity - Using a trial and error process, consult the topographic map to find the drainage area which produces the same flow (or slightly less) as the street capacity for the design storm. Design flow rates are calculated using the rational formula as described in Section 4.0.

Step 4. Locate Inlets - Locate the first inlet where the street is at capacity or at the location 500 feet from the street high point. Generally, the most upstream inlet on a storm sewer line is located at the point where the street gutter flows full based upon the 100-year design storm flow. However, inlets should generally be placed at the upstream side of an intersection to eliminate street flow across the intersection. For larger streets (≥53'), a single lane must remain above the water surface elevation during the design storm to allow passage of traffic.

Step 5. Determine Inlet Sizing Requirements - Unsubmerged curb opening inlets and area inlets without grates in a sump function as rectangular weirs with the coefficient of discharge of 3.0. Their capacity shall be based on the following equation:

$$Q = 3.0y^{1.5}L$$

where:

- Q = Capacity of curb opening inlet or of area inlet, cfs
- y = Head at the inlet, feet (should be measured from the gutter depression elevation to the water surface). See Figure 6-1 (A-H).
- L = Length of opening through which water enters the inlet, feet

Figure 6-4 provides for direct solution of the above equation.

The Engineer may decide that for economic reasons, an inlet large enough to capture all of the approaching flow is not feasible, however, the bypass flow must be accounted for in the gutter capacity downstream of the inlet.

Step 6. Low Point Inlets - Inlets are always located at street low points unless some other means (such as curb cuts) is provided to drain the streets.

6.4 STORM SEWERS

The City of Longview requires that storm sewer systems be capable of handling runoff from the 100-year flood event without exceeding the system's capacity.

6.5 STORM SEWER DESIGN REQUIREMENTS

The following are minimum design requirements:

6.5.1 Design Frequency

Storm sewer shall be designed to convey the 100-year frequency storm event.

6.5.2 Minimum Storm Sewer Pipe Size

The minimum size of a pipe in a storm sewer line shall be 18," and shall be reinforced concrete.

6.5.3 Manning's "n" Value

The Manning's "n" value to be used in a reinforced concrete pipe storm sewer shall be 0.013. Alternative values for "n" must be approved by the Office of the City Engineer.

6.5.4 Maximum and Minimum Velocities

The minimum velocity of flow to be allowed in a section of storm sewer flowing full shall be 3 fps. The maximum velocity shall be 15 fps.

6.5.5 Storm Sewer Alignment

All concrete storm sewers shall follow the alignment of the right-of-way or easement.

6.5.6 Storm Sewer Location

Storm sewers shall be located in public streets rights-of-way or in easements adjoining and parallel to a street right-of-way. The location of storm sewers shall not be within side lot or back lot easements that prohibit future maintenance access unless approved by the Office of the City Engineer.

6.5.7 Storm Sewer Outfall Erosion Protection

All storm sewer outfalls shall be constructed, and all outfall channels shall be protected with channel lining, such that turbulence and excessive flow velocities do not cause erosion at the outfall.

6.5.8 Junction Box Location

Junction boxes shall be placed at the location of all pipe size changes, storm sewer junctions, and at maximum intervals of 500 feet measured along the centerline of the pipe sewer.

6.6 STORM SEWER DESIGN PROCEDURE

Calculation of water surface (or hydraulic grade line) levels in a storm sewer system may be carried out utilizing a variety of approaches. The preferable approach is with application of any one of the several "pipe system" hydraulics models available for computer use. Most of these models are capable of accurately determining water surface (hydraulic grade line) elevations in a complex pipe network including location of free surface and pressure flow regions and determination of head losses at junctions, manholes, and inlets. The use of a pipe model is especially recommended for larger or more complex storm sewer systems.

When a simple or smaller system is being designed, hand calculation is often preferable. In this case, the following design procedure will be utilized.

Step 1. Determine Tailwater Elevation - Determine the 25-year water surface elevation in the channel at the storm sewer outfall using appropriate backwater calculations.

Step 2. Determine Design Flow Rates in Storm Sewer - Determine the 100-year design flow rates for all sections of storm sewer based on drainage area size.

Step 3. Make Initial Sizing Estimates - Using the 100-year flow rates, make an initial estimate of the appropriate sizes for all sections of storm sewer assuming uniform, full flow conditions. This may be accomplished using Figure 5-4, 5-5, or the hydraulic computer routine "Normal Depth in A Circular Channel". Initially, storm sewers should be designed to flow at least 85% full, unless this is prohibited by minimum storm sewer pipe size restrictions.

Step 4. Calculate Profile of HGL - Begin calculation at the 25-year water surface elevation in the outfall channel and plot the hydraulic gradient for the 100-year storm. If the 100-year water surface elevation in the outlet channel is below the elevation of critical depth in the outletting storm sewer pipe, calculation of the hydraulic grade line should begin at the elevation of critical depth in the storm sewer. Critical depth for a storm sewer pipe can be calculated utilizing Chart 7-5. If the outfall of the storm sewer is submerged, begin calcuation of the HGL profile at the elevation of $V^2/2g$ above the water surface in the receiving channel where:

V	=	velocity of flow exiting the storm sewer (ft/sec)
g	=	32.2 ft/sec/sec

Under pressure flow conditions (i.e. there is no free air/water surface in the pipe), the slope of the HGL shall be calculated using Manning's Equation in the following form:

$$\text{Slope of HGL due to friction losses alone} = n^2 Q^2 / (1.49)^2 A^2 R^{4/3} \tag{6-1}$$

where :

n	=	Manning's friction coefficient
Q	=	Flow rate (cfs)
A	=	Cross-sectional area of the storm sewer pipe (ft^2)
R	=	Hydraulic radius (ft) (flow area/wetted perimeter)

The hydraulic computer routine "Slope of Hydraulic Grade Line" will allow for easy calculation of the HGL slope under pressure flow conditions.

When a pipe is not flowing under pressure conditions (i.e. there is free surface flow in the pipe), the elevation of the HGL is conservatively defined as the top of the pipe.

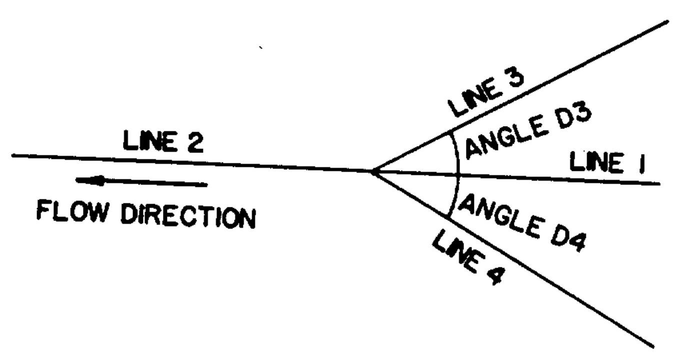
Step 5 - Calculate Junction and Bend Losses. Junctions and bends in conduits can cause major losses in both the energy grade and the hydraulic grade across the junction. If these losses are not included in the hydraulic design, the capacity of the conduit may be seriously restricted and underdesigned. The law of conservation of momentum, which equates the summation of all pressures acting at the junction with the summation of the momentums, affords a rational method of analyzing the hydraulic losses at a junction. Head losses at pipe junctions and bends may be calculated using the following formula which utilizes this methodology:

$$Y = \frac{Q_2 V_2 - Q_1 V_1 \cos D_1 - Q_3 V_3 \cos D_3 - Q_4 V_4 \cos D_4}{g(A_1 + A_2)/2} \tag{6-2}$$

where:

Y	=	head loss in feet across the bend or junction
Q_1	=	discharge in cfs in the largest pipe feeding into the junction or bend from upstream
Q_2	=	discharge in cfs in the outfall pipe leading away from the junction or bend downstream
Q_3, Q_4	=	discharge in cfs in the smaller pipes feeding into the junction or bend from upstream
V_1, V_2, V_3, V_4	=	estimated velocity in feet per second in the above-described pipes
D_1, D_3, D_4	=	the angle in degrees between the line of approach of the respective storm sewer pipe and a straight line extension of the outfall pipe. See diagram below.
A_2	=	cross-sectional area of the downstream outfall pipe
A_1	=	cross-sectional area of the largest pipe feeding into the junction or bend from upstream
g	=	gravitational constant (32.2 ft/sec/sec)

The hydraulic computer routine "Junction and Bend Losses" will easily calculate head losses by Equation 6-2. Note that bends are modeled like junctions with only one pipe coming in from upstream.



Step 6. Calculate Manhole and Inlet Losses.

In situations where a manhole or inlet is located on a storm sewer pipe at a point that is neither a bend nor a junction, the loss at that point is calculated by the following equation:

$$(0.05)\ V^2/2g \tag{6-3}$$

where:

V	=	average velocity of flow in the storm sewer (fps)

Average velocity is calculated one of two ways:

1. If the HGL exceeds the top of the pipe, V=Q/A where Q is the pipe flow rate (cfs) and A is the cross-sectional area of the pipe.

2. If the HGL does not exceed the top of pipe (i.e. not pressure flow), V is calculated from Figure 5-5 or from the hydraulic routine "Normal Depth in Circular Channel."

Losses at junctions, inlets and manholes are added to the elevation of the HGL at the point of the loss. Calculation of the HGL profile then proceeds upstream from the elevation after the head loss has been added.

Step 7. Adjust Storm Sewer System Configuration - Adjust pipe size and/or slope until the hydraulic grade line remains at least 1.5 feet below street inlet levels at all times.

6.7 MATERIALS TO BE PROVIDED FOR CITY REVIEW

The following materials must be provided for review to the Office of the City Engineer prior to approval of inlet and storm sewer designs:

a. A contour and drainage area map showing the proposed development, including storm sewer locations, inlet locations and drainage areas contributing to each inlet.

b. A table summarizing drainage area, time of concentration, design discharge (inlet intake), and size and number of inlets for each inlet(s) location and its contributing drainage area.

c. If storm sewer hydraulic design is carried out by hand, calculations of HGL profile and pipe friction and junction losses. If storm sewer design is carried out with a computer pipe model, a listing of the model input and output. Storm sewer plans submitted to the City Engineer for review shall have the HGL plotted on the profile.

7.0 CULVERTS

This section contains a discussion of required procedures for determining the culvert size and shape which will provide optimal economic and hydraulic design for a given design discharge and allowable headwater elevation.

7.1 GENERAL DESIGN REQUIREMENTS

The following general requirements apply to culvert design and construction within the City of Longview.

7.1.1 Design Frequency

All culverts in the City of Longview shall be designed to handle the 100-year flood flow for development conditions comparable to a C factor corresponding to a SF-4 residential development (C = 0.70) without causing upstream or downstream water surface profiles to exceed maximum levels. Maximum levels require that no water will overtop a bridge or road structure during the 100-year event and that a 1.0-foot freeboard shall be maintained in the channel. Exceptions to this requirement must be approved by the Office of the City Engineer

7.1.2 Culvert Length

Culverts shall be designed to span the road or railroad right-of-way unless prior approval for a shorter culvert is obtained from the Office of the City Engineer.

7.1.3 Headwalls

Headwalls and endwalls shall be utilized to control erosion and scour, to anchor the culvert against lateral pressures, and to insure bank stability. Headwalls shall be constructed of reinforced concrete or other materials as approved by the Office of the City Engineer and may be either straight and parallel to the channel, flared, or warped, with or without aprons, as required by site and hydraulic conditions. Protective guardrails should be included along culvert headwalls where necessary.

7.1.4 Minimum Culvert Sizes

The minimum pipe culvert diameter shall be 24 inches and the minimum box culvert dimension, horizontally or vertically, shall be 2 feet. These restrictions are made to guard against flow obstruction. Sizes less than these shall be considered on a case-by-case basis by the Office of the City Engineer.

7.1.5 Manning's "n" Values

Unless authorized by the Office of the City Engineer, the minimum Manning's "n" value to be used in concrete culverts shall be 0.013.

7.1.6 Erosion

Culverts, because of their hydraulic characteristics, generally increase the velocity of flow over that found in the natural channel. For this reason, the tendency for erosion, especially at the outlet, must be addressed, In general, culvert discharge velocities in unprotected channels should not exceed allowable channel velocities as defined in Table 5-1.

7.1.7 Structural Requirements

The following structural requirements must be met for culvert design in the City of Longview:

A. All precast reinforced concrete pipe should be Class III and ASTM C-76 (minimum).

B. All precast reinforced concrete box culverts with more than two feet of earth cover shall meet requirements of ASTM C789-79 or latest revision thereof.

C. All precast reinforced concrete box culverts with less than two feet of cover shall be ASTM 850-79 or latest revision thereof.

D. There shall be no corrugated metal pipe (CMP) used on City-approved projects.

E. ASSHTO HS20-44 loading should be used for all culverts.

F. Where guardrails are required at roadway culvert crossings, the approach ends of the guardrail shall be flared away from the roadway and properly anchored per Texas Department of Highways and Public Transportation Standards.

G. Joint sealing material for precast concrete culverts shall comply with "AASHTO Designation M-198 74 I, Type B, Flexible Plastic Gasket (Bitumen)," specification. All pipe joints will be wrapped and sealed with filter fabric.

H. Backfill requirements should be an integral part of culvert plans. Cohesive backfill around culverts should be a select material having a Plasticity Index of 17 or less.

I. The City of Longview promotes the use of geotextiles and geosynthetic as applicable to the design of drainage structures. Recommendations on acceptable practices and specifications can be obtained from the Office of the City Engineer.

7.2 CULVERT HYDRAULIC DESIGN

The fundamental objective of hydraulic design of culverts is to determine the most economical size at which the design discharge is passed without exceeding the allowable headwater elevation or causing erosion problems. However, there are numerous hydraulic considerations in culvert design which can make the design process somewhat complex.

7.2.1 Culvert Flow Types

The hydraulic character of a culvert is said to be either inlet-controlled or outlet-controlled. Inlet control means that the flow rate in the culvert is limited by the hydraulic and physical characteristics of the inlet of the culvert alone. These include headwater depth, barrel shape, barrel cross-sectional area, and/or the type of inlet edge. For inlet control, the barrel roughness, length, and slope are not factors in determining culvert capacity.

Under outlet control, the discharge capacity of the culvert is dependent on all of the hydraulic variables of the structure. These include headwater depth, tailwater depth, barrel shape, cross-sectional area, barrel roughness, slope, and length.

7.2.2 Headwater Depth

In all culvert design, headwater (depth of ponding at the entrance to the culvert) is an important factor in culvert capacity. The headwater depth (HW) is the vertical distance from the culvert entrance invert (flow line) to the energy line of the approaching flow. Due to low velocities in most entrance pools and the difficulty in determining velocity head in the flow, the energy line can often be assumed coincident with the water surface.

7.2.3 Tailwater Depth

For culverts under outlet control, tailwater depth is an important factor in computing both headwater depth and the hydraulic capacity of the culvert. If flow in the channel downstream of the culvert is subcritical, a backwater analysis or calculation of normal depth is warranted to determine the tailwater elevation. If the downstream flow is supercritical, tailwater is generally inconsequential to the culvert's hydraulic capacity.

7.2.4 Culvert Capacity Under Inlet Control Conditions

Under inlet control, the culvert entrance may or may not be submerged. However, in all cases inlet-controlled flow through the culvert barrel is free surface flow. When the culvert inlet is submerged under inlet control conditions, culvert capacity is best determined using standard empirical relationships based on laboratory research with models and full scale prototypes. Headwater vs. discharge relationships based on research of this kind are presented in the inlet control nomographs (Charts 7-1 and 7-2).

7.2.5 Culvert Capacity Under Outlet Control Conditions

Culverts flowing with outlet control can flow with the culvert barrel full or partially full for part or all of the barrel length. Both the headwater and tailwater may or may not submerge the culvert. In general, outlet control is most likely to occur when the vertical difference between the elevations of the headwater and the tailwater is small.

Outlet control nomographs are presented in Charts 7-3 and 7-4.

7.3 CULVERT DESIGN PROCEDURE

It is possible by involved hydraulic computations to determine the probable type of flow under which a culvert will operate for a given set of conditions. However, such computations can be avoided by determining the headwater necessary for a given discharge under both inlet and outlet flow conditions. The larger of the two will define the type of control which will actually exist and the corresponding headwater depth. Figure 7-1 provides a logical means for documenting culvert design and is required for City review of all culvert designs. The following is the recommended procedure for culvert design.

Step 1. <u>List design data</u> -

A. Design discharge (Q), in cfs, with return period.

B. Approximate length (L) of culvert, in feet.

C. Slope of culvert. If grade is given in percent, convert to slope in feet per feet.

D. Allowable headwater depth, in feet, which is the vertical distance between the culvert invert (flowline) at the culvert entrance and the permissible water surface elevation in the headwater pool or approach channel upstream from the culvert.

E. Flow velocities in the channel upstream and downstream of the proposed culvert location.

F. Type of culvert for first trial selection, including barrel material, barrel cross-sectional shape and entrance type.

Step 2. <u>Determine the first trial culvert size</u> -

Since the procedure given is one of trial and error, the initial trial size can be determined initially utilizing the inlet control nomographs (Charts 7-1 and 7-2) for the culvert type selected. An HW/D must be assumed along with the given Q to determine a trial size.

If any trial size is too large in dimension because of limited height of embankment or availability of size, multiple culverts may be used by dividing the discharge appropriately among the number of barrels used. Raising the embankment height or the use of box culverts with width greater than height should also be considered. Final selection should be based on applicability and costs.

Step 3. Find headwater depth for trial size culvert assuming inlet control -

A. Assuming inlet control and using the trial size from Step 2, find the headwater depth (HW_{inlet}) by use of the appropriate inlet control nomograph (Charts 7-1 and 7-2). Tailwater (TW) conditions are to be neglected in this determination. HW_{inlet} in this case is found by multiplying HW/D obtained from the nomographs by the height of culvert (D).

B. If HW_{inlet} is greater or less than allowable, try another trial size until HW_{inlet} is acceptable for inlet control before computing HW for outlet control.

Step 4. Find headwater depth for trial size culvert assuming outlet control

A. Estimate or calculate the depth of tailwater (TW), in feet, above the flowline at the culvert outlet for the design flow rate in the outlet channel. This is best accomplished with a water surface profile model using HEC-2. If the downstream channel is reasonably uniform in slope and configuration, Manning's Equation can be utilized with the hydraulic routine "Normal Depth in Non-Circular Channel" to determine tailwater elevation. If the flow in the downstream channel is supercritical, the tailwater elevation can generally be assumed to lie below the top of the culvert at the outlet.

B. For tailwater (TW) elevation equal to or greater than the top of the culvert at the outlet set d_2 equal to TW and find HW_{outlet} by the following equation:

$$HW_{outlet} = H + d_2 - LS_0 \tag{7-1}$$

where HW = vertical distance in feet from culvert invert (flowline) at entrance to the pool surface

H = head loss in feet as determined from the appropriate nomograph (Charts 7-3 and 7-4)

d_2 = vertical distance in feet from culvert invert at outlet to the water surface elevation (hydraulic grade line)

S_0 = slope of culvert barrel (feet/feet)

L = culvert length (in feet)

Note: For tailwater (TW) elevations less than the top of the culvert at the outlet, find headwater HW_{outlet} by Equation 7-1 as in Step 4 above except that

d_2 = d_c = D/2 or TW (whichever is greater)

where d_c = critical depth in feet (Charts 7-5 and 7-6)

Note: d_c cannot exceed D

D = height of culvert opening (feet)

Step 5. Compare the headwaters found in Step 3 and Step 4 (inlet control and outlet control) -

Comparing the headwater elevations (HW_{inlet} vs. HW_{outlet}) the higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected. If outlet control governs and the HW is higher than is acceptable, select a larger trial size and find HW as instructed under Step 2. (Inlet control need not be checked, since the smaller size was satisfactory for this control as determined under Step 4.

Step 6. Try other culvert types or shapes -

Try additional culvert types or shapes worthy of consideration, determine their size and HW by the above procedure.

Step 7. Compute outlet velocities -

Compute outlet velocities for size and type to be considered in selection and determine the need for channel protection.

If outlet control governs in Step 5 above, outlet velocity equals Q/A, where A is the cross-sectional area of flow in the culvert barrel at the outlet. If d_c or TW is less than the height of the culvert barrel, use A corresponding to d_c or TW depth, depending on whichever gives the greater area of flow. A can not exceed the total cross-sectional area A of the culvert barrel.

If inlet control governs in Step 5, outlet velocity can be assumed to equal mean velocity in open-channel type flow in the barrel as computed by Manning's Equation (Figure 5-5 or with the hydraulic computer routines "Normal Depth in Circular Channel" or "Normal Depth in Noncircular Channel" for the rate of flow, barrel size, roughness and slope of the culvert selected.

Step 8. Record culvert selection -

Record final selection of culvert with size, type, required and computed headwater, and outlet velocity. See Section 7.4 for required information to be submitted to the City.

7.4 MATERIALS TO BE PROVIDED FOR CITY REVIEW

The engineer shall be required to submit a summary of the technical calculations used in the design of any culvert. This includes at a minimum:

A. Calculations used to determine the design discharge for the culvert(s).

B. A summary table delineating design flow rate, culvert length, culvert slope, allowable headwater depth, tailwater depth, selected culvert type and size, flow velocity at culvert outlet, and designation of inlet or outlet control. See standard reporting sheet - Figure 7-1.

8.0 STORMWATER DETENTION/STORAGE

The introduction of impervious cover and improved runoff conveyance serves in many cases to increase flood peaks quite dramatically over those for existing conditions. When physical, topographic, and economic conditions allow it, channel improvements downstream of development are often used to prevent increased flooding. When this is not feasible, a widely used practice is runoff detention or retention storage wherein the storm volume is held back in the watershed and released at an acceptable rate. This section of the manual presents information on storage techniques, including guidance for the design of appropriate storm runoff storage facilities.

8.1 CITY OF LONGVIEW MASTER DRAINAGE PLAN

Development in a watershed can have complex and far-reaching consequences on the overall hydrologic regime. For this reason, careful plans for anticipating and meeting the long-term flood control and drainage needs of the City of Longview have been developed as part of the City's "master drainage plan." The goal of this work has been to provide the most practical and efficient basin-wide approach to the hydrologic consequences of ongoing or future development, including proper coordination of storm detention facilities and channel improvements. Accordingly, prior to the formulation of drainage plans, the Office of the City Engineer must be consulted concerning preferred watershed flood control strategies and alternatives.

In the City of Longview jurisdictional area, all drainage design shall assume a level of development equivalent to SF-4 zoning (5 units per acre) which equates to a Rational Method C-factor (or equivalent using an alternate method) of 0.70 for a 100-year return period (see Table 4-1). The determination of stormwater detention design requirements (e.g., storage volume, outflow) will depend on the status of Master Drainage Plan improvements in affected downstream drainageways. If the downstream Master Plan improvements have been built or funded and scheduled for construction in the near future (to the City of Longview's satisfaction), sufficient stormwater detention must be provided such that 5-, 25- and 100-year peak flow rates assuming a SF-4 level of development will not be exceeded in downstream areas due to the proposed development. If Master Plan improvements have <u>not</u> been built or funded and scheduled for

construction in the near future (to the City of Longview's satisfaction), sufficient stormwater detention must be provided such that 5-, 25- and 100-year peak flow rates for existing conditions will not be exceeded in downstream areas due to the proposed development.

The developer and the design engineer shall bear total responsibility for flooding occurring downstream of the subject property, even if stormwater detention facilities are constructed. In other words, meeting the requirements of the City of Longview Master Drainage Plan does not relieve the developer/engineer of responsibility for downstream flooding.

8.2 STORAGE CLASSIFICATIONS

Storage systems may be classified as either on-line or off-line facilities. They may be designed for either detention or retention of stormwater.

8.2.1 Retention Storage

In a retention storage facility, runoff is captured and released only after the storm event is over and the downstream water surface has subsided.

8.2.2 Detention Storage

The vast majority of flood control storage is handled by detention facilities. The purpose of detention storage is to hold storm runoff back but release it continuously at an acceptable rate through a flow-limiting outlet structure, thus controlling downstream peak flows.

8.2.3 On-line Storage

An on-line storage facility is one in which the total storm runoff volume passes through the retention or detention facility's outflow structure.

8.2.4 Off-line Storage

An off-line storage design is one in which storm runoff does not begin to flow into the storage facility until the discharge in the channel reaches some critical value above which unacceptable downstream flooding will occur. An off-line facility serves to store only the high flow rate portions of the flood event.

8.3 RECOMMENDED DESIGN METHODOLOGY

The stormwater detention design procedure recommended for use in the City of Longview employs the SCS dimensionless unit hydrograph methodology to develop a time-varying record of runoff flow rate from a subarea. This "hydrograph" is then "routed" through a detention facility with a particular physical configuration and outflow structure using the Modified Puls routing methodology. The term "routing" refers to keeping a running account of how much water enters the detention facility, how much exits, and how much stormwater volume is being stored in the facility.

To carry out this analysis, input procedures for the U.S. Corps of Engineers hydrologic model HEC-1 (which will perform a great many hydrologic tasks) have been modified to address only:

1. Hydrograph development in a single small subbasin.
2. Routing through a single detention facility with a user-supplied description of the facility's physical configuration and outflow characteristics.

Implementation of this detention design methodology is carried out utilizing menu item #5 in HYDROLIB (Appendix C).

8.3.1 Design Procedures

The following procedure should be followed for design of detention facilities in the City of Longview.

Step 1 - Locate a potential detention facility site keeping in mind that the goal of providing detention is to reduce post-development runoff rates to the C=0.70 level under post-development conditions.

Step 2 - Determine the 5-year, 25-year and 100-year flow rates at the basin outlet assuming a C value of 0.70. This task should be performed utilizing the rational method (Section 4.0). Then, using the proper CN value associated with the particular soil type (see Table 8-1). implement the "Detention Facility Design" routine for the subject area to determine the SF4 development peak flow as calculated using the SCS Method in HEC-1. The calculated peak flow rate is read from the line, "Peak Flow Rate Entering Detention Facility". The lesser of the two values (from the rational method and from the "Detention Facility Design" routine) will represent the SF4 development "target" flow rate to be matched through implementation of detention.

Step 3 - Determine a preliminary volume and configuration for the proposed detention facility and its outlet structure.

Step 4 - Invoke the computer routine - "Detention Facility Design" by entering "HYDROLIB".

Step 5 - Input the design storm frequency. In Longview, detention facilities must be designed to effectively mitigate peak flow rates during the 5-, 25- and 100-year events. In other words, during the 5-, 25-, and 100-year design storms, post-development peak flow rates must not exceed post-development peak flow rates assuming SF-4 development (C=0.70). The 5-year frequency event should usually be selected for the first detention routing trial. The pond should be somewhat oversized for the 5-year event in order not to exceed its capacity during the 100-year flood.

Step 6 - Input the contributing drainage area in acres.

Step 7 - Input the time of concentration (as calculated using Section 4.3) in minutes.

Step 8 - Input the appropriate CN value for the proposed development condition. See Table 8-1 for appropriate CN values.

Step 9 - Input the detention facility's preliminary physical configuration. This is accomplished by inputting a range of elevations along with corresponding water surface areas and pond outflow rates. Detention facility outflow rates for a given water surface elevation in the facility are determined using the procedures for culvert design described in Section 7.0 of this manual.

The first elevation should be the bottom of the pond and will usually have a water surface area of zero (i.e. no water in the facility) and an outflow rate of zero. The range of elevations should proceed up at intervals no greater than 1.0 or 2.0 feet and should be sufficiently close that the particular pond configuration is correctly represented. The maximum allowable number of elevations is 10.

Step 10 - Run the model.

Step 11 - Evaluate "Peak Water Surface Elevation in Detention Facility". If it exceeds the top of the pond berm, the facility is undersized. If it is far below the top of the pond berm, the facility may be oversized.

Step 12 - Evaluate "Peak Flow Rate Exiting Detention Facility". If it exceeds the peak flow rate for SF-4 development (C=0.70) as derived in Step 2, the pond configuration and/or outlet structure must be adjusted until the post-development flow-peak matches or is less than the SF-4 peak.

Step 13 - Perform the above procedure again for the 25-year and 100-year flood events so that the detention facility will meet its goals for all design storms between the 5- and 100-year events.

General Comment - In many cases, the detention facility will not serve the entire site of interest. Thus, careful consideration must be given to the requirement that flows exiting the entire subject site, and not just the detention facility, match the target SF-4 development peak flow rate.

8.4 GENERAL DETENTION FACILITY REQUIREMENTS

The following general requirements must be met for all detention facility design and construction in the City of Longview.

8.4.1 Detention Facility Outlet Structures

Several outlet structure types are available to the engineer. Most common is the pipe orifice exiting the facility through a berm or other flow containment structure at the elevation of the bottom of the detention facility. This type of outflow structure usually allows the facility to drain completely dry after a minimal period of no precipitation.

The sizing of a pipe orifice outflow structure involves determination of the pond outflow rate for varying water surface elevations in the facility. These outflow vs. water surface elevation relationships are generally established using the "inlet control" charts (Charts 7-1 and 7-2) from Section 7.0 of this manual. However, the engineer is warned that under certain circumstances, excessive tailwater elevations at the outlet structure outfall can cause the outflow structure to flow under outlet control conditions. If this is the case, the engineer must make conservative assumptions regarding the outflow characteristics of the facility and design according to the techniques for analysis of outlet control flow described in Section 7.0 (see Section 8.4.4).

In some cases, weir-type outflow structures (as opposed to orifice-type) may be utilized to meet the project's flood control objectives. Examples include simple rectangular weir spillways, V-notch weirs, and curb cuts in parking lots. Weir-type outflow structures function fundamentally differently from orifice-type outflow structures in that the outflow rate for weirs is a function of $H^{1.5}$ (as opposed to $H^{1/2}$ for orifices) where H is the depth of water above the flowline of the outflow structure. Thus, with weirs, an increase in depth yields a proportionately higher increase in outflow than for orifices.

Outflow vs. depth relationships for weir-type outflow structures are generally determined via an equation of the form:

$$Q = CLH^{1.5} \tag{8-1}$$

where:

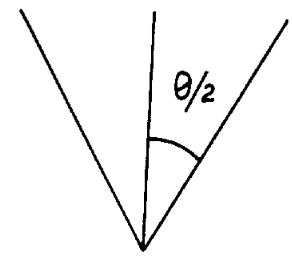
- Q = outflow rate (cfs)
- L = length of the weir perpendicular to the flow (ft)
- H = depth of water above the crest of the weir (ft).

Table 8-2 provides appropriate values of the weir coefficient "C" for varying water depths and weir crest thicknesses.

Outflow vs. depth relationships for V-notch weirs can be reasonably determined from the following equation:

$$Q = 2.5 \tan (1/2\theta)\, H^{5/2} \tag{8-2}$$

where:



Q = outflow rate (cfs)

θ/2 = V-notch weir angle as shown here:

H = water surface depth above the bottom of the V-notch.

Curb cut outflow structures can generally be designed utilizing the formula described above for broad-crested weirs.

8.4.2 Detention Basin Contour Grading

The detention basin can be constructed by berming a low-lying area, by full embankment construction, or by a balanced design utilizing excavated material for embankment. Whenever possible, the detaining pond should fit the natural contour of the land, be aesthetically pleasing and free-draining. Side slopes of 3:1 or flatter are generally acceptable for shallow ponds.

8.4.3 Sedimentation and Erosion

The detention basin should be spread with topsoil and seeded as soon as possible following final grading of the developed site. Sedimentation acquired during the construction operation may be redistributed over the site and stabilized by the planting of grass. Straw bale barriers may be used as filtering devices along embankment slopes or at the inlets of the outlet control structures. Any direct channels should be protected by using stone lining or mortared rubble. Side slopes accepting large quantities of sheetflow should be properly protected by mortared rubble or by using concrete lining. Proper erosion protection must especially be provided at pipe outfalls into the facility, pond outlet structures and overflow spillways where excessive turbulence and velocities will cause erosion. The entire developed tract should be stabilized as soon as possible to prevent any aggravated erosion caused by the disturbance of the construction operations.

8.4.4 Design Tailwater Depth for Outlet-controlled Flow

In order to determine peak pond outflow rate, a relationship must be established between the volume of storage (or depth) in the pond and the corresponding amount of discharge through the outflow structure.

For the purpose of establishing a peak outflow rate under outlet-control conditions, the engineer must make a judgment as to what will be the proper tailwater level for use in calculating the storage-discharge relationship.

In general, it should be noted that an unreasonably high choice for the tailwater depth will result in an oversized outflow structure and thus minimized attenuation of the peak outflow, especially for smaller storm events. An unreasonably low choice for the tailwater depth will result in an undersized outlet structure and the risk that the pond will be overtopped during the design storm event.

8.4.5 Allowances for Extreme Storm Events

Design consideration must be given to storm events in excess of the 100-year flood. An emergency spillway, overflow structure, or swale must be provided as necessary to effectively handle the extreme storm event. In places where a dam has been utilized to provide detention directly in the channel, due consideration must be given the consequences of a failure, and if a significant hazard exists, the dam must be adequately designed to prevent such hazards.

Detention facilities which measure greater than six feet in height are subject to 31 Texas Administrative Code (TAC) Chapter 299 (Subchapters A through E), which went into effect May 13, 1986, and all subsequent changes. The height of a detention facility or dam is defined as the distance from the lowest point on the crest of the dam (or embankment), excluding spillways, to the lowest elevation on the centerline or downstream toe of the dam (or embankment) including the natural stream channel. Subchapters A through E of Chapter 299 classify dam sizes and hazard potential and specify required failure analyses and spillway design flood criteria.

8.4.6 Storm Sewer Hydraulic Gradients

The hydraulic gradients in storm sewers shall be determined using procedures outlined in Section 6 of this manual. When storm sewers outlet into a detention facility, the starting water surface elevation for these calculations shall be the 25-year maximum pond elevation.

8.4.7 Detention Facility Maintenance

Once a detention facility is built, it is considered by the City of Longview to be a permanent facility. Regular maintenance of detention facilities is required.

8.5 MATERIALS TO BE PROVIDED FOR CITY REVIEW

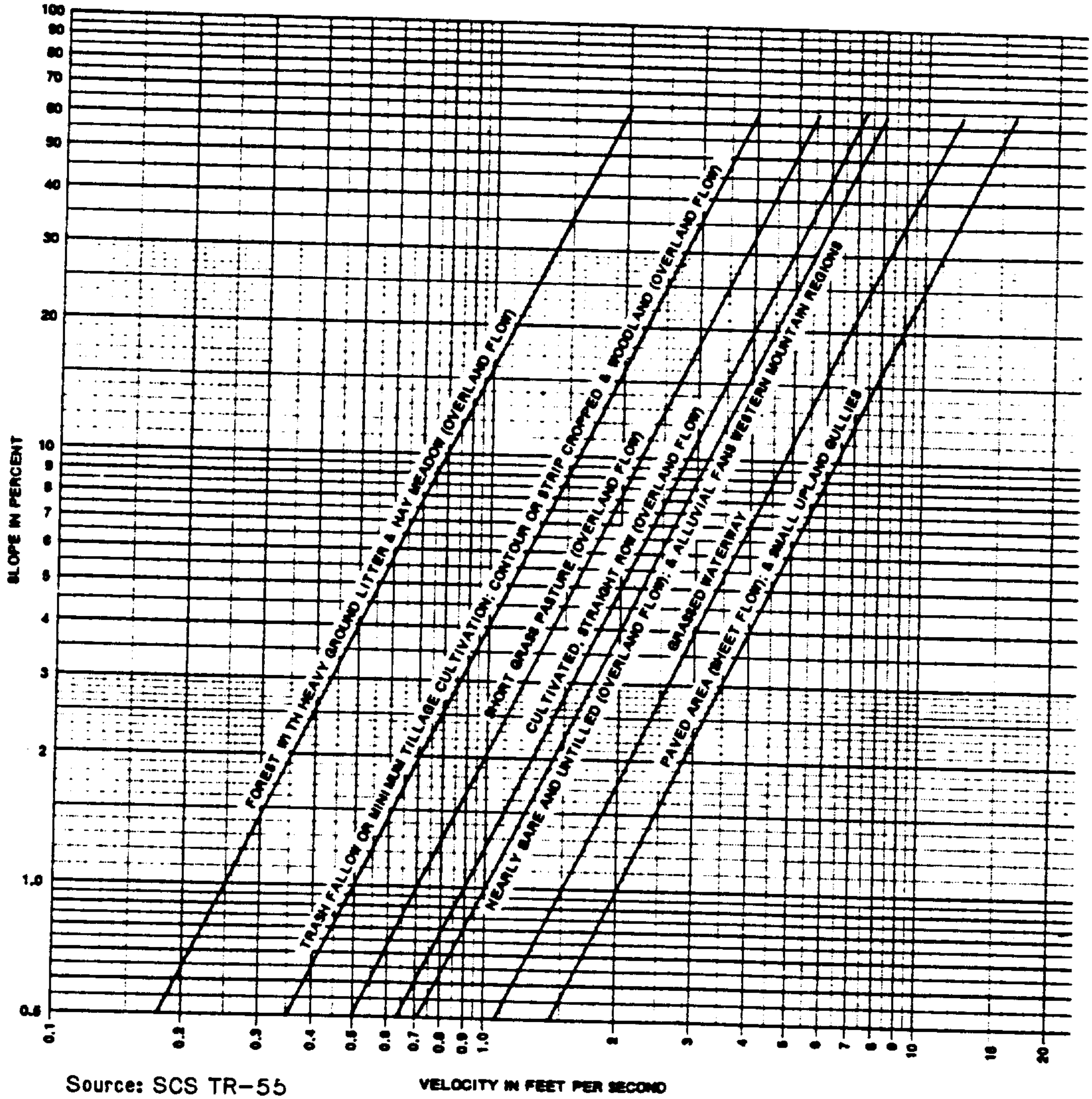
The engineer shall be required to submit a summary of the technical calculations used in the design of any stormwater storage facility. This shall include at a minimum:

1. Rational method calculations for the 5-, 25- and 100-year pre-development and post-development condition flows:

2. The HEC-1 output utilized for final design including output for the SF-4 conditions HEC-1 run.

3. Calculations used to determine outflow structure rating curves.

APPENDIX A
DESIGN CHARTS, NOMOGRAPHS
FIGURES, TABLES



Source: SCS TR-55

FIGURE 4-1

OVERLAND FLOW VELOCITY

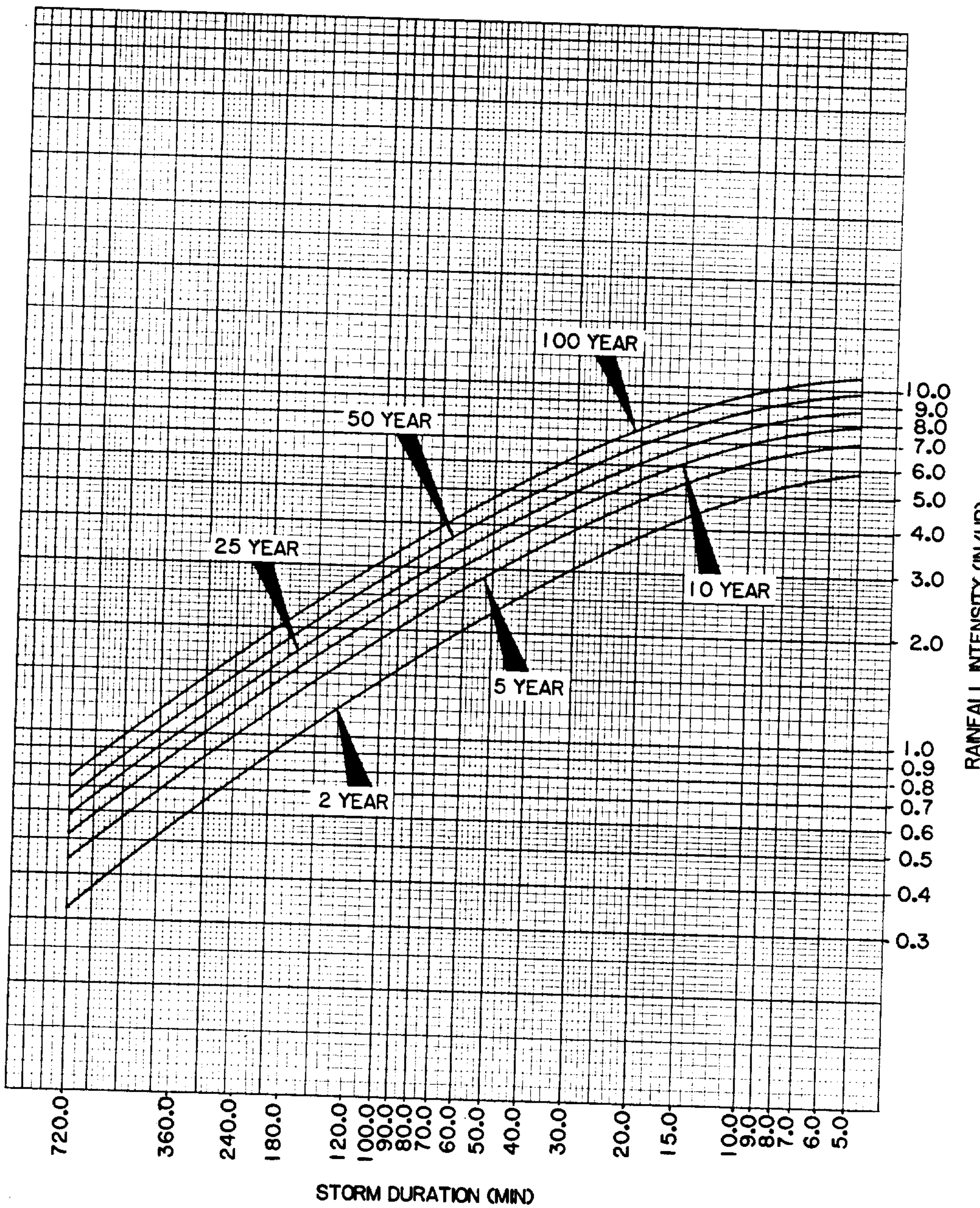


FIGURE 4-2
FREQUENCY/INTENSITY/DURATION CURVES FOR LONGVIEW, TEXAS

TABLE 4-1

RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS
RUNOFF COEFFICIENT (C)

	Return Period		
	5 Years	25 Years	100 Years
DEVELOPED			
Asphaltic	0.77	0.86	0.95
Concrete/Roof	0.80	0.88	0.97
Grass Areas (Lawns, Parks, etc.)			
Poor Condition (grass cover less than 50 percent of the area)			
Flat, 0-2%	0.34	0.40	0.47
Average, 2-7%	0.40	0.46	0.53
Steep, Over 7%	0.43	0.49	0.55
Fair Condition (grass cover on 50 to 75 percent of the area)			
Flat, 0-2%	0.28	0.34	0.41
Average, 2-7%	0.36	0.42	0.49
Steep, Over 7%	0.40	0.46	0.53
Good Condition (grass cover larger than 75 percent of the area)			
Flat, 0-2%	0.23	0.29	0.36
Average, 2-7%	0.32	0.39	0.46
Steep, Over 7%	0.37	0.44	0.51
UNDEVELOPED			
Cultivated Land			
Flat, 0-2%	0.34	0.40	0.47
Average, 2-7%	0.38	0.44	0.51
Steep, Over 7%	0.42	0.48	0.54
Pasture/Range			
Flat, 0-2%	0.28	0.34	0.41
Average, 2-7%	0.36	0.42	0.49
Steep, Over 7%	0.40	0.46	0.53
Forest/Woodlands			
Flat, 0-2%	0.25	0.31	0.39
Average, 2-7%	0.34	0.40	0.47
Steep, Over 7%	0.39	0.45	0.52

TABLE 4-1 (Concluded)

	Zoning*	5 Years	25 Years	Return Period 100 Years
RESIDENTIAL AREAS				
1 unit per 5.0 acres (10% impervious)	Rural Estate	0.37	0.44	0.50
1 unit per 2.0 acres (15% impervious)	N/A	0.39	0.46	0.54
1 unit per 1.0 acre (20% impervious)	A	0.42	0.49	0.56
2 units per 1.0 acre (25% impervious)	SF 1	0.44	0.51	0.59
4 units per 1.0 acre (38% impervious)	SF 2, SF 3, TF 1, MF 1	0.50	0.58	0.65
5 units per 1.0 acres 45% impervious	SF 4			0.70
8 units per 1.0 acre (65% impervious)	SF 5, TF 2, TF 3, MF 2, MF 3	0.63	0.71	0.79
12 units per 1.0 acre (80% impervious)		0.70	0.78	0.87
SUBURBAN SHOPPING CENTER (90% impervious)		0.76	0.83	0.92
CENTRAL BUSINESS DISTRICT (95% impervious)		0.78	0.86	0.95
INDUSTRIAL				
60% impervious		0.62	0.70	0.78
80% impervious		0.71	0.79	0.87

* See City of Longview Zoning Ordinance for more information.

TABLE 4-2

DETERMINATION OF DESIGN DISCHARGE

SUBAREA ____________________

DRAINAGE AREA (A)

Critical contributing drainage area to point of interest (acres) = ____________________

OVERLAND FLOW TIME (OR TIME TO CHANNELIZATION OF FLOW)

A. Overland flow distance (feet) for surface	B. Average slope (ft/ft) for surface	C. From Figure 4-1, average velocity (ft/sec) for surface	Overland flow time for surface (minutes) Col. A/(60 x Col. C)
1. ________	________	________	________/60 x ________ = ________
2. ________	________	________	________/60 x ________ = ________
3. ________	________	________	________/60 x ________ = ________

Total overland flow time (min) = ____________

TIME OF CHANNELIZED FLOW

A. Channelized flow distance (feet) for conduit	B. Average Slope (ft/ft) for conduit	C. Average velocity (ft/sec) for conduit	Time of channelized flow (minutes) Col. A/(60 x Col. C)
1. ________	________	________	________/60 x ________ = ________
2. ________	________	________	________/60 x ________ = ________
3. ________	________	________	________/60 x ________ = ________

Total Time of Channelized Flow (min) = ________

TIME OF CONCENTRATION

Tc = Overland flow Time + Time of Channelized Flow (min.) = ____________ + ________ = ________

DESIGN RAINFALL INTENSITY (i)

Design Duration = Tc = ____________ minutes
Storm Frequency = ____________ years

From Figure 4-2
Design Rainfall Intensity (in./hr) = ______________

C FACTOR (C)

Land Use	A. Area (acres)	B. C Value for Land Use	Component C Value for Land Use (Col. A/Total Area) x Col. B
1. ________	________	________	____/____ x ________ = ________
2. ________	________	________	____/____ x ________ = ________
3. ________	________	________	____/____ x ________ = ________
4. ________	________	________	____/____ x ________ = ________
5. ________	________	________	________ x ________ = ________

Composite C value for total drainage area = ________

DESIGN FLOW RATE

Application of Rational Formula:

Design Discharge (Q) = Composite C x Design Rain Intensity (i) x Area (A)

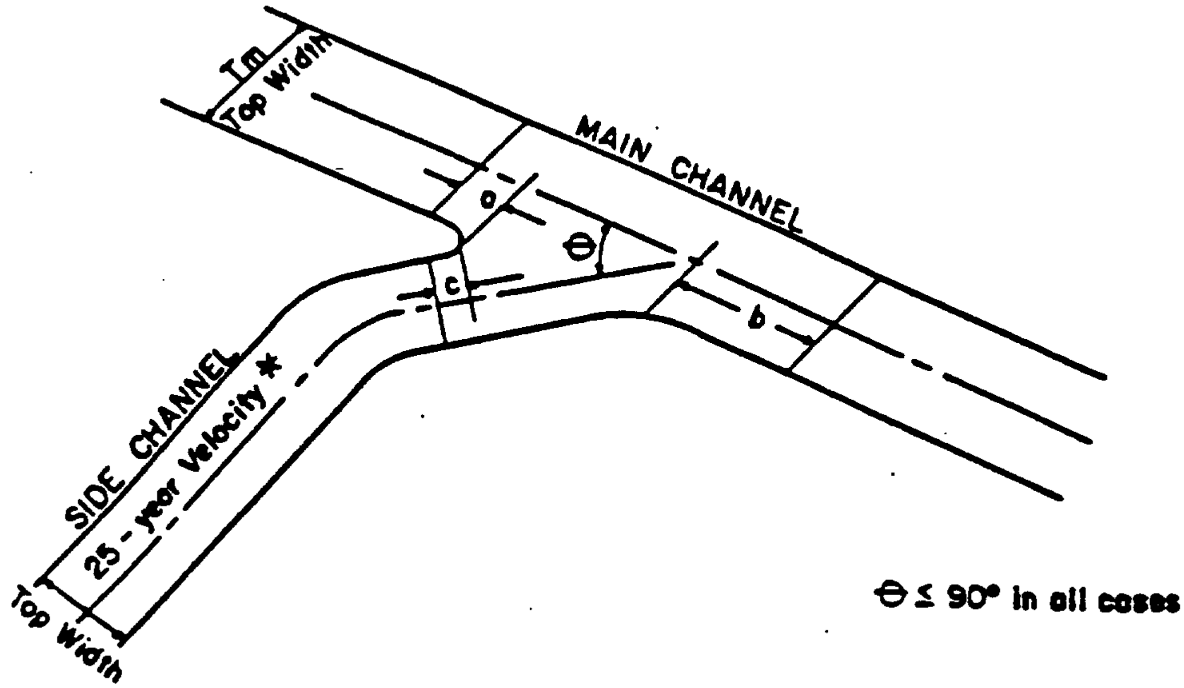
Q (cfs) = C x i(in/hr) x A(acres) = ______ x ______ x ______ = ______cfs

---

TABLE 4-3

DESIGN FLOW CALCULATIONS SUMMARY TABLE

Flow Calculation Point I.D.	Description	Drainage Area	Overland Flow Time	Channelized Flow Time	Time of Concentration	Design Rainfall Intensity(i)	C-Factor	Design Flow Rate (CiA)



MINIMUM EXTENT OF EROSION PROTECTION

Location	Distance (ft.)
a	20
b	longer of 50' or 0.75 x Tm ÷ Tan θ
c	20'

25-year Velocity * in Side Channel (feet per second)	ANGLE OF INTERSECTION θ	
	15°—45°	45°—90°
4 or more	Protection	Protection
2 - 4	No Protection	Protection
2 or less	No Protection	No Protection

* Note: 25-year velocity in side channel assuming no backwater from main channel

FIGURE 5-1
REQUIRED EROSION PROTECTION
AT CHANNEL CONFLUENCE

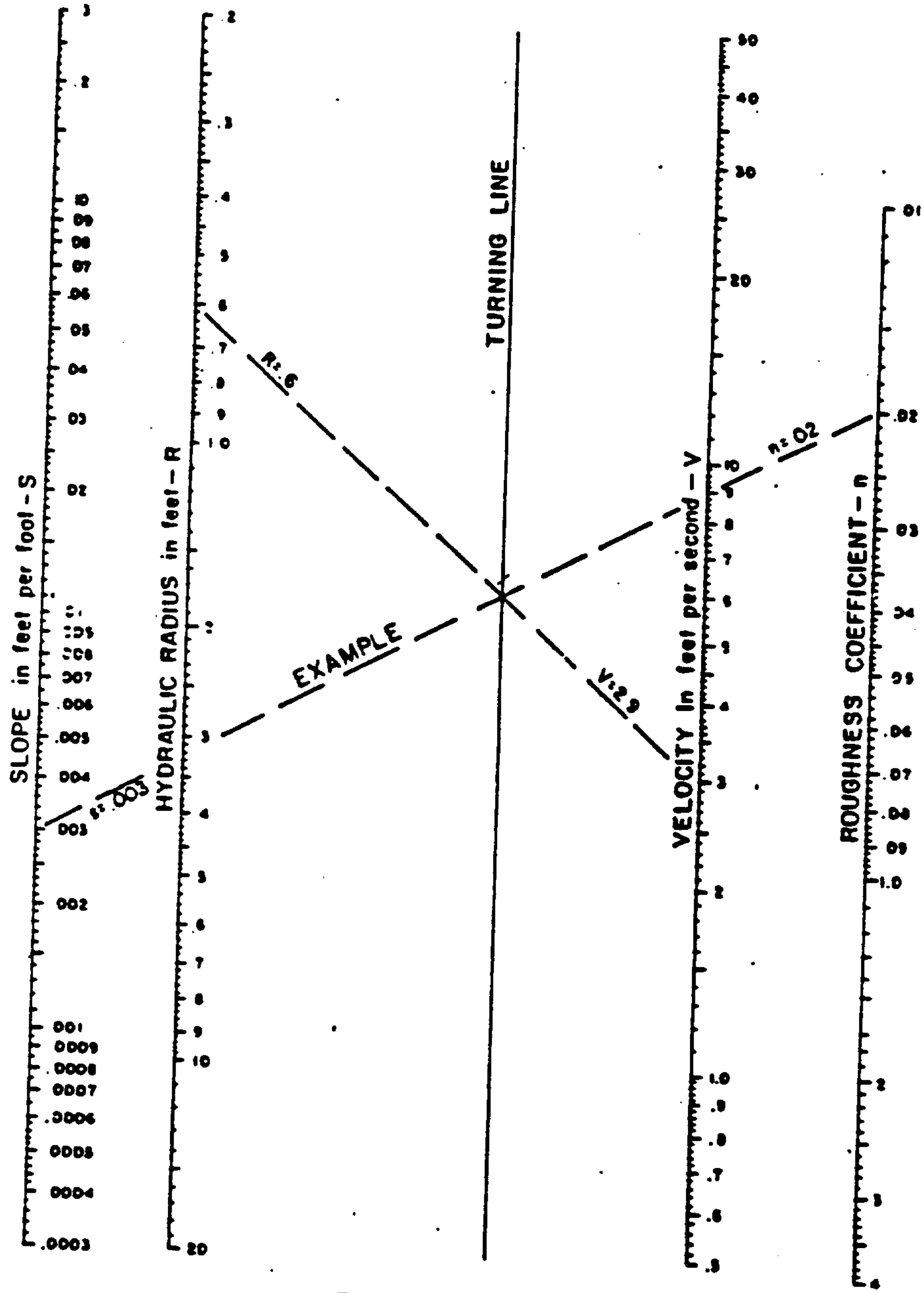


FIGURE 5-2
NOMOGRAPH FOR SOLUTION
OF MANNING EQUATION

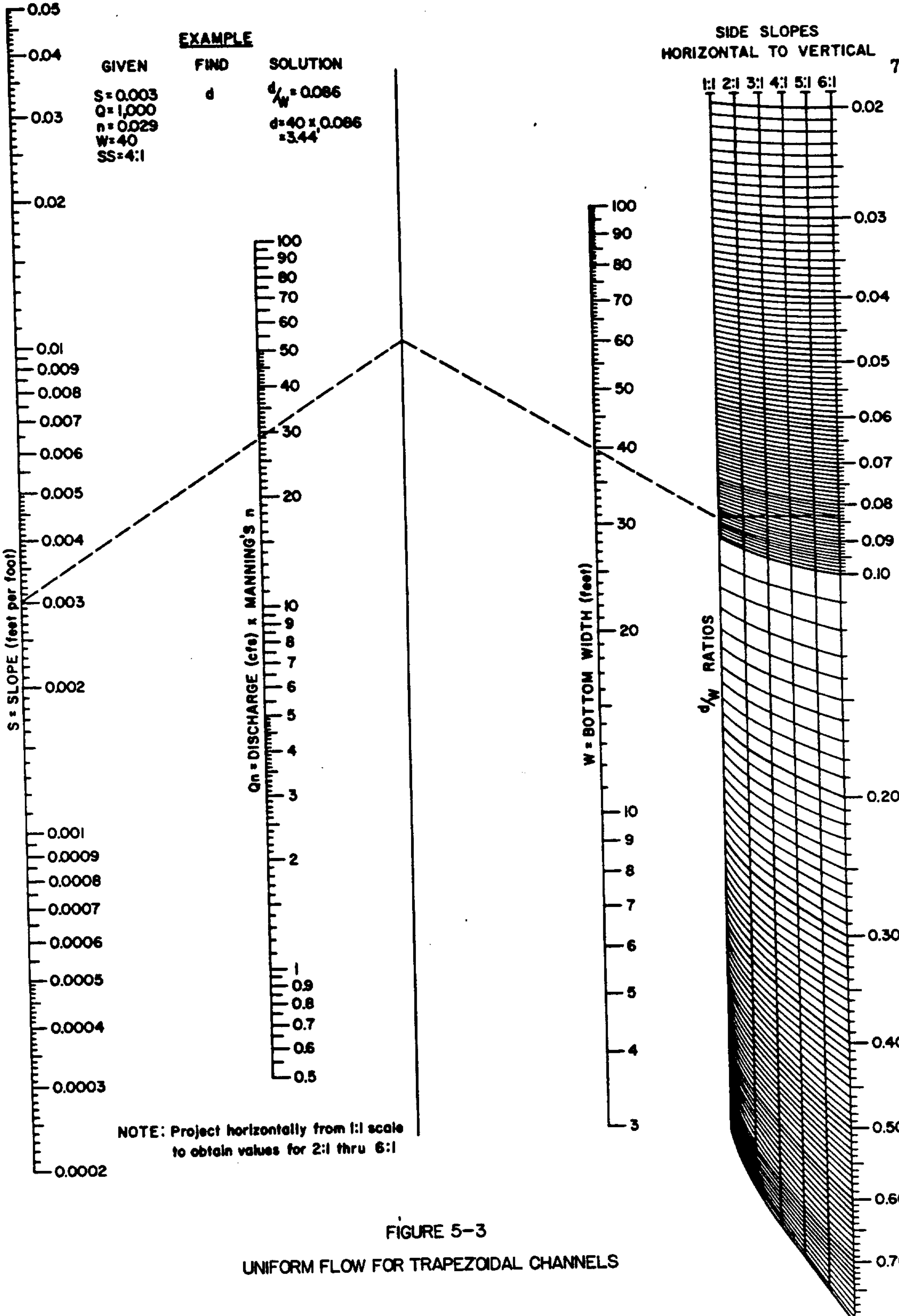


FIGURE 5-3

UNIFORM FLOW FOR TRAPEZOIDAL CHANNELS

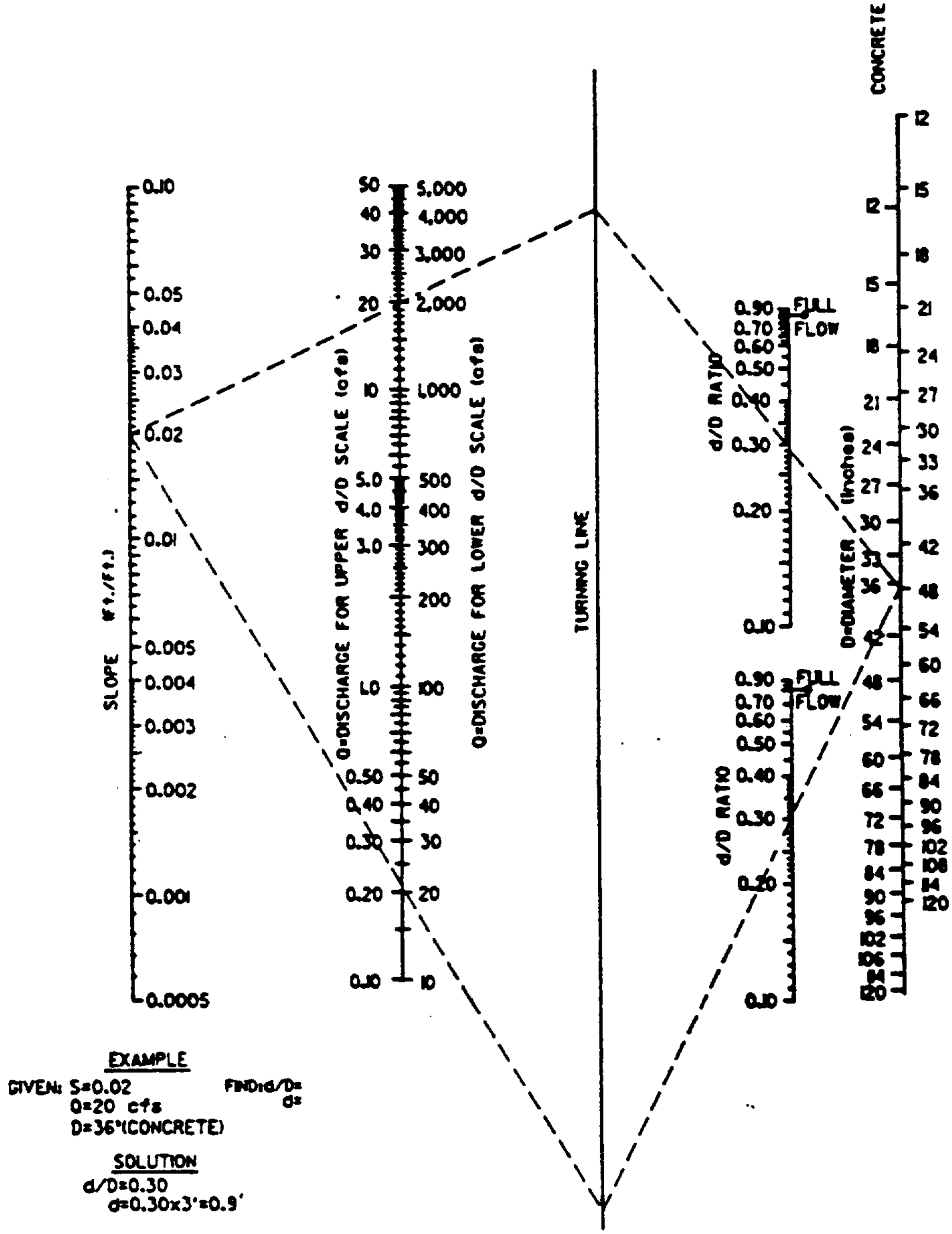
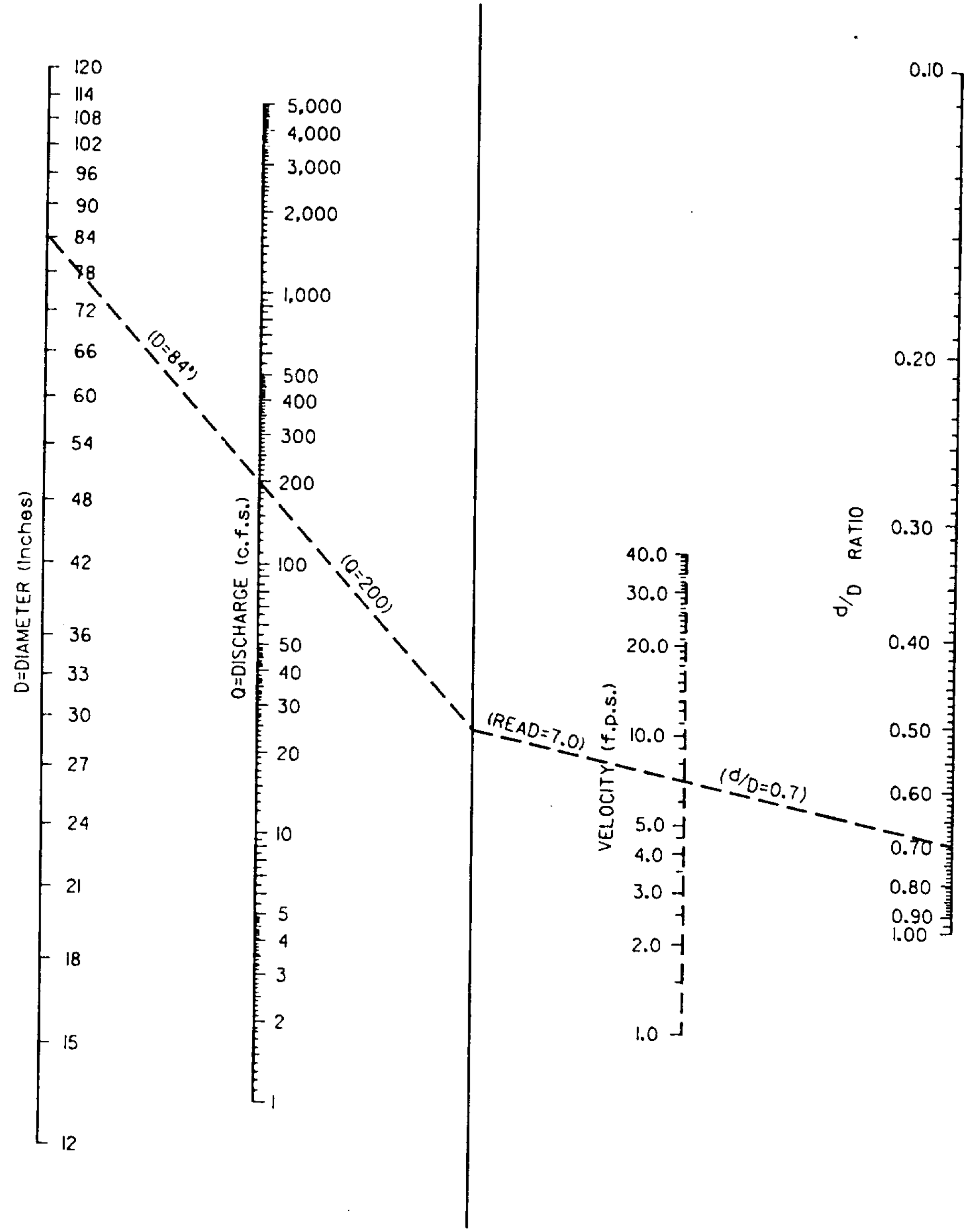


FIGURE 5–4
UNIFORM FLOW FOR PIPE CULVERTS

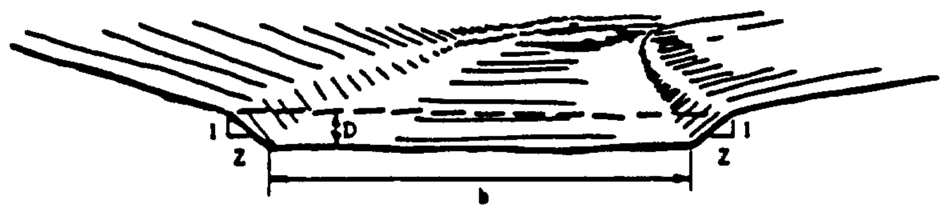


CITY OF LONGVIEW

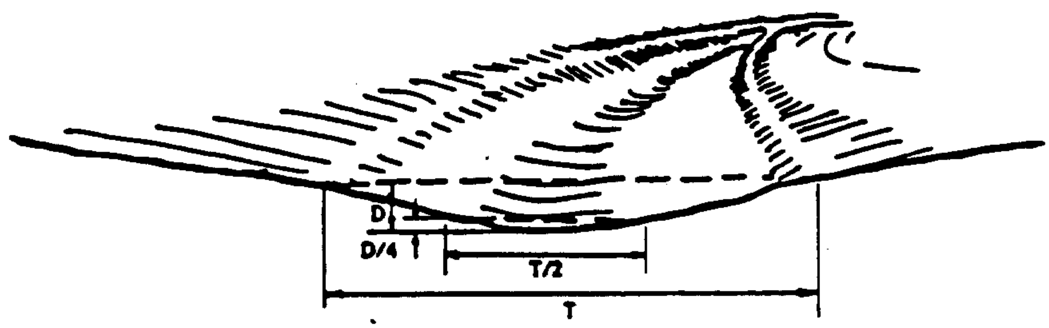
FIGURE 5–5

VELOCITY IN PIPE CONDUITS

GRASS-LINED SWALE



Trapezoidal Cross-Section



Parabolic Cross-Section

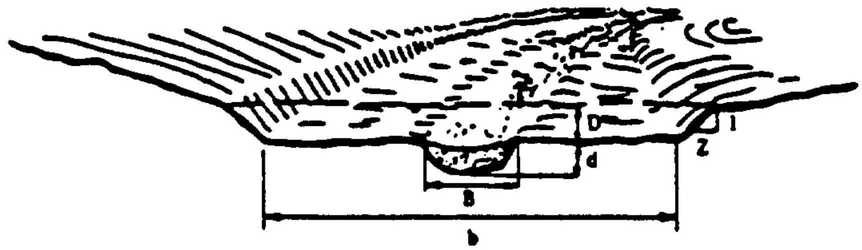
GENERAL NOTES:

1. Except as noted on construction plans or designated on-site, all trees, brush, stumps, obstructions, and other objectionable material shall be removed and disposed of so as not to interfere with the proper functioning of the waterway.
2. The waterway shall be excavated or shaped to line, grade, and cross-section, as required to meet the criteria specified herein, and be free of bank projections or other irregularities which will impede normal flow.
3. Fills shall be compacted as needed to prevent unequal settlement that would cause damage in the completed waterway.
4. All earth removed and not needed in construction shall be spread or disposed of so that it will not interfere with the functioning of the waterway.
5. Stabilization shall be DONE AS FOLLOWS:
   a. For design velocities of less than 3.5 feet per second, seeding and mulching may be used for the establishment of the vegetation. It is recommended that, when conditions permit, temporary diversions or other means should be used to prevent water from entering the waterway during the establishment of the vegetation.
   b. For design velocities of more than 3.5 feet per second, the waterway shall be stabilized with sod, with seeding protected by jute or other blanket products, including temporary diversion of the water until the vegetation is established.

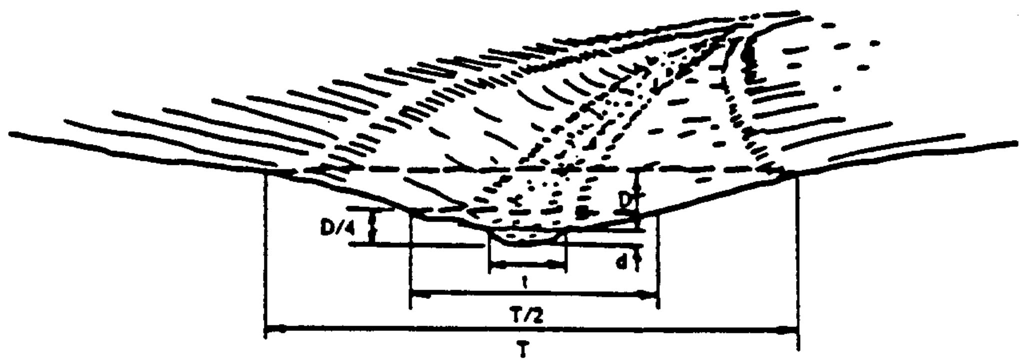
FIGURE 5-6

GRASS-LINED SWALE

GRASS-LINED SWALE WITH STONE CENTER



Trapezoidal Cross-Section



Parabolic Cross-Section

GENERAL NOTES:

1. Except as noted on construction plans or designated on-site, all trees, brush, stumps, obstructions, and other objectionable material shall be removed and disposed of so as not to interfere with the proper functioning of the waterway.
2. The waterway shall be excavated or shaped to line, grade, and cross-section, as required to meet the criteria specified herein, and be free of bank projections or other irregularities which will impede normal flow.
3. Fills shall be compacted as needed to prevent unequal settlement that would cause damage in the completed waterway.
4. All earth removed and not needed in construction shall be spread or disposed of so that it will not interfere with the functioning of the waterway.
5. Stabilization shall be done AS FOLLOWS:
   a. For design velocities of less than 3.5 feet per second, seeding and mulching may be used for the establishment of the vegetation. It is recommended that when conditions permit, temporary diversions or other means should be used to prevent water from entering the waterway during the establishment of the vegetation.
   b. For design velocities of more than 3.5 feet per second, the waterway shall be stabilized with sod, with seeding protected by jute or other blanket products, including temporary diversion of the water until the vegetation is established.
6. Low flow channels shall be constructed as shown above. The low flow section shall be stabilized with concrete, stone or a gabion mattress.

FIGURE 5-7
GRASS-LINED SWALE
WITH STONE CENTER

Location	Area (acres)	Q100 (cfs)	Typical Section	Channel Slope (So)	Manning's "n" Value	Velocity (fps)	Depth (ft)	Lining Depth	Type Lining	Remarks

FIGURE 5-8

CHANNEL DESIGN FORM

TABLE 5-1

MAXIMUM DESIGN FLOW VELOCITIES
FOR OPEN CHANNELS

Channel Lining Type	Maximum Velocity (ft/sec)
Grass*	8.0
Concrete	15.0
R-Rap	15.0
Concrete Pavers	15.0
Stone Riprap	15.0
Gabions	15.0

* For design velocities of less than 3.5 feet per second, seeding and mulching may be used for the establishment of the vegetation. It is recommended that when conditions permit, temporary diversions or other means should be used to prevent water from entering the waterway.

For design velocities of more than 3.5 feet per second, the waterway should be stabilized with sod, with seeding protected by jute, Excellsior Mat, Enkamat or with other comparable blanket products, including temporary diversion of the water until the vegetation is established.

All sideslope vegetation and associated vegetation stabilization or protection materials shall be adequately staked to the channel sideslope.

TABLE 5-2

VALUES OF THE MANNING ROUGHNESS COEFFICIENT - (n)

Type of Channel and Description	Minimum	Normal	Maximum
A. Lined or Built-up Channels			
A1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
A2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.015	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	--
8. On irregular excavated rock	0.022	0.027	--
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035

TABLE 5-2 (Cont'd)

			Type of Channel and Description	Minimum	Normal	Maximum
	h.		Dressed ashlar	0.013	0.015	0.017
		1.	Smooth	0.013	0.013	--
		2.	Rough	0.016	0.016	--
	j.		Vegetal lining	0.030	--	0.500
B.			Excavated or Dredged			
	a.		Earth, straight and uniform			
		1.	Clean, recently completed	0.016	0.018	0.020
		2.	Clean, after weathering	0.018	0.022	0.025
		3.	Gravel, uniform section, clean	0.022	0.025	0.030
		4.	With short grass, few weeds	0.022	0.027	0.033
	b.		Earth, winding and sluggish			
		1.	No vegetation	0.023	0.025	0.030
		2.	Grass, some weeds	0.025	0.030	0.033
		3.	Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
		4.	Earth bottom and rubble sides	0.028	0.030	0.035
		5.	Stony bottom and weedy banks	0.025	0.035	0.040
			Cobble bottom and clean sides	0.030	0.040	0.050
	c.		Dragline - excavated or dredged			
		1.	No vegetation	0.025	0.028	0.033
		2.	Light brush or banks	0.035	0.050	0.060
	d.		Rock cuts			
		1.	Smooth and uniform	0.025	0.035	0.040
		2.	Jagged and irregular	0.035	0.040	0.050
	e.		Channels not maintained, weeds and brush uncut			
		1.	Dense weeds, high as flow depth	0.050	0.080	0.120
		2.	Clean bottom, brush on sides	0.040	0.050	0.080
		3.	Same, highest stage of flow	0.045	0.070	0.110
		4.	Dense brush, high stage	0.080	0.100	0.140
C.			Natural Streams			
C1.			Minor streams (top width at flood stage <100 ft)			
	a.		Streams on plain			
		1.	Clean, straight, full stage no rifts or deep pools	0.025	0.030	0.033
		2.	Same as above, but more stones and weeds	0.030	0.035	0.040
		3.	Clean, winding, some pools and shoals	0.033	0.040	0.045
		4.	Same as above, but some weeds and stones	0.035	0.045	0.050
		5.	Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055

TABLE 5-2 (Cont'd)

Type of Channel and Description	Minimum	Normal	Maximum
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
C2. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160

TABLE 5-2 (Concluded)

Type of Channel and Description	Minimum	Normal	Maximum
C3. Major streams (top width at flood stage >100 ft). The n value is less than that for minor streams of similar description because banks offer less effective resistance			
a. Regular section with no boulders or brush	0.025	--	0.060
b. Irregular and rough section	0.035	--	0.100

TABLE 5-3

COMPUTATION OF COMPOSITE ROUGHNESS COEFFICIENT
FOR EXCAVATED AND NATURAL CHANNELS

$$n = (n^o + n^1 + n^2 = n^3 + n^4)\ m$$

	Channel Conditions	Value
Material Involved n^o	Earth	0.020
	Rockcut	0.025
	Fine Gravel	0.024
	Coarse Gravel	0.028
Degree of Irregularity n^1	Smooth	0.000
	Minor	0.005
	Moderate	0.010
	Severe	0.020
Variation of Channel Cross-Section n^2	Gradual	0.000
	Alternating Occasionally	0.005
	Alternating Frequently	0.010-0.015
Relative Effect of Obstructions n^3	Negligible	0.000
	Minor	0.010-0.015
	Appreciable	0.020-0.030
	Severe	0.040-0.060
Vegetation n^4	Low	0.005-0.010
	Medium	0.010-0.025
	High	0.025-0.050
	Very High	0.050-0.100
Degree of Meandering m	Minor	1.000
	Appreciable	1.150
	Severe	1.300

Source: Ven Te Chow, Open Channel Hydraulics, McGraw-Hill, 1959.

CITY OF LONGVIEW

29 FOOT STREET - DRAINAGE CURVES

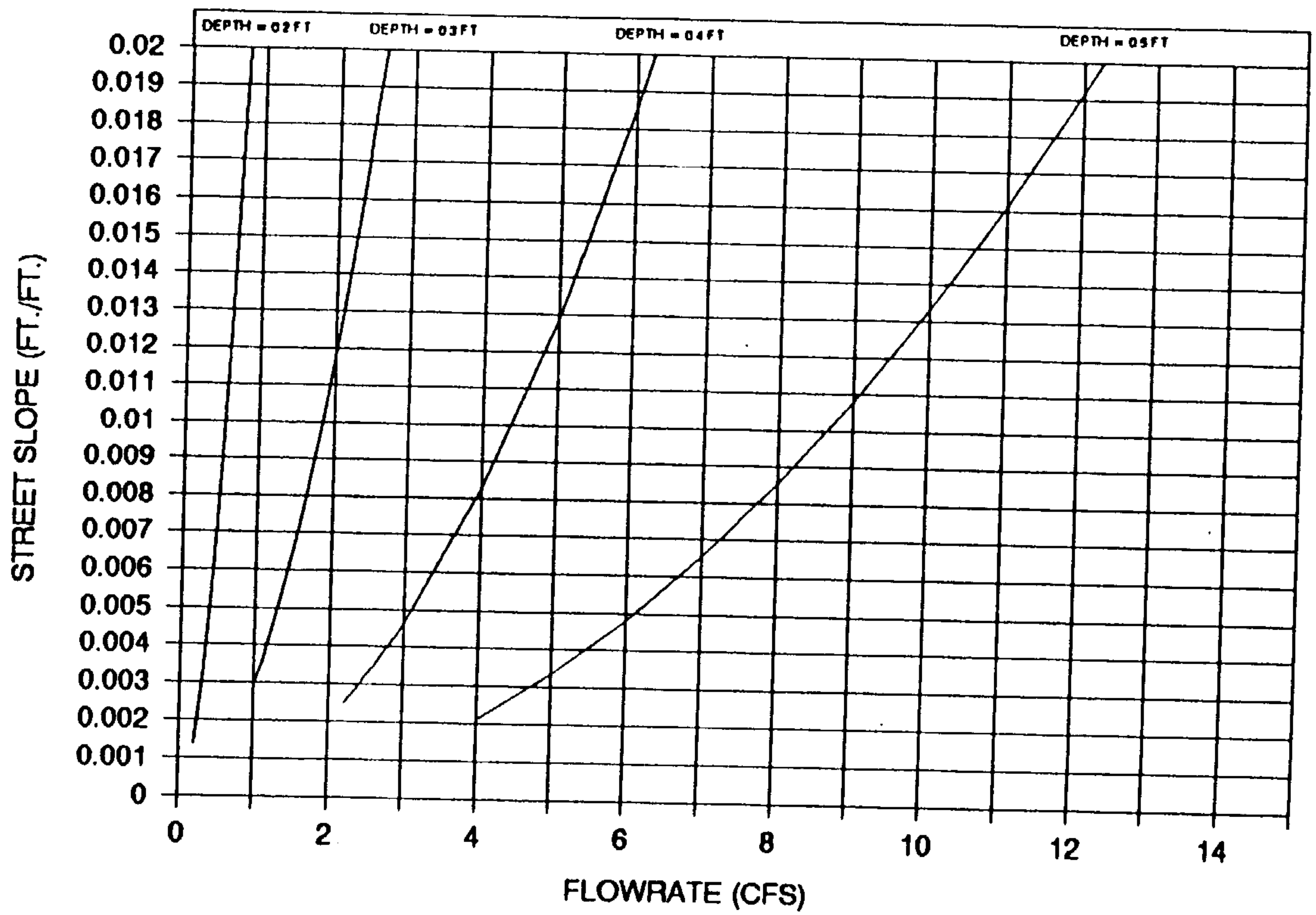


FIGURE 6-I-A

CITY OF LONGVIEW
29 FOOT STREET – DRAINAGE CURVES

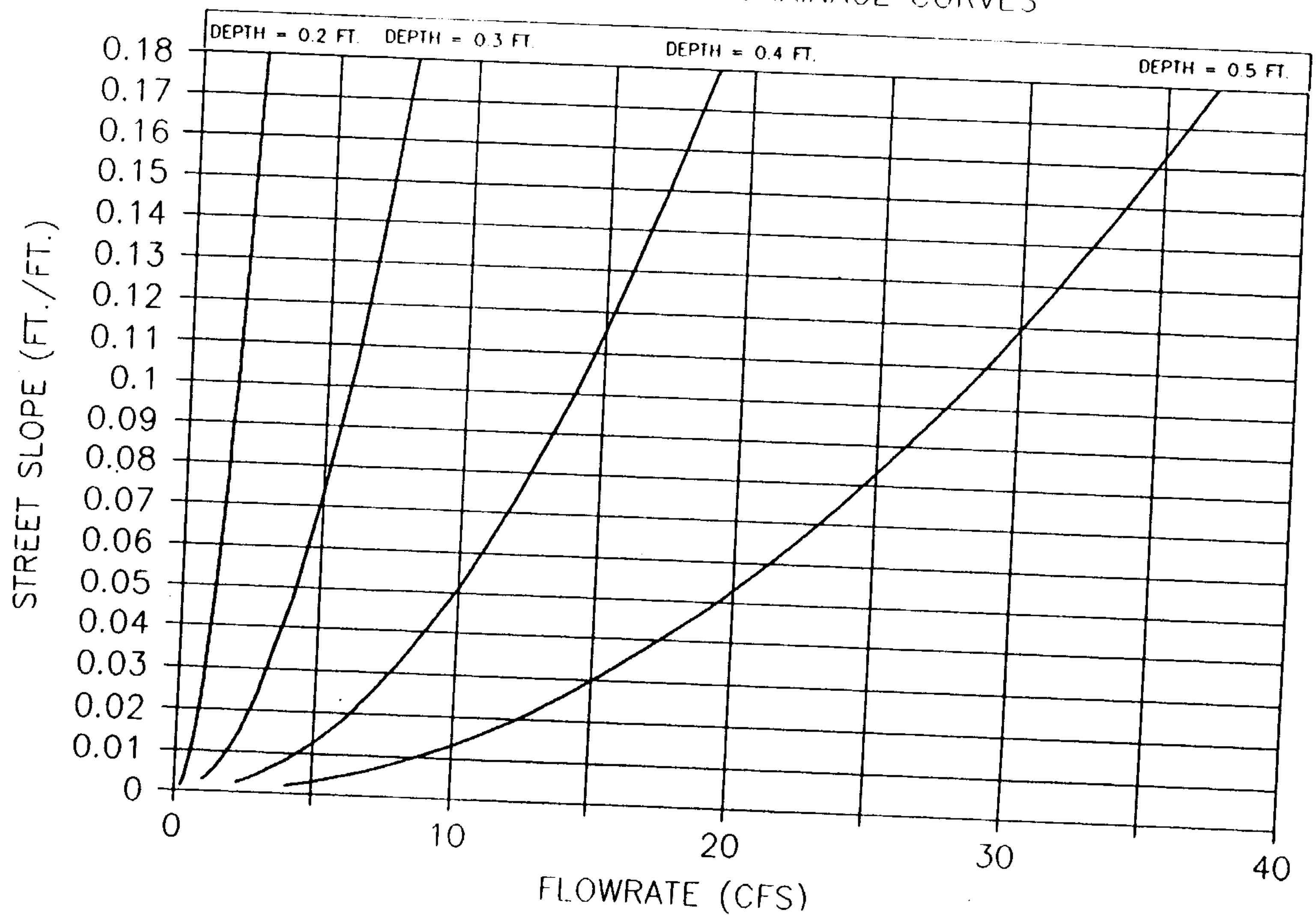


FIGURE 6-1-B

CITY OF LONGVIEW

37 FOOT STREET - DRAINAGE CURVES

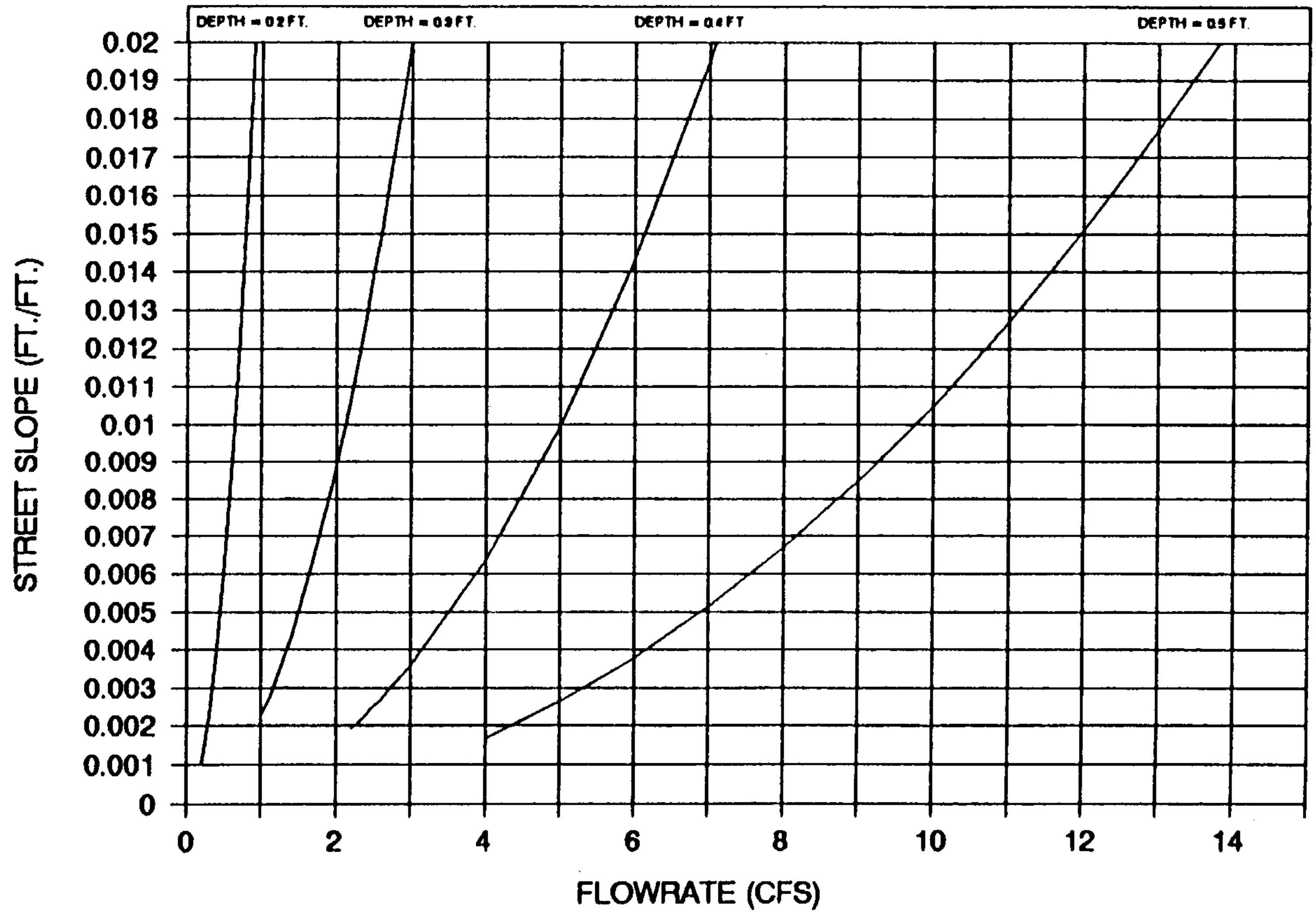


FIGURE 6- I-C

CITY OF LONGVIEW
37 FOOT STREET – DRAINAGE CURVES

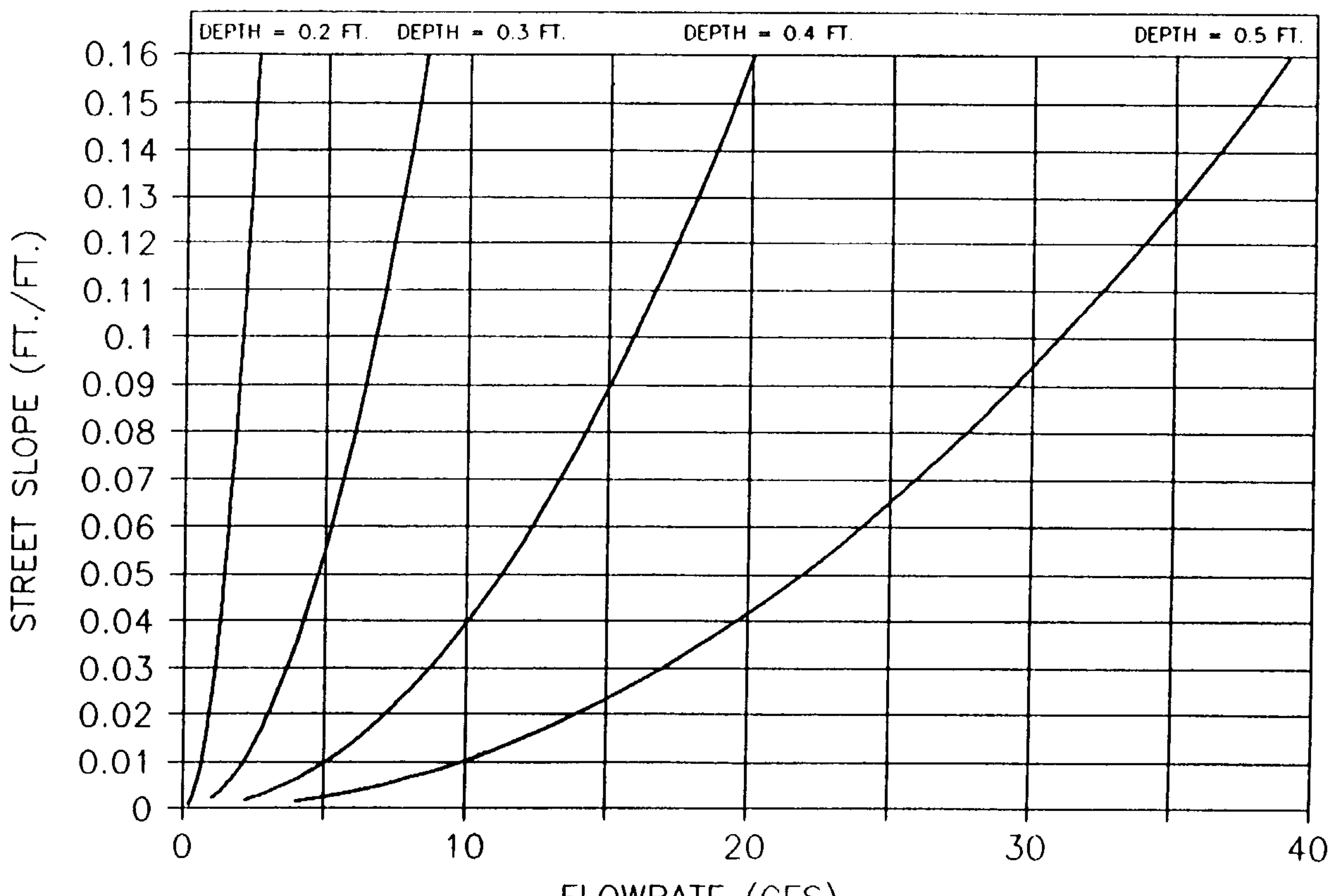


FIGURE 6-1-D

CITY OF LONGVIEW

53 OR 57 FOOT STREETS - DRAINAGE CURVES

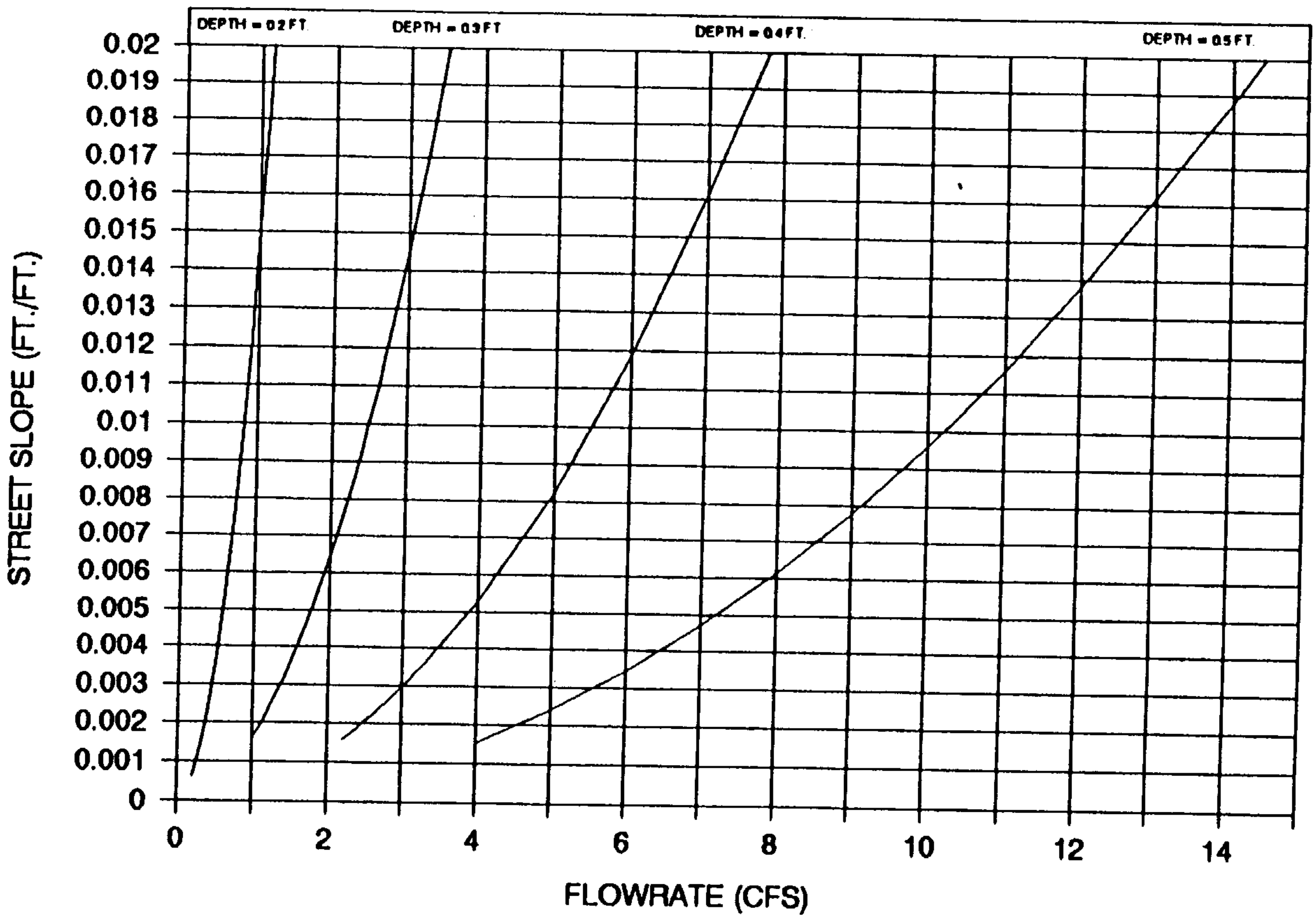


FIGURE 6-I-E

CITY OF LONGVIEW
53 OR 57 FOOT STREETS – DRAINAGE CURVES

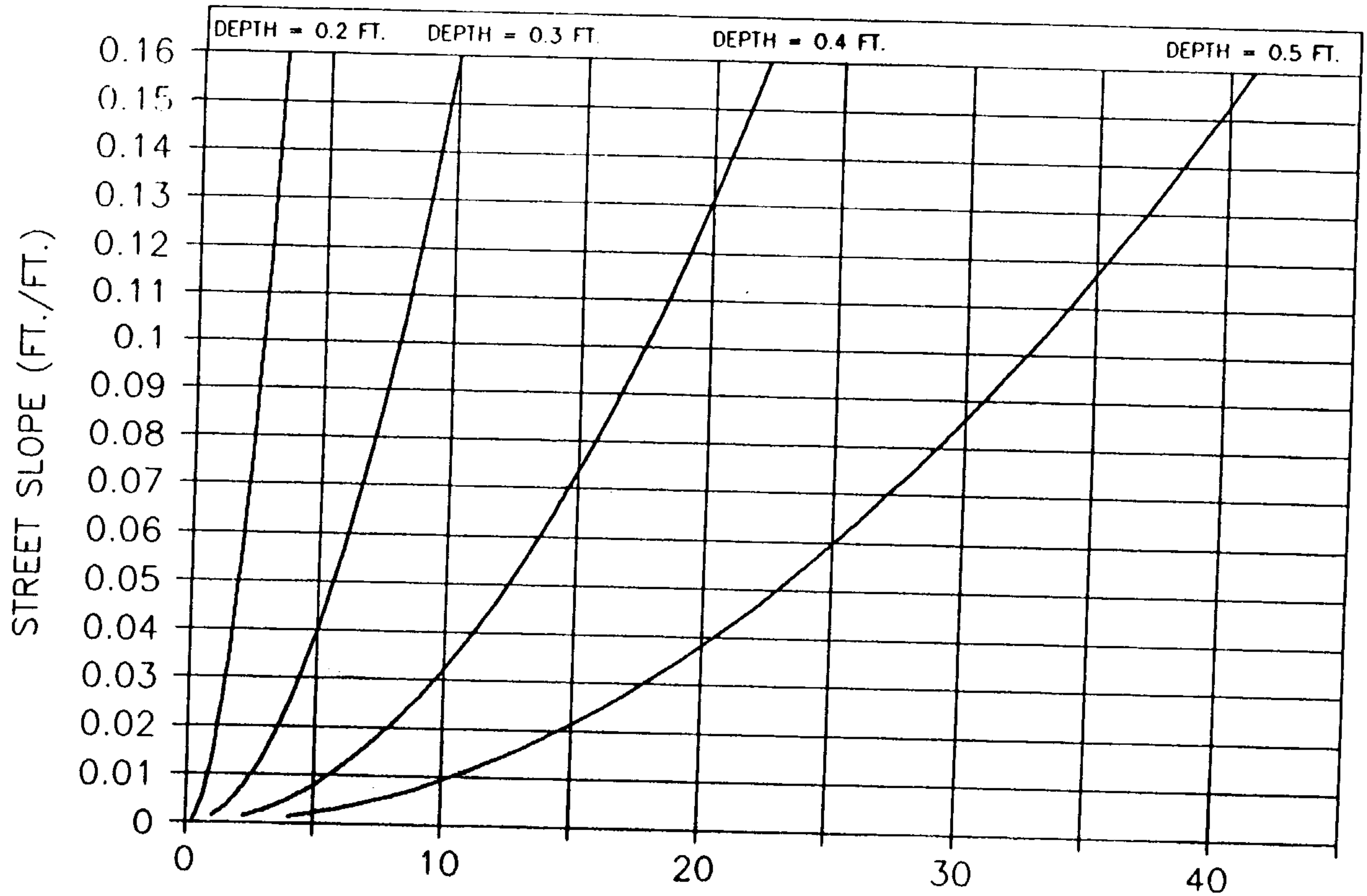


FIGURE 6-1-F

CITY OF LONGVIEW

65 FOOT STREET - DRAINAGE CURVES

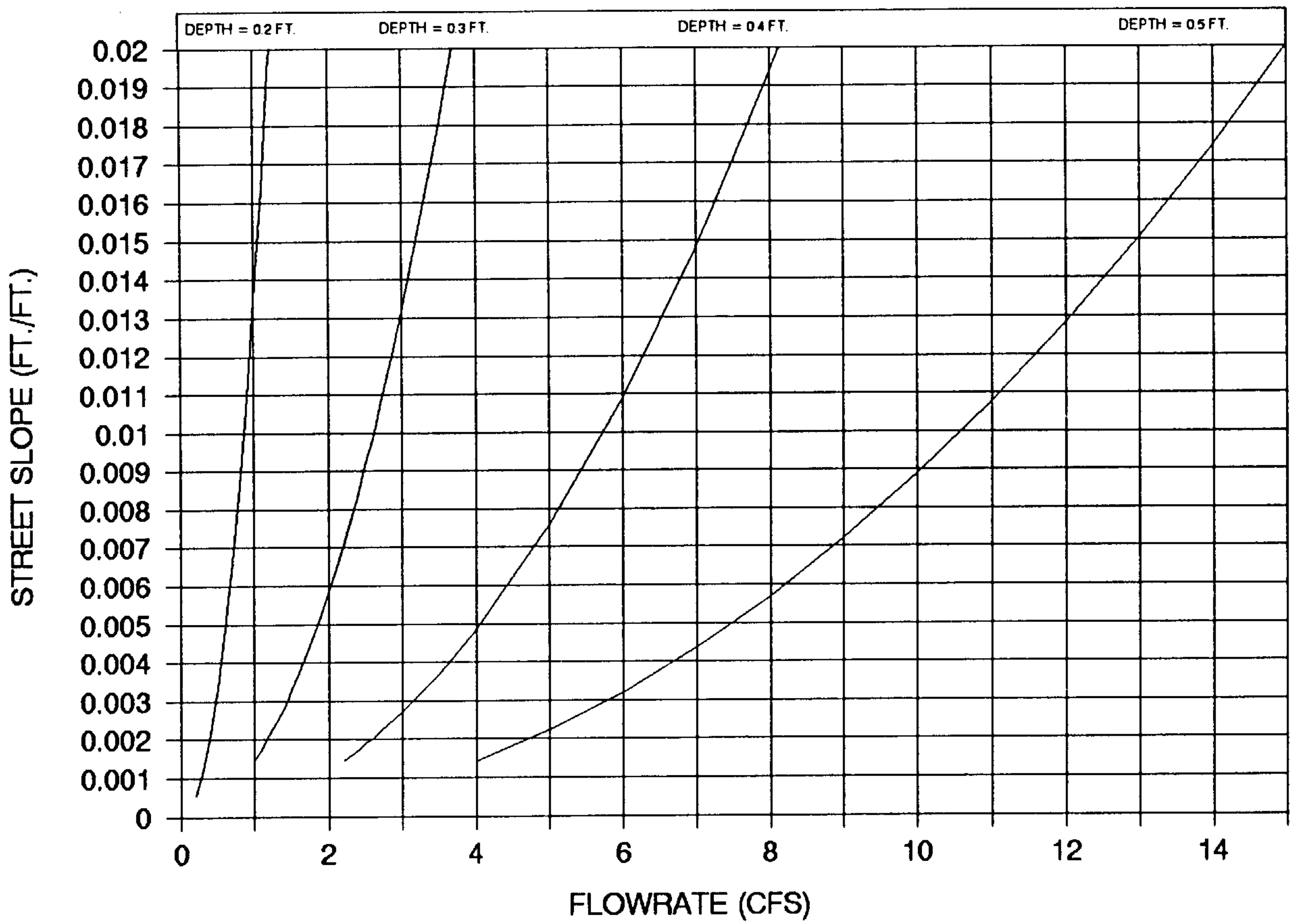


FIGURE 6-1-G

CITY OF LONGVIEW
65 FOOT STREET – DRAINAGE CURVES

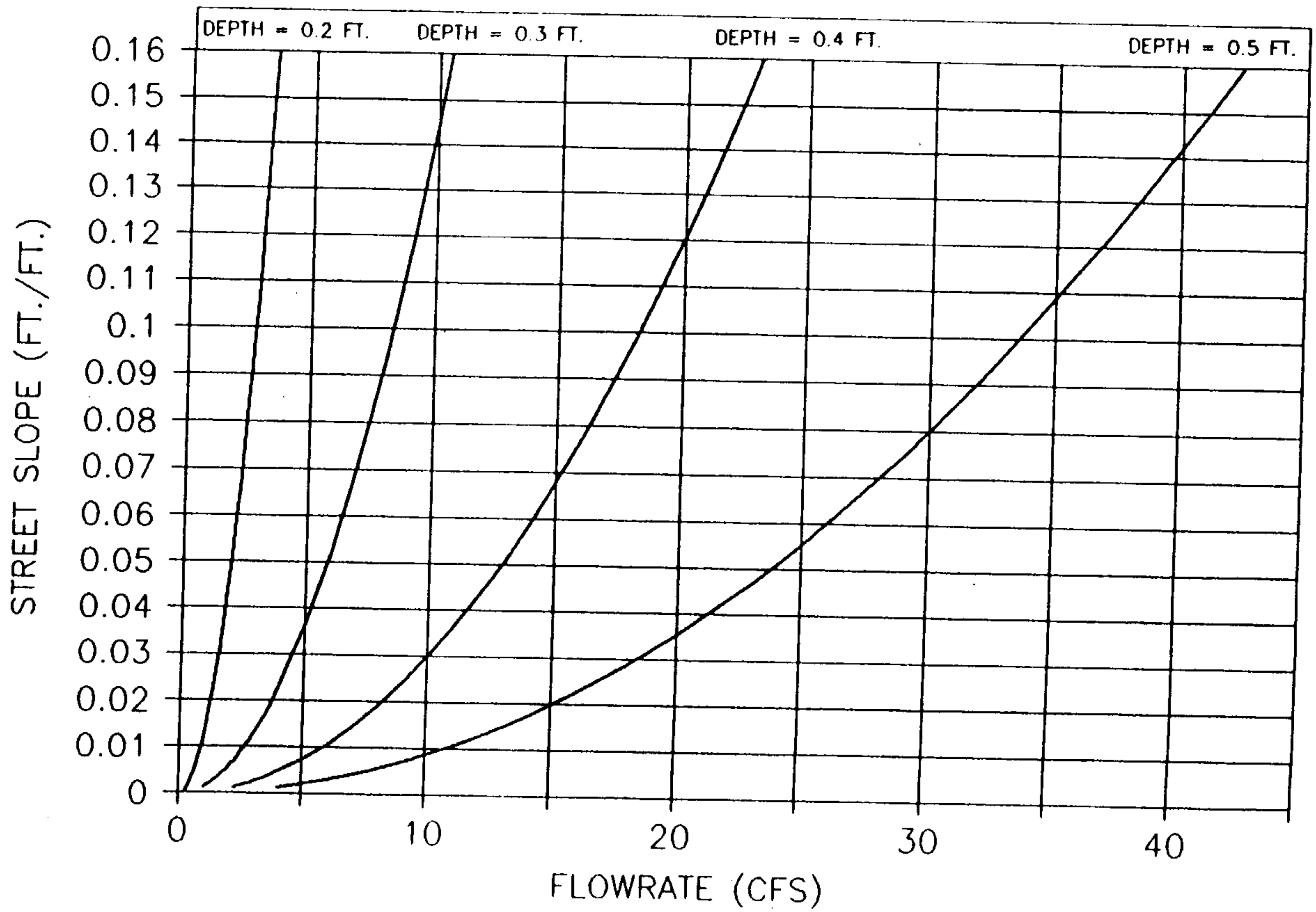


FIGURE 6-I-H

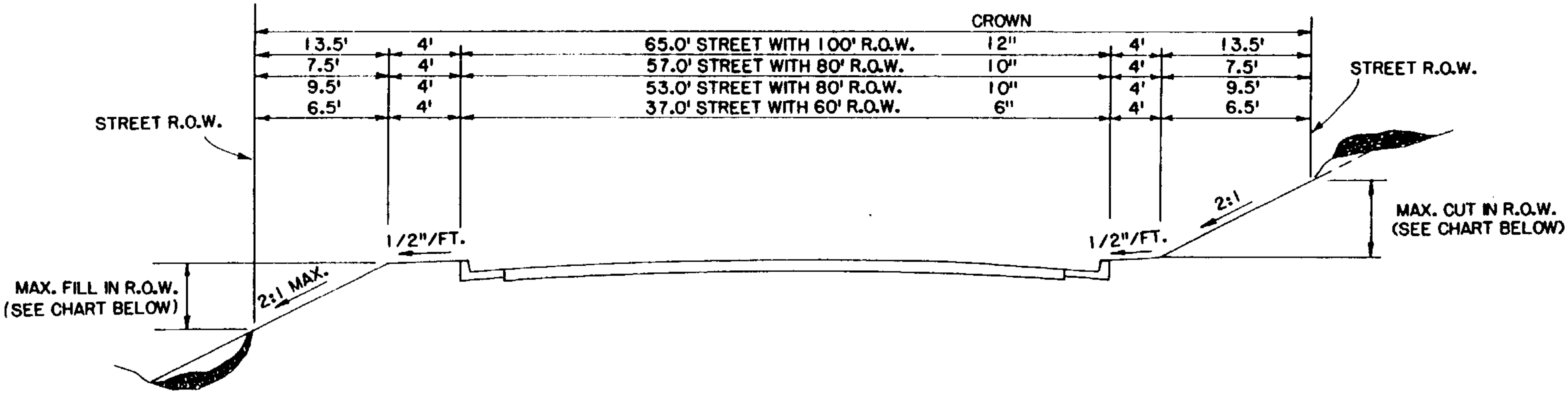


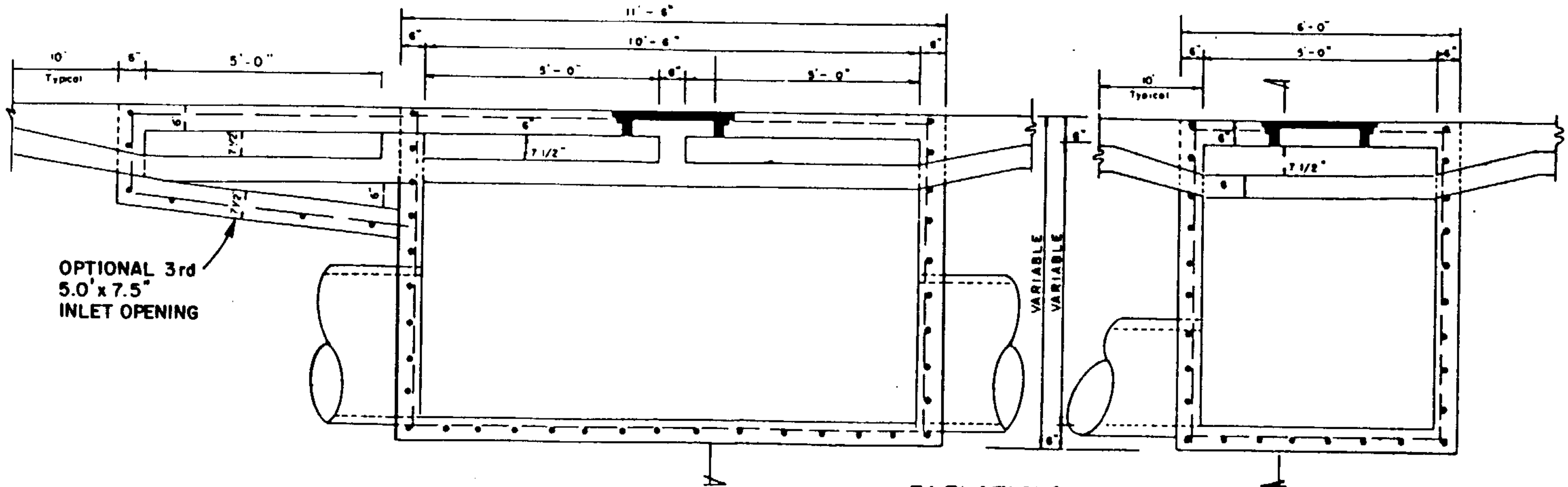
CHART OF MAXIMUM CUT or FILL WITHIN STREET R.O.W WITH MAX 2:1 BACK SLOPE			
STREET WIDTH	STREET ROW WIDTH	MAX CUT OR FILL NOT TO EXCEED ROW WIDTH at 2:1 max slope	
		With 4' Sidewalk at B C	Without 4' Walk at B C
29	50	3.25	5.25
33	60	4.75	6.75
37	60	3.75	5.75
57	80	3.75	5.75
50	60	0.50	2.50

NOTE When above cuts or fills are exceeded for 2:1 Max back slope, slope easements will be required adjacent to street Right-Of-Way

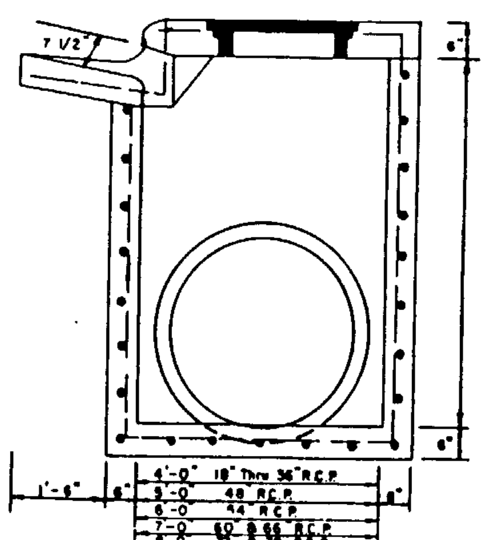
CITY OF LONGVIEW

FIGURE 6–2

STANDARD STREET SECTION



ELEVATIONS



SECTION AA

ALL REINFORCING STEEL TO BE 1/2" Ø DEFORMED BARS, 6" ON CTR., AND MAY BE CUT IN THE FIELD TO FIT.

CITY OF LONGVIEW

FIGURE 6-3

STANDARD INLET DESIGN

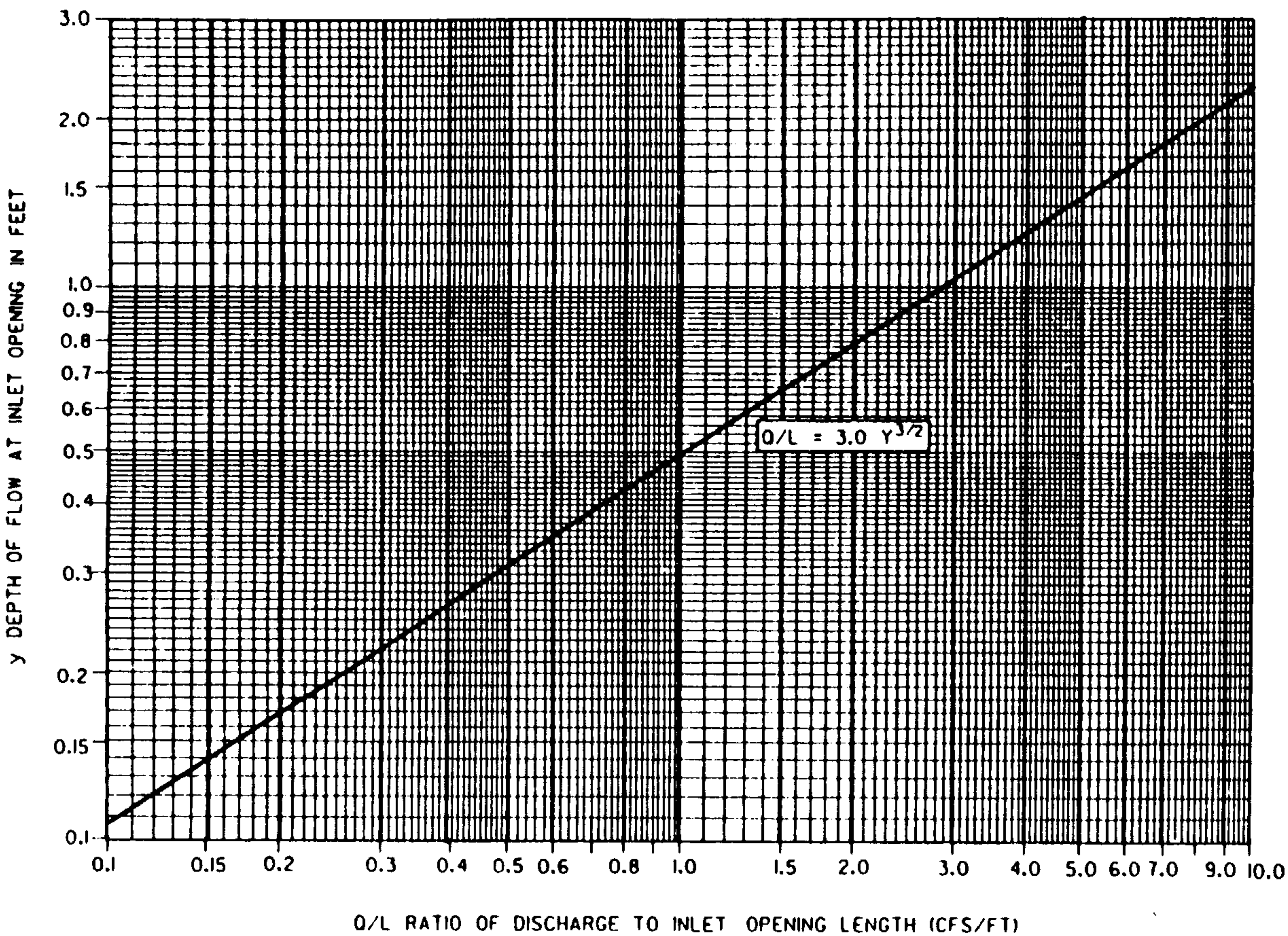


FIGURE 6-4
INLET CAPACITY

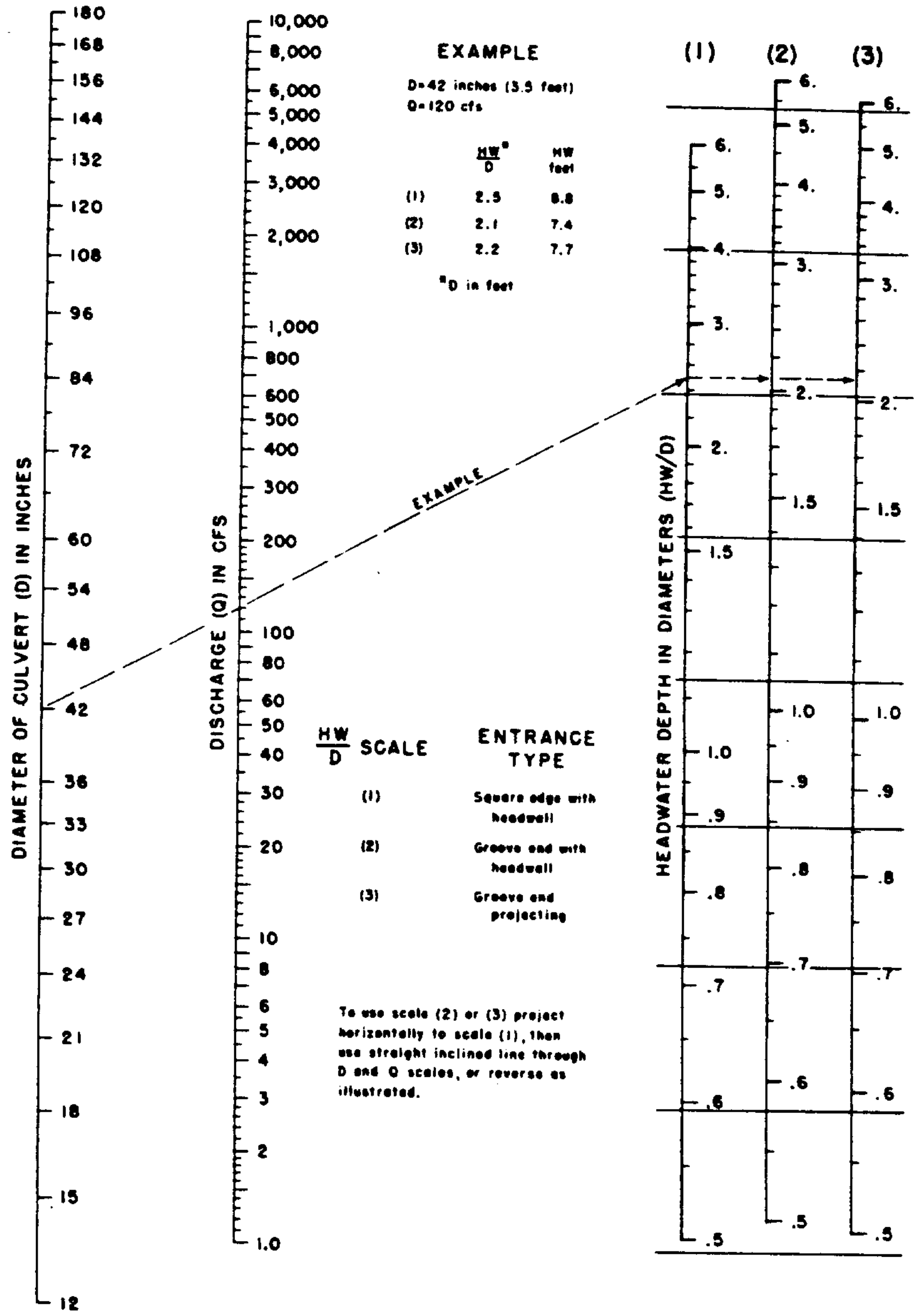
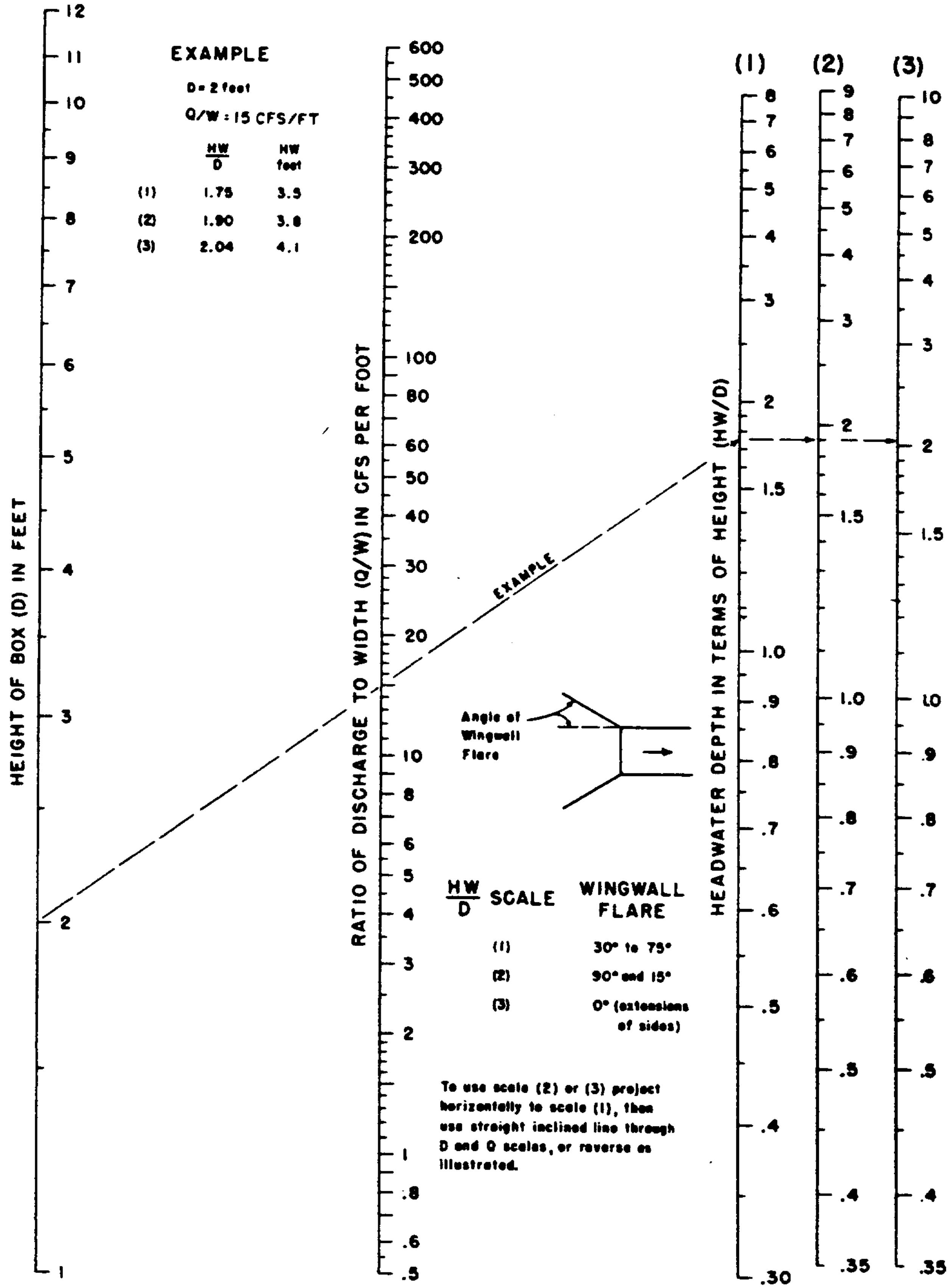


CHART 7-1
HEADWATER DEPTH FOR CONCRETE
PIPE CULVERTS WITH INLET CONTROL



FHWA

CHART 7–2
HEADWATER DEPTH FOR BOX
CULVERTS WITH INLET CONTROL

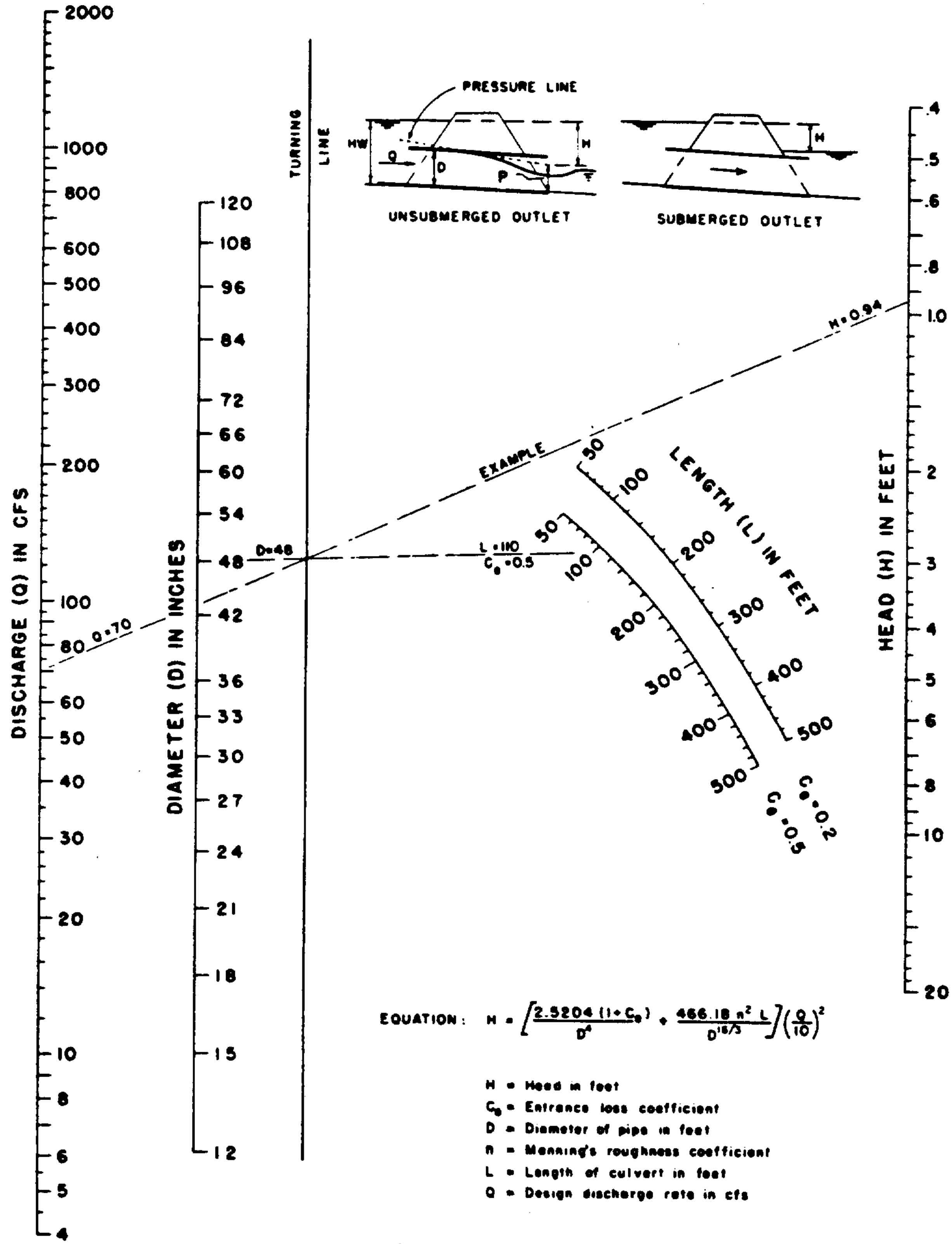


CHART 7–3

HEAD FOR CONCRETE PIPE CULVERTS
FLOWING FULL

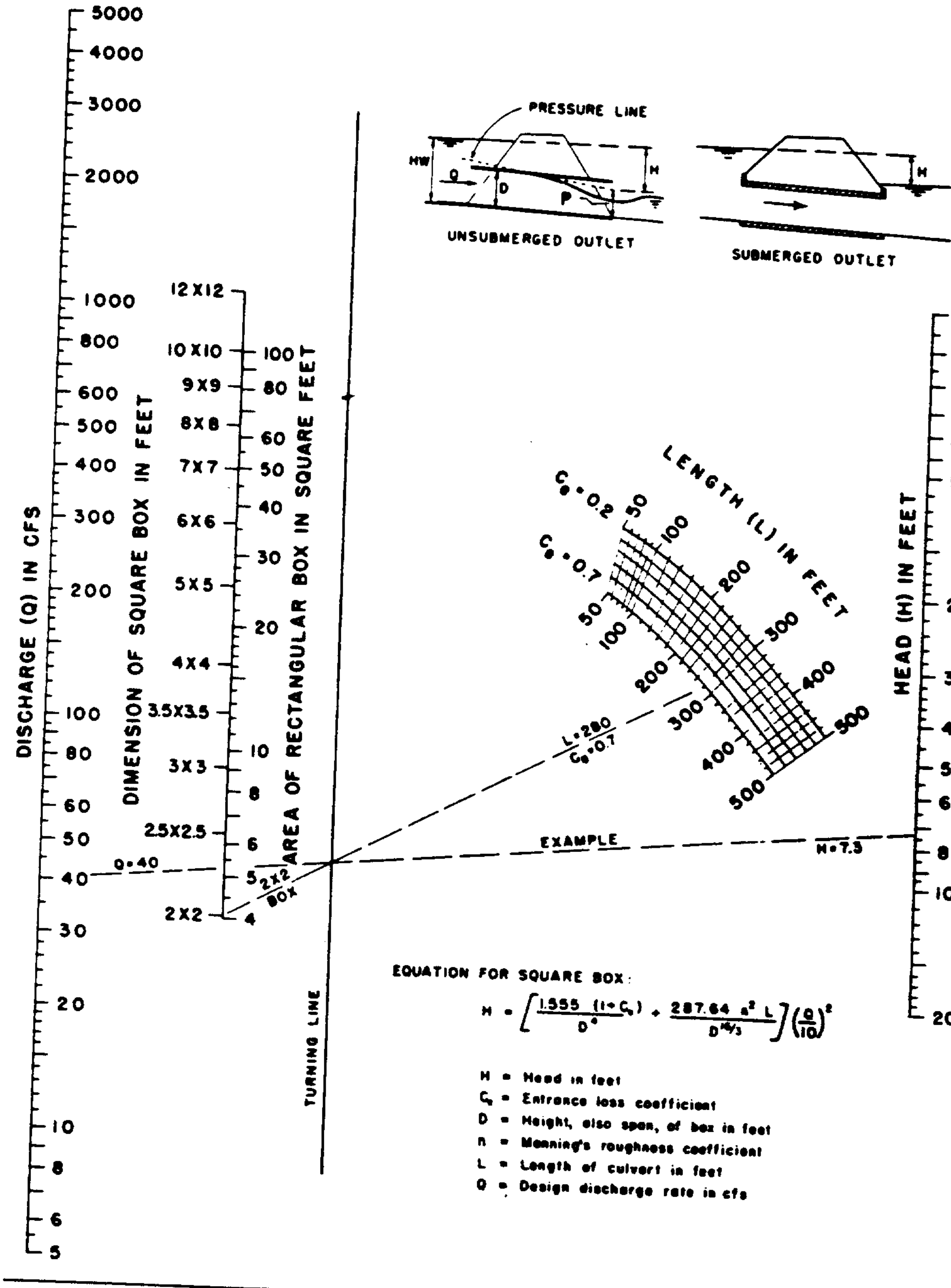


CHART 7–4
HEAD FOR CONCRETE BOX CULVERTS
FLOWING FULL

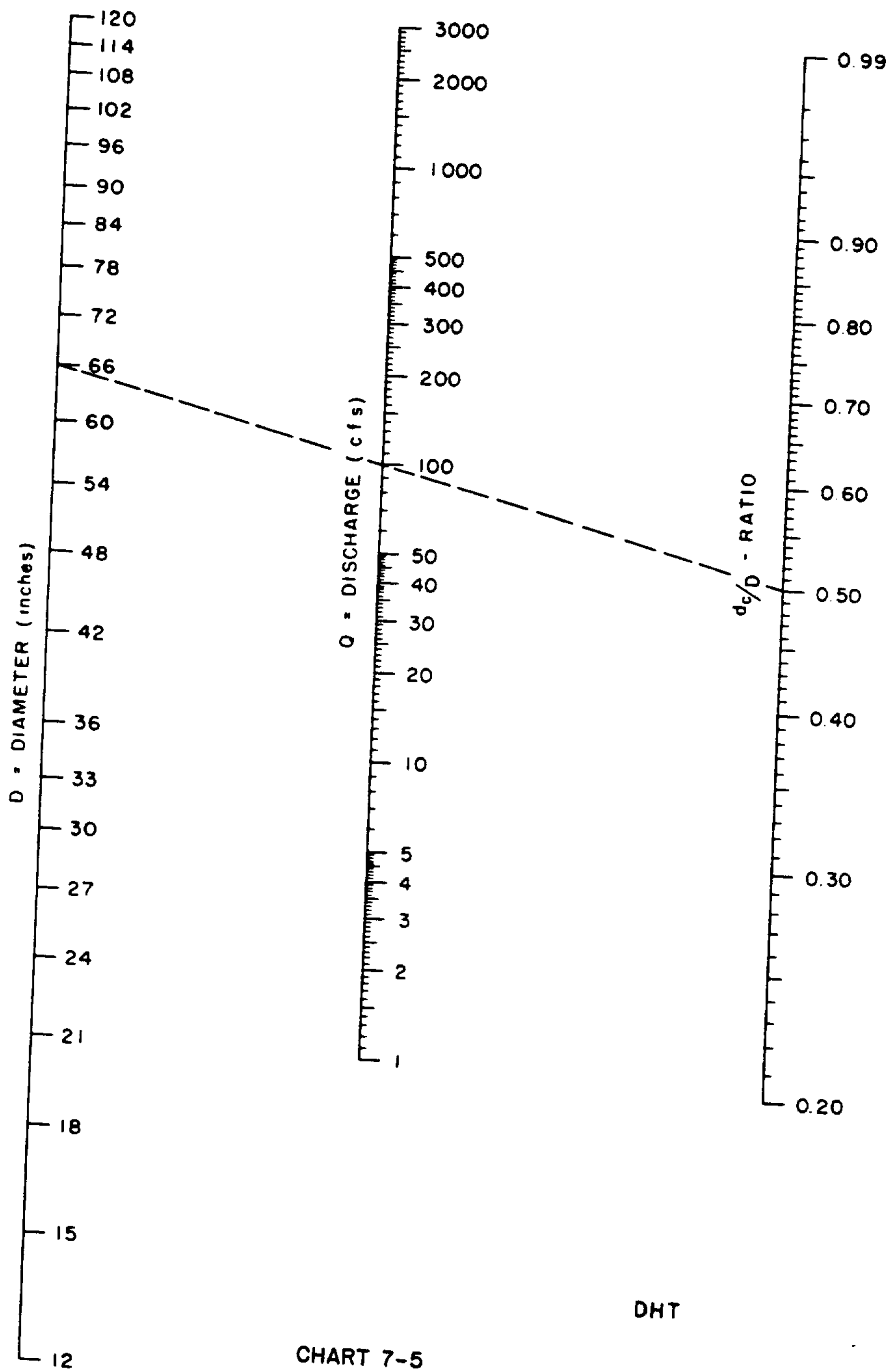
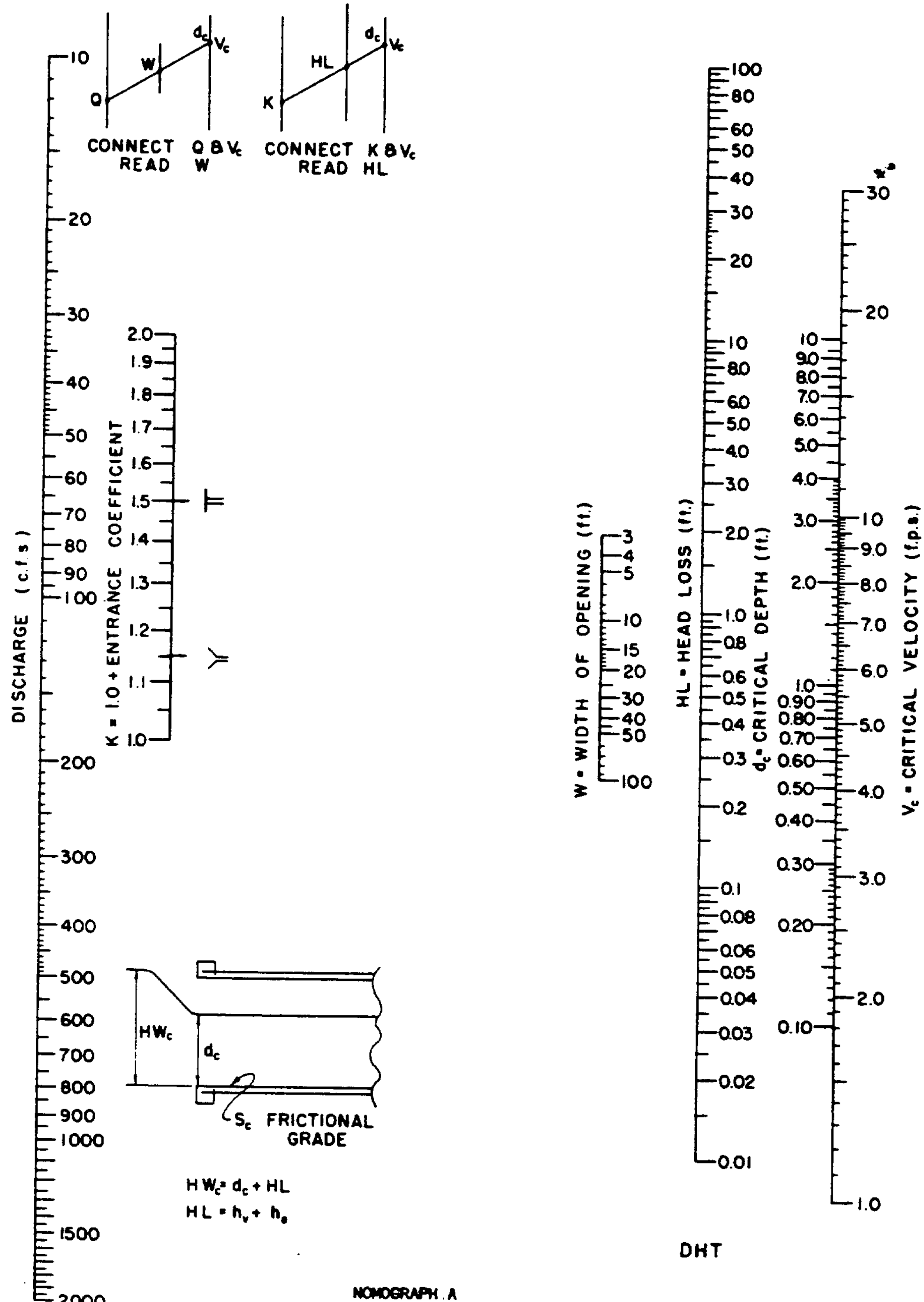


CHART 7-5
CRITICAL DEPTH OF FLOW
FOR CIRCULAR CONDUITS



NOMOGRAPH A

CHART 7-6

CRITICAL FLOW FOR BOX CULVERTS

PROJECT :

DESIGN REQUIREMENTS						SELECTED CULVERT DESIGN			
CULVERT LOCATION	DESIGN DISCHARGE (cfs)	CULVERT LENGTH (ft)	CULVERT SLOPE (ft/ft)	ALLOWABLE HEADWATER DEPTH (ft)	TAILWATER DEPTH FOR 100-YEAR FLOW	CULVERT TYPE	CULVERT SIZE	100-YEAR VELOCITY OF CULVERT OUTLET	INLET CONTROL OR OUTLET CONTROL

FIGURE 7-1
CULVERT DESIGN REPORTING SHEET

FIGURE 7-1

CULVERT DESIGN REPORTING SHEET

Project: ______________________________

Culvert: ____________________

Design Requirements

Design Discharge: __________ cfs

Culvert Length: __________ ft

Culvert Slope: __________ ft/ft

Allowable headwater depth _______ ft

Tailwater depth for 100-year flow ________ ft

Selected Culvert Design

Culvert type __________ (box, RCP, etc.)

Culvert size __________

100-year flow velocity at culvert outlet __________

Inlet control or outlet control __________

TABLE 8-1

RUNOFF CURVE NUMBERS FOR SELECTED AGRICULTURAL, SUBURBAN, AND URBAN LAND USES
(ANTECEDENT MOISTURE CONDITION II, $I_a = 0.2S$)

Land Use Description		Hydrologic Soil Group A	B	C	D
Cultivated land[1]:	without conservation treatment	72	81	88	91
	with conservation treatment	62	71	78	81
Pasture or range land:	poor condition	68	79	86	89
	good condition	39	61	74	80
Meadow:	good condition	30	58	71	78
Wood or forest land:	thin stand, poor cover, no mulch	45	66	77	83
	good cover[2]	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.					
	good condition: grass cover on 75% or more of the area	39	61	74	80
	fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (72% impervious)		81	88	91	93
Residential[3]:					
Average lot size:	Average % of impervious[4]:				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5]		98	98	98	98
Streets and roads:					
paved with curbs and storm sewers[5]		98	98	98	98
gravel		76	85	89	91
dirt		72	82	87	89

1 For a more detailed description of agricultural land use curve numbers, refer to Soil Conservation Service, 1972, Chapter 9.

2 Good cover is protected from grazing and litter and brush cover soil.

3 Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

4 The remaining pervious area (lawn) are considered to be in good pasture condition for these curve numbers.

5 In some warmer climates of the country a curve number of 95 may be used.

SOURCE: Applied Hydrology (McGraw-Hill, 1989)

TABLE 8-2

APPROPRIATE VALUES OF THE WEIR COEFFICIENT "C" FOR BROAD-CRESTED WEIRS

Measured head in feet, H	Breadth of crest of weir in feet 0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

Source: Brater and King (McGraw-Hill, 1976).

APPENDIX B

EXAMPLE
DESIGN CALCULATIONS

1. EXAMPLE HYDROLOGIC CALCULATION

The following example is intended as a guide to proper application of hydrologic methodologies required by the City of Longview for smaller development tracts with drainage areas less than 100 acres.

EXAMPLE #1

Task: A 98.4-acre subdivision is to be constructed within the Longview City Limits (see Figure E in this section). The tract will include 8.1 acres of multi-family units, 52.3 acres of medium-density single family residential property, a 14.6-acre commercial tract, and a 23.4-acre natural area to be left undeveloped. The subdivision is to be laid out as presented in Figure E. For the purposes of this example, the engineer's job is to determine the flow rate to design channel improvements along a section of Green Creek and the culvert at the Moore Road bridge (see Figure E1 in this section).

Procedure:

Determine critical drainage area - Since the contributing drainage area does not exhibit an unusual shape, the critical drainage area is assumed to include the entire area draining to the point of interest. The critical drainage area is determined to be 37.6 acres. Of this, 4.6 acres is from offsite drainage. (see Figure E1)

Determine Overland Flow Time: Overland flow distance = 250 ft

Average slope of overland flow = 0.005 ft/ft

From Figure 4-1 (short grass pasture) average velocity of overland flow = 0.5 ft/sec

Overland flow time (time to channelization) = 250/[60 (0.5)] = 8.3 minutes

Determine Time of Channelized Flow: Channelized flow distance has two components.

Storm sewer distance = 370 ft. Open channel distance = 903 ft.

The average slope of the storm sewer is determined to be 0.0075 ft/ft. The average slope of the open channel is determined to be 0.005 ft/ft.

The hydraulic computer routine "Normal Depth in a Circular Channel" is used to estimate average velocity in the storm sewer assuming:

slope = 0.0075 ft/ft

estimated average flow rate = 12 cfs
(from Q = CiA)

pipe size = 24 inches

Manning's "n" value = 0.013

The resulting flow velocity for flow in the storm sewer is 6.5 ft/sec.

The hydraulic computer routine "Normal Depth in a Non-Circular Channel" is used to esimate average velocity in the channel section along Green Creek assuming:

slope = 0.005 ft/ft

estimated average flow rate = 120 cfs
(from Q = CiA)

estimated sideslopes = 2H:1V

estimated bottom width = 4 ft.

estimated Manning's "n" value = 0.040

The resulting flow velocity for flow in Green Creek is 3.8 ft/sec.

Time of channelized flow = 370/[60 (6.5)] + 903/[60 (3.8)] = 0.95 + 3.96 = 4.9 minutes

Determine Time of Concentration: Tc = Overland Flow Time + Time of Channelized Flow = 8.3 + 4.9 = 13.2 minutes

Determine Design Rainfall Intensity: Assume a storm duration equal to the time of concentration (13.2 minutes) and a 100-year storm frequency.

From Figure 4-2, i = 8.0 in/hr.

Determine C Factor: The contributing drainage area is made up of the following land uses with associated areas and C values (Table 4-1):

Land Use	Area	C value from Table 4-1
Single Family (5 units/ac)	21.7 acres	0.70
Multi-family (8 units/ac)	0.5 acres	0.79
Commercial	6.1 acres	0.92
Natural Area	4.7 acres	0.45
Offsite	4.6 acres	0.70

Composite C value:

Single Family $\frac{\text{21.7 acres single family}}{\text{37.6 acres total area}} \times$ 0.70 C value for single family $= 0.404$

Multifamily (0.5/37.6)(0.79) =	0.01
Commercial (6.1/37.6)(0.92) =	0.15
Natural Area (4.7/37.6)(0.45) =	0.06
Offsite (4.6/37.6)(0.70) =	0.086
COMPOSITE C VALUE =	0.72

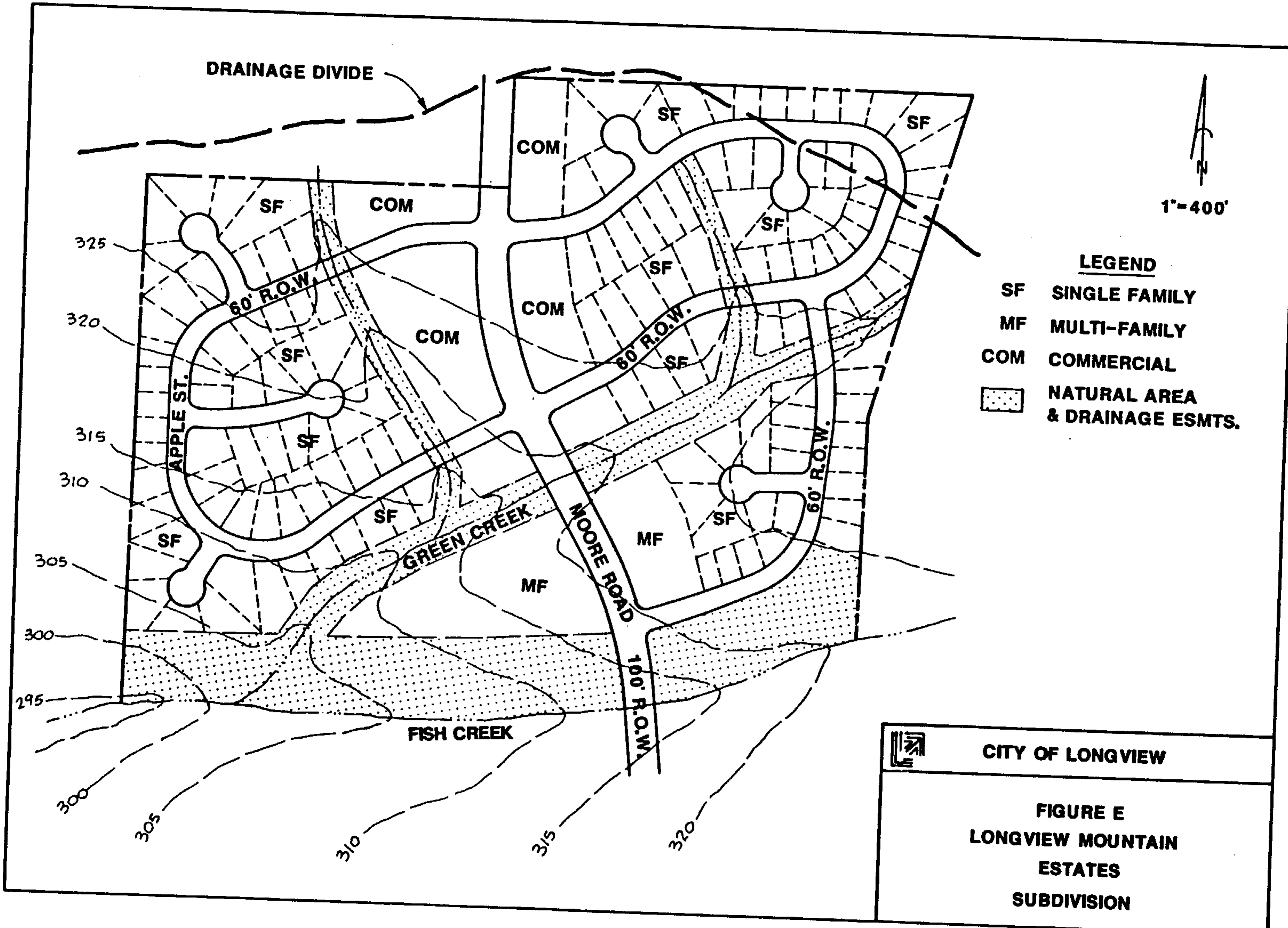
Determine Design Flow Rate: Apply the rational formula

$$Q = CiA$$

217 cfs = (0.72)(8.0 in/hr)(37.6 acres)

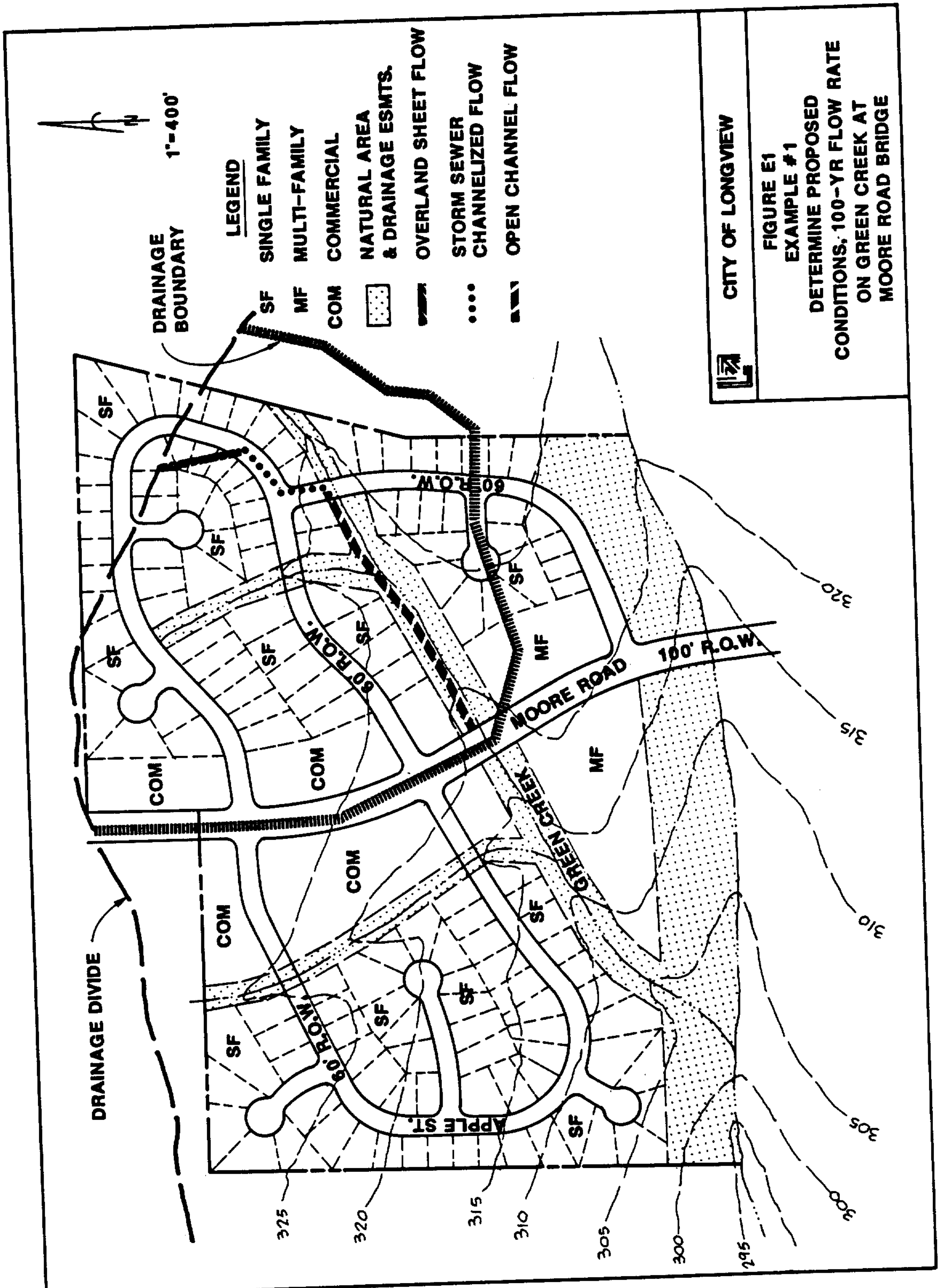
Thus, design channel and culvert to accommodate 217 cfs.

Note: Design of smaller drainage areas such as for inlets and storm sewers proceeds in a similar way.



CITY OF LONGVIEW

FIGURE E
LONGVIEW MOUNTAIN
ESTATES
SUBDIVISION


DRAINAGE BOUNDARY
LEGEND
SF SINGLE FAMILY
MF MULTI-FAMILY
COM COMMERCIAL
NATURAL AREA & DRAINAGE ESMTS.
OVERLAND SHEET FLOW
STORM SEWER CHANNELIZED FLOW
OPEN CHANNEL FLOW
N
1"=400'
CITY OF LONGVIEW
FIGURE E1
EXAMPLE #1
DETERMINE PROPOSED CONDITIONS, 100-YR FLOW RATE ON GREEN CREEK AT MOORE ROAD BRIDGE
DRAINAGE DIVIDE
MOORE ROAD
100' R.O.W.
60' R.O.W.
GREEN CREEK
APPLE ST.
SF
MF
COM
325
320
315
310
305
300
295

2. EXAMPLE CHANNEL IMPROVEMENT DESIGN CALCULATION

EXAMPLE #2

Task: The goal is to design channel improvements for the channel section shown on Figure E2.

Known: Design flow rate = 217 cfs from Example No. 1.

Procedure: Select design methodology - Since the proposed channel configuration and slope will be relatively uniform in this area, it is acceptable to assume uniform flow and design using Manning's Equation. However, it is important to note that calculation of flow depth using Manning's Equation is not applicable in the channel area immediately upstream of Moore Road. This is because the channel will be constricted by the culvert passing under Moore Road and, thus, flow can not be considered uniform. Flow depths in this area can be calculated either by applying HEC-2 on Green Creek or by estimating the headwater elevation during design of the Moore Road culverts. If the configuration of the proposed channel is designed so as not to change beneath Moore Road, uniform flow can be assumed and the use of Manning's Equation is acceptable.

Choose a channel lining type - Since the natural area bordering the creek provides adequate width, and since grassed channels are generally less expensive and aesthetically more pleasing than other sideslope treatments, utilize a grassed channel with 3:1 sideslopes. Proposed sideslopes will not exceed 3:1, thus, it is not necessary to be concerned about slope stability.

Select proposed channel depth - The existing natural channel is determined to be on average about 4.0 feet deep. Thus, to minimize the depth of the proposed channel, it is decided to build it only slightly deeper. A 4.5 foot depth is selected.

Establish freeboard requirement - Maintain a recommended freeboard of 1.0 foot. Thus the required flow depth is 3.5 feet (4.5 foot channel depth - 1.0 foot freeboard).

Establish known design requirements - The following flow design requirements are established.

- Design flow rate = 217 cfs
- Channel slope = 0.005 ft/ft
- Manning's "n" value = 0.04 (from Section 5.3.9)
- Channel depth = 4.5 feet
- Required flow depth = 3.5 feet
- Channel sideslope = 3:1
- Channel bottom width = ??

Determine final channel configuration - The only design parameter not yet established is channel bottom width. When channel bottom width is tied down it is possible to determine flow depth.

Utilizing the hydraulic routine, "Normal Depth in a Non-Circular Channel", it is determined by trial and error that a grassed channel with a bottom width of 5.0 feet, 3:1 sideslopes, and a channel slope of 0.005 ft/ft will carry 217 cfs (Q_{100}) with a depth of approximately 3.4 feet.

Check Design Velocity - Using "Normal Depth in a Non-Circular Channel" the design flow velocity is determined to be 4.1 ft/sec. This is an acceptable velocity for grassed channels.

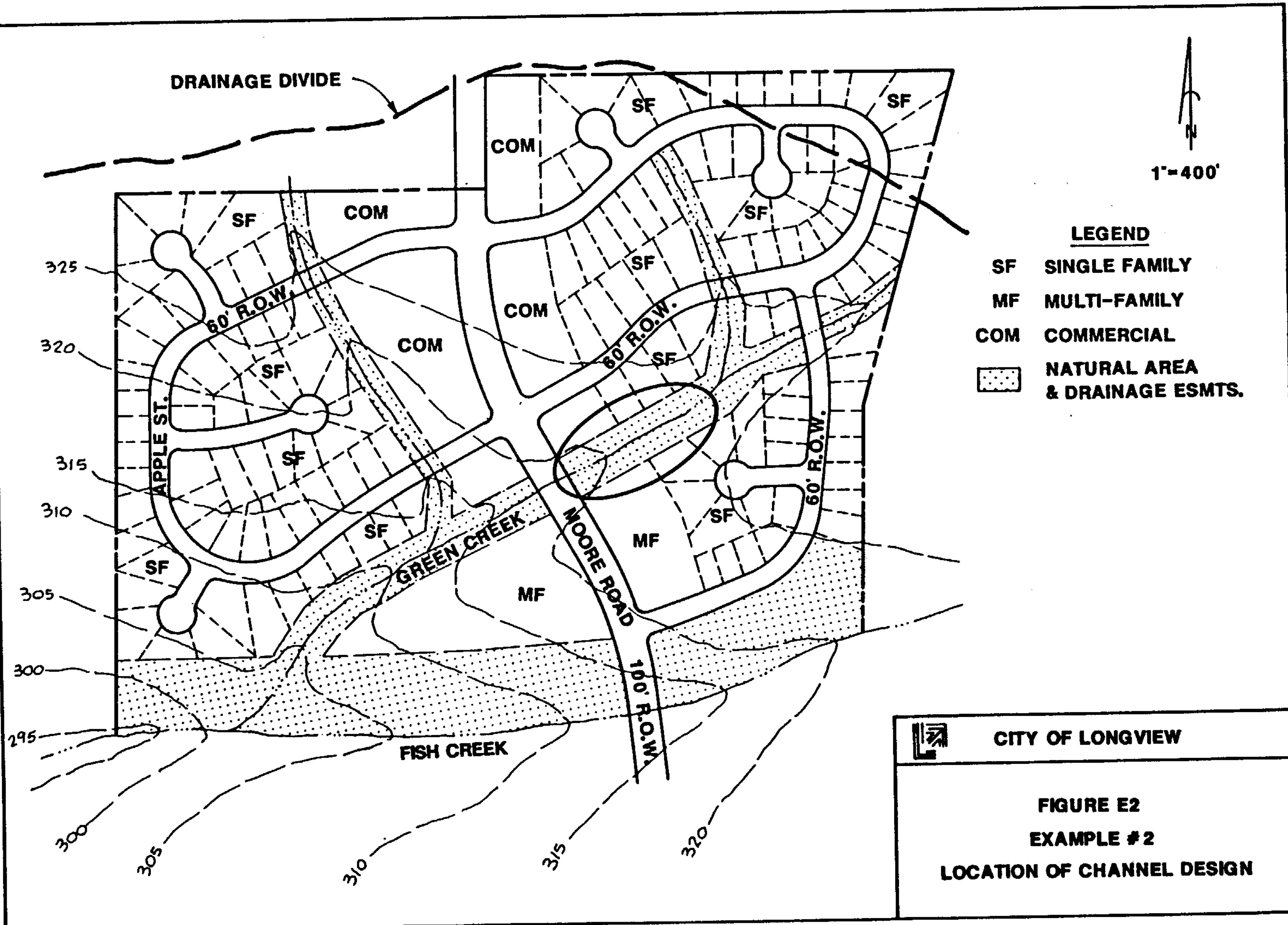


FIGURE E2
EXAMPLE # 2
LOCATION OF CHANNEL DESIGN

3. EXAMPLE OF INLET DESIGN

EXAMPLE #3

Task: It is necessary to design inlets for the single-family section of the proposed development (Figure E3). For this example an inlet will be designed on Apple Street in the southwest part of the development. Apple Street will be constructed to be 53 feet wide.

Known: The street configuration has been established as shown in Figure E3. The allowable depth of flow at the gutter for Apple Street is determined to be 0.5 feet in order to leave one lane open.

Procedure: Determine street drainage capacities - The proposed slope of Apple Street is 0.01 ft/ft. From Figure 6-1 (c), Apple Street's flow capacity is determined to be 10 cfs.

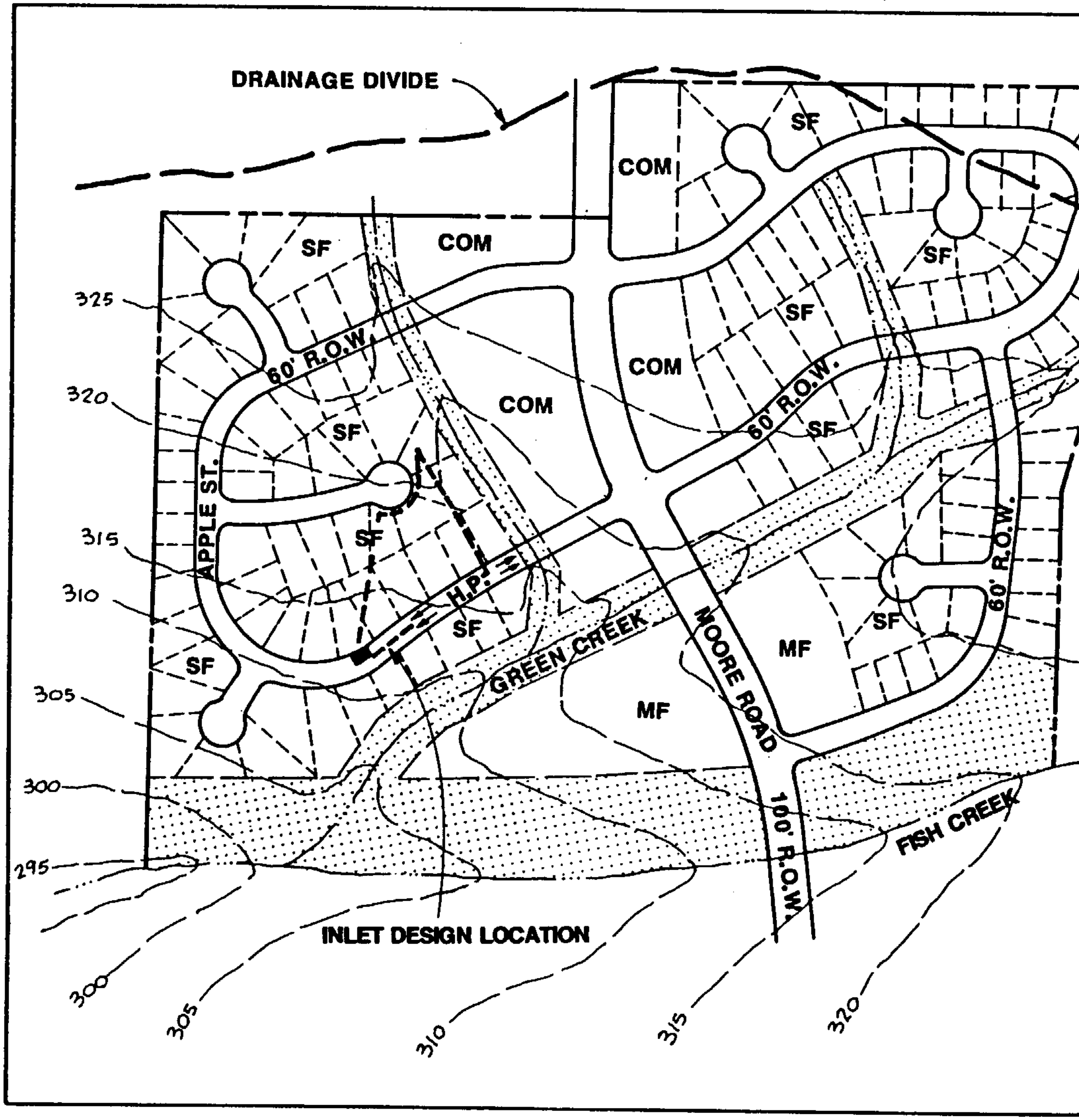
Determine the point where the gutter is flowing full - Begin at the high point and estimate the downstream point where the gutter will flow full in the design storm. This may be done on a 1" = 100′ drainage area map. For the purpose of this example, a portion of Figure E3 has been drawn at 1" = 100′ and is shown as Figure E3A.

A C value of 0.72 is calculated for this drainage area in the 100-year storm. To determine the time of concentration and therefore intensity (i), the path of runoff flow from the uppermost reach of the drainage area to the inlet is drawn. Using the method explained in Section 4.5 of this manual, the time of concentration is determined to be 10 minutes.

Using the rational method (Q=CiA) and a trial and error process, a drainage area is plotted on the drainage area map which will produce 10 cfs in the north gutter of Apple Street during the 100-year event. For inlet design purposes, this value is increased by 10 percent to 11 cfs to account for possible clogging of the inlet. At this point, the gutter is flowing full and an inlet is drawn on the drainage area map.

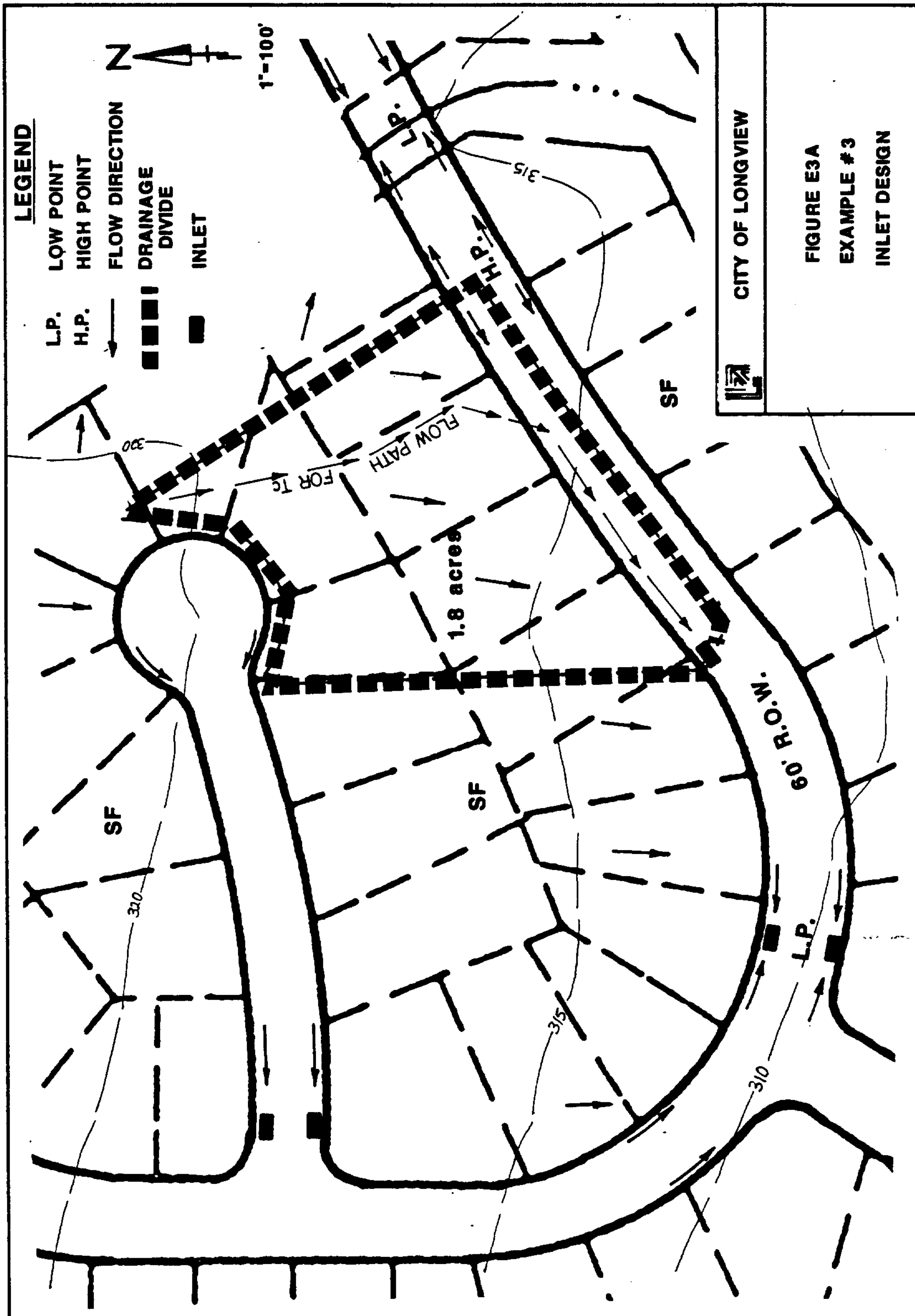
Size the inlet - Using the weir equation described in Section 6.3 (step 5), determine the inlet size which will capture all or most of the approaching flow. A standard 10-foot inlet at a depth of 0.5 feet is found to capture 10.6 cfs. Thus, a remaining 0.4 cfs (11 cfs - 10.6 cfs) is assumed to bypass the inlet and must be accounted for at the next downstream inlet.

The process is repeated for the next downstream inlet. Generally, inlets are placed at the upstream corner of all intersections to eliminate (or minimize) flow across the intersection.



CITY OF LONGVIEW

FIGURE E3
EXAMPLE #3
LOCATION OF DRAINAGE
AREA FOR INLET DESIGN


LEGEND
L.P. LOW POINT
H.P. HIGH POINT
FLOW DIRECTION
DRAINAGE DIVIDE
INLET
N
1"=100'
CITY OF LONGVIEW
FIGURE E3A
EXAMPLE #3
INLET DESIGN
L.P.
H.P.
315
320
310
FLOW PATH FOR Tc
1.8 acres
SF
SF
SF
60' R.O.W.
L.P.

4. EXAMPLE OF STORM SEWER DESIGN

EXAMPLE #4

Task: It is necessary to design a short reach of storm sewer (with one lateral) which will serve to drain a portion of the single-family section of the proposed development.

Known: The proposed storm sewer layout is as shown in Figure E4. The necessary pipe capacity has been determined (utilizing the rational method from Section 4.0) to be 50 cfs.

Procedure: Establish downstream starting water surface elevation - From backwater profiles established using HEC-2 on Fish Creek for the 25-year flow rate, the 25-year water surface elevation at the storm sewer pipe outlet is determined to be 304.0 ft msl (see Figure E4A).

Make initial sizing estimate - Given the natural grade, the probable pipe slope is determined to be 0.022 ft/ft. The hydraulic routine "Normal Depth in Circular Channel" indicates that a 36-inch diameter RCP will carry the design flow (50 cfs) under near full conditions at the required slope.

Calculate the profile of the HGL - Calculation of the HGL profile begins at the downstream water/surface elevation - 304 ft. Since the outlet of the storm sewer is submerged during a 25-year flow on Fish Creek, an outlet loss must be calculated.

$$\text{outlet loss} = V^2/2g = (Q/A)^2/2g = (50/7.07)^2/64.4 = 0.78'$$

where:

V = flow velocity in the storm sewer at the pipe outlet (ft/sec)

Since the HGL elevation exceeds the top of the pipe at the outlet, the velocity can be calculated assuming pressure flow conditions using the continuity equation:

$$Q = VA \text{ or } V = Q/A$$

where:

Q = flow rate (ft^3/sec)
A = cross-sectional area of the pipe (ft^2)

The starting elevation for the HGL is then equal to the tailwater elevation (304') plus the outlet loss (0.78') = 304.78'

Since this elevation inundates the top of the storm sewer outfall, the pipe is assumed to flow full at this point. Thus, Equation 6-1 is used to determine the slope of the HGL. The computer routine "Slope of Hydraulic Grade Line" will solve equation 6-1.

$$\text{Slope of HGL} = n^2\, Q^2/(1.49)^2\, A^2\, R^{4/3}$$

In a circular pipe flowing full, $R = \dfrac{\text{pipe diameter}}{4}$

In RCP, "n" is assumed to be 0.013. Thus:

$$\text{Slope of the HGL} = (0.013)^2\,(50)^2/(1.49)^2\,(7.068)^2\,(0.75)^{4/3} = 0.0056 \text{ ft/ft}$$

Beginning at 304.78', the HGL is plotted at a slope of 0.0056 ft/ft. It is determined that, at this slope, the HGL will intersect the top of the pipe approximately 260 feet from the outfall. This point is estimated to be the location of the beginning of full flow conditions (no longer pressure flow) as you proceed upstream.

Beyond this point, as the HGL is plotted moving upstream, the HGL is conservatively assumed to lie coincident with the top of the storm sewer pipe. At locations where head losses will occur (i.e., inlets, manholes, junctions) the HGL will once again exceed the level of the top of the pipe by jumping vertically an amount equal to the magnitude of the head loss. Beyond the point of the loss, where the HGL exceeds the top of the pipe, the slope of the HGL is once again calculated via Equation 6-1 until it again intersects the top of the pipe.

Calculate Junction, Inlet and Manhole Losses - At the inlet, the HGL experiences a head loss calculated as 0.05 ($V^2/2g$). If the HGL exceeds the elevation of the top of the pipe, the velocity V is calculated as Q/A where:

Q	=	flow rate in pipe (cfs)
A	=	cross-sectional area of pipe (ft^2)

If the HGL does not exceed the top of the pipe, the velocity is calculated either with the hydraulic computer routine "Normal Depth in Circular Channel" or with Figure 6-4. In this case, the HGL does not exceed the top of the pipe at the location of the inlet and the velocity is calculated to be 14.03 ft/sec. The above equation is used to calculate head loss at the inlet.

$$\text{Inlet loss} = [(0.05)(14.03)^2]/2(32.2) = 0.153'$$

Thus, at the point of the inlet loss, the HGL is caused to increase vertically by the amount of the loss (0.153 ft).

Beyond this point, the HGL once again exhibits a slope of 0.0056 ft/ft as calculated with Equation 6-1 until the HGL again intersects the top of pipe another 26.0 feet upstream.

At the bend, and again at the junction, the loss is calculated using the hydraulic computer routine "Junction and Bend Losses" which solves Equation 6-3.

At the bend, the following conditions apply:

Q_1	=	Q_2	=	50 cfs
V_1	=	V_2	=	14.03 ft/sec (from Computer Routine "Normal Depth in Circular Channel")

D_1 = 15°

A_2 = A_1 = 3.0 ft (diameter of pipe)^2 $\pi/4$ = 7.07 ft^2

The resulting head loss from the "Junction and Bend Losses" routine is 0.105.

At the junction, the loss is again calculated using the hydraulic routine "Junction and Bend Losses". The following conditions apply at the junction:

Q_1 = 30 cfs

Q_2 = 50 cfs

Q_3 = 20 cfs (note that the sum of $Q_1 + Q_3$ must equal Q_2 to preserve continuity)

V_1 = 6.11 fps

V_2 = 7.07 fps

V_3 = 6.37 fps (note that velocities are calculated assuming full flow in all pipes thus $V = Q/A$).

D_1 = 55°

D_3 = 71° (see Figure E4)

A_1 = 2.5 ft. (diameter of pipe)^2 $\pi/4$ = 4.91 ft^2

A_2 = 3.0 ft. (diameter of pipe)^2 $\pi/4$ = 7.07 ft^2

A_3 = 2.0 ft. (diameter of pipe)^2 $\pi/4$ = 3.14 ft^2

The resulting head loss from the "Junction and Bend Losses" routine is 1.07 ft.

PROJECT NO.

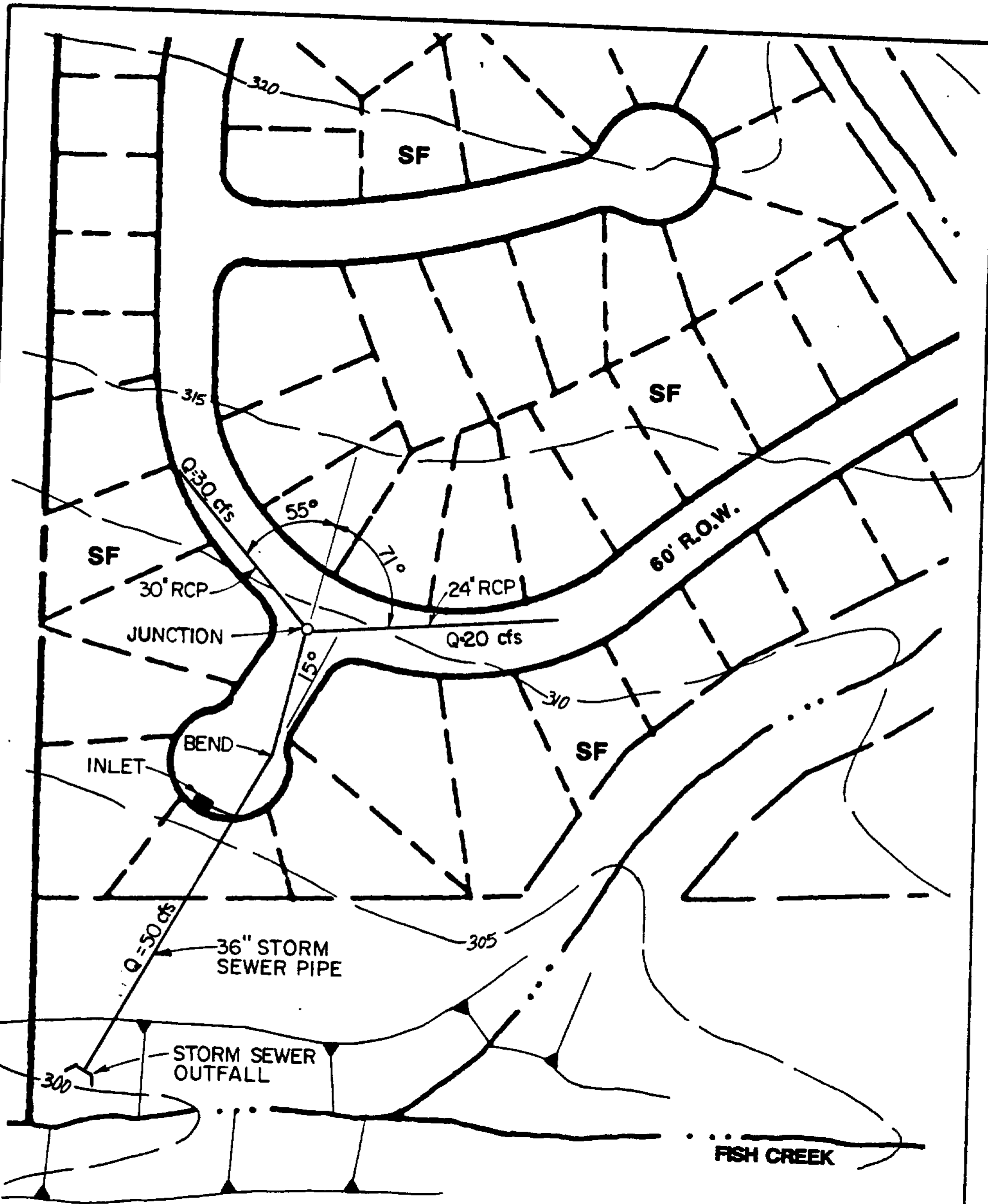


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FIGURE E4
EXAMPLE # 4
DESIGN LOWER PORTION OF PROPOSED STORM SEWER

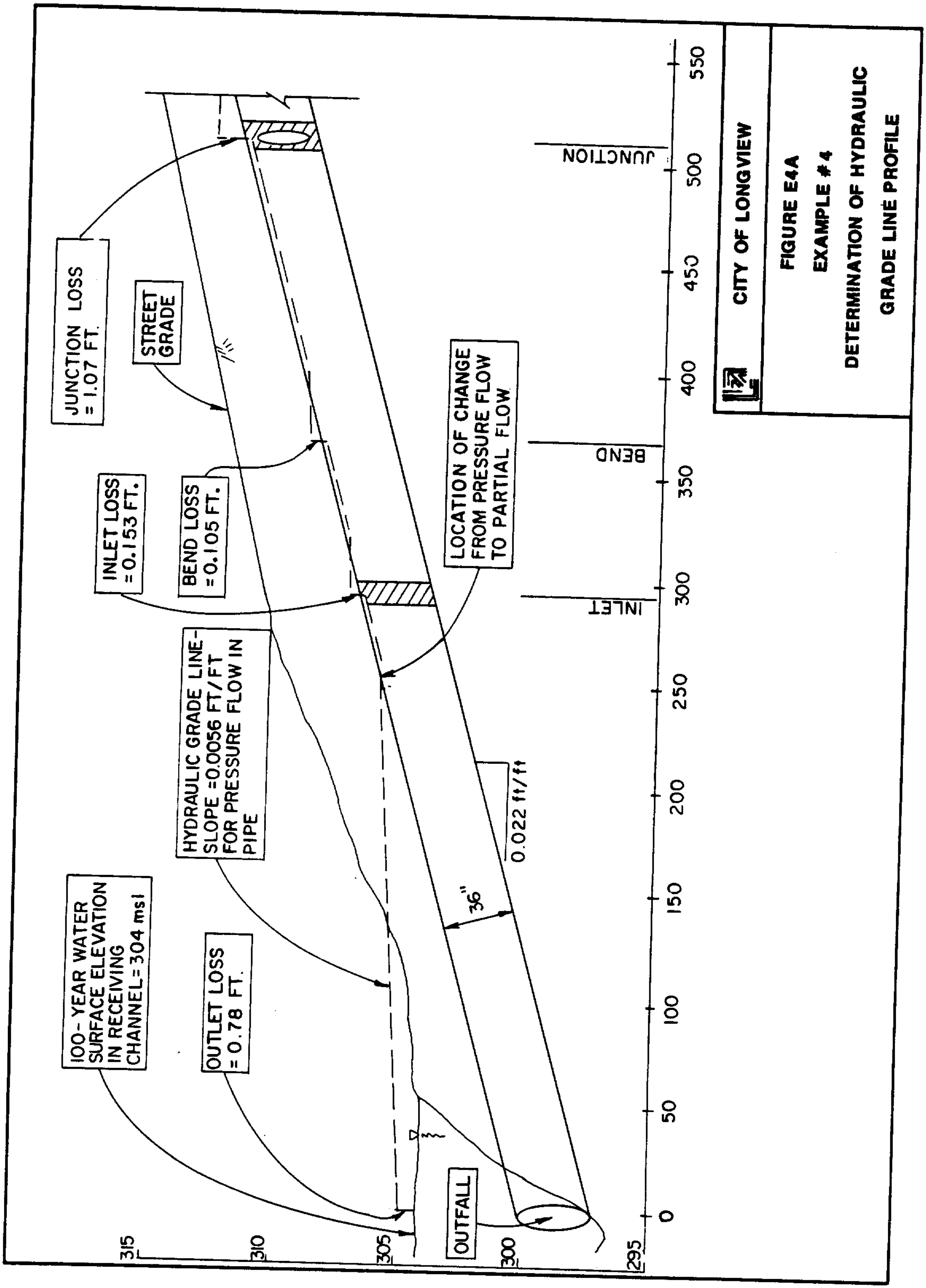

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FIGURE E4A
EXAMPLE # 4
DETERMINATION OF HYDRAULIC GRADE LINE PROFILE
JUNCTION LOSS = 1.07 FT.
STREET GRADE
INLET LOSS = 0.153 FT.
BEND LOSS = 0.105 FT.
LOCATION OF CHANGE FROM PRESSURE FLOW TO PARTIAL FLOW
HYDRAULIC GRADE LINE-SLOPE = 0.0056 FT/FT FOR PRESSURE FLOW IN PIPE
100-YEAR WATER SURFACE ELEVATION IN RECEIVING CHANNEL = 304 msl
OUTLET LOSS = 0.78 FT.
OUTFALL
36"
0.022 ft/ft
JUNCTION
BEND
INLET
315
310
305
300
295
0
50
100
150
200
250
300
350
400
450
500
550

5. EXAMPLE CULVERT DESIGN

EXAMPLE #5

Given: It is necessary to design a culvert for the road crossing Green Creek as shown on Figure E-5.

Known: From previous hydrologic calculations in Example #1, it is determined that the design flow rate at the culvert (100-year flow) is 217 cfs.

Given the physical configuration of the proposed channel near the culvert it is determined that the maximum headwater depth (HW) at the proposed culvert is 6.0 feet.

Given the proposed road/bridge right-of-way width, it is determined that the culvert must be 100 feet long (L = 100).

It is determined that the proposed culvert slope should match the existing average channel slope: $S_0 = 0.005$.

From previous hydraulic calculations, it is determined that the average velocity of flow for the 100-year discharge in Green Creek is 4.0 ft/sec.

It is decided that a reinforced concrete, multiple box culvert will be utilized.

Procedure: Select a first trial culvert size. This is accomplished by assuming inlet control and consulting the inlet control nomograph for box culverts (Chart 7-1). As a first cut, it is decided the culvert opening should be 4.0 feet in height. The maximum headwater depth is determined to be 6.0 feet given the proposed road and channel configuration. The headwater depth is assumed to be at its maximum (6.0 feet) during the 100-year flow.

Determine the required culvert width - Chart 7-2 (for inlet control) is consulted. HW/D is determined to be 1.5 (6.0/4.0). From Chart 7-2, it is seen that a 4.0 foot high box culvert with a 6.0 foot headwater depth will pass 40 cfs for every foot of width. Thus, the required box culvert width must be about 5.4 feet (217/40) wide. 6.0 feet will represent the headwater depth under inlet control conditions.

Determine headwater depth assuming outlet control - From a HEC-2 flow profile run on Green Creek, it is determined that the flow depth at the culvert outlet will be 4.5 feet during the 100-year flood. This represents the tailwater depth (TW) for the culvert. Since TW is greater than the culvert height, the culvert is submerged at its outlet. Thus D_2 is set equal to the tailwater (TW) (d_2 = TW = 4.5 feet).

If the tailwater had been less than the culvert height, HW would have been set equal to the greater of $(d_c + D)/2$ and TW. The value of d_c (critical depth) is determined from charts 7-5 or 7-6.

HW_{outlet} is now determined from Equation 7-1: $HW_{outlet} = H + d_2 - LS_0$

where:

H = energy loss (head) in feet as determined from the appropriate nomograph (charts 7-3 and 7-4). In this case, Chart 7-4 ("Head for Concrete Box Culverts Flowing Full") is consulted. The inlet loss coefficient k_e is assumed to be 0.5. From Chart 7-3, it is determined that H = 2.8 feet.

d_2 = TW, as determined above. $d_2 = 4.5$ feet.

L, S_0 = L is the culvert length (100 feet) and S_0 is the culvert slope (0.005 ft/ft), thus $LS_0 = 0.50$ feet.

Thus $HW_{outlet} = 2.8 + 4.5 - 0.50 = 6.8$ feet.

Compare HWs for inlet and outlet control - The headwater depth under inlet control is 6.0 feet. HW for outlet control is 6.80 feet. Therefore, since HW is greater for the outlet control condition, the proposed culvert will in reality flow under outlet control with a headwater depth of 6.80 (0.80 above the allowable).

Adjust culvert configuration - To reduce headwater depth the trial culvert size is increased to 4 ft x 8 ft. This causes the head loss (H) calculated from Chart 7-3 to decrease to 1.35 feet. Thus, for outlet control,

$$HW_{outlet} = 1.35 + 4.5 - 0.50 = 5.35 \text{ feet}$$

For inlet control (from Chart 7-2),

$$HW_{outlet} = 4.9 \text{ feet}$$

Since 5.35 >4.9, the flow is still outlet-controlled, but the headwater depth of 5.35 feet is not excessive and the design is acceptable. It is recommended that a variety of culvert sizes be evaluated and "fine-tuned" prior to selection of a final culvert configuration.

Evaluate outlet velocity - Since the culvert is flowing full, it is possible to determine the outlet velocity directly from the continuity equation, Q = VA, where Q is the design flow rate and A is the culvert cross-sectional area. In this case,

$$\text{Velocity} = 217 \text{ cfs}/32 \text{ ft}^2 = 6.78 \text{ fps.}$$

Since the calculated outlet velocity (6.78 fps) does not exceed the value specified as a maximum for natural channels (Table 5-1), this design would not require structural erosion protection at the outlet.

Note, however, that if during lower flow events the culvert is not flowing full, the computer routine "Normal Depth in a Non-Circular Channel" should be implemented to determine velocity at the culvert outlet.

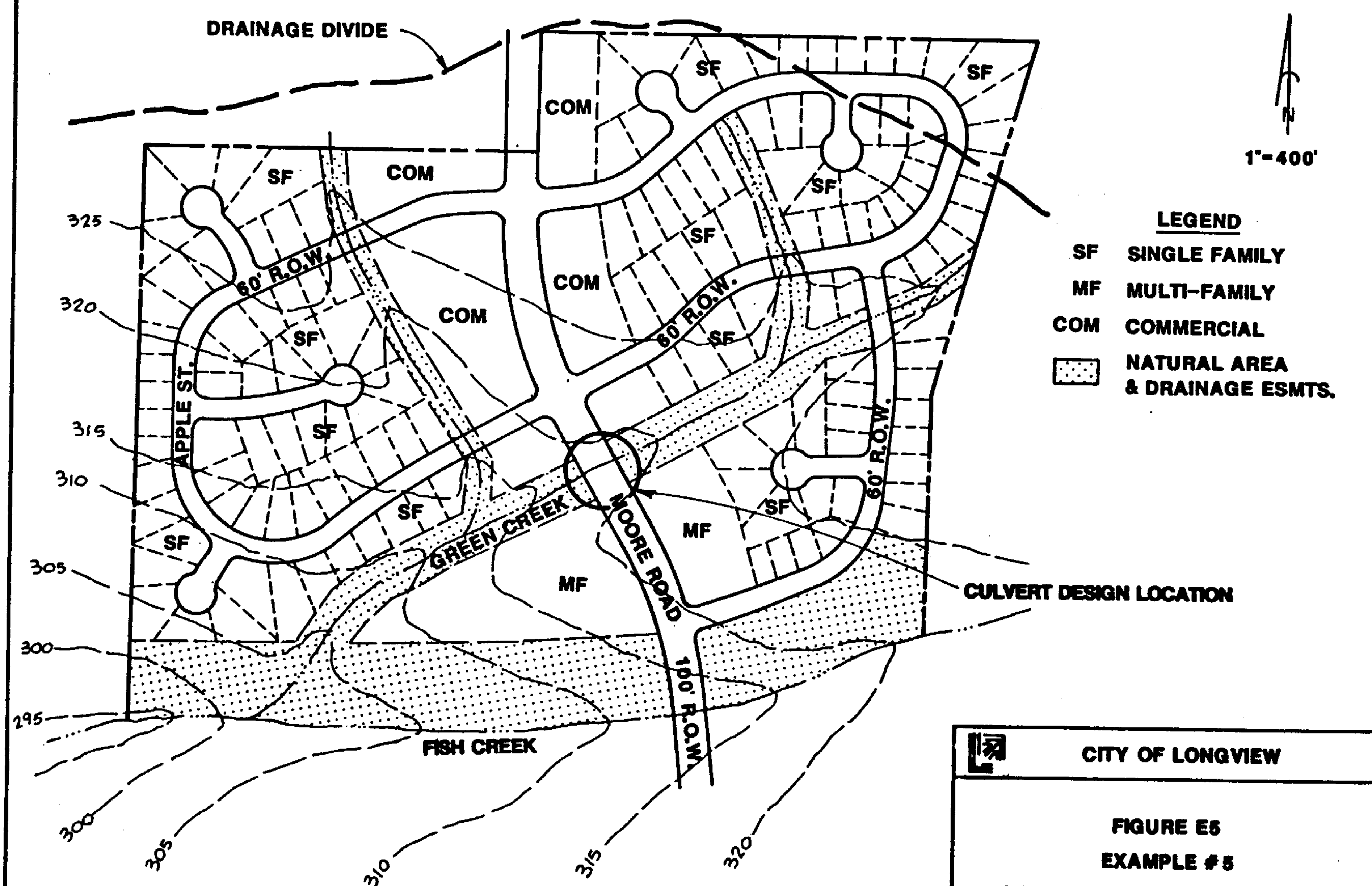

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FIGURE E5
EXAMPLE #5
LOCATION OF CULVERT DESIGN

6. EXAMPLE DETENTION FACILITY DESIGN CALCULATION

EXAMPLE #6

Given - Due to significant increases in impervious cover and drainage efficiency related to the proposed development, it is necessary to provide detention to prevent flooding on Fish Creek downstream of the proposed development. It is decided to place one detention facility to serve the entire proposed development in the natural area near the confluence of Green Creek and Fish Creek.

Step 1 - It is decided to design an online detention facility by placing a berm across Green Creek near its confluence with Fish Creek. The outflow structure will consist of a pipe orifice exiting the facility at the elevation of the pond flowline along with a 1.0-ft-deep, weir-type spillway at elevation 309.0.

Step 2 - The area draining to the detention facility is seen to be 80 acres. The time of concentration for this area is calculated to be 18 minutes. Calculations utilizing the rational method for the area draining to the detention facility indicate the peak 5-, 25-, and 100-year flow rates are as shown below. The "HYDROLIB" model is then invoked to determine the 5-, 25-, and 100-year peak flow rates for SF-4 development draining to the pond using the SCS method in HEC-1. Table 8-1 indicates that for the given combination of soil types and land uses, the CN value associated with SF-4 development is 84. Using Option 5 in HYDROLIB, the peak 5-, 25-, and 100-year flow rates are as shown below:

	Rational Method	SCS Method
100-year	403	464
25-year	336	354
5-year	269	247

The target flow rates from the entire site for SF-4 development are then developed by adding the lesser of the values shown above for each frequency storm to the comparable SF-4 flow rates exiting the site area not draining to the pond:

	SF-4 Development for Area Not Draining to Detention Facility (cfs)	SF-4 Development for Area Draining to Detention Facility (cfs)	Target Flow for Entire Site (cfs)
5-year	71	247	318
25-year	95	336	431
100-year	117	403	520

Step 3 - A preliminary detention configuration is determined assuming some excavation in the Green Creek drainageway to provide extra storage. The first cut configuration is:

Elevation (msl)	Surface (acres)	Outflow (cfs)
304	0	0
305	0.55	30
306	1.12	96
307	1.60	162
308	1.80	222
309	2.00	270
310	2.20	425

Step 4 - Invoke "HYDROLIB" Option 5.

Step 5 - Input the proposed conditions hydrologic data. Begin, by inputting the 5-year event rainfall. The contributing drainage area is 80 acres. The time of concentration is 18 minutes. The SCS curve number is determined for the actual proposed development to be 94. The preliminary detention facility described in Step 3 is input.

Step 6 - The model is run.

Step 7 - The model indicates that the "Peak Water Surface Elevation in the Detention Facility" is 308.15. This is 0.85 ft less than the elevation of the spillway crest. The "Peak Flow Rate Exiting the Detention Facility" is 229. The actual development conditions, 5-year peak flow from the area not draining to the detention facility is determined to be 88 cfs. Thus, the peak 5-year flow exiting the entire site is 229+88 = 317 cfs. This value is 1.0 cfs less than the target of 318 cfs as shown above, thus for the 5-year detention facility, this pond configuration will succeed.

Step 8 - Check the other frequency storm events. The 100-year rainfall event is input to the model. Now, the model indicates that the peak water surface elevation in the facility reaches 309.98. Thus, the spillway is overtopped by 0.98 ft. The peak 100-year flow rate exiting the detention facility is seen to be 422.0. The actual development conditions, 100-year peak flow rate from the area not draining to the pond is determined to be 145 cfs (higher than for SF-4). Thus, the post-detention, 100-year peak flow exiting the entire site is 145+422=567 cfs.

This value exceeds the target 100-year peak of 520 cfs. Thus, the pond volume should be adjusted upward until the 100-year proposed conditions flow for the entire site does not exceed the 100-year SF-4 development conditions flow rate for the entire site.

Step 9 - Adjustments to the pond configuration are made and the above procedure is performed again for the 5-, and 25-year events until the facility performs adequately for all three sizes of design storms.

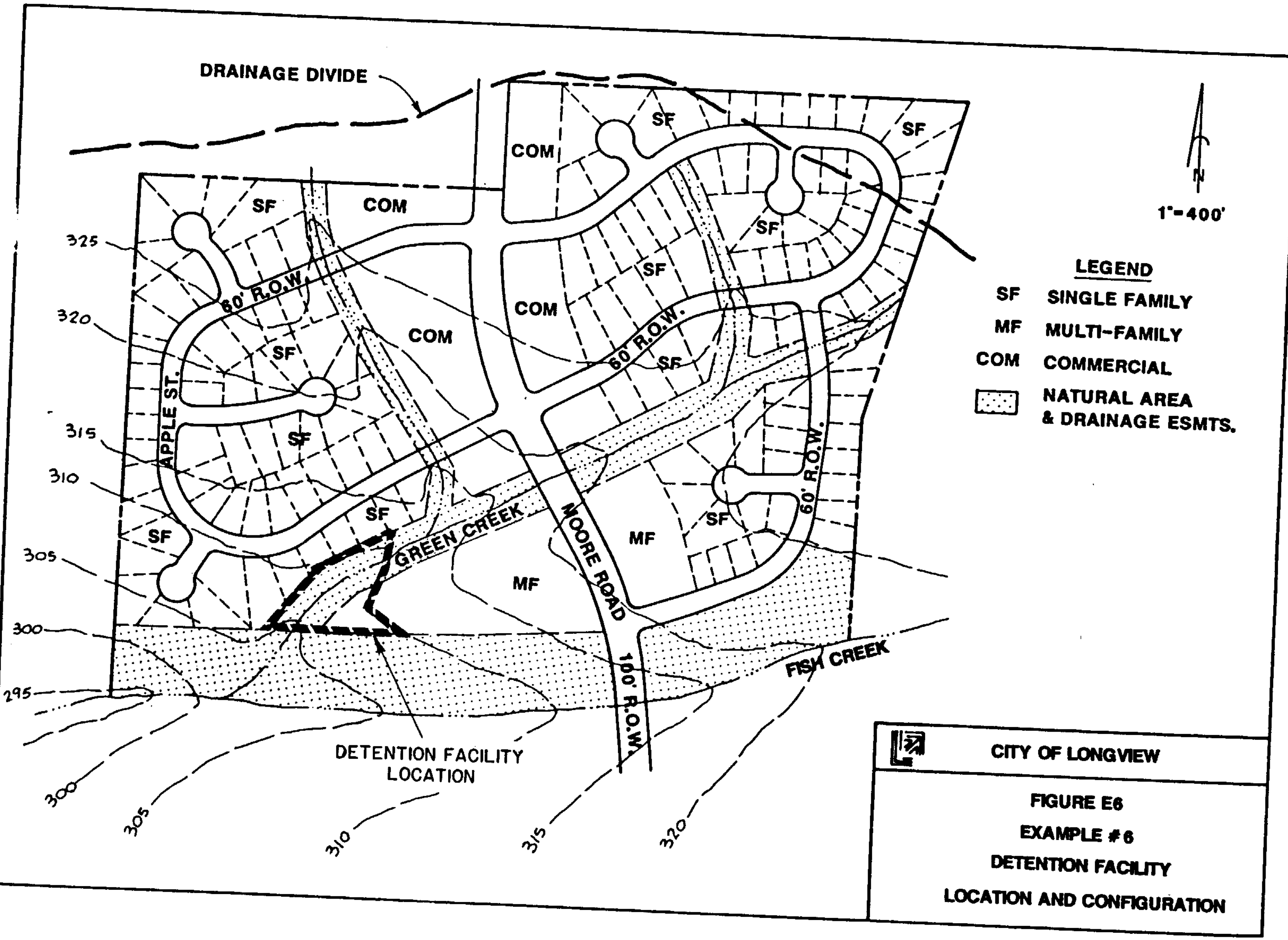

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FIGURE E6

EXAMPLE # 6

DETENTION FACILITY

LOCATION AND CONFIGURATION

APPENDIX C
HYDROLIB

APPENDIX C
HYDROLIB

TOOLS AVAILABLE FOR COMPLIANCE WITH CITY OF LONGVIEW DRAINAGE DESIGN REQUIREMENTS

Provided with this manual is a floppy disk containing several simple hydraulic routines. These easy-to-use, interactive programs will serve to greatly facilitate the engineering design process. Each is referenced in the appropriate sections of this manual and the design engineer is encouraged to utilize them.

The hydraulics library can be invoked by typing HYDROLIB. The user will then see a menu from which to select the appropriate routine. The menu will appear as follows:

1. Slope of hydraulic grade line
2. Junction and bend losses
3. Normal depth in non-circular channel
4. Normal depth in circular channel
5. Detention facility design
6. Exit

Routine #1, "Slope of Hydraulic Grade Line", will calculate the friction slope of the HGL assuming pressure flow conditions in a pipe conduit (storm sewer). Its technical basis and appropriate application are referenced and explained in Section 6.0, "Inlets and Storm Sewers".

Routine #2, "Junction and Bend Losses", utilizes an equation developed by the City of Los Angeles, Bureau of Engineering, Storm Drain Design Division (1968) to calculate head losses at junctions and bends in a storm sewer system. The technical basis for this methodology and its appropriate application are discussed in Section 6.6 (Step 5) of this manual.

Routine #3, "Normal Depth in Non-Circular Channel", carries out an iterative solution of Manning's Equation for uniform flow to determine depth, velocity, Froude Number, velocity head and specific energy in a trapezoidal channel flowing at normal depth (uniform flow). This routine is referenced and recommended for use in several sections of this manual.

Routine #4, "Normal Depth in Circular Channel", carries out an iterative solution of Manning's Equation for uniform flow as described above, except that the channel is a circular pipe conduit. This routine is referenced and recommended for use in several sections of this manual.

Routine #5, "Detention Facility Design", incorporates the U.S. Corps of Engineers hydrologic analysis program, HEC-1, to perform a simple hydrograph determination in a small subarea and to route the hydrograph through a detention facility of known configuration. This easy-to-use, interactive application of HEC-1 will insure proper design of stormwater detention facilities.