STORMWATER MANAGEMENT DESIGN MANUAL

CITY OF RUSSELLVILLE, ARKANSAS

POPE COUNTY

PREPARED BY:

FTN ASSOCIATES, LTD.

July 2001

COPY NO. _______

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FORWARD

This document sets forth the administrative and technical criteria to be used for the design, submittals, and evaluations of stormwater management facilities associated with land disturbing activities within the City of Russellville, Arkansas and its Planning Area. These criteria are not intended to limit the initiative and resourcefulness involved with developing drainage plans but rather to standardize major requirements and procedures. This edition was developed in compliance with Stormwater Management Drainage Ordinance #1675 formerly adopted by the City of Russellville on June 15, 2000 and is intended to be used in conjunction with that Ordinance.

A "New User Information Form" is supplied for the User of this copy of the Russellville STORMWATER MANAGEMENT DRAINAGE MANUAL. A copy of this form should be completed by the holder of this copy and submitted to address shown on the form. This information will be used to notify the registered holder of any corrections or amendments to the MANUAL.

A "Change-of-User Information Form" is also supplied for the User of this copy of the Russellville STORMWATER MANAGEMENT DRAINAGE MANUAL. A copy of this form should be completed by the holder of this copy and submitted to address shown on the form should any of the original information be changed. This information will insure the proper notification of the registered holder of any corrections or amendments to the MANUAL.

A copy of the Russellville Stormwater Management Drainage Ordinance (SMDO) is included for the User as it is the basis for this MANUAL. All criteria presented in this MANUAL shall be in agreement with the provisions of this document. Any provisions herein that conflict with the provisions of the SMDO shall yield to the Ordinance.

A "Purpose of the Manual" section is included for the User to present the purpose of this MANUAL and its relevancy to the land disturbing activities within the City of Russellville and its Planning Area.

A "Use of the Manual" section is included for the User to better understand how the MANUAL is to be used and its general layout.

NEW USER INFORMATION FORM

TO: THE USER OF "THE STORMWATER MANAGEMENT DESIGN MANUAL"

In order to insure you receive future changes and additions to this manual, please complete the form below and mail to the address below:

Name ____

Address ____

____

e-mail Address _______________________________________

Telephone No. ____________________

Copy No. ________

Mail to: **City of Russellville**
City Engineer
P.O. Box 428
Russellville, Arkansas 72801

CHANGE-OF-USER INFORMATION FORM

To insure you are notified of all corrections, amendments, and/or updates to this manual, please complete the form below for any change of name, address, telephone number, or other contact information.

PREVIOUS INFORMATION:

Old Name ____

Old Address ____

____

Old e-mail Address ____________________________________

Old Telephone No. ____________________

Copy No. _______

NEW INFORMATION:

New Name ____

New Address ____

____

New e-mail Address ____________________________________

New Telephone No. ____________________

Mail to: **City of Russellville**
City Engineer
P.O. Box 428
Russellville, Arkansas 72801

Stormwater Management and Drainage Ordinance

AN ORDINANCE ADOPTING REGULATIONS DESIGNED TO LESSEN OR AVOID HAZARDS TO PERSONS AND PROPERTY CAUSED BY INCREASED STORM WATER RUNOFF OR BY OBSTRUCTION TO DRAINAGE, AND TO OTHERWISE PROMOTE THE PUBLIC HEALTH, SAFETY AND GENERAL WELFARE.

ARTICLE I. GENERAL PROVISIONS

SECTION A. Title; purpose

1. The provisions of this ordinance shall constitute and be known as the "Stormwater Management Ordinance for the City of Russellville, AR."
2. The purpose of this Ordinance is to protect, maintain, and enhance the public health, safety, and general welfare by establishing minimum requirements and procedures to control the adverse effects of increased stormwater runoff associated with both future land development and existing developed land within the City. Proper management of stormwater runoff will minimize damage to public and private property, ensure a functional drainage system, reduce local flooding, maintain as nearly as possible the pre-developed runoff characteristics of the area, and facilitate economic development while mitigating associated flooding and drainage impacts.
3. The application of this Ordinance and the provisions expressed herein shall be the minimum stormwater management requirements and shall not be deemed a limitation or repeal of any other powers granted by State statute. In addition, if site characteristics indicate that complying with these minimum requirements will not provide adequate designs or protection for local property or residents, it is the designer's responsibility to exceed the minimum requirements as necessary. The CITY ENGINEER or designee shall be responsible for the coordination and enforcement of the provisions of this ordinance.

SECTION B. Definitions

For the purpose of this Ordinance, the following terms, phrases and words, and their derivatives, shall have the meaning given herein:

1. As-built plan shall mean a set of engineering or site drawings that delineate the specific permitted stormwater management facility as actually constructed.
2. Best management practices shall mean a wide range of management procedures, schedules of activities, prohibitions on practices, and other management practices which have been demonstrated to effectively control the quality and/or quantity of stormwater runoff and which are compatible with the planned land use.
3. City Engineer shall mean the duly designated Head of the Engineering Department or department of public works, or his duly authorized agent.
4. City Engineering Department shall mean the department responsible for all stormwater management activities and implementation of the provisions of this ordinance.
5. Cross-drain culvert shall mean a culvert located under a roadway.
6. Design report shall mean the report that accompanies the Stormwater Management Plan and includes data used for engineering analysis, results of all analysis, design and analysis calculations (including input files and results obtained from computer programs), and other engineering data that would assist the City Engineer in evaluating proposed stormwater management facilities.
7. Designer shall mean a professional who is permitted to prepare plans and studies required by this ordinance.
8. Detention structure shall mean a permanent stormwater management structure whose primary purpose is to temporarily store stormwater runoff and release the stored runoff at controlled rates.
9. Development should generally mean any of the following actions undertaken by a public or private individual or entity:
    - the division of a lot, tract or parcel of land into two (2) or more lots, plots, sites, tracts, parcels or other divisions by plat or deed, or
    - any land change, including, without limitation, clearing, tree removal, grubbing, stripping, dredging, grading, excavating, transporting and filling of land.
10. Develop land shall mean to change the runoff characteristics of a parcel of land in conjunction with residential, commercial, industrial, or institutional construction or alteration.
11. Develop land use conditions shall mean the land use conditions according to the current City Land Use Map or proposed development plan.
12. Easement shall mean a grant or reservation by the owner of land for the use of such land by others for a specific purpose or purposes, and which must be included in the conveyance of land affected by such easement.
13. Erosion shall mean the wearing away of land surface by the action of wind, water, gravity, ice, or any combination of those forces.
14. Erosion and sediment control shall mean the control of solid material, both mineral and organic, during a land disturbing activity to prevent its transport out of the disturbed area by means of wind, water, gravity, or ice.

15. Existing land use conditions shall mean the land use conditions existing at the time of the most recent official aerial photography available from the City.
16. Four percent annual chance (4%) storm shall mean a storm that is capable of producing rainfall expected to have a 4% chance of being equaled or exceeded in any given year.
17. Grading shall mean excavating, filling (including hydraulic fill), or stockpiling of earth material, or any combination thereof, including the land in its excavated or filled condition.
18. Impervious shall mean the condition of being impenetrable by water.
19. Imperviousness shall mean the degree to which a site is impervious.
20. Infiltration shall mean the passage or movement of water through the soil profile.
21. Interior culvert shall mean a culvert that is not located under a roadway.
22. Land disturbing activity shall mean any use of the land by any person that results in a change in the natural cover or topography that may cause erosion and contribute to sediment and alter the quantity of stormwater runoff.
23. Maintenance shall mean any action necessary to preserve stormwater management facilities in proper working condition, in order to serve the intended purposes set forth in Article I of this Ordinance and to prevent structural failure of such facilities. Maintenance shall not include actions taken solely for the purpose of enhancing the aesthetics associated with stormwater management facilities.
24. Natural waterways shall mean waterways that are part of the natural topography. They usually maintain a continuous or seasonal flow during the year and are characterized as being irregular in cross-section with a meandering course. Construction channels such as drainage ditches shall not be considered natural waterways.
25. Nonerodible shall mean a material, e.g., natural rock, riprap, concrete, plastic, etc., that will not experience surface wear due to natural forces of wind, water, ice, gravity, or a combination of those forces.
26. On-site stormwater management shall mean the design and construction of a facility necessary to control stormwater runoff within and for a single development.
27. One percent annual chance (1%) storm shall mean a storm that is capable of producing rainfall expected to have a 1% chance of being equaled or exceeded in any given year.
28. Person responsible for the land disturbing activity shall mean:
    a. the person who has or represents having financial or operational control over the land disturbing activity; and/or
    b. the landowner or person in possession or control of the land who directly or indirectly allowed the land disturbing activity or has benefited from it or who has failed to comply with any provision of this ordinance.
29. Post-development conditions shall mean the conditions which exist following the completion of the land disturbing activity in terms of topography, vegetation, or land use and rate, volume, or direction of stormwater runoff.
30. Pre-developed conditions shall mean those land use conditions that existed prior to the initiation of the land disturbing activity in terms of topography, vegetation, or land use and rate, volume, or direction of stormwater runoff.

31. Preliminary plat shall mean the preliminary plat of a residential subdivision submitted pursuant to the City's Subdivision Regulations.
32. Record survey shall mean a final field survey which locates the visible surface features of a constructed stormwater facility on the ground, but without locating non-visible or subsurface features such as the actual route and elevation of buried pipe.
33. Regional stormwater management shall mean the design and construction of a facility necessary to control stormwater runoff within or outside a development and for one or more developments.
34. Registered Civil Engineer shall mean a civil engineer properly registered and licensed to conduct work within the City.
35. Registered Land Surveyor shall mean a land surveyor properly registered and licensed to conduct work within the City.
36. Registered Landscape Architect shall mean a landscape architect properly registered and licensed to conduct work within the City.
37. Responsible personnel shall mean any foreman, superintendent, or similar individual who is the on-site person in charge of land disturbing activities.
38. Retention structure shall mean a permanent structure whose primary purpose is to permanently store a given volume of stormwater runoff. Release of the given volume is by infiltration and/or evaporation.
39. Sediment shall mean solid particulate matter, both mineral and organic, that has been or is being transported by water, air, ice, or gravity from its site of origin.
40. Stabilization shall mean the installation of vegetative or structural measures to establish a soil cover to reduce soil erosion by stormwater runoff, wind, ice and gravity
41. Stage work or stage construction shall mean a plan for the staged construction of stormwater facilities where portions of the facilities will be constructed as different stages of the proposed development are started or completed.
42. Stormwater Concept Plan shall mean the overall proposal for a storm drainage system, including stormwater management structures, and supporting documentation as specified in
the Stormwater Management Design Manual, for each proposed private or public development to the extent permitted by law. Also included are the supporting engineering calculations and results of any computer analysis, if necessary.
43. Stormwater management shall mean the collection, conveyance, storage, treatment, and disposal of stormwater runoff in a manner to minimize accelerated channel erosion and/or increased flood damage, and in a manner to enhance and ensure the public health, safety, and general welfare, which shall include a system of vegetative or structural measures, or both, that control the increased volume and rate of stormwater runoff caused by manmade changes to the land.
44. Stormwater Management Design Manual shall mean the manual of design, performance, and review criteria for stormwater management practices, prepared under the direction of the City Engineer. Copies of this manual can be obtained from the City Engineering Department.

45. Stormwater management facilities shall mean those structures and facilities that are designed for the collection, conveyance, storage, and disposal of stormwater runoff into and through the drainage system.
46. Stormwater Management Plan (SMP) shall mean the set of drawings and other documents that comprise all of the information and specifications for the drainage systems, structures, concepts and techniques that will be used to control stormwater as required by this Ordinance and the Stormwater Management Design Manual. Also included are the supporting engineering calculations and results of any computer analysis.
47. Stormwater management qualitative control shall mean a system of vegetative, structural, or other measures that reduce or eliminate pollutants that might otherwise be carried by stormwater runoff.
48. Stormwater runoff shall mean the direct response of a watershed to precipitation and includes the surface and subsurface runoff that enters a ditch, stream, storm drain or other concentrated flow during and following the precipitation.
49. Subdivision shall mean (1) The creation of one or more new streets, alleys or other public ways; or, the changing of any rights-of-way of any existing streets, alleys or other public ways. (2) Any division or redivision of lot, tract, or parcel or land, regardless of its prospective use. Such subdivision may be accomplished by platting or by description of metes and bounds or otherwise into two (2) or more lots or other divisions for sale or improvement. The following are not defined as subdivisions:
    a. The combination or recombination of portions of previously platted lots where the total number of lots is not increased and the resultant lots are in accordance with the rules and regulations contained in the City's Subdivision Regulations and with the City's Zoning Ordinance.
    b. Division or sale of land by judicial decree which shall not be deemed a division for purposes of this ordinance.
    c. The acquisition of land for the purpose of widening or opening of streets when the acquisition and work is done by the City, State, or other governmental agency.
    d. The division of land into parcels greater than five (5) acres where no street right-of-way dedication is involved.
50. Swale shall mean a structural measure with a lining of grass, riprap, or other materials which can function as a detention structure and convey stormwater runoff without causing erosion.
51. Variance shall mean the modification of the minimum stormwater management requirements for specific circumstances where strict adherence of the requirements would result in unnecessary hardship and not fulfill the intent of this ordinance.
52. Waiver shall mean the relinquishment from stormwater management requirements by the City Engineer for a specific land disturbing activity on a case-by-case review basis.

53. Water quality shall mean those characteristics of stormwater runoff from a land disturbing activity that relate to the physical, chemical, biological, or radiological integrity of water.
54. Water quantity shall mean those characteristics of stormwater runoff that relate to the rate and volume of the stormwater runoff to downstream areas resulting from land disturbing activities.
55. Watershed shall mean the drainage area contributing stormwater runoff to a single point.

SECTION C. Scope of Ordinance

No person shall develop any land, realign any channel, place fill or debris in the channel without having provided for appropriate stormwater management measures that control or manage runoff, in compliance with this Ordinance, unless exempted in Article I, Section D below.

SECTION D. Exemptions

All construction, subdivision approvals or remodeling activities shall have a stormwater management and drainage plan approved before a building permit is issued or subdivision is approved except for the following:

- One - new or existing single family structure.
- One - new or existing duplex family structure.
- One - existing commercial or industrial structure where additional structural improvements or additional impervious areas are less than 500 square feet.
- Residential subdivisions which were approved prior to the effective date of these regulations are exempt from these requirements. Development of new phases of existing subdivisions which were not previously approved shall comply with the provisions of these regulations.

SECTION E. Stormwater Management Design Manual

To assist in the design and evaluation of stormwater management facilities in the City, a Stormwater Management Design Manual will be developed. Recommended design procedures and criteria are presented for conducting hydrologic and hydraulic evaluations. Although the intention of the manual is to establish uniform design practices, it neither replaces the need for engineering judgment nor precludes the use of information not presented. Other accepted engineering procedures may be used to conduct hydrologic and hydraulic studies if approved by the City Engineer.

ARTICLE II. STORM WATER CONCEPT AND PRELIMINARY DEVELOPMENT PLANS

SECTION A. Scope of development plans

1. a. In developing plans for residential subdivisions, individual lots in a residential subdivision development shall not be considered to be separate land disturbing activities and shall not require individual permits. Instead the residential subdivision development, as a whole, shall be considered to be a single land disturbing activity. Hydrologic parameters that reflect the ultimate subdivision development shall be used in all engineering calculations.

b. If individual lots or sections in a residential subdivision are being developed by different property owners, all land disturbing activities related to the residential subdivision shall be covered by the approved Stormwater Management Plan for the residential subdivision. Individual lot owners or developers shall sign a certificate of compliance that all activities on that lot will be carried out in accordance with the approved Stormwater Management Plan for the residential subdivision.

2. Unless otherwise deemed necessary by the City Engineer, for land disturbing activities involving two and one half (2.5) acres or less of actual land disturbance or development of less than 40,000 square feet of impervious area which are not part of a larger common plan of development or sale, the person responsible for the land disturbing activity shall submit a simplified stormwater management control plan meeting the requirements listed below. This plan does not require preparation or certification by the designers specified in Section J of Article II unless deemed necessary by the City Engineer. The requirements for the simplified stormwater management control plan include:

   a. A narrative description of the stormwater management facilities to be used.
   b. A general description of topographic and soil conditions of the development site.
   c. A general description of adjacent property and a description of existing structures, buildings, and other fixed improvements located on surrounding properties.
   d. A sketch plan to accompany the narrative which shall contain:
      - a site location drawing of the proposed project, indicating the location of the proposed project in relation to roadways, jurisdictional boundaries, streams and rivers;
      - the boundary lines of the site on which the work is to be performed;
      - all areas within the site which will be included in the land disturbing activities shall be identified and the total disturbed area calculated;
      - a topographic map of the site;
      - anticipated starting and completion dates of the various stages of land disturbing activities and the expected date the final stabilization will be completed.
      - the location of temporary and permanent vegetative and structural stormwater management control measures.
   e. Stormwater Management Plans shall contain certification by the persons responsible for the land disturbing activity that the land disturbing activity will be accomplished pursuant to the plan.
   f. Stormwater Management Plans shall contain certification by the person responsible for the land disturbing activity of the right of the City Engineer to conduct on-site inspections.

3. For land disturbing activities disturbing more than two and one-half (2.5) acres or development of greater than 40,000 square feet of impervious area, the requirements of Article II, Sections B - J shall apply.

SECTION B. Stormwater Concept and Stormwater Management Plans

1. A Stormwater Concept Plan for each development shall be submitted for review by the City Engineer prior to submission of the Stormwater Management Plan and construction plans for the entire development, or any portion thereof.
2. All preliminary plats of the development shall be consistent with the Stormwater Concept Plan required in Paragraph 1 above.
3. Upon approval of the concept plan, the applicant shall submit a final Stormwater Management Plan (as part of the construction plans) to the City Engineer for review and approval; provided that the City Engineer may accept and submit into the review process a Stormwater Concept Plan if it identifies the location and type of facilities to be constructed in sufficient detail to accurately estimate construction costs and the City Engineer determines that a Stormwater Management Plan is not needed. If accepted under this provision, the Stormwater Concept Plan then becomes the Stormwater Management Plan for this development.
4. Should any Stormwater Management Plan involve any stormwater management facilities or land to be dedicated to public use, the same information shall also be submitted for review and approval to the department having jurisdiction over the land or other appropriate departments or agencies identified by the City Engineer for review and approval. This Stormwater Management Plan shall serve as the basis for all subsequent construction.
5. The Stormwater Concept Plan may be reviewed, if needed, with the designer, after City review, where it will be approved, approved with changes, or rejected. If rejected, changes, additional analysis, or other information needed to approve the next submittal of the concept plan shall be identified. The City review of the Stormwater Concept Plan will be completed within five (5) working days from and after the receipt of the plan.
6. Within ten (10) working days from and after the receipt of the Stormwater Management Plan, the City Engineer shall issue a decision approving, rejecting or conditionally approving the plan with modification.

SECTION C. Stormwater Management requirements

1. For purposes of obtaining approval of a Stormwater Management Plan, a plan for the site meeting the requirements established in the Stormwater Management Design Manual shall be submitted to the City Engineer for review and approval. All design criteria plan details shall be in conformance with the Stormwater Management Design Manual.
2. Construction of stormwater management facilities shall be in conformance with the approved Stormwater Management Plan for the site.

3. The Stormwater Management Plan, including on-site stormwater detention facilities, shall be reviewed and approved by the City Engineer prior to issuance of building permits for the site. The improvements shall be constructed prior to the issuance of final certificates of occupancy. The requirements of this paragraph may be deferred at the discretion of the City Engineer.
4. For sites on which privately owned and maintained stormwater detention and/or conveyance facilities are located, the property owner shall be responsible for the following:
    a. All future grading, repairs, and maintenance.
    b. Maintenance of the minimum stormwater detention volume, as approved by the City Engineer.
    c. Maintenance of the detention basin control structure(s) and discharge pipe(s) to insure the maximum theoretical stormwater release rate, as approved by the City Engineer, is not increased.
5. The property owner shall place no fill material, or erect any buildings, obstructions, or other improvements on the area reserved for stormwater detention purposes, unless otherwise approved by the City Engineer.
6. The property owner shall dedicate to the City of Russellville, by instrument or final platting, any property on which public stormwater detention basins will be located. Ingress-egress easements for maintenance of public facilities shall be provided prior to final site approval.
7. All public storm sewers shall be dedicated to the City.
8. All stormwater drainage facilities serving more than one lot which are not dedicated to the city shall be covered under a drainage easement. Such easements shall grant to the City the authority for operation, maintenance, and inspection.
9. Upon determination that a site is not in compliance with these regulations, the City Engineer may issue an order to comply. The order shall describe the problem and specify a date whereby the work must be completed, and indicate the penalties to be assessed for further noncompliance.
10. Except as provided in this Ordinance, no person shall engage in construction of stormwater management facilities, unless a Stormwater Management Plan has been reviewed and approved by the City Engineer.
11. Compliance – Compliance with this Section is achieved when:
    a. The site plan has been approved.
    b. The approved stormwater drainage facilities have been implemented and are demonstrably in conformance with the approved site plan and Stormwater Management Design Manual.
12. Coordination with Building Permit – It is the intent of this Section that review of the stormwater drainage system be carried out simultaneously with the review of the request for a building permit. The site plan required under this chapter may be submitted in a form which will satisfy the site plan requirements set forth in the Building Code and Zoning Ordinance.
13. Other Permits – Before starting on construction regulated by this chapter, the applicant shall comply with the requirements set forth in other applicable ordinances with respect to submission and approval of subdivision plats, plans of improvements,

building permits, inspections, appeals and similar matters, as well as requirements of state statutes and the regulations of any Department of the State of Arkansas.

14. Alternatives to On-Site Detention
    a. Alternative Methods – where on-site detention is deemed inappropriate due to local topographical or other physical conditions, alternate methods for accommodating increases in stormwater runoff shall be permitted. The methods may include:
        1) Off-site detention or comparable improvements.
        2) In-lieu monetary contributions for drainage system improvements by the City. Channel improvements shall only be used if they are an integral part of a detailed watershed study.
    b. In-Lieu Contributions to Regional or Sub-Regional Detention – An owner may contribute to drainage system improvements to be constructed in lieu of constructing on-site detention. However, no in-lieu contributions are allowed when existing flooding occurs downstream from the development, or if the development will cause downstream flooding.
    c. In-Lieu Fees – The in-lieu fee contribution of shall be based upon an amount of $15,000 per Acre-Foot of stormwater storage.
    d. Excess Stormwater Storage Credit – An owner may receive credit for excess stormwater storage (in Acre-Feet) created on one site that may be applied to another site within the same watershed. The transfer of storage volume credit (in Acre-Feet) shall not be allowed if the site where credited storage is proposed to be transferred has an existing flooding condition downstream or the proposed development will produce downstream flooding.
    e. Drainage System Improvements – Monies contributed by the owners as above provided shall be used for the construction of drainage improvements; facilities thereon will be financed by the City.

SECTION D. Permit requirements

1. No final occupancy permit shall be issued without the following:
    a. Recorded easements for stormwater management facilities.
    b. Receipt of an as-built plan, which includes a certification of the storm drainage system.
2. No site grading permit shall be issued or modified without the following:
    a. Right of entry for emergency maintenance if necessary.
    b. Right of entry for inspections.
    c. Any off-site easements needed.
    d. An approved Stormwater Concept Plan or Stormwater Management Plan, as appropriate.
3. The approved Stormwater Management Plan shall contain certification by the applicant that all land clearing, construction, development and drainage will be done according to the Stormwater Management Plan or previously approved revisions.

Any and all site grading permits may be revoked at any time if the construction of stormwater management facilities is not in strict accordance with approved plans.

5. In addition to the plans and permits required from the City, applicants shall obtain all state and federal permits required for the proposed development.

SECTION E. Fees

A list of fees for plan review and other fees associated with this ordinance can be obtained from the City Engineering Department.

SECTION F. Permit suspension and revocation

1. A site grading permit may be suspended or revoked if one or more of the following violations have been committed:
   a. Violation(s) of the conditions of the Stormwater Management Plan approval;
   b. Construction not in accordance with the intent of the approved plans;
   c. Non-compliance with correction notice(s) or stop work orders(s); or
   d. The existence of an immediate danger in a downstream area in the judgment of the City Engineer.

   If one or more of these conditions is found, a written notice of violations shall be served upon the owner or authorized representative and an immediate stop-work order may be issued. The notice shall set forth the measures necessary to achieve compliance with the plan. Correction of these violations must be started immediately or the owner shall be deemed in violation of this Ordinance.

SECTION G. Minimum runoff control requirements

1. The minimum stormwater control requirements shall provide management measures necessary to accomplish the following:
   a. Install stormwater management facilities to limit the 4% annual chance storm developed peak discharge rates to pre-developed peak discharge rates. The design of these facilities shall be based on procedures contained in the Stormwater Management Design Manual or approved by the City Engineer.
   b. The requirements, or portions thereof, of item (a.) may be waived by the City Engineer if it can be shown by detailed engineering calculations and analysis which are acceptable to the City Engineer that one of the following exists:
      1) the installation of stormwater management facilities would have insignificant effects on reducing downstream flood peaks; or
      2) stormwater management facilities are not needed to protect downstream developments and the downstream drainage system has sufficient capacity to receive any increase in runoff for the design storm; or
      3) it is not necessary to install stormwater management facilities to control developed peak discharge rates at the exit to a proposed development and installing such facilities would increase flood peaks at some downstream locations; or

    4) the City Engineer determines that stormwater management facilities are not needed to control developed peak discharge rates and installing such facilities would not be in the best interest of the City.

   c. The requirements, or portions thereof, of item (a.) may not be waived if the City Engineer determines that not controlling downstream flood peaks would increase known flooding problems, or exceed the capacity of the downstream drainage system.
   d. A waiver shall only be granted after a written request is submitted by the applicant containing descriptions, drawings, and any other information that is necessary to evaluate the proposed land disturbing activity. A separate written waiver request shall be required if there are subsequent additions, extensions, or modifications which would alter the approved stormwater runoff characteristics to a land disturbing activity receiving a waiver. The City Engineer will conduct a review of the request for a waiver within ten (10) working days. Failure of the City Engineer to act by the end of the tenth working day will result in the automatic approval of the waiver.
   e. Discharge velocities shall be reduced to provide a nonerosive velocity flow from a structure, channel, or other control measure or the velocity of the 4 percent annual chance storm runoff in the receiving waterway prior to the land disturbing activity, whichever is greater.
2. For all stormwater management facilities, a hydrologic-hydraulic study shall be done showing how the drainage system will function with and without the proposed facilities. Existing land use data shall be taken from the most recent aerial photograph and field checked and updated. For such studies the following land use conditions shall be used:
   a. For the design of the facility outlet structure, use developed land use conditions for the area within the proposed development and existing land use conditions for upstream areas draining to the facility.
   b. For any analysis of flood flows downstream from the proposed facility, use existing land use conditions for all downstream areas.
   c. All stormwater management facilities' emergency spillways shall be checked using the 1% annual chance storm and routing flows through the facility and emergency spillways. For this analysis, developed land use conditions shall be used for all areas within the analysis.
   d. If accepted for municipal maintenance, the effects of existing upstream detention facilities can be considered in the hydrologic-hydraulic study.

SECTION H. Stormwater management facilities

1. Stormwater management facilities may include both structural and nonstructural elements. Natural swales and other natural runoff conduits shall be retained where practicable.
2. Where additional stormwater management facilities are required to satisfy the minimum control requirements, the following measures are examples of what may be used:

a. Stormwater detention structures (dry basins);
b. Stormwater retention structures (wet ponds);
c. Facilities designed to encourage overland flow, slow velocities of flow, and flow through buffer zones;
d. Infiltration practices.

3. Where detention and retention structures are used, designs which consolidate these facilities into a limited number of large structures will be preferred over designs which utilize a large number of small structures.
4. Stormwater Management Plans can be rejected by the City Engineer if they incorporate structures and facilities that will demand considerable maintenance, will be difficult to maintain, or utilize numerous small structures if other alternatives are physically possible.
5. The drainage system and all stormwater management structures within the City (including both public and private portions) will be designed to the same engineering and technical criteria and standards. The City Engineering Department's review will be the same whether the portion of the drainage system will be under public or private control or ownership.
6. All stormwater management measures shall be designed in accordance with the design criteria contained in the Stormwater Management Design Manual using procedures contained in this manual or procedures approved by the City Engineer.

SECTION I. Plan requirements

Stormwater Management Plans shall include as a minimum the following.

1. A vicinity map indicating a north arrow, scale, boundary lines of the site, and other information necessary to locate the development site.
2. The existing and proposed topography of the development site except for individual lot grading plans in single family subdivisions.
3. Physical improvements on the site, including present development and proposed development.
4. Location, dimensions, elevations, and characteristics of all stormwater management facilities.
5. All areas within the site which will be included in the land disturbing activities shall be identified and the total disturbed area calculated.
6. The location of temporary and permanent vegetative and structural stormwater management control measures.
7. An anticipated starting and completion date of the various stages of land disturbing activities and the expected date the final stabilization will be completed.
8. Stormwater Management Plans shall include designation of all easements needed for inspection and maintenance of the drainage system and stormwater management facilities. As a minimum, easements shall have the following characteristics:
   a. Provide adequate access to all portions of the drainage system and structures.
   b. Provide sufficient land area for maintenance equipment and personnel to adequately and efficiently maintain the system with a minimum of ten (10) feet along both sides of all drainage ways, streams, channels, etc., and around the

perimeter of all detention and retention facilities, or sufficient land area for equipment access for maintenance of all stormwater management facilities. This distance shall be measured from the top of the bank or toe of the downstream side of the dam whichever is applicable.

   c. Restriction on easements shall include prohibiting all fences and structures which would interfere with access to the easement areas and/or the maintenance function of the drainage system.

9. To improve the aesthetic aspects of the drainage system, a landscape plan for all portions of the drainage system shall be part of the Stormwater Management Plan. This landscape plan shall address the following:
   a. Tree saving and planting plan.
   b. Types of vegetation that will be used for stream bank, stabilization, erosion control, sediment control, aesthetics, and water quality improvement.
   c. Any special requirements related to the landscaping of the drainage system and efforts necessary to preserve the natural aspects of the drainage system.

SECTION J. Plan hydrologic criteria

The hydrologic criteria to be used for the Stormwater Concept And Stormwater Management Plans shall be as follows:

1. Four percent (4%) annual chance design storm for all cross-drain culverts and drainage designs.
2. Ten percent (10%) annual chance design storm for drainage design for all interior culverts.
3. Four percent (4%) annual design storm for all detention and retention basins using procedures contained in the Stormwater Management Design Manual or approved by the City Engineer.
4. All hydrologic analysis will be based on land use conditions as specified in Article II Section G 2.
6. For the design of storage facilities, a secondary outlet device or emergency spillway shall be provided to discharge the excess runoff in such a way that no danger of loss of life or facility failure is created. The size of the outlet device or emergency spillway shall be designed to pass the one percent (1%) chance storm as a minimum requirement.
7. All storms listed above are to be analyzed assuming a 24-hour duration.

SECTION K. Professional registration requirements

Stormwater concept and Stormwater Management Plans and design reports that are incidental to the overall or ongoing site design shall be prepared, certified, and stamped/sealed by a qualified registered Professional Engineer, Land Surveyor, or Landscape Architect, as applicable, using acceptable engineering standards and practices. All other stormwater concept and Stormwater Management Plans and design reports shall be prepared, certified, and stamped/sealed by a qualified registered Professional Engineer, using acceptable engineering standards and practices.

The engineer, surveyor, or landscape architect shall perform services only in areas of his/her competence, and shall undertake to perform engineering or land surveying assignments only when qualified by education and/or experience in the specific technical field. In addition, the engineer, surveyor, or landscape architect must verify that the plans have been designed in accordance with this ordinance and the standards and criteria stated or referred to in this ordinance.

ARTICLE III. OWNERSHIP AND CITY PARTICIPATION

SECTION A. Ownership of stormwater management facilities

1. All stormwater management facilities shall be privately owned and maintained unless the City accepts the facility for City ownership and maintenance. The owner of all private facilities shall grant to the City, a perpetual, non-exclusive easement which allows for public inspection and emergency repair.
2. All stormwater management measures relying on designated vegetated areas or special site features shall be privately owned and maintained as defined on the Stormwater Management Plan.
3. Regional stormwater management facilities will be publicly owned and/or maintained.

SECTION B. City participation

When the City Engineer determines that additional storage capacity beyond that required by the applicant for on-site stormwater management is necessary in order to enhance or provide for the public health, safety and general welfare, to correct unacceptable or undesirable existing conditions or to provide protection in a more desirable fashion for future development, the City Engineer may:

a. Require that the applicant grant any necessary easements over, through or under the applicant's property to provide access to or drainage for such a facility;
b. Require that the applicant attempt to obtain from the owners of property over, through or under where the stormwater management facility is to be located, any easements necessary for the construction and maintenance of same (and failing the obtaining of such easement the City may, at its option, assist in such matter by purchase, condemnation, dedication or otherwise, and subject to (c) below, with any cost incurred thereby to be paid by the City); and/or
c. Participate financially in the construction of such facility to the extent that such facility exceeds the required on-site stormwater management as determined by the City Engineer.

To implement this provision both the City and developer must be in agreement with the proposed facility that includes the additional storage capacity and jointly develop a cost sharing plan which is agreeable to all parties.

ARTICLE IV. MAINTENANCE, CONSTRUCTION AND INSPECTION

SECTION A. Maintenance

1. Any stormwater discharge control facility which services a single lot or commercial and industrial development shall be privately owned and maintained; provided, however, the owner thereof shall grant to the City, a perpetual, non-exclusive easement which allows for public inspection and emergency repair, in accordance with the terms of the maintenance agreement set forth in Article IV, Section B, below.
2. All regional stormwater discharge control facilities, identified on municipal stormwater discharge control masterplans, shall be publicly owned and/or maintained.
3. All other stormwater discharge control facilities shall be publicly owned and/or maintained only if accepted for maintenance by the City.
4. Private maintenance requirements shall be a part of the deed to the affected property.

SECTION B. Maintenance agreement (privately owned facilities only)

1. A proposed inspection and maintenance agreement shall be submitted to the City Engineer for all private on-site stormwater discharge control facilities prior to the approval of the Stormwater Management Plan. Such agreement shall be in form and content acceptable to the City Engineer and shall be the responsibility of the private owner. Such agreement shall provide for access to the facility by virtue of a non-exclusive perpetual easement in favor of the City at reasonable times for regular inspection by the City Engineer. The agreement will identify who will have the maintenance responsibility. Possible arrangements for this maintenance responsibility might include the following:
   - Use of homeowner associations,
   - Arrangements to pay the City for maintenance,
   - Private maintenance by development owner(s), or
   - Contracts with private maintenance companies.

   All maintenance agreements shall contain without limitation the following provisions:
   a. A description of the property on which the stormwater management facility is located and all easements from the site to the facility;
   b. Size and configuration of the facility;
   c. A statement that properties which will be served by the facility are granted rights to construct, use, reconstruct, repair, and maintain access to the facility;

d. A statement that each lot served by the facility is responsible for repairs and maintenance of the facility and any unpaid ad valorem taxes, public assessments for improvements, and unsafe building and public nuisance abatement liens charged against the facility, including all interest charges together with attorney fees, cost and expenses of collection. If an association is delegated these responsibilities, then membership into the association shall be mandatory for each parcel served by the facility and any successive buyer, the association shall have the power to levy assessments for these obligations, and that all unpaid assessments levied by the association shall become a lien on the individual parcel; and

e. A statement that no amendments to the agreement will become effective unless approved by the City.

2. The agreement shall provide that preventive maintenance inspections of stormwater management facilities may be made by the City Engineer, at his option. Without limiting the generality of the foregoing, the City Engineer's inspection schedule may include an inspection during the first year of operation and once every year thereafter, and after major storm events.
3. Inspection reports shall be maintained by the City Engineer.
4. The agreement shall provide that if, after an inspection, the condition of a facility presents an immediate danger to the public health, safety or general welfare because of unsafe conditions or improper maintenance, the City shall have the right, but not the duty, to take such action as may be necessary to protect the public and make the facility safe. Any cost incurred by the City shall be paid by the owner.
5. The agreement shall be recorded by the owner in the Register of Deeds prior to the final inspection and approval.
6. The agreement shall provide that the City Engineer shall notify the owner(s) of the facility of any violation, deficiency, or failure to comply with this Ordinance. The agreement shall also provide that upon a failure to correct violations requiring maintenance work, within ten (10) days after notice thereof, the City Engineer may provide for all necessary work to place the facility in proper working condition. The owner(s) of the facility shall be assessed the costs of the work performed by the City Engineer pursuant to this subsection and subsection 4 above and there shall be a lien on all property of the owner which property utilizes or will utilize such facility in achieving discharge control, which lien, when filed in the Register of Deeds, shall have the same status and priority as liens for ad valorem taxes. Should such a lien be filed, portions of the affected property may be released by the City following the payments by the owner of such owner's pro-rata share of the lien amount based upon the acreage to be released with such release amount to be determined by the City Engineer, in his reasonable discretion.
7. The City Engineer, at his sole discretion, may accept the certification of a registered engineer in lieu of any inspection required by this Ordinance.

SECTION C. Construction and inspection

1. Prior to the approval of the Stormwater Management Plan, the applicant shall submit a proposed staged construction and inspection control schedule. This plan shall indicate a phase line for approval; otherwise the construction and inspection control schedule will be for the entire drainage system.
2. No stage work, related to the construction of stormwater management facilities, shall proceed until the next preceding stage of work, according to the sequence specified in the approved staged construction and inspection control schedule, is inspected and approved.
3. Any portion of the work that does not comply with the Stormwater Management Plan shall be promptly corrected by the permittee.
4. The permittee shall notify the City Engineer before commencing any work to implement the Stormwater Management Plan and upon completion of the work.
5. The permittee shall provide an "as-built" plan certified by a registered professional (as outlined in Article II, Section J) to be submitted upon completing of the stormwater management facilities included in the Stormwater Management Plan. The registered professional shall certify that:
   a. The facilities have been constructed as shown on the "as-built" plan, and
   b. The facilities meet the approved Stormwater Management Plan and specifications or achieves the function for which they were designed.
6. A final inspection shall be conducted by the City Engineer upon completion of the work included in the approved Stormwater Management Plan to determine if the completed work is constructed in accordance with the plan.
7. The City Engineer shall maintain a file of inspection reports and provide copies of all inspection reports to the permittee that include the following:
   a. The date and location of the site inspection.
   b. Whether the approved plan has been properly implemented.
   c. Any approved plan deficiencies and any actions taken.
8. The City Engineer will notify the person responsible for the land disturbing activity in writing when violations are observed describing the following:
   a. Nature of the violation.
   b. Required corrective actions.
   c. The time period for violation correction.

ARTICLE V. MISCELLANEOUS PROVISIONS

SECTION A. Variances from requirements

1. The City Engineer may grant a variance from the requirements of this Ordinance if there are exceptional circumstances applicable to the site such that strict adherence to the provisions of the Ordinance will result in unnecessary hardship and not fulfill the intent of the Ordinance.
2. A written request for a variance shall be required and shall state the specific variance sought and the reasons, with supporting data, for their granting. The request shall

include descriptions, drawings, calculations, and any other information that is necessary to evaluate the proposed variance.

3. Any substantial variance from the Stormwater Management Plan or concept plan shall be referred to all agencies that reviewed the original plan.
4. The City Engineer will conduct a review of the request for a variance within ten (10) working days. Failure of the City Engineer to act by the end of the tenth working day will result in the automatic approval of the variance.

SECTION B. Appeals

Any person aggrieved by a decision of the City Engineer (including any decision with reference to the granting or denial of a variance from the terms of this Ordinance) may appeal same by filing a written notice of appeal with the City Engineer within thirty (30) calendar days of the issuance of said decision by the City Engineer. The City Engineer can then reverse his/her decision or send this notice to the Appeals Board with comments. A notice of appeal shall state the specific reasons why the decision of the City Engineer is alleged to be in error and the City Engineer shall prepare and send to the Appeals Board and Appellant, within fifteen (15) days of receipt of the notice of appeal, a written response to said notice of appeal.

All such appeals shall be heard by the Appeals Board that is hereby granted specific authority to hear and determine such appeals in a quasi-judicial capacity. Said appeal shall be heard by the Appeals Board at its next regularly scheduled meeting date, not to exceed thirty (30) days after receipt of the notice of appeal, or at such other time as may be mutually agreed upon in writing by the Appellant and the Chairperson of the Appeals Board. The Appeals Board will then render a decision within fifteen (15) days after the appeal has been heard.

Each party to the appeal shall be entitled to a hearing before the Appeals Board under judicial forms of procedure, at which hearing each party shall have the right to present evidence and sworn testimony of witnesses, to cross-examine witnesses, and to cause a transcription of the proceedings to be prepared.

Should either party be dissatisfied with the decision of the Appeals Board, any appeal of said decision may be appealed to the Superior Court by writ of certiorari.

SECTION C. Penalties

1. Upon determination that a violation of this ordinance has occurred the owner shall be given a written notice of the violations and the time in which to correct the deficiencies.
2. If construction violations of the approved plan are occurring, an immediate stop-work order may be issued by the City Engineer. If the City issues a stop work order, the City must show cause within forty-eight (48) hours.

3. Any person violating this ordinance or any part thereof, including failing to stop work upon order, shall upon conviction thereof, be fined not more than two hundred dollars or imprisoned not more than thirty (30) days for each offense. Each separate interval of 24 hours, or every day, such violations shall be continued, committed or existing, shall constitute a new and separate offense and be punished, as aforesaid, for each separate period of violation.
4. The City Attorney may institute injunctive, mandamus, or other appropriate action or proceedings at law or equity for the enforcement of this Ordinance or to correct violations of this Ordinance, and any court of competent jurisdiction shall have the right to issue restraining orders, temporary or permanent injunctions, mandamus or other appropriate forms of remedy or relief.

SECTION D. Grandfather clause

Any applicant or owner of a parcel of land within the jurisdiction of the City who has constructed the required stormwater management facility or who is in the process of meeting the stormwater management requirements of the law at the time of the effective date of this Ordinance may elect to apply to the City Engineer for reconsideration under the provisions of this ordinance.

SECTION E. Conflict with other laws

Whenever the provisions of this ordinance impose more restrictive standards than are required in or under any other ordinance, the regulations herein contained shall prevail. Whenever the provisions of any other ordinance require more restrictive standards than are required herein, the requirements of such shall prevail.

SECTION F. Severability

If any term, requirement or provision of this Ordinance or the application thereof to any person or circumstance shall, to any extent, be invalid or unenforceable, the remainder of this Ordinance or the application of such terms, requirements, and provisions to persons or circumstances other than those to which it is held invalid or unenforceable, shall not be affected thereby and each term, requirement, or provision of this Ordinance shall be valid and be enforced to the fullest extent permitted by law.

SECTION G. Amendments

This ordinance may be amended in the manner as prescribed by law for its original adoption. Before the City governing body amends this Ordinance, it must seek the advice of the City Engineer who will make a recommendation for each amendment within thirty (30) days of this request.

SECTION H. Liability

Neither the approval of a plan under the provisions of this Ordinance nor the compliance with the provisions of this Ordinance shall relieve any person from the responsibility for damage to any person or property otherwise imposed by law nor shall it impose any liability upon the City for damage to any person or property.

SECTION I. Effective date

The Ordinance shall be effective immediately after adoption of this Ordinance by the City and by reference to the Stormwater Management Design Manual.

Purpose of the Manual

The purpose of this Stormwater Management Design Manual (MANUAL) is to assist in the design and evaluation of stormwater management facilities in the City of Russellville, Arkansas and its Planning Area. It has been developed in compliance with the policies and requirements of the Stormwater Management and Drainage Ordinance (SMDO) #1675 adopted by the City.

As an extension of the Ordinance, its purpose is to protect, maintain, and enhance the public health, safety, and general welfare by establishing minimum requirements and procedures to control the adverse effects of increased stormwater runoff associated with land disturbing activities within the City of Russellville and its Planning Area.

Proper management of stormwater runoff will minimize damage to public and private property, ensure a functional drainage system, reduce local flooding, maintain as nearly as possible the pre-developed runoff characteristics of the area, and facilitate economic development while mitigating associated flooding and drainage impacts.

Use of the Manual

The use of this MANUAL and the provisions expressed herein shall be the minimum stormwater management requirements and shall not be deemed a limitation or repeal of any other powers granted by State statute. In addition, if site characteristics indicate that complying with these minimum requirements will not provide adequate designs or protection for local property or residents, it is the designer's responsibility to exceed the minimum requirements as necessary. The CITY ENGINEER or his/her designee shall be responsible for the coordination and enforcement of the provisions of this MANUAL and shall make the final determination of all questions related to this MANUAL and the Stormwater Management and Drainage Ordinance (SMDO). Should any criteria in this MANUAL conflict with criteria specifically defined by the SMDO, the criteria of the SMDO shall prevail.

The MANUAL presents information relative to drainage policies, submittal and review of drainage investigations/plans, procedures for hydrologic and hydraulic analyses and design. Although the intention of this MANUAL is to establish uniform design practices, it neither replaces the need for engineering judgment nor precludes the use of information not presented. Other accepted engineering procedures may be used to conduct hydrologic and hydraulic studies if approved by the CITY ENGINEER. Approval by the CITY ENGINEER or his/her designee will be obtained in writing, prior to the commencement of any alternative procedure and/or analysis, for any deviation from the criteria expressed in this MANUAL.

For the ease of those proposing land disturbing activities in the City of Russellville and within its Planning Area and for City personnel charged with the responsibility of reviewing such proposed activities for compliance with the Russellville SMDO, this edition of the Stormwater Management Drainage Manual is divided into five major divisions. The divisions are: DIVISION I – STORMWATER MANAGEMENT CONCEPT PLAN; DIVISION II – MINOR DEVELOPMENTS; DIVISION III – MAJOR DEVELOPMENTS; DIVISION IV – ASSOCIATED DRAINAGE; and DIVISION V – DEFINITIONS, ABBREVIATIONS, AND SYMBOLS.

DIVISIONS II and III are divided into the following sections: Submittal Procedures, Hydrology, Hydraulics, Floodplain/Floodway Policies, Erosion and Sedimentation Control, and Maintenance. Each section of each DIVISION addresses requirements of this MANUAL and the SMDO for the type of land disturbing activity proposed.

DIVISION IV presents criteria for drainage components often associated with either type of development. DIVISION V presents definition of terms and abbreviations used in this MANUAL and commonly used in discussions regarding the topics discussed herein. It also provides a list of symbols to be used in the preparation of all plans, maps, and other documents related to submittals required by the SMDO and this MANUAL.

Appendices A, B, and C discuss issues that should be considered during the planning of stormwater management facilities in the City of Russellville and its Planning Area.

The information provided therein is generally for the consideration of the users of this MANUAL and does not necessarily represent any additional requirements not already defined in the SMDO.

Appendix A presents a copy of the Arkansas Stormwater Law. Appendix B presents an overview of Floodplain Management issues. Appendix C presents an overview of Water Quality and Environmental issues.

DIVISION I. DEVELOPMENT CLASSIFICATION

1.0 GENERAL

For the purpose of this MANUAL, developments or land disturbing activities within the City of Russellville and its Planning Area shall be classified as either MINOR DEVELOPMENTS or MAJOR DEVELOPMENTS.

A land disturbing activity involving 2.5 acres or less of actual land disturbance or development of less than 40,000 square feet of impervious area shall be classified as a MINOR DEVELOPMENT unless determined otherwise by the CITY ENGINEER.

A land disturbing activity of more than 2.5 acres of actual land disturbance or development of greater than 40,000 square feet of impervious area shall be classified as a MAJOR DEVELOPMENT.

Prior to the production of any detailed plans for a land disturbing activity, including a Stormwater Management Plan, the proposed developer shall submit a completed Development Classification Form to the CITY ENGINEER for review and approval.

2.0 PURPOSE

The purpose of the Development Classification Form is to minimize the effort of the developer in the preparation of submittals and the design of drainage facilities in the planning of the proposed land disturbing activity. It will determine the appropriate classification of development and define the procedures and criteria to be used in the further planning of the proposed development.

3.0 REVIEW

The CITY ENGINEER or his/her designee will review the Development Classification Form to determine the appropriateness of the proposed classification of the development and the design methods proposed by the developer. An office and/or field review of the proposed development may be required before a final determination can be made.

4.0 ACTIONS

After all reviews are complete by the CITY ENGINEER, the developer shall receive a copy of the signed Development Classification Form with a determination from the CITY ENGINEER of the classification of proposed land development and the applicable requirements. The original copy of that Form will be retained in the records of the CITY ENGINEER until all features of the proposed land disturbing activities and Stormwater Management Plan have been completed, inspected by the CITY ENGINEER, and accepted by the City. An official determination by the CITY ENGINEER is required prior to the commencement of any further activities pertaining to the proposed land disturbing activity.

5.0 APPEALS

If, after receipt of the decision of the CITY ENGINEER, the developer can not reach an agreement with the CITY ENGINEER'S decision, he may appeal that decision pursuant to SECTION B of the Stormwater Management and Drainage Ordinance (SMDO).

DEVELOPMENT CLASSIFICATION FORM
City of Russellville, Arkansas

Project Name: ____
General Location: ____
Project Developer: ____
Address: ____
Address: ____
Telephone: ____

Please provide the following information:

Will this development disturb more than 2.5 acres? ______Yes ______No

Will this development involve more than 40,000 sq ft of impervious area (buildings, pavement, sidewalks, etc.)? ______Yes ______No

Is this development located within an area shown on a Flood Insurance Rate Map as a Floodplain and/or Floodway? ______Yes ______No

Brief narrative description of the proposed development: ______________________
____
____
____
____
____

Provide a sketch of the site and proposed development with the following information:

⇒ Site boundaries with total area shown in acres
⇒ Location by Township, Range, Section, ¼ Section
⇒ Development "footprint" (anticipated building outline) with anticipated square footage shown
⇒ Parking areas, walkways, and other impervious areas
⇒ Adjacent streets
⇒ General existing drainage flow paths of storm runoff on site
⇒ Existing drainage from adjacent property
⇒ Existing streams, ditches or other channels adjacent to or across any portion of the proposed site
⇒ Existing bridges, culverts, and/or storm sewers adjacent to or across any portion of the proposed site; please note size, type, and material
⇒ Proposed drainage flow paths on site and anticipated discharge offsite

DEVELOPMENT CLASSIFICATION FORM

Project Name:______________________________
Project Developer:__________________________

Proposed Development Classification: ______ MINOR ______ MAJOR

Action: Approved / Disapproved By: ______________________________ Date: ______________

CITY ENGINEER, City of Russellville

Recommendations/Additional Requirements: ____

DIVISION II - MINOR DEVELOPMENTS

1.0 GENERAL

Land disturbing activities in the City of Russellville and its Planning Area involving 2.5 acres or less of actual land disturbance or development of less than 40,000 square feet of impervious area which are not part of a larger common plan of development or sale, shall be considered as a MINOR DEVELOPMENT. In no case will a proposed development that is situated partially or totally within a delineated Floodway be classified as a MINOR DEVELOPMENT. A Development Classification Form discussed in DIVISION I, shall be submitted to the CITY ENGINEER for the establishment of the appropriate classification of development and the associated requirements. Unless otherwise approved by the CITY ENGINEER in writing, all MINOR DEVELOPMENTS shall conform to the requirements of this DIVISION.

2.0 SUBMITTAL REQUIREMENTS

Unless otherwise deemed necessary by the CITY ENGINEER, the person responsible for the land disturbing activity classified as a MINOR DEVELOPMENT shall submit a Simplified Stormwater Management Plan meeting the requirements listed in Section II-2.1. This plan does not require preparation or certification by a Professional Engineer unless deemed necessary by the CITY ENGINEER.

2.1 Simplified Stormwater Management Plan Requirements

The requirements for the Simplified Stormwater Management Plan include:

1. A narrative description of the stormwater management facilities to be used.
2. A general description of topographic and soil conditions of the development site.
3. A general description of adjacent property and a description of existing structures, buildings, and other fixed improvements located on surrounding properties.
4. A sketch plan that contains:
    - a site location drawing of the proposed project, indicating the location of the proposed project in relation to roadways, jurisdictional boundaries, streams, and rivers;
    - the boundary lines of the site on which the work is to be performed;
    - all areas within the site that will be included in the land disturbing activities shall be identified and the total disturbed area calculated;
    - a topographic map of the site;
    - anticipated starting and completion dates of the various stages of land disturbing activities and the expected date the final stabilization will be completed; and

- the location of temporary and permanent vegetative and structural stormwater management measures.

5. Certification by the persons responsible for the land disturbing activity that the land disturbing activity will be accomplished pursuant to the plan.
6. Certification by the persons responsible for the land disturbing activity that the CITY ENGINEER has the right to conduct on-site inspections.

3.0 HYDROLOGY

3.1 General

A detailed hydrologic analysis is generally not required for developments classified as MINOR DEVELOPMENTS. A Development Classification Form shall be submitted to the CITY ENGINEER for the establishment of the appropriate classification of development and the associated requirements. Unless otherwise specified by the CITY ENGINEER, the method, criteria, and other requirements for the discharge computations for a MINOR DEVELOPMENT shall be presented in this DIVISION. No analyses shall be performed nor submittals made prior to the determination and/or approval of the appropriate development classification by the CITY ENGINEER.

3.2 Hydrologic Criteria

Specific hydrologic criteria to be used for developing discharges with the Rational Method (see Section II-3.3.1) for the design of drainage facilities associated with a MINOR DEVELOPMENT shall be as follows:

1. Cross–Drain Drainage facilities - 4% annual chance design storm runoff.
2. Interior Drainage Facilities - 10% annual chance design storm runoff.
3. Detention/Retention - 4% annual chance design storm runoff.

   All hydrologic analyses will be based on land use conditions as follows:

   a. Outlet structures - Use developed land use conditions for the area within the proposed development and existing land use conditions for upstream areas draining to the facility.

   b. For an evaluation of the impact of the runoff from the site on flood flows downstream from the proposed facility, use existing land use conditions for all downstream areas.

   c. Emergency Spillways - Use the 1% annual chance storm and route flows through the facility and emergency spillways. For this analysis, developed land use conditions shall be used for all areas within the analysis.

4. All storms listed above are to be analyzed assuming a 24-hour duration.

3.3 Storm Runoff Computation-Rational Method

The method of storm runoff computations on which the design of storm drainage for MINOR DEVELOPMENTS shall be based is the Rational Method. However, the CITY ENGINEER may approve other engineering methods of analysis for calculation of stormwater runoff when they are shown to be comparable to the required method. Approval of any alternative method shall be acquired at the submittal of the Development Classification Form. Criteria for the Rational Method are specified in the following sections.

The Rational Method is probably the most frequently used rainfall-runoff method in urban hydrology in the United States. The Rational Method formula is expressed as:

$$\mathbf{Q = C\,I\,A} \qquad \textbf{(II-1)}$$

Q is defined as the peak rate of runoff in cubic feet per second (cfs). Actually, Q is in units of acre-inches per hour, but calculated results differ from cubic feet by less than 1%. Since the difference is so small, the Q value calculated by the equation is universally taken as cfs.

C is the dimensionless coefficient of runoff represented as the ratio of the amount of runoff to the amount of rainfall.

I is the average intensity of rainfall in inches per hour for a period of time equal to the critical time of full contribution of the drainage area under consideration. This critical time for full contribution is commonly referred to as the Time of Concentration (T_c).

A is the area in acres that contributes to runoff at the point of design or the point under consideration.

Basic assumptions associated with use of the Rational Method are as follows:

1. The computed peak rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.

2. The time of concentration is the critical value in determining the design rainfall intensity and is equal to the time required for water to flow from the hydraulically most distant point in the watershed to the point of design.

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

4. The rate of rainfall or rainfall intensity, I, is uniform for the entire duration of the storm event and is uniformly distributed over the entire watershed area.

3.3.1 Runoff Coefficient (C)

The runoff coefficient (C) is the proportion of the total rainfall that runs off an area. The amount of runoff depends on the relative porosity or imperviousness of the ground surface, the surface slope, and the ponding character of the surface. Impervious surfaces, such as asphalt pavements and building roofs, will be subject to nearly 100% runoff, regardless of the slope, after the surfaces have become thoroughly wet. Onsite inspection and aerial photographs are valuable in estimating the nature of the surfaces within the drainage area.

3.3.2 Soil

The runoff coefficient in the Rational Method formula is also dependent on the character of the soil. The type and condition of the soil determines its ability to absorb precipitation. The rate at which a soil absorbs precipitation generally decreases if the rainfall continues for an extended period of time. The soil absorption or infiltration rate is also influenced by the presence of soil moisture before a rain (antecedent condition), the rainfall intensity, the proximity of the groundwater table, the degree of soil compaction, the porosity of the subsoil, vegetation, ground slopes, and surface depressions.

3.3.3 Selection Of Runoff Coefficients

It should be noted that the runoff coefficient is the variable of the Rational Method that is least susceptible to precise determination. Proper selection requires judgement and experience on the part of the Engineer, and its use in the formula implies a fixed ratio for any given drainage area, which in reality is not the case. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff.

To standardize City design computations, Table II-1 represents runoff coefficient values to be used in the application of the Rational Method for determining peak runoff rates within the City of Russellville and its Planning Area. Artificially high C values will not be allowed to lessen the difference between pre-development versus post-development discharges.

Often a watershed or sub-watershed is composed of a number of different types of runoff conditions. For those situations, a Composite Runoff Coefficient Worksheet (Figure II-1) is provided to compute a weighted C value. Area calculations are completed for each land use type within the study area. Table II-2 lists C values to be used in the application of the Rational Method for composite areas. These values are used along with the area calculations to arrive at a weighted C for the watershed or sub-watershed under consideration. Areas can be measured either in acres or square miles but should be consistent for all areas being weighted. Weighted C values should be rounded to the nearest hundredth.

Runoff Coefficients and Percent Impervious

Land Use or Surface Characteristics	% Impervious	Frequency				
		2 yr	5 yr	10 yr	25 yr	100 yr
Business:						
Commercial Area	95	.87	.87	.88	.89	.89
Neighborhood Areas	70	.60	.65	.70	.75	.80
Residential:						
Single-family	*	.40	.45	.50	.55	.60
Multi-Unit (detached)	50	.45	.50	.60	.65	.70
Multi-Unit (attached)	70	.60	.65	.70	.75	.80
½ Acre Lot or Larger	*	.30	.35	.40	.50	.60
Apartments	70	.65	.70	.70	.75	.80
Industrial:						
Light Areas	80	.71	.72	.76	.78	.82
Heavy Areas	90	.80	.80	.85	.87	.90
Parks, Cemetaries	7	.10	.18	.25	.35	.45
Playgrounds	13	.15	.20	.30	.40	.50
Schools	50	.45	.50	.60	.65	.70
Railroad Yard Areas	20	.20	.25	.35	.40	.45
Undeveloped Areas:						
Historic Flow Analysis	2	See "Lawns"				
Offsite Flow Analysis (when land use not defined)	45	.43	.47	.55	.60	.65
Streets:						
Paved	100	.87	.88	.90	.92	.93
Gravel (Packed)	40	.40	.45	.50	.55	.60
Drive and Walks	96	.87	.87	.88	.88	.89
Roofs	90	.80	.85	.90	.90	.90
Lawns, Sandy Soil	0	.00	.01	.05	.15	.20
Lawns, Clayey Soil	0	.05	.15	.25	.40	.50

Note: These Rational Formula coefficients may not be valid for large basins. A composite coefficient should be developed from sub-basins of similar surface characteristics.

Source: Arkansas Highway and Transportation Department Drainage Design Manual

City of
RUSSELLVILLE
Arkansas

RUNOFF COEFFICIENT VALUES

TABLE II-1

Composite Runoff Coefficient Worksheet

Development:____

Sub-Watershed:____

Performed by:________________________Date:____________

Land Use	C	Acres	C x Acres
	Total		

$$\text{Weighted C} = \frac{\text{Total (C x Acres)}}{\text{Total Acres}} =$$

City of **RUSSELLVILLE** Arkansas	**RUNOFF COEFFICIENT WORKSHEET** **FIGURE II-1**

Typical Composite Runoff Coefficients, by Land Use*

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single Family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yards	0.20 to 0.35
Unimproved	0

* The ranges of C values presented are typical for return periods of 2-10 years. Higher values are appropriate for larger design storms. For storm events outside of this range, a multiplier may be required. See the discussion at the end of this section.

Normal Range of Runoff Coefficients*

Character Surface	Runoff Coefficients
Pavement	
Asphalt and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, Sandy Soil	
Flat (2%)	0.05 to 0.10
Average (2 to 7%)	0.10 to 0.15
Steep (>7%)	0.15 to 0.20
Lawns, Heavy Soil	
Flat (2%)	0.13 to 0.17
Average (2 to 7%)	0.18 to 0.22
Steep (>7 %)	0.25 to 0.35

* The ranges of C values presented are typical for return periods of 2-10 years. Higher values are appropriate for larger design storms. For storm events outside of this range, a multiplier may be required. See the discussion at the end of this section.

Source: ASCE Manual 77, Design and Construction of Urban Stormwater Management Systems

City of **RUSSELLVILLE** Arkansas	**RUNOFF COEFFICIENT VALUES BY LAND USE** **TABLE II-2**

3.3.4 Rainfall Intensity (I)

Rainfall intensity (I) is the design rainfall rate in inches per hour for a particular drainage basin or sub-basin. The rainfall instensity is selected on the basis of the design rainfall duration and frequency of occurrence. The design duration is equal to the time of concentration for a drainage area under consideration. Once the time of concentration is known, the design intensity of rainfall may be determined from the rainfall Intensity-Duration-Frequency Chart (see Table II-3) or the rainfall Intensity-Duration-Frequency curves for the City of Russellville (see Figure II-2). The frequency of occurrence is a statistical variable that may be established by the City standards or chosen by the Engineer as a design parameter.

3.3.5 Time Of Concentration (T_c)

The time of concentration (T_c), sometimes referred to as "travel time", used in the Rational Method is a measure of the time of travel required for runoff to reach the design point or the point under consideration along a flow path from the highest elevation of the watershed or sub-watershed. This relationship and the Rational Method formula state that the rate of runoff is equal to the rate of supply (rainfall excess) if the rainfall event lasts long enough to permit the entire watershed to contribute. These assumptions may not involve significant errors for watersheds several acres in size. However, errors may be involved with significant channel and overland flow storage effects.

There are two types of flow used in calculating the design time of concentration: overland flow and channelized flow. Overland flow is defined as that portion of the flow pattern that results in thin sheet flow across a given area. Channelized flow is characterized by significant depth accumulation either in a swale, ditch, natural channel, improved channel, or pipe system.

For urban areas, the time of concentration consists of an inlet time or overland flow time (t_I) plus the time of travel (t_T) in the storm sewer, paved gutter, roadside drainage ditch, or drainage channel.

For non-urban areas, the time of concentration consists of an overland flow time (t_I) plus the time of travel (t_T) in a combined form, such as a small swale, channel, or drainageway. The latter portion of the time of concentration can be estimated from the hydraulic properties of the storm sewer, gutter, swale, or other channel, and will vary with surface slope, surface cover, and distance of surface flow. Thus, the time of concentration can be calculated for both urban and non-urban areas:

$$\mathbf{T_c = t_I + t_T} \qquad \textbf{(II-2)}$$

Where:

T_c = Time of concentration (minutes)
t_I = Initial, inlet, or overland flow time (minutes)
t_T = Travel time in the ditch, channel, gutter, storm sewer (minutes)

The Engineer may use either Equation II-3 or Equation II-4 for the initial time of concentration.

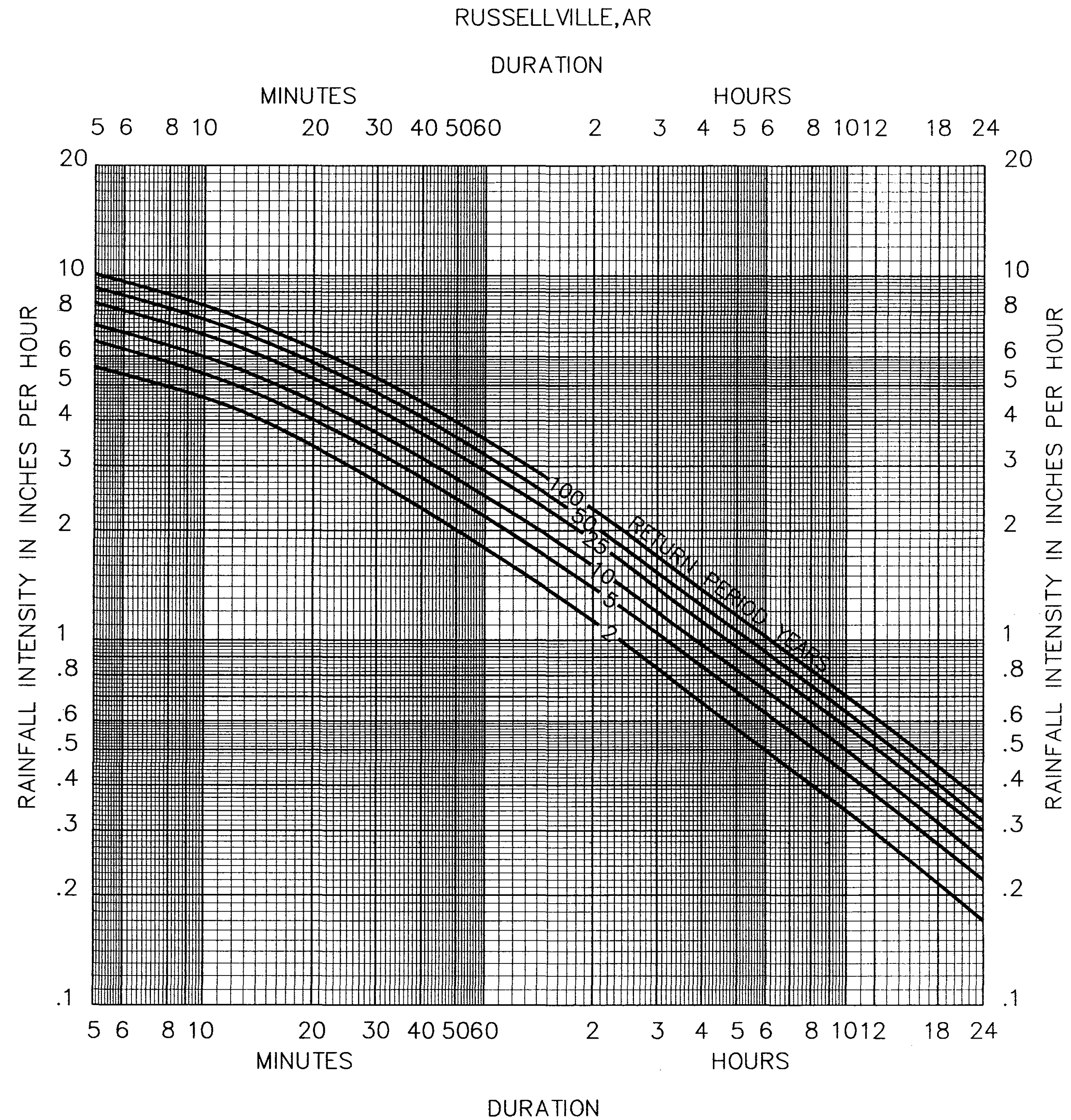
Duration Minutes	2 Years	5 Years	10 Years	25 Years	50 Years	100 Years
5	5.60	6.60	7.32	8.40	9.20	10.08
6	5.35	6.30	6.98	8.00	8.80	9.60
7	5.23	6.00	6.70	7.65	8.45	9.20
8	4.95	5.75	6.40	7.35	8.10	8.90
9	4.80	5.55	6.20	7.10	7.80	8.55
10	4.63	5.45	6.00	6.90	7.60	8.30
11	4.45	5.20	5.80	6.65	7.30	8.00
12	4.30	5.05	5.60	6.45	7.10	7.80
13	4.20	4.90	5.45	6.30	6.90	7.55
14	4.05	4.70	5.30	6.10	6.70	7.30
15	3.92	4.60	5.10	5.90	6.55	7.10
16	3.80	4.45	5.00	5.75	6.35	6.95
17	3.70	4.35	4.85	5.60	6.20	6.75
18	3.60	4.25	4.75	5.45	6.05	6.60
19	3.50	4.15	4.60	5.35	5.90	6.40
20	3.40	4.00	4.50	5.20	5.75	6.30
21	3.30	3.90	4.40	5.10	5.65	6.20
22	3.25	3.85	4.30	5.00	5.52	6.05
23	3.17	3.77	4.22	4.90	5.40	5.90
24	3.10	3.70	4.15	4.80	5.30	5.80
25	3.00	3.60	4.05	4.70	5.20	5.70
26	2.95	3.52	4.00	4.60	5.10	5.60
27	2.90	3.47	3.90	4.52	5.00	5.50
28	2.82	3.40	3.82	4.42	4.92	5.40
29	2.77	3.35	3.75	4.37	4.85	5.30
30	2.72	3.30	3.70	4.30	4.78	5.20
31	2.67	3.22	3.65	4.22	4.70	5.15
32	2.62	3.17	3.57	4.15	4.60	5.07
33	2.57	3.10	3.50	4.10	4.55	5.00
34	2.52	3.05	3.46	4.02	4.47	4.90
35	2.47	3.00	3.40	3.95	4.40	4.80
36	2.45	2.95	3.35	3.90	4.32	4.75
37	2.40	2.90	3.30	3.85	4.27	4.70
38	2.37	2.87	3.25	3.80	4.20	4.62
39	2.33	2.82	3.20	3.73	4.15	4.55
40	2.30	2.80	3.15	3.70	4.10	4.50
41	2.27	2.75	3.10	3.62	4.03	4.45
42	2.23	2.70	3.07	3.57	3.97	4.37
43	2.20	2.67	3.02	3.52	3.90	4.30
44	2.17	2.63	2.97	3.48	3.87	4.27
45	2.15	2.60	2.93	3.45	3.82	4.20
46	2.12	2.57	2.90	3.40	3.78	4.17
47	2.07	2.52	2.87	3.36	3.72	4.10
48	2.05	2.50	2.83	3.30	3.68	4.07
49	2.03	2.47	2.80	3.27	3.65	4.00
50	2.00	2.45	2.77	3.25	3.60	3.97
51	1.98	2.40	2.73	3.20	3.57	3.92
52	1.96	2.37	2.70	3.17	3.52	3.87
53	1.94	2.35	2.67	3.13	3.47	3.82
54	1.92	2.32	2.65	3.10	3.43	3.78
55	1.89	2.30	2.62	3.07	3.40	3.75
56	1.86	2.28	2.58	3.03	3.37	3.72
57	1.84	2.26	2.56	3.00	3.32	3.67
58	1.82	2.23	2.53	2.97	3.30	3.62
59	1.80	2.20	2.51	2.92	3.27	3.60
1 hr	1.78	2.18	2.49	2.90	3.23	3.55
2 hr	1.13	1.40	1.60	1.91	2.08	2.30
3 hr	0.80	1.05	1.19	1.38	1.53	1.70
6 hr	0.49	0.63	0.73	0.84	0.92	1.02
12 hr	0.31	0.38	0.44	0.51	0.56	0.61
24 hr	0.17	0.22	0.25	0.30	0.32	0.36

Source: 5-60 min. NOAA HYDRO-35; 120 min. – 24 hr. Technical Paper No. 40

City of
RUSSELLVILLE
Arkansas

RAINFALL INTENSITY TABLE
(INCHES PER HOUR)

TABLE II-3



SOURCE: NOAA HYDRO-35
NOAA Technical Paper No. 40

City of **RUSSELLVILLE** Arkansas	**INTENSITY - DURATION - FREQUENCY CURVES** **FIGURE II-2**

A limitation on the time of concentration is usually placed on the calculations for the sub-basins in the watershed. Typical ranges for T_c are 5 to 30 minutes. For paved areas, the minimum shall be 5 minutes. For overland flow areas, the minimum T_c shall be 10 minutes subject to individual design conditions. The minimum T_c recommended for the first residential basin is 10 minutes and 5 minutes for the first industrial/commercial basin. The size of the basin may require adjustment until the minimum time is achieved.

If a watershed or basin involves a design time of concentration in excess of 30 minutes then the applicability of the Rational Method must be checked.

3.3.5.1 Non-Urbanized Watershed

The initial or overland flow time (t_I) in non-urbanized watersheds may be obtained by using Figure II-3 or calculated by the following Federal Aviation Administration (FAA) equation (FAA 1983):

$$\mathbf{t_I = 1.8\ (1.1\text{-}C)\ (L^{0.5})/(S^{0.333})} \qquad \textbf{(II-3)}$$

Where:

t_I = Initial or overland flow time (minutes)
C = Runoff coefficient
L = Length of overland flow (feet, 500-foot maximum)
S = Average basin slope (%)

Equation II-3 is considered generally appropriate for distances up to 500 feet. For longer basin lengths, the runoff will combine into drainage paths and the sheet flow assumption is no longer valid.

For non-urban watersheds, the FAA equation (Equation II-3) will be appropriate for the computation of overland flow time. The time of concentration would then be overland flow in combination with the travel time (t_T), which is calculated using the hydraulic properties of the swale, ditch, or channel. The travel time can be calculated by the SCS method with the following equation:

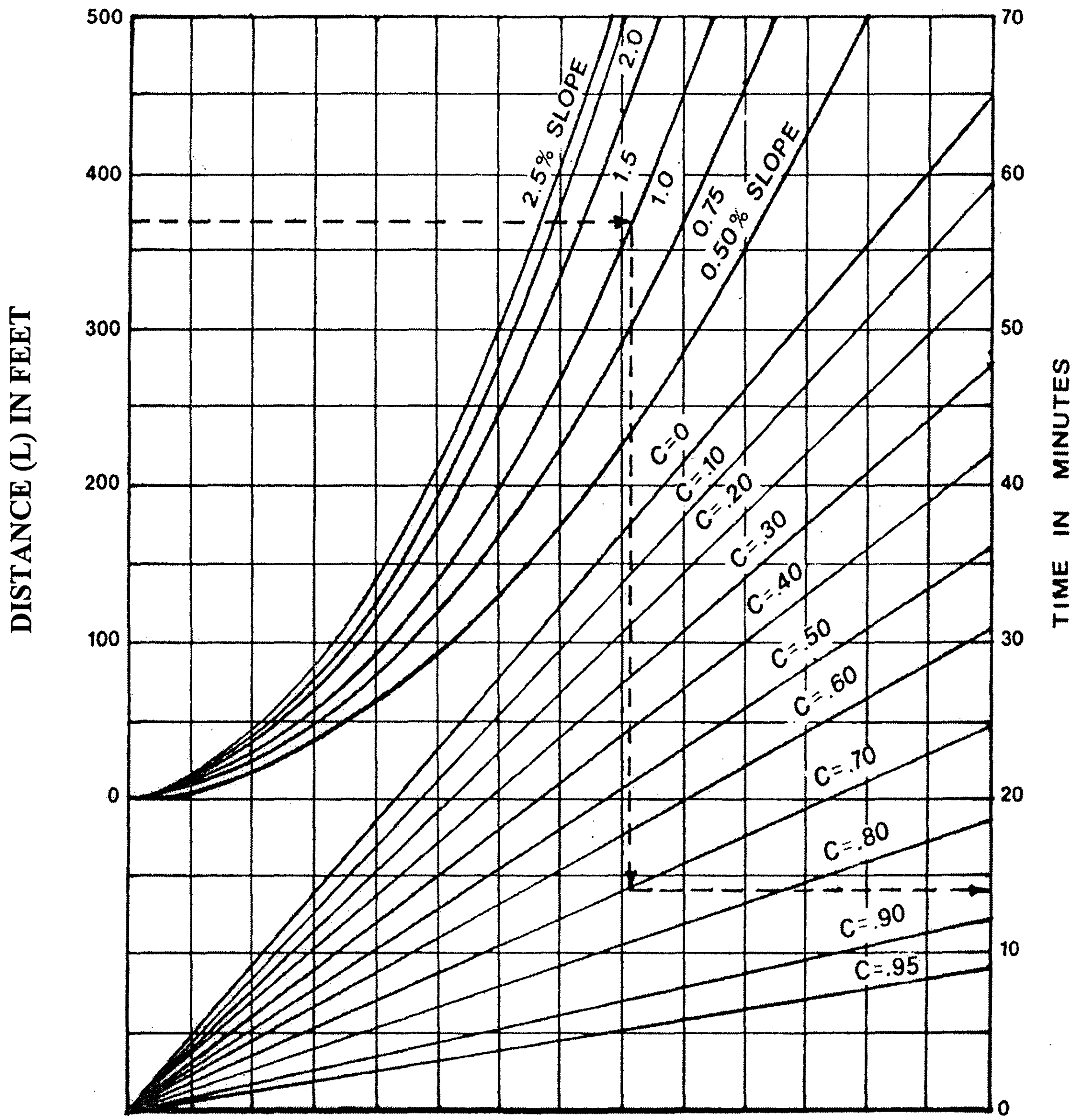
$$\mathbf{t_T = L/[60*V]} \qquad \textbf{(II-4)}$$

Where:

t_T = Travel time (minutes)
L = Length of channelized flow (ft)
V = Average velocity calculated for the channel (fps)

The constant 60 is a conversion value to convert from seconds to minutes. The travel time (t_T) for non-urbanized watersheds can also be estimated with the help of Figure II-3 or calculated with Equation II-4. The time of concentration is then the sum of the initial flow time (t_I) and the travel time (t_T).

In no case should a time of concentration of less than 5 minutes be used.



$$t = 1.8(1.1-C)(L^{0.5})/(S^{0.333})$$

Source: Urban Storm Drainage Criteria Manual, UDFCO, 1984

City of **RUSSELLVILLE** Arkansas	**OVERLAND TIME OF FLOW CURVES** **FIGURE II-3**

The process of calculating the time of concentration in a non-urbanized watershed is illustrated in the following example:

Example No. 1: Time of Concentration in Non-Urbanized Watershed

Given: A 15-acre non-urbanized range land watershed with generally clayey soils. Watershed has a length of 660 feet and an average slope of 1.0%. Uppermost 400 feet of watershed has an average land slope of 2.0%.

Required: Time of concentration, T_c

Solution:

Step 1: For this example the runoff coefficient for this site was determined to be:

$$C = 0.50$$

Step 2: Find the initial (overland) flow time for the uppermost 400 feet of the watershed. From Figure II-3 or Equation II-3, t_I for the 400-foot sub-watershed length, slope, and C value is:

$$t_I = 1.8(1.1\text{-}0.50)(400^{0.5})/(2.00^{0.333})$$
$$t_I = 17 \text{ minutes}$$

Step 3: Find the travel time for the remaining 260 feet of the watershed length. From Figure II-4, the average travel velocity for the given watershed, slope, and Short Grass Pasture curve is:

$$V = 0.7 \text{ fps}$$

The travel time is calculated using this velocity and 260 feet of travel length in Equation II-4.

$$t_T = L/[60*V] = 260 \text{ ft}/[(60\text{sec/min})(0.7 \text{ fps})]$$

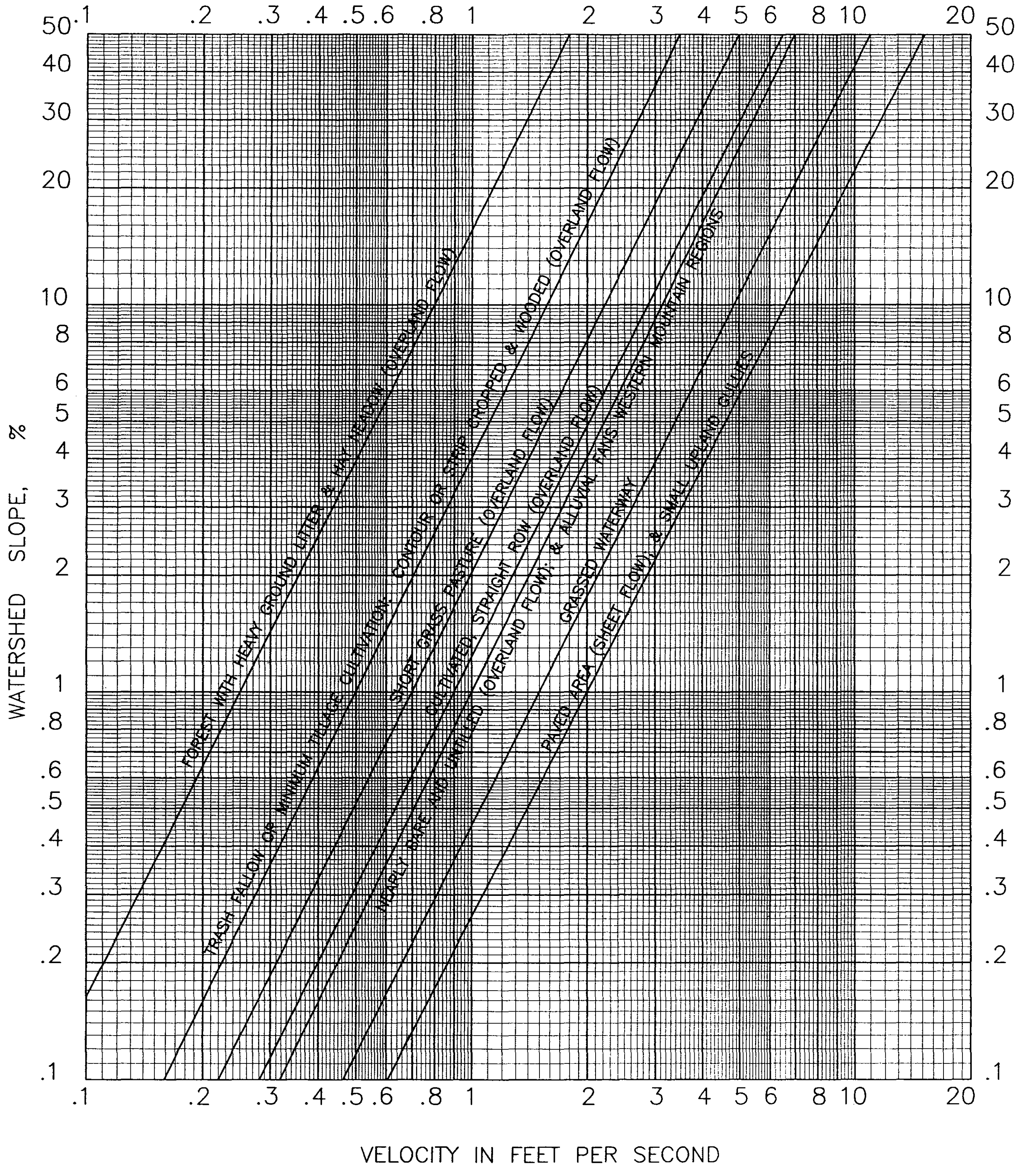
$$t_T = 6 \text{ minutes}$$

Step 4: Combine $t_I + t_T$

$$T_c = 17 + 6 = 23 \text{ minutes}$$

3.3.5.2 Urbanized Watershed

Overland flow in urbanized basins can occur from the back of the lot to the street, in parking lots, in greenbelt areas, or within park areas. The flow can be calculated using the procedure described in Section II-3.3.1, except the travel time t_T to the first design point or inlet is



SOURCE: National Resources Conservation Service
Technical Release No. 55

City of **RUSSELLVILLE** Arkansas	**RATIONAL METHOD** **VELOCITY FOR ESTIMATING TIME OF CONCENTRATION, T_c** **FIGURE II-4**

estimated using the "Paved Area (Sheet Flow) & Shallow Gutter Flow" line in Figure II-4. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream drainageway reaches.

The Design Engineer may use the following kinematic wave equation to calculate the initial time of concentration where applicable, especially in highly developed areas.

$$t_I = .938\,(n^{0.6} * L^{0.6})/(I^{0.4} * S^{0.3}) \qquad \text{(II-5)}$$
(English Units)

Where:

t_I = Initial or overland flow time (minutes)
n = Mannings roughness coefficient
L = Length of overland flow (ft)
I = Rainfall intensity (inches/hour)
S = Average basin slope (%)

The kinematic wave equation may be appropriate for highly developed areas but involves an iterative process since I and t_I are both undefined. In no case should a time of concentration of less than 5 minutes be used.

The process of calculating the time of concentration in an urbanized watershed is illustrated by the following example:

<u>Example No. 2: Time of Concentration in Urbanized Watershed</u>

Given: A 100-acre single-family residential development. The first watershed has a total length of 300 feet. The upper overland flow portion is from the back of a lot and is 100 feet in length at an average slope of 2.0%. The overland flow is mostly over grass and a short driveway, which have a composite runoff coefficient of 0.50. The lower 200 feet of the first watershed has an average slope of 1.0%.

Required: Time of concentration at the lower end of first area located 300 feet from the top of the first watershed.

Solution:

Step 1: Find the initial (overland) flow time for the uppermost 100 feet at 2.0% slope. From Figure II-3 or Equation II-3.

$$t_I = 1.8(1.1 - 0.50)(100^{0.5})/(2.0^{0.333})$$
$$t_I = 8.6 \text{ minutes}$$

Step 2: Using "Paved Area (Sheet Flow) & Shallow Gutter Flow" curve on Figure II-4, find the average flow velocity for the remaining 200 feet at 1.0% slope:

$$V = 2.0 \text{ fps}$$

Step 3: Calculate the travel time t_T using the velocity found in Step 2 and Equation II-4.

$$t_T = L/[60*V] = 200 \text{ feet } /[(60 \text{ sec/min})(2.0 \text{ fps})]$$
$$t_T = 1.7 \text{ minutes}$$

Step 4: Calculate the time of concentration to the first inlet using Equation II-2.

$$T_c = t_I + t_T$$
$$T_c = 8.6 + 1.7 = 10.3 \text{ minutes}$$

Step 5: The time of concentration calculations are continued in the downstream direction. The flow calculated at each design point is used to calculate the flow velocity in the downstream pipe, gutter, swale, or channel. This flow velocity is then used to calculate the time of travel to the next downstream design point.

Individual travel times are accumulated in a downstream direction to calculate the time of concentration at each successive downstream design point.

3.3.6 Drainage Area (A)

The drainage area (A) is generally measured in acres when using the Rational Method. Drainage areas should be delineated and calculated using maps with a 2-foot contour interval. This information may be obtained through the CITY ENGINEER.

4.0 HYDRAULICS

A detailed hydraulic analysis is generally not required for a development classified as a MINOR DEVELOPMENT. A Development Classification Form shall be submitted to the CITY ENGINEER for the establishment of the appropriate classification of development and the associated requirements. No analyses shall be performed nor submittals made prior to the determination and/or approval of the appropriate development classification by the CITY ENGINEER. Should hydraulic computations be required or performed, they shall comply with the criteria presented in Division III of this MANUAL, including all design computations required for detention/retention facilities, where applicable to watershed/sub-watershed areas of 2.5 acres or less.

5.0 FLOODPLAIN/FLOODWAY POLICES

Generally a proposed development situated partially or totally within a delineated "100-year Flood Boundary" (1% annual chance flood event) area in a Flood Insurance Study will not be classified as a MINOR DEVELOPMENT. The CITY ENGINEER may approve such a classification with the requirement of more detailed hydrologic and hydraulic analyses to demonstrate that the proposed development will not violate City and/or Federal Emergency Management Agency regulations. In no case will a proposed development that is situated partially or totally within a delineated Floodway be classified as a MINOR DEVELOPMENT. Further discussion regarding Floodplain Management is presented in Appendix B. ?

6.0 EROSION AND SEDIMENTATION CONTROL

A proposed development classified as a MINOR DEVELOPMENT will not be required to submit a detailed Erosion and Sedimentation Control Plan. The developer shall submit a narrative description of the control measures that will be instituted during the construction of the proposed development. The CITY ENGINEER may impose additional requirements as conditions for the approval of the Development Classification Form. An assurance in writing shall be submitted by the OWNER to the CITY ENGINEER that control measures will be instituted to minimize erosion/sedimentation during the land disturbing activities. The OWNER shall be responsible for reasonably re-vegetating the disturbed areas so as to minimize erosion/sedimentation after construction is completed. No action, or lack thereof, by the CITY ENGINEER shall relieve the OWNER of the responsibility to minimize erosion/sedimentation problems that may develop during and following the proposed land disturbing activities.

7.0 MAINTENANCE

A proposed development classified as a MINOR DEVELOPMENT will not be required to submit a detailed Maintenance Plan. The developer shall submit a narrative description of the maintenance program that will be instituted after the development is in place to insure that it is maintained and functioning properly. The CITY ENGINEER may impose additional requirements as conditions for the approval of the Development Classification Form. An assurance in writing shall be submitted by the OWNER to the CITY ENGINEER that maintenance of the Stormwater Management facilities will be performed. The OWNER shall grant the right of the CITY ENGINEER and/or his representative to inspect the Stormwater Management facilities at his discretion. No action taken or not taken by the CITY ENGINEER shall relieve the OWNER of the responsibility to provide regular maintenance to the Stormwater Management facility serving the proposed development.

DIVISION III - MAJOR DEVELOPMENTS

1.0 GENERAL

Land disturbing activities involving more than 2.5 acres of actual land disturbance or development of greater than 40,000 square feet of impervious area shall be considered as a MAJOR DEVELOPMENT. A Development Classification Form discussed in DIVISION I shall be submitted to the CITY ENGINEER for the establishment of the appropriate classification of development and the associated requirements. No further plans, designs, analyses, or other actions in regard to the proposed development shall be performed until the Development Classification Form has been submitted and approved by the CITY ENGINEER.

2.0 SUBMITTAL REQUIREMENTS

The OWNER or the OWNER'S representative responsible for a proposed land disturbing activity classified as a MAJOR DEVELOPMENT shall submit a detailed Stormwater Management Plan meeting the requirements listed in the following sections. This Plan requires preparation (or supervision) and certification by a Professional Engineer registered in the State of Arkansas, unless approved otherwise by the CITY ENGINEER.

2.1 Stormwater Concept Plan

Prior to any other plan development, the OWNER shall submit a Stormwater Concept Plan for review and approval by the CITY ENGINEER. The Concept Plan shall include a narrative description of the concept of the drainage activities proposed, the anticipated methodologies to be used, and sufficient details of the anticipated stormwater management facilities to be provided. The Concept Plan shall be prepared (or supervised) by a registered Professional Engineer.

After review of the Stormwater Concept Plan, the CITY ENGINEER may require an additional review with the OWNER's Engineer to clarify elements of the anticipated Stormwater Management Plan. Written approval of the Stormwater Concept Plan must be received prior to the preparation of a Stormwater Management Plan or any other plans regarding the proposed MAJOR DEVELOPMENT.

2.2 Stormwater Management Plan

Only after the review and approval of the Stormwater Concept Plan by the CITY ENGINEER shall the OWNER proceed with the development of the Stormwater Management Plan. No site development plans are to be prepared until the Stormwater Management Plan has been prepared (or supervised) by a Professional Engineer registered in the State of Arkansas and submitted and approved by the CITY ENGINEER. The CITY ENGINEER may require a review meeting to clarify elements of the Plan. The Stormwater Management Plan shall become a part of the Site Development Plans.

2.2.1 Stormwater Management Plan Requirements

The Stormwater Management Plan shall be prepared (or supervised) by a Professional Engineer registered in the State of Arkansas and shall include the requirements described in the following sections of this MANUAL and Article II, Sections B through J of the SMDO.

2.2.2 Detailed Site Plan

The OWNER's Engineer will prepare, as a part of the Stormwater Management Plan, a detailed Site Plan that shall include the following minimum information:

1. A General Location Map at a scale of 1 inch = 200 feet to 1 inch = 2000 feet showing the following:

    a. A "True North" arrow of adequate size and properly located so as to clearly reflect the correct orientation of the site.

    b. A scale, including both a graphical scale bar and a written scale (i.e., 1 inch = 500 feet).

    c. The complete property boundaries of the site, including all easements with their defined purposes shown.

    d. A delineation of the overall drainage area boundary and drainage sub-area boundaries above and through the site.

    e. Other information necessary to locate the site including streets, roads, railroads, and highways as well as defined water courses.

2. The General Location Map may be a separate drawing or a part of an overall Site Plan drawing. The Site Plan drawing shall include the following:

    a. Existing and proposed contours at maximum of 2-foot contour interval. If the topography is too flat to reflect the slope of the terrain, spot elevations shall be provided with slope arrows to define the direction of slope.

    b. Existing and proposed physical developments on the site and within 200 feet of the perimeter of the site shall be clearly noted and identified.

    c. Existing drainage features, natural and man-made, including natural streams, ditches, storm sewers, culverts, and swales, or drainageways. The direction of flow will be shown for each drainage feature. The information for all regular shaped features, such as culverts and man-made ditches, will include the material, size, shape, and slope.

d. A clear physical description of all proposed stormwater management facilities included in the Stormwater Management Plan. The information to be shown will include that specified above for "existing" drainage features and the point of outfall to a receiving stream or stormwater collector.

e. All proposed land disturbing activities will be clearly identified and quantified.

f. All proposed temporary and permanent vegetative and structural stormwater management control measures (erosion and sediment control measures).

g. A bar graph shall be included representing the anticipated construction schedule for each item or phase of the proposed land disturbing activities.

h. The expected or planned date of final stabilization of the site after the completion of all proposed land disturbing activities.

i. As required by the proposed stormwater management features and to comply with the requirements of the SMDO and this MANUAL, all proposed Right-of-Way shall be clearly shown and identified.

j. The OWNER shall provide the following statement and signature:

"I hereby certify that the drainage and/or grading plans approved by the CITY ENGINEER as shown on the Stormwater Management Plan will be implemented under the direct supervision of a Professional Engineer registered in the State of Arkansas."

Date: ____________ (__________________________)
Signature
(__________________________)
Printed Name

2.2.3 Detailed Drainage Report

In addition to the drawing(s) specified above, the OWNER shall submit a detailed drainage report describing all existing and proposed hydrologic analyses and hydraulic analyses/designs. The analyses/designs and the subsequent report shall be performed/prepared by a Professional Engineer registered in the State of Arkansas. The report will include a cover letter prepared, signed, and stamped (with his/her Professional Engineer's seal) by the Professional Engineer responsible for the analyses and designs for the Stormwater Management Plan. The report shall include a copy of the "approved" Development Classification Form. The report will include all assumptions, data, data sources, and design computations used in the analyses and designs. If electronic computations are performed, the report shall include an electronic copy of all input and output files and a printed listing of all data utilized in the analyses and designs. All deviations and/or variances from the requirements of the SMDO and this MANUAL shall be described and justified by written approval from the CITY ENGINEER.

3.0 HYDROLOGY

3.1 General

A detailed hydrologic analysis is required for a development classified as a MAJOR DEVELOPMENT. A Development Classification Form shall be submitted to the CITY ENGINEER for the establishment of the appropriate classification of development and the associated requirements. Unless otherwise specified by the CITY ENGINEER, the method, criteria, and other requirements for the discharge computations for a MAJOR DEVELOPMENT shall be presented in this SECTION. No analyses shall be performed nor submittals made prior to the determination and/or approval of the appropriate development classification by the CITY ENGINEER.

3.2 Hydrologic Criteria

Specific hydrologic criteria to be used for developing discharges for the design of drainage facilities associated with a MAJOR DEVELOPMENT shall be as follows:

1. Cross – Drain Drainage Facilities - 4% annual chance storm runoff.

2. Interior Drainage Facilities - 10% annual chance storm runoff.

3. Detention/Retention - 4% annual chance design storm runoff.

4. All hydrologic analyses will be based on land use conditions as follows:

    a. Outlet Structures - Use developed land use conditions for the area within the proposed development and existing land use conditions for upstream areas draining to the facility.

    b. For an evaluation of the impact of the runoff from the site on flood flows downstream from the proposed facility, use existing land use conditions for all downstream areas.

    c. Emergency Spillways - Use the 1% annual chance storm and route flows through the facility and emergency spillways. For this analysis, developed land use conditions shall be used for all areas within the analysis.

5. A secondary outlet device or emergency spillway shall be provided for in the design of storage facilities to discharge the excess runoff in such a way that no

danger of loss of life or facility failure is created. The size of the outlet device or emergency spillway shall be designed to pass the 1% annual chance storm as a minimum requirement.

6. All storms listed above are to be analyzed assuming a 24-hour duration.

3.3 Storm Runoff Computation-Rational Method

The methods of storm runoff computations on which the design of storm drainage for MAJOR DEVELOPMENT within the City of Russellville and its Planning Area shall be based are the Rational Method, Soil Conservation Service (SCS) Method TR-55, and HEC-1. However, the CITY ENGINEER may approve other engineering methods of analysis for calculation of stormwater runoff when they are shown to be comparable to the required methods. Approval of any alternative method shall be acquired at the submittal of the Development Classification Form. Criteria for each of these methods are specified in the following sections.

The Rational Method is probably the most frequently used rainfall-runoff method in urban hydrology in the United States. The Rational Method formula is expressed as:

$$\mathbf{Q = C\ I\ A} \qquad \textbf{(III-1)}$$

Q is defined as the peak rate of runoff in cubic feet per second (cfs). Actually, Q is in units of acre-inches per hour, but calculated results differ from cubic feet by less than 1%. Since the difference is so small, the Q value calculated by the equation is universally taken as cfs.

C is the dimensionless coefficient of runoff represented as the ratio of the amount of runoff to the amount of rainfall.

I is the average intensity of rainfall in inches per hour for a period of time equal to the critical time of full contribution of the drainage area under consideration. This critical time for full contribution is commonly referred to as the Time of Concentration (T_c).

A is the area in acres that contributes to runoff at the point of design or the point under consideration.

Basic assumptions associated with use of the Rational Method are as follows:

1. The computed peak rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.
2. The time of concentration is the critical value in determining the design rainfall intensity and is equal to the time required for water to flow from the hydraulically most distant point in the watershed to the point of design.
3. The ratio of runoff to rainfall, C is uniform during the entire duration of the storm event.

4. The rate of rainfall or rainfall intensity, I is uniform for the entire duration of the storm event and is uniformly distributed over the entire watershed area.

3.3.1 Runoff Coefficient (C)

The runoff coefficient (C) is the proportion of the total rainfall that runs off an area. The amount of runoff depends on the relative porosity or imperviousness of the ground surface, the surface slope, and the ponding character of the surface. Impervious surfaces, such as asphalt pavements and building roofs, will be subject to nearly 100% runoff, regardless of the slope, after the surfaces have become thoroughly wet. Onsite inspection and aerial photographs are valuable in estimating the nature of the surfaces within the drainage area.

3.3.2 Soil

The runoff coefficient in the Rational Method formula is also dependent on the character of the soil. The type and condition of the soil determines its ability to absorb precipitation. The rate at which a soil absorbs precipitation generally decreases if the rainfall continues for an extended period of time. The soil absorption or infiltration rate is also influenced by the presence of soil moisture before a rain (antecedent condition), the rainfall intensity, the proximity of the groundwater table, the degree of soil compaction, the porosity of the subsoil, vegetation, ground slopes, and surface depressions.

3.3.3 Selection of Runoff Coefficients

It should be noted that the runoff coefficient is the variable of the Rational Method that is least susceptible to precise determination. Proper selection requires judgement and experience on the part of the Engineer, and its use in the formula implies a fixed ratio for any given drainage area, which in reality is not the case. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff.

To standardize City design computations, Table III-1 represents standard runoff coefficient values to be used in the application of the Rational Method for determining peak runoff rates within the City of Russellville and its Planning Area. Artificially high C values will not be allowed to lessen the difference between pre-development versus post-development discharges.

Often a watershed or sub-watershed is composed of a number of different types of runoff conditions. For those situations, a Composite Runoff Coefficient Worksheet (Figure III-1) is provided to compute a weighted C value. Area calculations are completed for each land use type within the study area. Table III-1 lists C values to be used in the application of the Rational Method. These values are used along with the area calculations to arrive at a weighted C for the watershed or sub-watershed under consideration. Areas can be measured either in acres or square miles but should be consistent for all areas being weighted. Weighted C values should be rounded to the nearest hundredth.

Runoff Coefficients and Percent Impervious

Land Use or Surface Characteristics	% Impervious	Frequency 2 yr	5 yr	10 yr	25 yr	100 yr
Business:						
Commercial Area	95	.87	.87	.88	.89	.89
Neighborhood Areas	70	.60	.65	.70	.75	.80
Residential:						
Single-family	*	.40	.45	.50	.55	.60
Multi-Unit (detached)	50	.45	.50	.60	.65	.70
Multi-Unit (attached)	70	.60	.65	.70	.75	.80
½ Acre Lot or Larger	*	.30	.35	.40	.50	.60
Apartments	70	.65	.70	.70	.75	.80
Industrial:						
Light Areas	80	.71	.72	.76	.78	.82
Heavy Areas	90	.80	.80	.85	.87	.90
Parks, Cemetaries	7	.10	.18	.25	.35	.45
Playgrounds	13	.15	.20	.30	.40	.50
Schools	50	.45	.50	.60	.65	.70
Railroad Yard Areas	20	.20	.25	.35	.40	.45
Undeveloped Areas:						
Historic Flow Analysis	2	See "Lawns"				
Offsite Flow Analysis (when land use not defined)	45	.43	.47	.55	.60	.65
Streets:						
Paved	100	.87	.88	.90	.92	.93
Gravel (Packed)	40	.40	.45	.50	.55	.60
Drive and Walks	96	.87	.87	.88	.88	.89
Roofs	90	.80	.85	.90	.90	.90
Lawns, Sandy Soil	0	.00	.01	.05	.15	.20
Lawns, Clayey Soil	0	.05	.15	.25	.40	.50

Note: These Rational Formula coefficients may not be valid for large basins. A composite coefficient should be developed from sub-basins of similar surface characteristics.

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RUNOFF COEFFICIENTS AND PERCENT IMPERVIOUS

TABLE III-1

Composite Runoff Coefficient Worksheet

Development:____

Sub-Watershed:____

Performed by:____________________________Date:______________

Area No.	**Land Use**	**C**	**Acres**	**C x Acres**
	Total			

$$\text{Weighted C} = \frac{\text{Total (C x Acres)}}{\text{Total Acres}} =$$

City of **RUSSELLVILLE** Arkansas	**COMPOSITE RUNOFF COEFFICIENT WORKSHEET** **FIGURE III-1**

3.3.4 Rainfall Intensity (I)

Rainfall intensity (I), is the design rainfall rate in inches per hour for a particular drainage basin or sub-basin. The rainfall intensity is selected on the basis of the design rainfall duration and frequency of occurrence. The design duration is equal to the time of concentration for the drainage area under consideration. Once the time of concentration is known, the design intensity of rainfall may be determined from the rainfall Intensity-Duration-Frequency Table (Table III-2) or the rainfall Intensity-Duration-Frequency Curves for the City of Russellville (Figure III-2). The frequency of occurrence is a statistical variable that may be established by the City standards or chosen by the Engineer as a design parameter.

3.3.5 Drainage Area

The drainage area (A) is measured in acres when using the Rational Method. Drainage areas should be delineated and calculated using 2-foot contour information. Such information may be obtained through the CITY ENGINEER.

3.3.6 Time of Concentration (T_c)

The time of concentration (T_c), sometimes referred to as "travel time", used in the Rational Method is a measure of the time of travel required for runoff to reach the design point, or the point under consideration, along a flow path from the highest elevation of the watershed or sub-watershed. This relation and the Rational Method formula state that the rate of runoff is equal to the rate of supply (rainfall excess) if the rainfall event lasts long enough to permit the entire watershed to contribute. These assumptions may not involve significant errors for watersheds several acres in size. However, errors may be involved with significant channel and overland flow storage effects.

There are two types of flow used in calculating the design time of concentration: overland flow and channelized flow. Overland flow is defined as that portion of the flow pattern that results in thin sheet flow across a given area. Channelized flow is characterized by significant depth accumulation either in a swale, ditch, natural channel, improved channel, or pipe system.

For urban areas, the time of concentration consists of an inlet time or overland flow time (t_I) plus the time of travel (t_T) in the storm sewer, paved gutter, roadside drainage ditch, or drainage channel.

For non-urban areas, the time of concentration consists of an overland flow time (t_I) plus the time of travel (t_T) in a combined form, such as a small swale, channel, or drainageway. The latter portion of the time of concentration can be estimated from the hydraulic properties of the storm sewer, gutter, swale, or other channel, and will vary with surface slope, surface cover, and distance of surface flow. Thus, the time of concentration can be calculated for both urban and non-urban areas:

Duration Minutes	2 Years	5 Years	10 Years	25 Years	50 Years	100 Years
5	5.60	6.60	7.32	8.40	9.20	10.08
6	5.35	6.30	6.98	8.00	8.80	9.60
7	5.23	6.00	6.70	7.65	8.45	9.20
8	4.95	5.75	6.40	7.35	8.10	8.90
9	4.80	5.55	6.20	7.10	7.80	8.55
10	4.63	5.45	6.00	6.90	7.60	8.30
11	4.45	5.20	5.80	6.65	7.30	8.00
12	4.30	5.05	5.60	6.45	7.10	7.80
13	4.20	4.90	5.45	6.30	6.90	7.55
14	4.05	4.70	5.30	6.10	6.70	7.30
15	3.92	4.60	5.10	5.90	6.55	7.10
16	3.80	4.45	5.00	5.75	6.35	6.95
17	3.70	4.35	4.85	5.60	6.20	6.75
18	3.60	4.25	4.75	5.45	6.05	6.60
19	3.50	4.15	4.60	5.35	5.90	6.40
20	3.40	4.00	4.50	5.20	5.75	6.30
21	3.30	3.90	4.40	5.10	5.65	6.20
22	3.25	3.85	4.30	5.00	5.52	6.05
23	3.17	3.77	4.22	4.90	5.40	5.90
24	3.10	3.70	4.15	4.80	5.30	5.80
25	3.00	3.60	4.05	4.70	5.20	5.70
26	2.95	3.52	4.00	4.60	5.10	5.60
27	2.90	3.47	3.90	4.52	5.00	5.50
28	2.82	3.40	3.82	4.42	4.92	5.40
29	2.77	3.35	3.75	4.37	4.85	5.30
30	2.72	3.30	3.70	4.30	4.78	5.20
31	2.67	3.22	3.65	4.22	4.70	5.15
32	2.62	3.17	3.57	4.15	4.60	5.07
33	2.57	3.10	3.50	4.10	4.55	5.00
34	2.52	3.05	3.46	4.02	4.47	4.90
35	2.47	3.00	3.40	3.95	4.40	4.80
36	2.45	2.95	3.35	3.90	4.32	4.75
37	2.40	2.90	3.30	3.85	4.27	4.70
38	2.37	2.87	3.25	3.80	4.20	4.62
39	2.33	2.82	3.20	3.73	4.15	4.55
40	2.30	2.80	3.15	3.70	4.10	4.50
41	2.27	2.75	3.10	3.62	4.03	4.45
42	2.23	2.70	3.07	3.57	3.97	4.37
43	2.20	2.67	3.02	3.52	3.90	4.30
44	2.17	2.63	2.97	3.48	3.87	4.27
45	2.15	2.60	2.93	3.45	3.82	4.20
46	2.12	2.57	2.90	3.40	3.78	4.17
47	2.07	2.52	2.87	3.36	3.72	4.10
48	2.05	2.50	2.83	3.30	3.68	4.07
49	2.03	2.47	2.80	3.27	3.65	4.00
50	2.00	2.45	2.77	3.25	3.60	3.97
51	1.98	2.40	2.73	3.20	3.57	3.92
52	1.96	2.37	2.70	3.17	3.52	3.87
53	1.94	2.35	2.67	3.13	3.47	3.82
54	1.92	2.32	2.65	3.10	3.43	3.78
55	1.89	2.30	2.62	3.07	3.40	3.75
56	1.86	2.28	2.58	3.03	3.37	3.72
57	1.84	2.26	2.56	3.00	3.32	3.67
58	1.82	2.23	2.53	2.97	3.30	3.62
59	1.80	2.20	2.51	2.92	3.27	3.60
1 hr	1.78	2.18	2.49	2.90	3.23	3.55
2 hr	1.13	1.40	1.60	1.91	2.08	2.30
3 hr	0.80	1.05	1.19	1.38	1.53	1.70
6 hr	0.49	0.63	0.73	0.84	0.92	1.02
12 hr	0.31	0.38	0.44	0.51	0.56	0.61
24 hr	0.17	0.22	0.25	0.30	0.32	0.36

Source: 5-60 min. NOAA HYDRO-35; 120 min. – 24 hr. Technical Paper No. 40

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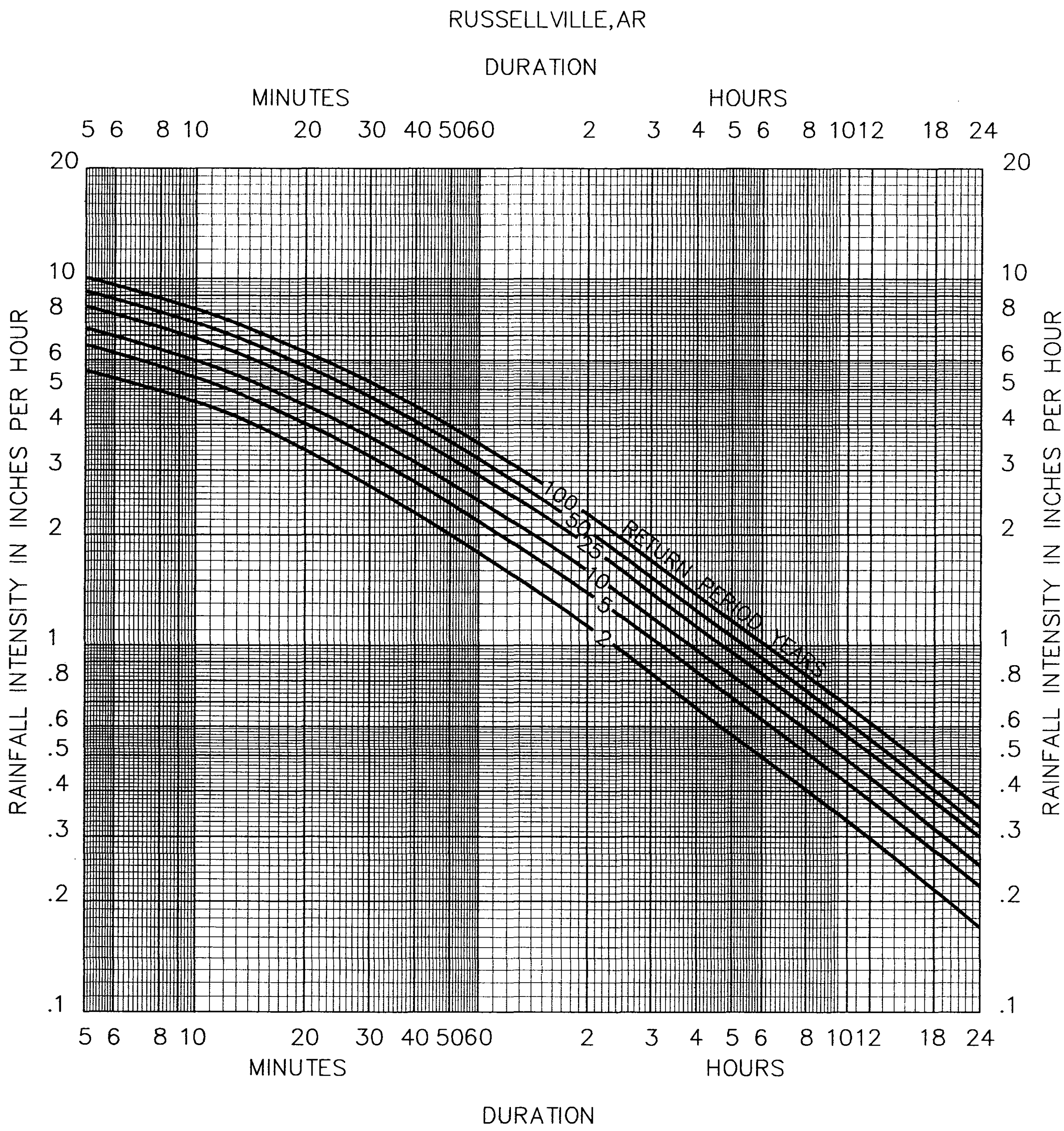
RAINFALL INTENSITY TABLE
(INCHES PER HOUR)

TABLE III-2



SOURCE: NOAA HYDRO-35
NOAA Technical Paper No. 40

City of **RUSSELLVILLE** Arkansas	**INTENSITY - DURATION - FREQUENCY CURVES** **FIGURE III-2**

$$T_c = t_I + t_T \quad \textbf{(III-2)}$$

Where:

T_c = Time of concentration (minutes)
t_I = Initial, inlet, or overland flow time (minutes)
t_T = Travel time in the ditch, channel, gutter, storm sewer (minutes)

The Engineer may use either Equation III-3 or Equation III-4 for the initial time of concentration. A limitation on the time of concentration is usually placed on the calculations for the sub-basins in the watershed. Typical ranges for T_c are 5 to 30 minutes. For paved areas, the minimum shall be 5 minutes. For overland flow areas, the minimum T_c shall be 10 minutes subject to individual design conditions. The minimum T_c recommended for the first residential basin is 10 minutes and 5 minutes for the first industrial/commercial basin. The size of the basin may require adjustment until the minimum time is achieved.

If a watershed or basin involves a design time of concentration in excess of 30 minutes then the applicability of the Rational Method must be checked.

3.3.6.1 Non-Urbanized Watershed

The initial or overland flow time (t_I) in non-urbanized watersheds may be obtained by using Figure III-3 or calculated by the following Federal Aviation Administration (FAA) equation (FAA 1983):

$$t_I = 1.8\,(1.1 - C)(L^{0.5})/(S^{0.333}) \quad \textbf{(III-3)}$$

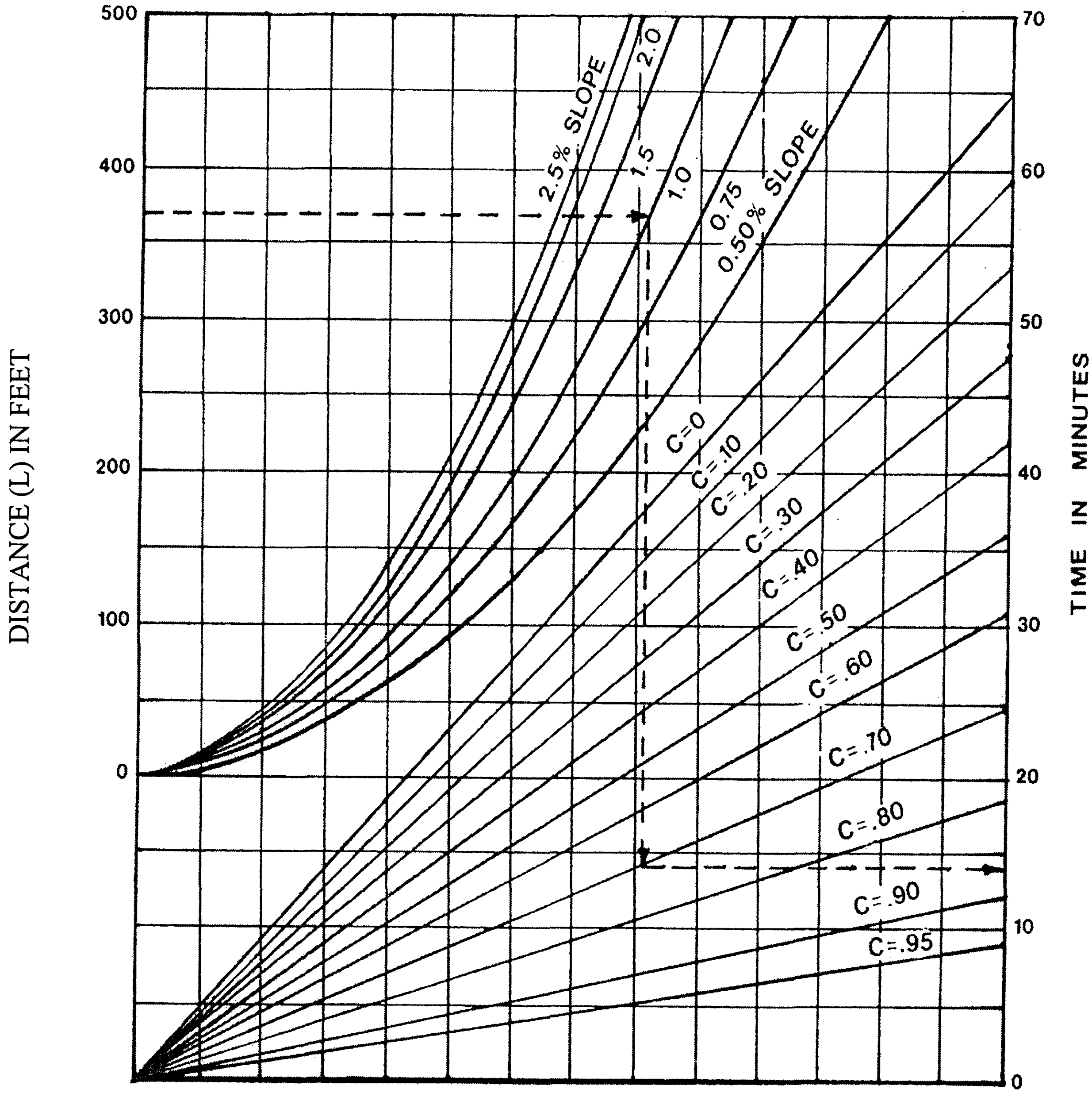
Where:

t_I = Initial or overland flow time (minutes)
C = Runoff coefficient
L = Length of overland flow (ft, 500-foot maximum)
S = Average basin slope (%)

Equation III-3 is considered generally appropriate for distances up to 500 feet. For longer basin lengths, the runoff will combine and the sheet flow assumption is no longer valid.

For non-urban watersheds, the FAA equation (Equation III-3) will be appropriate for the computation of overland flow time. The time of concentration would then be overland flow in combination with the travel time (t_T), which is calculated using the hydraulic properties of the swale, ditch, or channel. The travel time can be calculated by the SCS method with the following equation:

$$t_T = L/[60*V] \quad \textbf{(III-4)}$$



Source: Urban Storm Water Design Criteria Manual, DROG, Denver, Colorado, 1969

City of **RUSSELLVILLE** Arkansas	**OVERLAND TIME OF FLOW CURVES** **FIGURE III-3**

Where:

t_T = Travel time (minutes)
L = Length of channelized flow (ft)
V = Average velocity calculated for the channel (fps)

The constant 60 is a conversion value to convert from seconds to minutes. The travel time (t_T) for non-urbanized watersheds can also be estimated with the help of Figure III-3 or calculated with Equation III-4. The time of concentration is then the sum of the initial flow time (t_I) and the travel time (t_T).

In no case should a time of concentration of less than 5 minutes be used.

The process of calculating the time of concentration in a non-urbanized watershed is illustrated in the following example:

Example No. 1: Time of Concentration in Non-urbanized Watershed

Given: A 15-acre non-urbanized range land watershed with generally cohesive soils. Watershed has a length of 660 feet and an average slope of 1.0%. Uppermost 400 feet of watershed has an average land slope of 2.0%.

Required: Time of concentration, T_c

Solution:

Step 1: For this example, the runoff coefficient for this site was determined to be:

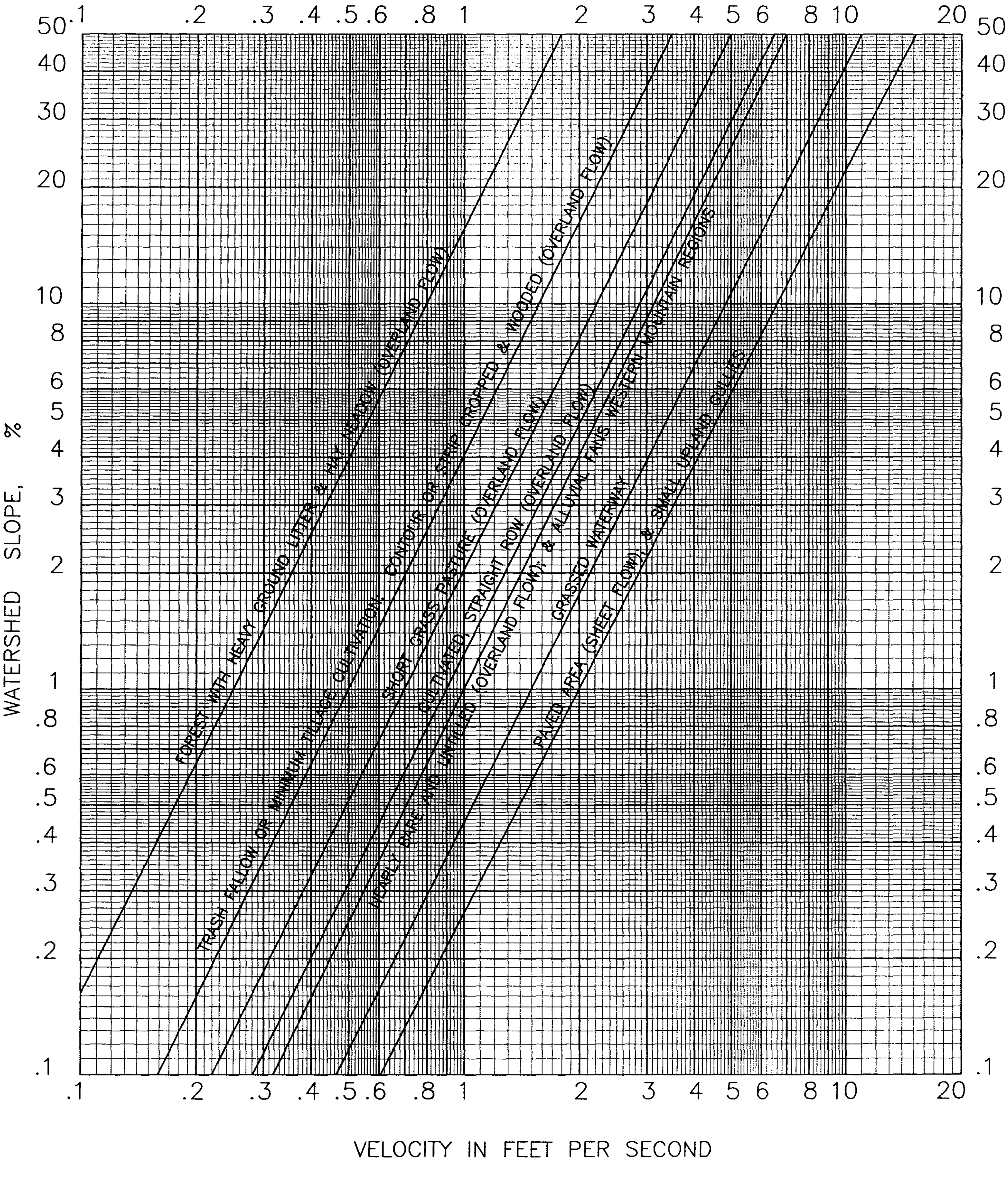
C = 0.50

Step2: Find the initial (overland) flow time for the uppermost 400 feet of the watershed. From Figure III-3 or Equation III-3, t_I for the 400-foot sub-watershed length, slope, and C value is:

$t_I = 1.8(1.1\text{-}0.50)(400^{0.5})/(2.00^{0.333})$
t_I = 17 minutes

Step3: Find the travel time for the remaining 260 feet of the watershed length. From Figure III-4, the average travel velocity for the given watershed, slope, and "Short Grass Pasture" curve is:

V = 0.7 fps



SOURCE: National Resources Conservation Service
Technical Release No. 55

City of **RUSSELLVILLE** Arkansas	**RATIONAL METHOD** **VELOCITY FOR ESTIMATING TIME OF CONCENTRATION, T_c** **FIGURE III-4**

The travel time is calculated using this velocity and 260 feet of travel length in Equation III-4.

$t_T = L/[60*V] = 260$ ft. / [(60sec/min) (0.7 fps)]
$t_T = 6$ minutes

Step 4: Combine $t_I + t_T$

$T_c = 17 + 6 = 23$ minutes

3.3.6.2 Urbanized Watershed

Overland flow in urbanized basins can occur from the back of the lot to the street, in parking lots, in greenbelt areas, or within park areas. The flow can be calculated using the procedure described in Section III-3.3.5.1 except the travel time t_T to the first design point or inlet which is estimated using the "Paved Area (Sheet Flow) & Shallow Gutter Flow" line on Figure III-4. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream drainageway reaches.

The Design Engineer may use the following kinematic wave equation to calculate the initial time of concentration where applicable, especially in highly developed areas.

$$t_I = .938\,(n^{0.6} * L^{0.6})/(I^{0.4} * S^{0.3}) \qquad \textbf{(III-5)}$$

(English Units)

Where:

t_I = Initial or overland flow time (minutes)
n = Mannings roughness coefficient
L = Length of overland flow (ft)
I = Rainfall intensity (inches/hour)
S = Average basin slope (%)

The kinematic wave equation may be appropriate for highly developed areas but involves an iterative process since I and t_I are both undefined. In no case should a time of concentration of less than 5 minutes be used.

The process of calculating the time of concentration in an urbanized watershed is illustrated by the following example:

Example No. 2: Time of Concentration in Urbanized Watershed

Given: A 100-acre single-family residential development. The first watershed has a total length of 300 feet. The upper overland flow portion is from the back of a lot and is 100 feet in length at an average slope of 2.0%. The overland flow is mostly over grass and a short driveway, which have a composite runoff coefficient of 0.50. The lower 200 feet of the first watershed has an average slope of 1.0%.

Required: Time of concentration at the lower end of first area located 300 feet from the top of the first watershed.

Solution:

Step 1: Find the initial (overland) flow time for the uppermost 100 feet at 2.0% slope. From Figure III-3 or Equation III-3.

$$t_I = 1.8(1.1\text{-}0.50)(100^{0.5})/(2.0^{0.333})$$
$$t_I = 8.6 \text{ minutes}$$

Step 2: Using "Paved Area (Sheet Flow) & Shallow Gutter Flow" curve on Figure III-4, find the average flow velocity for the remaining 200 feet at 1.0% slope:

$$V = 2.0 \text{ fps}$$

Step 3: Calculate the travel time t_T using the velocity found in Step 2 and Equation III-4.

$$t_T = L/[60{*}V] = 200 \text{ feet}/[(60 \text{ sec/min})(2.0 \text{ fps})]$$
$$t_T = 1.7 \text{ minutes}$$

Step 4: Calculate the time of concentration to the first inlet using Equation III-2.

$$T_c = t_I + t_T$$
$$T_c = 8.6 + 1.7 = 10.3 \text{ minutes}$$

Step 5: The time of concentration calculations are continued in the downstream direction. The flow calculated at each design point is used to calculate the flow velocity in the downstream pipe, gutter, swale, or channel. This flow velocity is then used to calculate the time of travel to the next downstream design point.

Individual travel times are accumulated in a downstream direction to calculate the time of concentration at each successive downstream design point.

3.4 Soil Conservation Service Method (TR-55)

3.4.1 General

The Soil Conservation Service (SCS) tabular method (SCS 1986) is a synthetic hydrograph method developed specifically for use in urbanized and urbanizing areas. This method is similar to the Rational Method in that runoff is directly related to rainfall amounts through use of runoff curve numbers (CNs) (See Tables III-3 and III-4 for urban, cultivated agricultural, and other agricultural areas, respectively). The basic equation used with the tabular method is also very similar to the one used for the Rational Method:

$$\mathbf{q = (DRO)(DA)(HDO)} \qquad \textbf{(III-6)}$$

Runoff Curve Numbers (CN) for Urban Areas[a]

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover type and Hydrologic Condition	Average Percent Impervious Area[b]	A	B	C	D
Fully developed urban area (vegetation established)[c]					
Open space (lawns, parks, golf courses, cemetries, etc.):					
Poor conditions (grass cover < 50%)		68	79	86	89
Fair conditions (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding ROW)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding ROW)		98	98	98	98
Paved; open ditches (including ROW)		83	89	92	93
Gravel (including ROW)		76	85	89	91
Dirt (including ROW)		72	82	87	89
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	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
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded area (pervious areas only, no vegetation)		77	86	91	94

[a]Average runoff condition and $I_2 = 0.2S$.

[b]The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious area are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

[c]CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Notes: CN = Curve number
ROW = Right of way

Source: Soil Conservation Service (SCS) Technical Release No. 55 (SCS 1986)

City of **RUSSELLVILLE** Arkansas	**RUNOFF CURVE NUMBERS FOR URBAN AREAS** **TABLE III-3**

Runoff Curve Numbers (CN) for Agricultural Areas[a]

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover type	Hydrologic Condition[c]	A	B	C	D
Pasture, grassland, or range – continuous forage for grazing[b]	Poor Fair Good	68 49 39	79 69 61	86 79 74	89 84 80
Meadow – continuous grass, protected from grazing and generally mowed for hay	--	30	58	71	78
Brush – brush-weed-grass mixture with brush the major element[c]	Poor Fair Good	48 35 30[d]	67 56 48	77 70 65	83 77 73
Woods – grass combination (orchard or tree farm)[e]	Poor Fair Good	57 43 32	73 65 58	82 76 72	86 82 79
Woods[f]	Poor Fair Good	45 36 30[d]	66 60 55	77 73 70	83 79 77
Farmsteads – buildings, lanes, driveways, and surrounding lots	--	59	74	82	86

[a]Average runoff condition and $I_2 = 0.2S$.
[b]*Poor* = < 50% ground cover or heavily grazed with no mulch
Fair = 50 to 75% ground cover and not heavily grazed.
Good = 75% ground cover and lightly or only occasionally grazed.
[c]*Poor* = < 50% ground cover.
Fair = 50 to 75% ground cover.
Good = 75% ground cover.
[d]Actual curve number is less than 30; use CN = 30 for runoff computations.
[e]CNs shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CNs for woods and pasture.
[f]*Poor* = Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair = Woods are grazed but not burned, and some forest litter covers the soil.
Good = Woods are protected from grazing, litter and brush adequately cover the soil.

Notes: CN = Curve number
ROW = Right of way

Source: Soil Conservation Service (SCS), Technical Release No. 55 (SCS 1986)

City of **RUSSELLVILLE** Arkansas	**RUNOFF CURVE NUMBERS FOR AGRICULTURAL AREAS** **TABLE III-4**

Where:

q = Hydrograph coordinate discharge (cfs)
DRO = Direct Runoff Amount (in)
DA = Drainage Area (square miles)
HDO = Hydrograph Distribution Ordinate (csm/inch)
csm/inch = Cubic feet per second per square mile per inch of runoff

Hydrograph coordinates are computed from the hydrograph distribution data in TR-55. A coordinated value is computed for each time shown in the distribution data. The calculated q results, when plotted against the corresponding times, constitute the runoff hydrograph.

The tabular method is useful in analyzing watersheds involving several sub-areas with complex runoff patterns. The method is most useful in analyzing changes in runoff volume due to development and in evaluating runoff control measures. The SCS tabular method, as described here, shall be used in all cases where watershed problems involve two or more interacting sub-areas. The SCS tabular method is the suggested method to be used in evaluating the runoff effects of urbanization and the evaluation/design of runoff control measures.

3.4.2 Method Fundamentals

The SCS has completed extensive research in the runoff potential from native soils under specific conditions of pre-wetting and rainfall events. This research has also extended to correlation of native soil types and land uses in assessing runoff potential. Runoff curve number values have been developed that approximate the runoff potential from various types of development with respect to native soils. These CN values are similar to runoff coefficient values used in the Rational Method in that they can be used to estimate the amount of rainfall that will actually result in runoff. The amount of runoff that will occur for a given CN value is a function of the design rainfall and is termed direct runoff amount (DRO).

The CN values differ from runoff coefficients in that:

1. Their development encompasses a wide range of land uses.
2. Runoff potentials from native soil types are taken into account.
3. The amount of runoff that will occur is the function of both the CN value and the design rainfall.

Design rainfalls used with the tabular method are 24-hour rainfall amounts taken from the US Weather Bureau data for each frequency event considered. The data include recurrence intervals or frequencies of occurrence of 10, 25, 50, and 100 years (10%, 4%, 2%, and 1% annual chance events, respectively).

Hydrograph distribution ordinates used in the tabular method were developed by computer analysis of many watersheds of various sizes and configurations. The distribution data published in TR-55 were developed specifically by computing hydrographs for a 1-square-mile drainage area for a range of times of concentration and routing of the hydrographs through stream reaches with a range of travel times.

One advantage of using the empirically based hydrograph distributions over simpler methods is that the channel storage and overland flow storage effects are taken into account. This feature is particularly useful in the cases involving larger, more complex watersheds.

The biggest advantage of the tabular method over simpler methods is that the runoff effects of different development patterns (both in land use and in drainage facilities) can be easily measured. The effects of a wide variety of runoff control measures can also be measured since the method's work result is in hydrograph form. These features are extremely valuable in watershed management efforts since differences in flow magnitudes are often more important in design decisions than are determinations of precise peak flow values for given conditions. Also, volumetric effects of runoff can be considered with hydrograph methods.

3.4.2.1 Limitations on Tabular Method Use

The tabular method should not be used when large changes in CN values occur among watershed sub-areas and when runoff volumes are less than about 1½ inches for CN values less than 60.

The tabular method should not be used for watersheds that have several sub-areas with times of concentration below 6 minutes. In these cases, sub-areas should be combined to produce a time of concentration of at least 6 minutes (0.10 hour) for the combined areas.

3.4.3 Determination of Runoff Curve Number (CN)

The CN determines the amount of runoff that will occur with the given rainfall. Soil types and land use are used to determine the runoff potential.

Calculation of the CN values for a watershed or sub-area proceeds in the same manner as the calculation of weighted runoff coefficients used in the Rational Method. Area calculations are completed for each land use type within the study area. Tables III-3 and III-4 list CNs for various land uses. A more complete table listing CN values for specific soil types and land coverages can be found in the TR-55 Manual. These values are used along with the area calculations to arrive at a weighted CN value for the watershed or sub-area under consideration. Figure III-5 is a worksheet that is useful in tabulating weighted CNs for watersheds and watershed sub-areas. Areas can be measured either in acres or square miles. Weighted CN values should be rounded to the nearest whole number.

3.4.4 Design Storm Data

The tabular method is based on 24-hour rainfall amounts for various design recurrence intervals or frequency of occurrence. These rainfall amounts for the City of Russellville are taken from the US Weather Bureau Technical Paper No. 40 (TP-40) (US Weather Bureau) and are as follows: 4.08 inches for the 2-year frequency; 6.0 inches for the 10-year frequency; 7.2 inches for the

Composite Curve Number Worksheet

Development:____

Sub-Watershed:____

Performed by:___________________________Date:______________

Area No.	Land Use	CN	Acres	CN x Acres
	Total			

$$\text{Weighted CN} = \frac{\text{Total (CN x Acres)}}{\text{Total Acres}} =$$

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COMPOSITE CURVE NUMBER WORKSHEET

FIGURE III-5

25-year frequency; 7.68 inches for the 50-year frequency; and 8.64 inches for the 100-year frequency.

3.4.5 Direct Runoff Amounts for Design Storm (DRO Values)

Table III-5 is a generalized table of direct runoff amounts for given rainfalls and CNs. This table can be used to interpolate runoff amounts (DRO values) from any combination of CN between 60 and 98 and rainfall amounts between 1 and 12 inches.

3.4.6 Tabular Method Computations

The methodology for the tabular method uses the tabular discharge values listed in tables for various times of concentration presented in Chapter 5 of TR-55, (SCS) and other publications addressing the tabular method such as McCuen (1982).

A table segment is provided for selected times of concentration (T_c): 0.1, 0.2, 0.3, 0.4, 0.5, 0.75, 1.0, 1.25, 1.5, and 2.0 hours. For values other than these select values, the closest value can be used. Additional precision can be achieved through interpolation.

Each table segment is further subdivided by the total travel time (t_T) from the subwatershed outlet to the design point. For each T_c and t_T, discharge rates are given in csm/inch of runoff for hydrograph times (i.e., the time from the beginning of precipitation) ranging from 11.0 hours to 20.0 hours in various time increments.

The main procedure used in solving hydrologic problems with the tabular discharge hydrograph method is to divide the watershed into appropriate sub-areas and identify the necessary input for each sub-area and channel reach. The hydrograph at the design point due to runoff from any sub-area is determined by entering the appropriate table in TR-55 for the computed sub-area T_c and the total travel time from the outlet of the sub-area to the design point. The total hydrograph is determined by summing the sub-area hydrographs. The solution procedure is best illustrated by examples taken from McCuen (1982).

Example 1

Given: A 1.1 square mile watershed is subdivided into two sub-areas. The upper portion of the watershed has a drainage area of 0.6 mi^2, a time of concentration of 2.0 hours, and a CN of 70. The lower portion has a drainage area of 0.5 mi^2, a time of concentration of 1.5 hours, and a CN of 75. The travel time from the outlet of the upper portion of the watershed through the channel in the lower portion of the watershed is 1.0 hours. A precipitation total of 7 inches was assumed for this example.

The hydrograph computations are illustrated in the following table. The runoff volumes (DRO), which were obtained from Table III-5, are 3.62 inches and 4.15 inches for the upper and lower portions of the watershed, respectively.

DIRECT RUNOFF VALUES
BY CN'S AND RAINFALL AMOUNTS

Rainfall	Runoff Curve Number (CN)								
(inches)	60	65	70	75	80	85	90	95	98
1.0	0	0	0	0.03	0.08	0.17	0.32	.56	.79
1.2	0	0	0.03	0.07	0.15	0.28	0.46	.74	.99
1.4	0	0.02	0.06	0.13	0.24	0.39	0.61	.92	1.18
1.6	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5	0.17	0.30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.33	0.51	0.72	0.96	1.25	1.59	1.98	2.45	2.78
4.0	0.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
5.0	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	1.92	2.35	2.80	3.28	3.78	4.31	4.85	5.41	5.76
7.0	2.60	3.10	3.62	4.15	4.69	5.26	5.82	6.41	6.76
8.0	3.33	3.90	4.47	5.04	5.62	6.22	6.81	7.40	7.76
9.0	4.10	4.72	5.34	5.95	6.57	7.19	7.79	8.40	8.76
10.0	4.90	5.57	6.23	6.88	7.52	8.16	8.78	9.40	9.76
11.0	5.72	6.44	7.13	7.82	8.48	9.14	9.77	10.39	10.76
12.0	6.56	7.32	8.05	8.76	9.45	10.12	10.76	11.39	11.76

NOTE: To obtain runoff depths for CN's and other rainfall amounts not shown in this table, use an arithmetic interpolation.

Source: Soil Conservation Service, Technical Release No. 55 (SCS 1968)

City of RUSSELLVILLE Arkansas	DIRECT RUNOFF VALUES TABLE III-5

Hydrograph Computations

Sub-area	Hydrograph Time Hours				
	12.8	13.0	13.2	13.5	14.0
1	2(3.6)(0.6)=91	4 (3.6)(0.6)=138	2(3.6)(0.6)=199	36(3.6)(0.6)=294	80(3.6)(0.6)=389
2	36(4.1)(0.5)=484	25(4.1)(0.5)=461	01(4.1)(0.5)=412	53(4.1)(0.5)=314	9(4.1)(0.5)=203
Total	575 cfs	599 cfs	611 cfs	608 cfs	529 cfs

The hydrograph computations for the upper portion (i.e., sub-area 1) yield the hydrograph from the precipitation excess for sub-area 1 as estimated for the outlet from sub-area 2 (i.e., the hydrograph is routed through the reach in sub-area 2). The computations show the unit discharges (csm/in) multiplied by the product of the runoff volume (in) and the drainage area (mi^2). The total flows indicate that the peak discharge of 611 cfs for the total watershed occurs at the hydrograph time of 13.2 hours. The peak from the upper and lower portions of the watershed occurred at 14.0 and 12.8 hours, respectively.

Example 2

Given: The watershed shown in Figure III-6 has been subdivided into four sub-watersheds, each having similar land use and soil type. The pertinent characteristics for each sub-watershed are given in the following table.

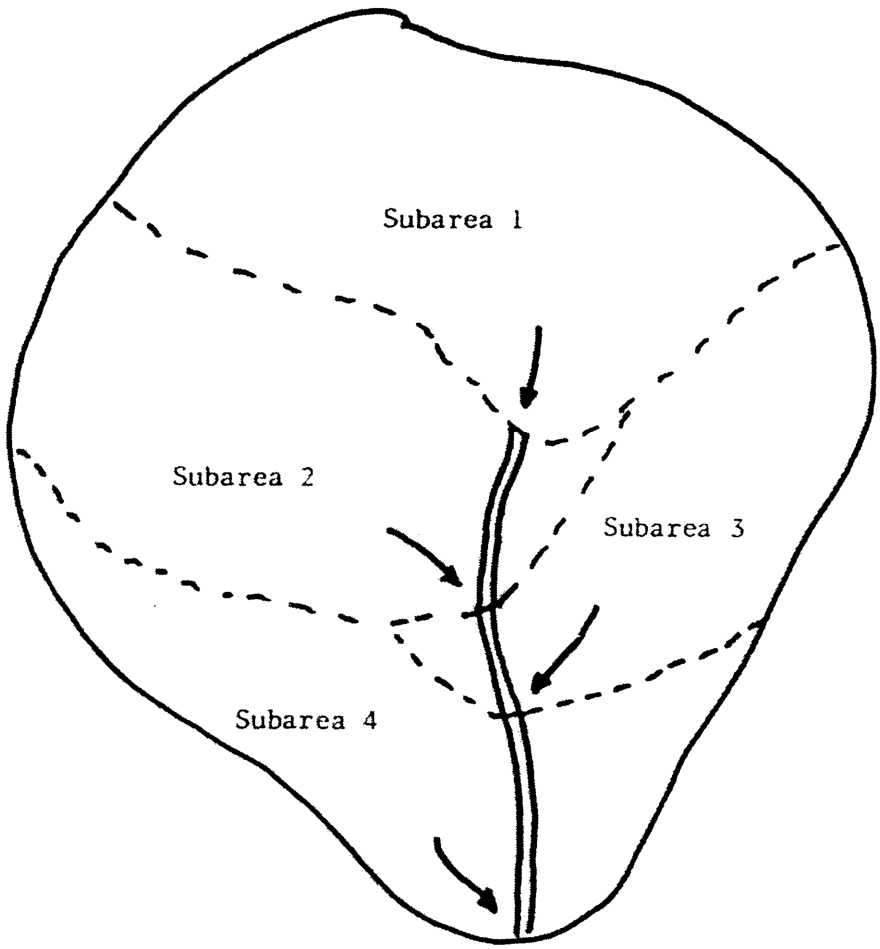
Watershed and Runoff Characteristics for P = 7 inches

Sub-area	Drainage Area (A) (mi^2)	T_c (hrs)	Runoff Curve Number (CN)	Direct Runoff (DRO) (inches)	Reach Travel Time (hrs)	AQ
1	0.40	2.0	67	3.3	-	1.32
2	0.25	1.5	71	3.7	0.75	0.92
3	0.20	1.0	75	4.1	0.25	0.82
4	0.30	1.25	81	4.8	1.00	1.44

The peak discharge as measured at the outfall of the total watershed but resulting from rainfall on the upstream sub-area 1 occurred at a hydrograph time of 15.0 hours while the peak discharge from sub-area 4 occurred at time 12.6 hours. The peak discharge of 574 cfs for the entire watershed occurred at 13.5 hours as shown in the following table.

Computation of Composite Flood Hydrograph

Sub-area	Hydrograph Time (hours)							
	12.6	12.8	13.0	13.2	13.5	14.0	14.5	15.0
1	8	9	13	18	34	94	176	224
2	24	40	64	95	141	186	163	115
3	53	99	153	197	215	150	88	54
4	390	369	315	255	184	117	81	60
Total	475 cfs	517 cfs	545 cfs	565 cfs	574 cfs	547 cfs	508 cfs	453 cfs



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WATERSHED DELINEATION EXAMPLE

FIGURE III-6

Figure III-7 presents a TR-55 Worksheet to facilitate the computation of runoff hydrographs for each sub-watershed and the combination of those hydrographs to produce the final discharge hydrograph at the point of design/analysis.

3.4.7 Computer Methods

Use of computer methods by experienced engineers is encouraged. Several hydrologic software developers have included the TR-55 methodology in their hydrologic design/analysis package. One example is PondPack developed by Haestad Methods. While this is primarily a software package for the design/analysis of detention ponds, it has a hydrologic component that includes the TR-55 methodology. Any computer software proposed to be utilized in the computation of peak discharges must be approved in writing by the CITY ENGINEER prior to its use.

3.5 HEC-1 Computer Model

3.5.1 General

A unit hydrograph method must be used for watersheds over 200 acres, which is the upper limit for the Rational Method. HEC-1 (Snyder or SCS method) is suitable for these and smaller watersheds (60 acres suggested minimum).

The HEC-1 computer program is a hydrologic simulation model developed by the US Army Corps of Engineers Hydrologic Engineering Center in Davis, California. The program has numerous options for determining and routing runoff hydrographs and is well documented. A user's manual is available from the Corps website <www.hec.usace.army.mil> and should be used by anyone performing this type of hydrologic modeling.

The HEC-1 model is designed to simulate the surface runoff response of a river basin to precipitation by representing the watershed as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a portion of the watershed, commonly referred to as a sub-watershed. A component may represent a surface runoff entity, a stream channel, or a reservoir. Representation of a component requires a set of parameters that specify the particular characteristics of the component and mathematical relations that describe the physical processes. The result of the modeling process is the computation of stream flow hydrographs at desired locations in the river basin.

3.5.2 Stream Network Model

A river basin is subdivided into an interconnected system of stream network components using topographic maps and other geographic information. Basin components are developed by the following steps:

1. The study area basin boundary is delineated first. In a natural or open area, this can be done from a topographic map. However, supplementary information such as development drainage maps may be necessary to obtain an accurate depiction of an urban basin's extent.

SUB AREA	T_c HR	T_T HR	AREA MI^2	RCN	DIRECT RUNOFF INCHES	DISCHARGE CSM/IN - CFS											
						HR	HR	HR	HR	HR	HR	HR	HR	HR	HR	HR	HR

Source: Technical Release No. 55, Urban Hydrology of Small Watersheds

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TR-55 WORKSHEET

FIGURE III-7

2. Segmentation of the basin into a number of sub-basins determines the number and types of stream network components to be used in the model. Two factors impact the basin segmentation: the study purpose and the hydrometeorological variability throughout the basin. First, the study purpose defines the areas of interest in the basin and hence, the points where sub-basin boundaries should occur.

   Second, the variability of the hydrometeorological processes and basin characteristics impact the number and location of sub-basins. Each sub-basin is intended to represent an area of the basin that, on the average, has the same hydraulic/hydrologic properties. Further, the assumption of uniform precipitation and infiltration over a basin becomes less accurate as the area becomes larger. Consequently, if the basins are chosen appropriately, the average parameters used in the components will be more representative, and the model results will be more accurate.

3. Each sub-basin is represented by a combination of model components. Sub-basin runoff, river routing, reservoir, and diversion components are available to the user.

4. The sub-basin and its components are linked together to represent the continuity of the river basin. HEC-1 has a number of methods for combining or linking outflow from different components. This step finalizes the basin schematic.

3.5.3 Subwatershed Hydrograph Model Component

The sub-watershed land surface runoff component is used to represent the movement of water over the land surface and in stream channels. The input to this component is a precipitation hyetograph. Precipitation excess is computed by subtracting infiltration and detention losses based on a soil-water infiltration rate function. Note that the rainfall and infiltration are assumed to be uniform over the watershed. The resulting rainfall excesses are then routed by the unit hydrograph to the outlet of the watershed producing a runoff hydrograph. The unit hydrograph technique produces a runoff hydrograph at the most downstream point in the watershed. If a location for the runoff computation is not appropriate, it may be necessary to further subdivide the area.

3.5.4 River Routing Model Component

A river routing component is used to represent flood wave movement in a river channel. The input to the component is an upstream hydrograph resulting from individual or combined contributions of sub-watershed runoff hydrographs, river routing, or diversions. The hydrograph is routed to a downstream point based on the characteristics of the channel.

A number of techniques are available to route the runoff hydrographs. The direct translation and Modified Puls methods are recommend. The direct translation routing should be used when the flood is in an improved channel or a natural channel that is significantly contained within its banks. Storage routing (Modified Puls method) should be used when a significant portion of the flood is carried in the overbank area resulting in large amounts of transient storage, or for long channel reaches where the lag time is about 15 minutes or more.

3.5.5 Reservoir Routing Model Component

Use of the reservoir routing component is similar to that of the river routing component. The reservoir routing component can be used to represent the storage-outflow characteristics of a reservoir, lake, detention pond, highway culvert, etc. (as well as channel storage). The reservoir component functions by receiving upstream inflows and routing these inflows through a reservoir using storage routing methods. Reservoir outflow is assumed to be solely a function of storage (or water surface elevation) in the reservoir and not dependent on downstream controls.

3.5.6 Routing and Combining Hydrographs

A suitable combination of the sub-watershed runoff and river routing components can be used to represent the intricacies of most rainfall-runoff and stream routing problems. The continuity of the stream network components is implied by the order in which the data components are arranged. Simulation must always begin at the uppermost sub-watershed in a branch of the stream network. The simulation (succeeding data components) proceeds downstream until a confluence is reached. Before simulating below the confluence, all flows above that confluence must be computed and routed to that confluence. The flows are combined at the confluence, and the combined flows can then be routed downstream.

3.5.7 Design Storm Precipitation

The peak rainfall increment will be in accordance with the balanced rainfall distribution concept, which places the peak increment at the center of the storm. Because of the balanced rainfall distribution, the calculation of infiltration is less sensitive, and the methodology of using 0.5 inches/hour initial rate and 0.05 inches/hour final rate is acceptable (See Section III-3.5.8).

A precipitation hyetograph is used as the input for all runoff calculations. The specified precipitation is assumed to be uniformly distributed over the watershed. The hyetograph represents average precipitation depths over a computation interval.

It is convenient to have the unit duration incremented in multiples of 2 or 5 minutes (e.g., 2, 5, 10, or 15 minutes) with the maximum unit duration recommended at 15 minutes under most circumstances. An acceptable unit storm duration should not exceed one-fifth of the time to peak of the watershed (t_p). As an example, if the watershed has a t_p of 35 minutes, then an appropriate unit storm duration would be 5 minutes.

Storm precipitation is input as incremental rainfall depths for each time increment in the design storm. Historic storm events can also be input by dividing the total rainfall into incremental depths over the computational interval.

3.5.8 Interception/Infiltration

Land surface interception, depression storage, and infiltration are referred to in the HEC-l model as precipitation losses. Interception and depression storage are intended to represent the surface storage of water by trees or grass, local depressions in the ground surface, in cracks and crevices in parking lots or roofs, or in a surface area where water is not free to move as overland flow. Infiltration represents the movement of water to areas beneath the land surface.

Two important factors should be noted about the precipitation loss computation in the model. First, precipitation that does not contribute to the runoff process is considered to be lost from the system. Second, the equations used to compute the losses do not provide for soil moisture or surface storage recovery. This fact dictates that the HEC-1 program is a single-event-oriented model.

The precipitation loss computations can be used with the unit hydrograph model components. In this case, the precipitation loss is considered to be a watershed average, uniformly distributed over an entire watershed.

In some instances, there are negligible precipitation losses for a portion of a watershed. This would be true for an area containing a lake, reservoir, or impervious area. In this case, precipitation losses will not be computed for a specified percentage of the area labeled as impervious.

An initial loss (0.5 inches/hour) and a constant loss rate (0.05 inches/hour) are to be specified. All rainfall is lost until the volume of initial loss is satisfied. After the initial loss is satisfied, rainfall is lost at the constant rate.

3.6 HEC-HMS

HEC-HMS is the successor to HEC-1. The Hydrologic Modeling System (HMS) is a Windows-based software package with a Graphical Users' Interface (GUI) that will provide the modeler with a "tool" to more easily model a hydrologic setting and be able to see a graphical representation of the watershed model and the resulting output. It is replacing HEC-1 except where HEC-1 is required for studies, such as Flood Insurance Studies where it is the "method of record".

4.0 HYDRAULICS

4.1 General

This section of the MANUAL contains a general description of the storm drainage facilities and their requirements that may be used to properly manage the storm runoff within the City of Russellville and its Planning Area. Facilities that are not properly designed are wasteful and often deceiving as to their ability to protect from damage resulting from certain storm runoff. It is the intent of this section of the MANUAL to discuss various types of drainage facilities and present design criteria that should be considered when developing a functional Stormwater Management Plan.

4.2 City of Russellville Hydraulic Design/Analysis Methods

Methods of hydraulic computations on which the design of drainage structures are developed are too numerous to generalize design/analysis procedures. This section will discuss typical drainage system components, basic design criteria, and typical design procedures. This aspect of stormwater management requires a thorough knowledge of hydraulic design and analysis and should be performed/directed by a Professional Engineer, registered in the State of Arkansas, and who has experience in the area of stormwater management design.

4.3 Open Channels

4.3.1 Introduction

Technical criteria for the hydraulic evaluation and hydraulic design of open channels including natural channels, grass, concrete or rock lined channels, composite channels, and roadside ditches is presented in this section. The individuals using this MANUAL are assumed to posses a working knowledge of hydraulics and have stormwater drainage evaluation experience. The user is encouraged to review the many referenced materials for additional information.

The open channel is usually the main facility for the drainage system (except for the roadside ditch). If the historic or natural drainage path is selected for the open channel route, the construction costs for the system can be minimized. The historic route along with the minimum alteration of the existing channel is recommended for all drainageways. In some instances, however, the land use of the property can be improved by relocating or straightening the natural drainage path, thereby improving the economic aspects of the project. Altering the channel alignment and shape may require the addition of grade control sections to control the flow velocities.

4.3.1.1 Channel Types

The channels in the City of Russellville area are defined as natural or artificial. Natural channels include all the primary water courses that have developed by the erosion process such as Prairie Creek, Whig Creek, and Engineers Ditch. Artificial channels are those constructed or significantly altered by human effort and include roadside ditches, grassed channels, and composite channels through the many subdivisions.

4.3.1.1.1 Natural Channels

The hydraulic properties of natural channels vary along the channel reach and can be either controlled to the extent desired or altered to meet given requirements. The initial decision to be made is whether or not the channel is to be protected from erosion due to high velocity flows, or protected from excessive silt deposition due to low velocities.

Many natural channels in un-urbanized areas have mild slopes, are reasonably stable, and are not in a state of serious degradation or aggradation. However, if a natural channel is to be used for carrying storm runoff from an urbanized area, the increased peaks, duration, and volume of runoff from urban development will cause erosion. Hydraulic analyses will be required for natural channels in order to identify the erosion tendencies. Some onsite modification of the natural channel may be required to assure a stabilized condition.

The investigations necessary to assure that the natural channels will be adequate are different for every waterway. The Engineer must prepare cross sections of the channel, define the water surface profile for the major design flood, investigate the bed and bank material to determine erosion tendencies, and study the bank slope stability of the channel under different flow conditions. Supercritical flow normally does not occur in natural channels, but calculations must be made to assure that the results do not reflect supercritical flow.

4.3.1.1.2 Grass-Lined Channels

Grass-lined channels are the most desirable of the artificial channels. The grass will stabilize the body of the channel, consolidate the soil mass of the bed, check the erosion on the channel surface, and control the movement of soil particles along the channel bottom. The channel storage, the lower velocities, and the greenbelt multiple-use-benefits obtained create significant advantages over other artificial channels.

The presence of grass in channels creates turbulence which results in loss of energy and increased flow retardance. Therefore, the Engineer must give full consideration to sediment deposition and scour, as well as hydraulics. Unless existing development restricts the availability of right-of-way, channels lined with grass should be given preference over other artificial types.

4.3.1.1.3 Concrete-Lined Channels

Concrete-lined channels are sometimes required where right-of-way restrictions within existing developments prohibit grass-lined channels. The lining must be designed to withstand the various forces and actions which tend to overtop the bank, deteriorate the lining, erode the soil beneath the lining, and erode unlined areas.
If the project constraints dictate the use of a concrete channel, such use shall be allowed only upon approval by the CITY ENGINEER.

4.3.1.1.4 Rock-Lined Channels

Rock-lined channels are constructed from ordinary riprap or wire enclosed riprap. The rock lining increases the turbulence resulting in a loss of energy and increased flow retardance. The rock lining also permits a higher design velocity and therefore a steeper design slope than for grass-lined channels. Rock linings will normally only be used for erosion control at culvert/storm sewer outlets, at sharp channel bends, at channel confluences, and at locally steepened channel sections.

4.3.1.1.5 Other Linings

The use of fabrics for drainage construction (i.e., erosion control liners) has increased over the past several years. The placement of slope revetment mattresses and erosion/revegetation mattresses are methods of erosion control and are not discussed in this section.

4.3.1.1.6 Composite-Lined Channels

In many cases, the physical constraints in a channel result in the channel being lined with various combinations of materials, including grass, rock, concrete, or other hard materials. These channels are called composite-lined channels, where composite refers to the various materials lining the channel. The same benefits and problems associated with a channel completely lined with grass, rock, or concrete also apply to a composite channel with these materials.

4.3.1.2 Hydraulics of Open Channels

An open channel is a conduit in which water flows with a free surface. The hydraulics of an open channel can be very complex, encompassing many different flow conditions from steady state uniform flow to unsteady, rapidly varied flow. Most of the problems in stormwater drainage involve uniform, gradually varied, or rapidly varied flow states. An example of these flow conditions is illustrated in Figure III-8. The calculations for uniform and gradually varied flow are relatively straight forward and are based upon similar assumptions (e.g., parallel streamlines). Rapidly varied flow computations, such as hydraulic jumps and flow over spillways, however, can be very complex and the solutions are generally empirical in nature.

Presented in this section are the basic equations and computational procedures for uniform, gradually varied, and rapidly varied flow. The user is encouraged to review the many hydraulics textbooks available for a more detailed discussion.

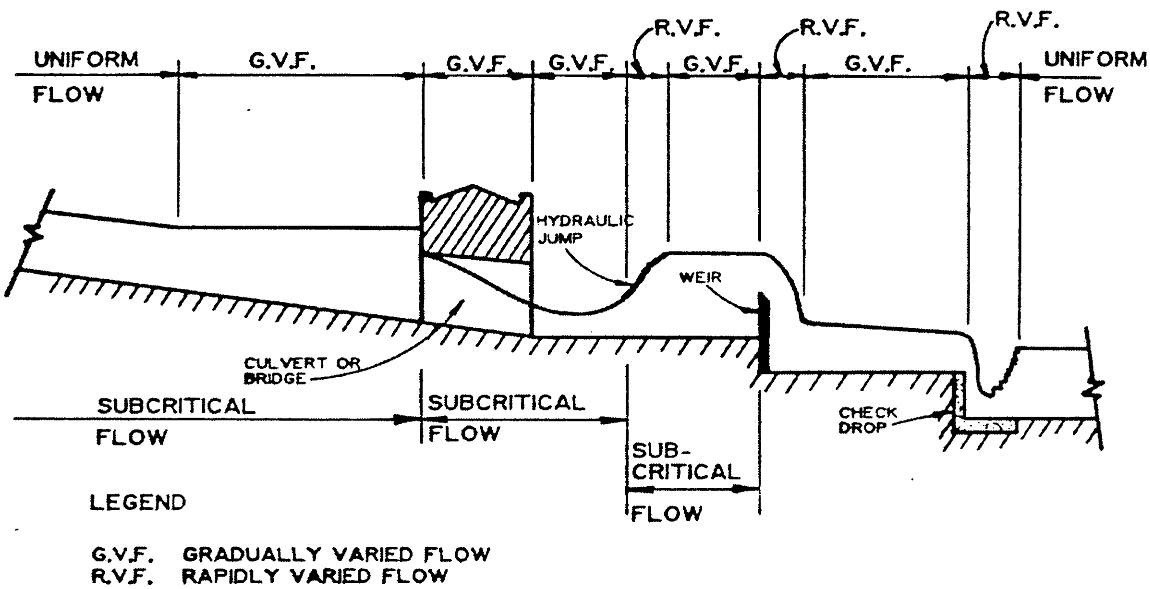
4.3.1.2.1 Uniform Flow

Open channel flow is said to be uniform if the depth of flow is the same at every section of the channel. For a given channel geometry, roughness, discharge and slope, there is only one possible depth for maintaining uniform flow: the normal depth. For a prismatic channel (i.e., uniform cross section) the water surface will be parallel to the channel bottom for uniform flow.

Uniform flow rarely occurs in nature and is difficult to achieve in a laboratory, because not all of the parameters remain exactly the same. However, channels are designed assuming uniform flow, which is an approximation and which is generally adequate for drainage purposes.

The computation of uniform depth shall be based upon the Manning's formula as follows:

$$\mathbf{Q = (1.49/n)\ AR^{2/3}S^{1/2}} \qquad \textbf{(III-7)}$$



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Arkansas

TYPICAL URBAN FLOW CONDITIONS

FIGURE III-8

Where:

Q = Discharge (cfs)
n= Roughness coefficient
A = Area (square feet)
R = Hydraulic radius, A/WP (ft)
WP = Wetted perimeter (ft)
S = Slope of the energy grade line (EGL) (ft/ft)

For prismatic channels, the EGL slope and the bottom slope are assumed to be the same.

4.3.1.2.2 Uniform/Critical Flow

The design of earth or rock channels in the critical flow regime (i.e., Froude numbers from 0.9 to 1.2) is not permitted in the Russellville area. Information presented in this section is for the purpose of identifying the critical flow state in existing channels or avoiding the condition for proposed channels.

The critical state of flow through a channel is characterized by several important conditions.

1. The specific energy is a minimum for a given discharge
2. The discharge is a maximum for a given specific energy
3. The specific force is a minimum for a given discharge
4. The velocity head is equal to half the hydraulic depth in a channel of small slope
5. The Froude Number is equal to 1.0

If the critical state of flow exists throughout an entire reach, the channel flow is critical and the channel slope is at critical slope, S_c. A slope less than S_c will cause subcritical flow. A slope greater than S_c will cause supercritical flow. A flow at or near the critical state is unstable, because major changes in specific energy, such as from channel debris, will cause a major change in depth.

The criteria of minimum specific energy for critical flow results in the definition of the Froude Number (F) as follows:

$$\mathbf{F = V/(gD)^{0.5}} \qquad \textbf{(III-8)}$$

Where:

F = Froude Number
V = Velocity (fps)
g = Acceleration of gravity (32.2 ft/sec^2)
D = Hydraulic Depth (ft) = A/T
A = Channel flow area (ft^2)
T = Top width of flow area (ft)

When F is equal to 1.0, the flow is critical. The Froude Number should be calculated for the design of all open channels to check the flow state.

The computation of the critical flow state for trapezoidal and circular sections can be performed with the use of Figure III-9.

4.3.1.2.3 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile must be computed using backwater techniques.

Backwater computation can be made using the method presented in V. T. Chow's, *Open Channel Hydraulics* (1959). Many computer programs are available for computation of backwater curves. The most general and widely used program is HEC-2, water-surface profiles, developed by the US Army Corps of Engineers and is the program recommended for floodwater profile computations if the proposed site is within a reach of a stream studied in a detailed Flood Insurance Study (FIS). This program will compute water-surface profiles for natural and man-made channels.

If the proposed development will not be located within a reach of a stream studied in a detailed FIS, the preferred program for modeling and computation of water surface profiles for design and/or analysis is the successor to HEC-2, HEC-RAS.

For prismatic channels, the backwater calculation can be computed manually using the Direct Step Method presented in Chow (1959). For an irregular non-uniform channel, the Standard Step Method is used, which is a more tedious iterative process. The use of HEC-2 or HEC-RAS is recommended for non-uniform channel analysis.

4.3.1.2.4 Rapidly Varied Flow

Rapidly varied flow (RVF) is characterized by very pronounced curvature of the streamlines. The change in curvature may become so abrupt that the flow profile is virtually broken, resulting in a state of high turbulence. The two cases of RVF (weir flow and hydraulic jump) occurring commonly in storm drainage are discussed in the following sections.

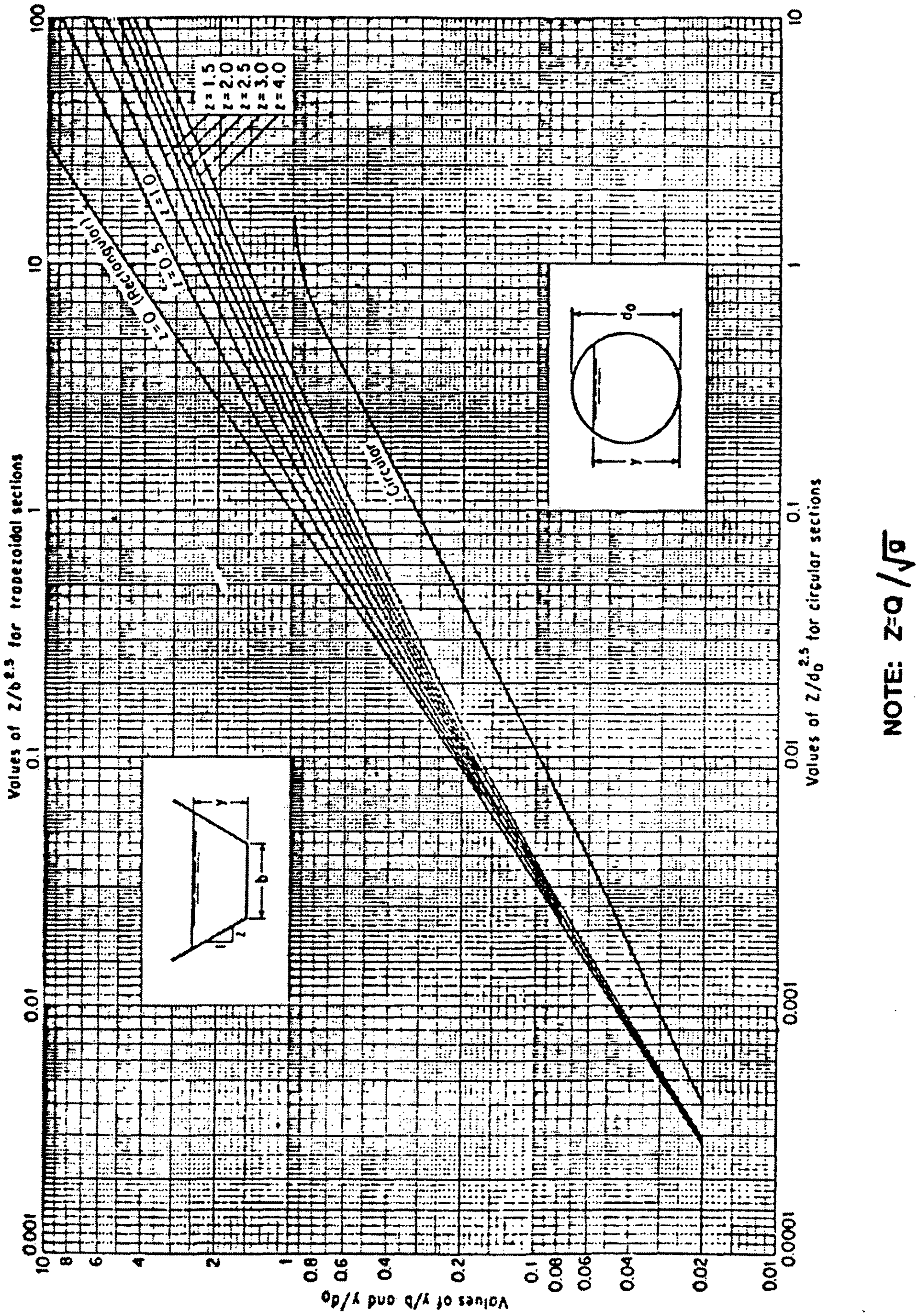
4.3.1.2.5 Weir Flow

The common use of weirs in storm drainage analysis is for spillway outlets in detention ponds (Section III-4.5, Storage). The general form of the equation for horizontal crested weirs is:

$$\mathbf{Q = CL(H)^{3/2}} \qquad \textbf{(III-9)}$$

Where:

Q = Discharge (cfs)
C = Weir coefficient
L = Horizontal length (ft)
H = Total energy head (ft)



Source: Open Channel Hydraulics, V. T. Chow

City of **RUSSELLVILLE** Arkansas	**CRITICAL DEPTH FOR TRAPEZOIDAL AND CIRCULAR SECTIONS** **FIGURE III-9**

Another common weir is the v-notch equation is as follows:

$$\mathbf{Q = 2.5 \tan (\theta/2)H^{5/2}} \qquad \textbf{(III-10)}$$

Where:

θ = angle of the notch at the apex (degrees)
(See Section III-4.5.3.3.7)

When designing or evaluating weir flow, the effects of submergence must be considered. A single check on submergence can be made by comparing the tailwater depth to the headwater depth as illustrated on Figure III-10 (see also Section III-4.3.1.2.3). The weir coefficients to be used with Equation III-9 are also listed on Figure III-10.

4.3.1.2.6 Hydraulic Jump

In urban hydraulics, a hydraulic jump may occur at grade control structures (i.e., check drops), inside of or at the outlet of storm sewers or concrete box culverts or at the outlet of an emergency spillway for detention ponds. The evaluation of hydraulic jumps is important since there is a loss of energy and erosive forces associated with a jump. For hard-lined facilities such as pipes or concrete channels, the forces and the change in energy can affect the structural stability or the hydraulic capacity. For grass-lined channels, the erosive forces at the outlet of culverts must be controlled to prevent serious damage. The control is usually obtained by confining the erosive forces to a riprap or concrete protected area.

4.3.1.2.7 Storm Sewers

The analysis of the jump inside of storm sewers is approximate due to the lack of data for circular, elliptical, or arch sections. The jump can be approximately located by intersecting the energy grade line of the supercritical and subcritical flow reaches. The primary concerns are: (1) can the pipe withstand the forces which may separate the joint or damage the pipe wall, and (2) will the jump affect the hydraulic characteristics. The stormwater velocities in storm sewers should be limited to 15 fps. Therefore, the hydraulic jump needs only to be located and the impact on the pipe capacity determined by storm sewer analyses as discussed above. The effect on pipe capacity can be determined by evaluating the energy grade line taking into account the energy lost by the jump. In general, for Froude Numbers less than 2.0, the loss of energy is less than 10%.

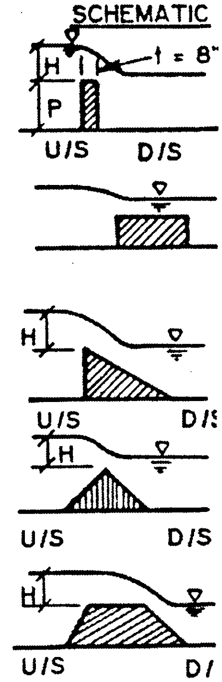
4.3.1.2.8 Box Culverts

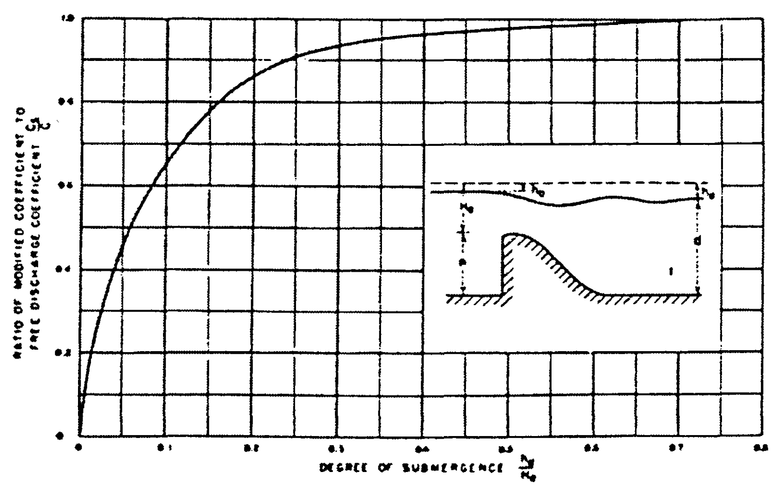
For long box culverts, with a concrete bottom, the concerns of the jump are the same as for storm sewers. However, the jump can be adequately defined for box culverts/sewers and for spillways using the jump characteristics of rectangular sections. A detailed evaluation of the hydraulic jump is beyond the scope of this MANUAL and the user is referred to Chow (1959) for computational procedures. The calculations are to be included with the required submittals.

WEIR FLOW COEFFICIENTS

SHAPE	COEFFICIENT	COMMENTS
Sharp Crested	-	
Projection Ratio (H/P = 0.4)	3.4	H≥1.0
Projection Ratio (H/P = 2.0)	4.0	H≥1.0
Broad Crested	-	
w/Sharp U/S Corner	2.6	Minimum Value
w/Rounded U/S Corner	3.1	Critical Depth
Triangular Section	-	
A) Vertical U/S Slope	-	
1:1 D/S Slope	3.8	H≥0.7
4:1 D/S Slope	3.2	H≥0.7
10:1 D/S Slope	2.9	H≥0.7
B) 1:1 U/S Slope	-	
1:1 D/S Slope	3.8	H≥1.0
3:1 D/S Slope	3.5	
Trapezoidal Section		
1:1 U/S Slope, 2:1 D/S Slope	3.4	H≥1.0
2:1 U/S Slope, 2:1 D/S Slope	3.4	H≥1.0
Road Crossings		
Gravel	3.0	H≥1.0
Paved	3.1	H≥1.0

SCHEMATIC





ADJUSTMENT FOR TAILWATER

Source: City of Tulsa, S.M.C.M

City of **RUSSELLVILLE** Arkansas	**WEIR FLOW COEFFICIENTS** **FIGURE III-10**

4.3.1.2.9 Vertical Drop Structures

The hydraulic jump conditions at vertical check drops (Section III-4.3) have been defined using experimental data found in Chow (1959). The aerated free-falling nappe in a vertical check drop will reverse the curvature and turn smoothly into supercritical flow on the apron (Figure III-11), which may form a hydraulic jump downstream. Using the experimental data, the flow geometry can be described by functions of the drop number (D_n) defined as:

$$\mathbf{D_n = (q^2/gh^3)} \tag{III-11}$$

Where q is the discharge per unit width of the crest (cfs/ft), g is the acceleration of gravity, and h is the height of drop (ft). The functions are:

$$\mathbf{L_d/h = 4/3\ D_n^{0.22}} \tag{III-12}$$
$$\mathbf{d_p/h = 1.00\ D_n^{0.22}} \tag{III-13}$$
$$\mathbf{d_1/h = 0.54\ S_n^{0.425}} \tag{III-14}$$
$$\mathbf{d_2/h = 1.66\ D_n^{0.27}} \tag{III-15}$$

Where:

L_d = Drop length (the distance from the drop wall to the position of the depth d_1)
d_p = Pool depth under the nappe
d_1 = Depth at the toe of the nappe or the beginning of the hydraulic jump
d_2 = Tailwater depth subsequent to d_1

L is the length of the hydraulic jump and may be determined as outlined for stilling basins in Chow (1959).

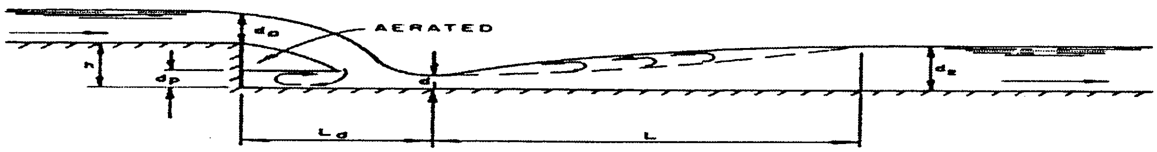
From these equations, the drop length and design tailwater depth may be determined. The above equations are contingent upon the length of the spillway crest being approximately the same width as the approach channel and were used to develop standard dimensions for check drops which are discussed in Section III-4.3.1.12.

4.3.1.2.10 Trapezoidal Cross Sections

Since the typical channel cross section in urban hydraulics is trapezoidal, the calculation of the hydraulic jump in trapezoidal channels is important.

There have been some attempts to define the characteristics of a jump in a trapezoidal channel. The experiments indicate a degree of uncertainty in locating the maximum downstream depth and therefore establishing the length of the jump. This uncertainty is somewhat related to the inability to identify the initial depth at the start of the jump.

The flatter the side slope, the longer the jump distance (or at least the distance required to reach normal depth). However, the flatter side slopes of the channel are less susceptible to erosion



h = height of drop (ft)
dp = pool depth under the nappe (ft)
L_D = drop length (ft)
L = length of hydraulic jump (ft)
d_0 = depth of flow over the crest (ft)
d_1 = depth of flow at the beginning of the hydraulic jump (ft)
d_2 = tailwater depth at the end of the hydraulic jump (ft)

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FLOW GEOMETRY OF A STRAIGHT DROP SPILLWAY

FIGURE III-11

from the turbulence. The need for protection of slopes less than 2:1, can be based on the 2:1 side slopes.

The suggested procedure for trapezoidal channels is based on the Federal Highway publication "Hydraulic Design of Energy Dissipators for Culverts and Channels", HEC-14 (FHWA 1975). The procedure is based on theoretical calculations using the momentum principle and many observations of jump distances. The procedure is illustrated by the following discussion of a jump created by the high velocity discharge from a culvert.

Illustrated in Figure III-12 is an example of a hydraulic jump created by the supercritical discharge from a culvert on a steep slope. The flow depth at the outlet is 1.3 feet, which is below critical depth of 2.5 feet. The tailwater depth is the normal depth in the channel (3.3 feet).

Since the conjugate depth for the flow depth at the outlet of the culvert is greater than the tailwater depth, the jump will not occur right at the outlet of the culvert. Instead the flow will decelerate downstream until the conjugate depth relationship is achieved, which is where the jump will occur.

If the tailwater depth was greater than the conjugate depth, then the jump would be submerged and would actually begin inside of the culvert. The flow profile and the conjugate depth are computed using the methods illustrated in Chow (1959). For either tailwater case, the location of the jump must first be estimated based on actual flow conditions in the channel.

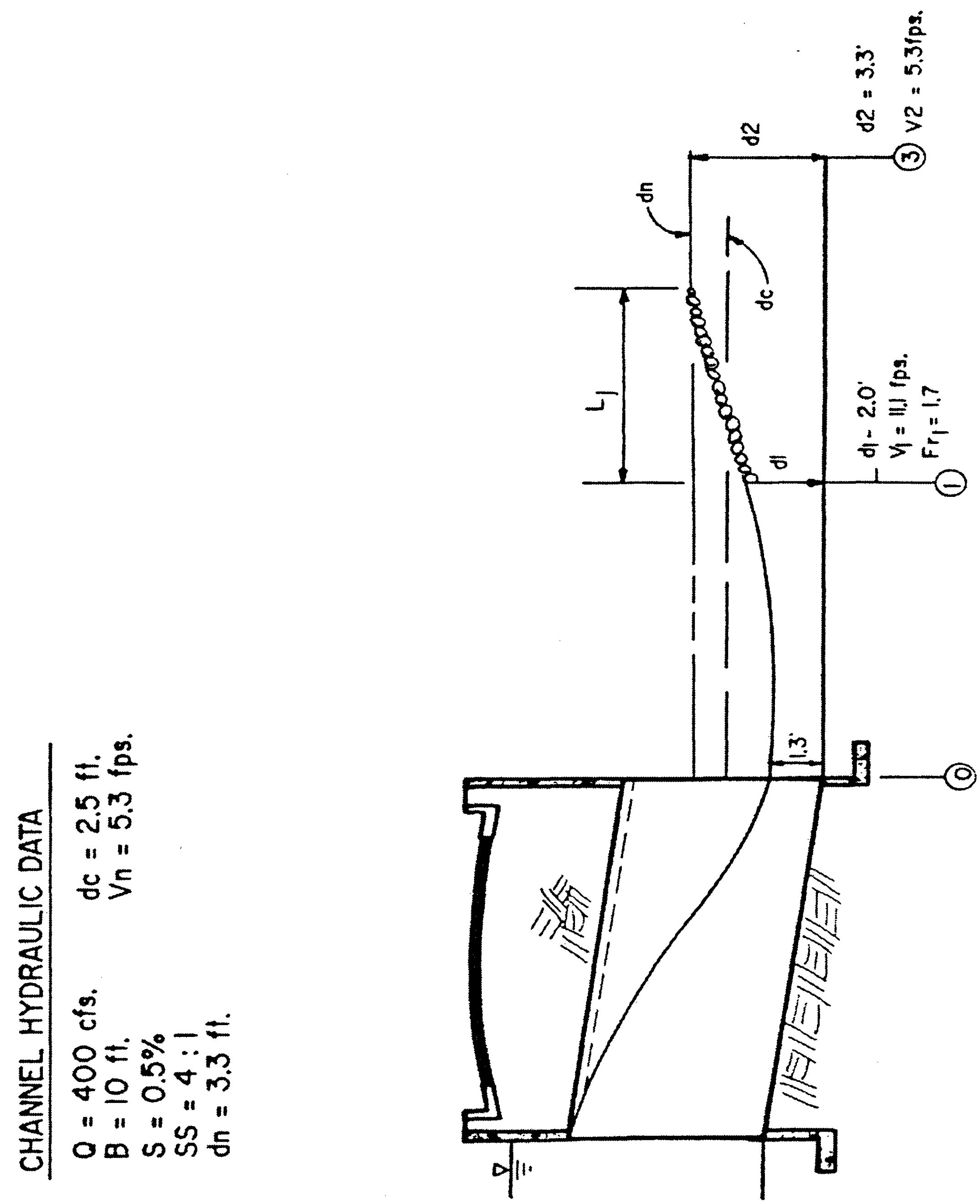
Once the location of the jump is computed, then the length of the jump (and therefore the length of the protection required) can be estimated using Figure III-13, which was obtained from the Federal Highway publication, "Hydraulic Design of Energy Dissipators for Culverts and Channels", HEC-14 (FHWA 1975). For the example in Figure III-13, the value of $z = 4$, $t = 1.25$, and the value of $Fr_1 = 1.7$. Since the example channel has side slopes flatter than 2:1, the curve for $t = 4$ and $z = 2$ can be used. Using these values, the ratio $L_1/y_1 = 27$ is read from the curve. The length of the jump would then be 2 feet times 27 or approximately 54 feet. This is the length from the start of the jump which should be protected from erosion.

4.3.1.3 Natural Channel Design Standards

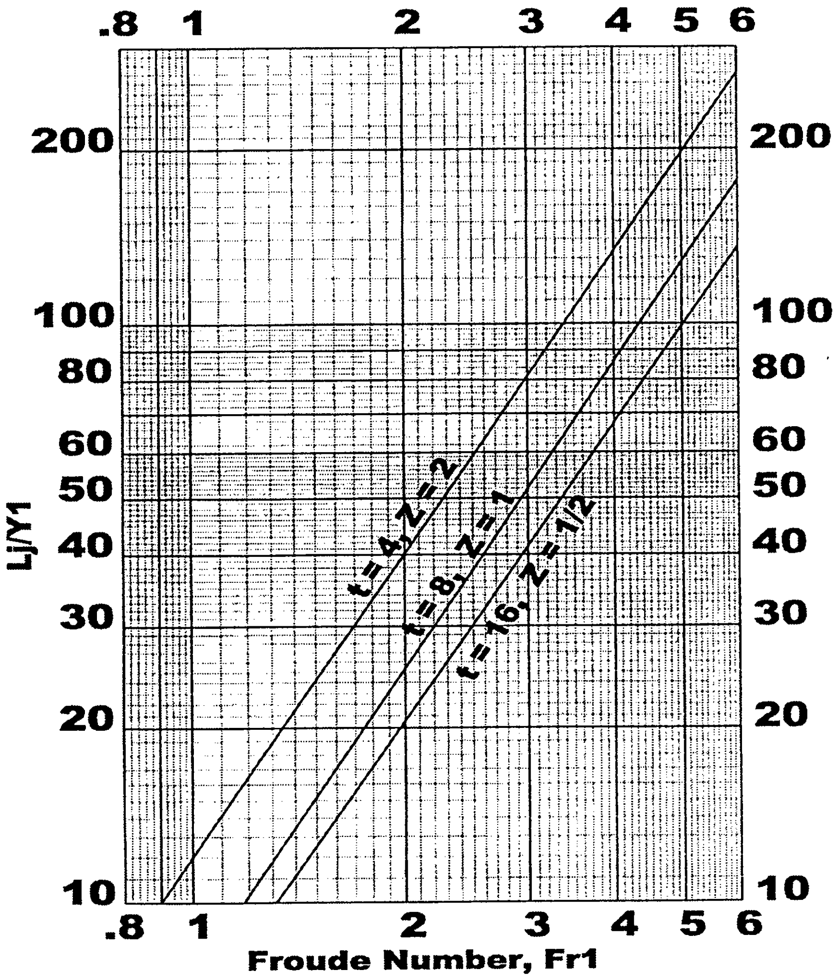
The design standards for open channel cannot be presented in a step by step fashion because of the wide range of options available to the Engineer. Certain planning and conceptual criteria are particularly useful in the preliminary design of a channel. These criteria, which have the greatest effect on the performance and cost of the channel, are discussed below.

The design criteria and evaluation techniques for natural channels are:

1. The channel and overbank areas shall have adequate capacity for the 100-year regulatory storm runoff with freeboard in accordance with Section III-4.3.1.5.1.
2. Natural channel segments which have a Froude Number greater than 0.95 for the 100-year flood peak shall be protected from erosion.



City of **RUSSELLVILLE** Arkansas	**HYDRAULIC JUMP EXAMPLE** **FIGURE III-12**



Legend

Fr1 = $V_1/(gY_1)^{.5}$
Lj = jump length, (ft.)
Y1 = Initial Flow Depth, (ft.)
t = b/Zy1
b = bottom width, (ft.)
Z = side slope
A = Flow area, @ Y1
T = Top width of flow @ Y1

Source: Hydraulic Design of Energy Dissipators for Culverts and Channels HEC-14, USDOT, Federal Highway Administration

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HYDRAULIC JUMP LENGTH IN TRAPEZOIDAL CHANNEL

FIGURE III-13

3. The water surface profiles shall be delineated so that the floodplain can be defined and protected.
4. Filling of the flood fringe reduces valuable channel storage capacity and tends to increase downstream runoff peaks. Filling of the flood fringe is subject to the restriction of floodplain regulations, which requires compensatory storage.
5. Proposed channel projects shall investigate the impacts on flood peaks and timing of the flood peaks within the downstream portion of the watershed.
6. Erosion control structures, such as riprap check drops or check dams, may be required to control flow velocities.
7. Plan and profile drawings of the floodplain shall be prepared. Appropriate allowances for future bridges or culverts, which can raise the water surface profile and cause the floodplain to be extended, shall be included in the analysis.

With most natural waterways, grade control structures should be constructed at regular intervals to decrease the thalweg slope and to control erosion. However, these channels should be left in as near a natural condition as possible. For that reason, extensive modifications should not be undertaken unless they are found to be necessary to avoid excessive erosion with subsequent deposition downstream. (Also, a modification of the channel within the normal high water line may require a US Army Corps of Engineers Section 404 permit for many of the streams.)

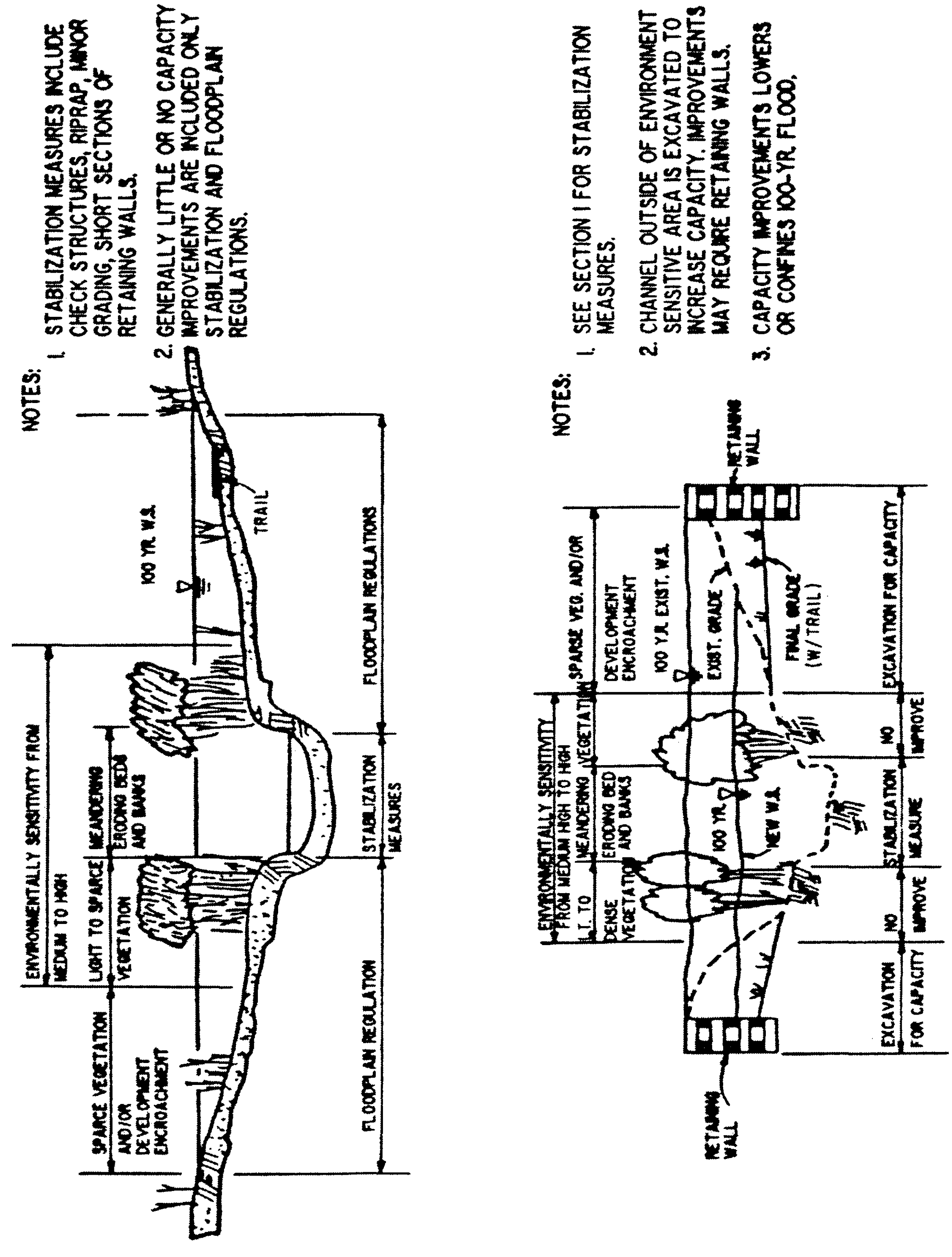
The usual rules of freeboard depth, curvature, and other guidelines that are applicable to artificial channels do not necessarily apply to natural channels. All structures constructed along the channel shall be elevated to a minimum of 1 foot above the water surface unless otherwise specified by the City of Russellville. There are significant advantages that may occur if the Engineer incorporates into his planning the overtopping of the channel and localized flooding of adjacent areas which are laid out and developed for the purpose of being inundated during the major storm runoff. The freeboard criteria can be used to advantage in gaging the adequacy of a natural channel for future changes in runoff.

One variation of the natural channel is to leave the main channel area undisturbed (i.e., that area containing the base flow plus the immediate vegetation area) and to improve the overbank conveyance capabilities by excavating the floodplain area. This "naturalized" channel, which is illustrated in Figure III-14, preserves the environmentally sensitive area adjacent to the base flow and increases the capacity of the total channel to convey the flood.

If a natural channel that has historic flood peaks in excess of 100 cfs is to be utilized as a drainageway for a development, the applicant shall meet with the CITY ENGINEER to discuss the concept and to obtain the requirements for planning and design documentation. Approval of the concept and design will be made in accordance with the requirements of DIVISION I.

4.3.1.4 Grass-Lined Channels Design Standards

Key parameters in grass-lined channel design include velocity, slopes, roughness coefficients, depth, freeboard, curvature, cross section shape, and lining materials. Other factors such as water



Source: City of Tulsa, S.M.C.M.

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TYPICAL NATURAL CHANNELS AND MINIMAL IMPACT ALTERNATIVES

FIGURE III-14

surface profile computation, erosion control, drop structures, and transitions also play an important role. A discussion of these parameters is presented below.

4.3.1.4.1 Flow Velocity and Capacity

The maximum normal depth velocity (average velocity) for the flood peak shall not exceed 6.0 fps for grass-lined channels, except in sandy soil where the maximum velocity shall not exceed 5.0 fps. The Froude Number (turbulence factor) shall be less that 0.8 for grass-lined channels. Grass-lined channels having a Froude Number greater than 0.8 are not permitted. The minimum velocity, wherever possible, shall be greater than 2.0 fps for the regular storm runoff.

4.3.1.4.2 Longitudinal Channel Slope

Grass-lined channels typically will have slopes less than 1.0%, but the slopes will be dictated by velocity and Froude Number requirements. Where the natural topography is steeper than desirable, channel drops shall be utilized to maintain design velocities (see Section III-4.3.1.12).

4.3.1.4.3 Freeboard

Except where localized overflow in certain areas is desirable for additional ponding benefits or other reasons, the freeboard shall be:

$$H_f = 0.5+V_c/2g \qquad \textbf{(III-16)}$$

Where:

H_f = Freeboard height (ft)
V = Average channel velocity (fps)
g = Acceleration of gravity = 32.2 ft/sec^2

The minimum freeboard shall be 1.0 feet above the computed water surface elevation. Freeboard shall not be obtained by the construction of levees.

An approximation of the super-elevation, H_{FB} (ft), at a channel bend with velocity, V (fps), centerline radius of curvature, r_c (ft), and top width of channel, Tw (ft), can be obtained from the following equation:

$$H_{FB} = V^2Tw/gr_c \qquad \textbf{(III-17)}$$

Where:

H_{FB} = Freeboard height (ft)
V = Average channel velocity (fps)
Tw = Top width of channel (ft)
g = Acceleration of gravity = 32.2 ft/sec^2
r_c = Centerline radius of curvature (ft)

The freeboard shall be measured above the super-elevated water surface.

4.3.1.4.4 Curvature

The centerline of curvature shall have a radius three times the top width of the design flow, but not less than 100 feet.

4.3.1.4.5 Roughness Coefficient

The variation of Manning's "n" with the retardance, and the product of mean velocity and hydraulic radius as presented in Figure III-15, shall be used in the capacity computation.

Retardance curve D shall be used to determine the channel capacity and velocity, since the ENGINEERING DEPARTMENT will maintain channels with a watershed of 40 acres and greater. Retardance curve C should be used to evaluate existing unmaintained channels or to determine the impact of heavy vegetation on the flooding in a channel segment.

4.3.1.4.6 Cross Section

The channel shape may be practically any section suitable to the location, the environmental conditions, or site-specific purpose. Often the shape can be chosen to suit open space and recreational needs (Figures III-16 through III-18). Limitations on the design are as follows:

(a) Trickle Channel

The base flow (i.e., runoff from lawn watering, low intensity rain showers, and snow melt) shall be carried in a trickle channel. The minimum capacity shall be 1.0% to 3.0% of the flow, but not less than 1 cfs. Trickle channels shall be constructed of concrete or other approved materials to minimize erosion, to facilitate maintenance, and to aesthetically blend with the adjacent vegetation and soils. Substitution of shape and materials may be made if approved by the CITY ENGINEER.

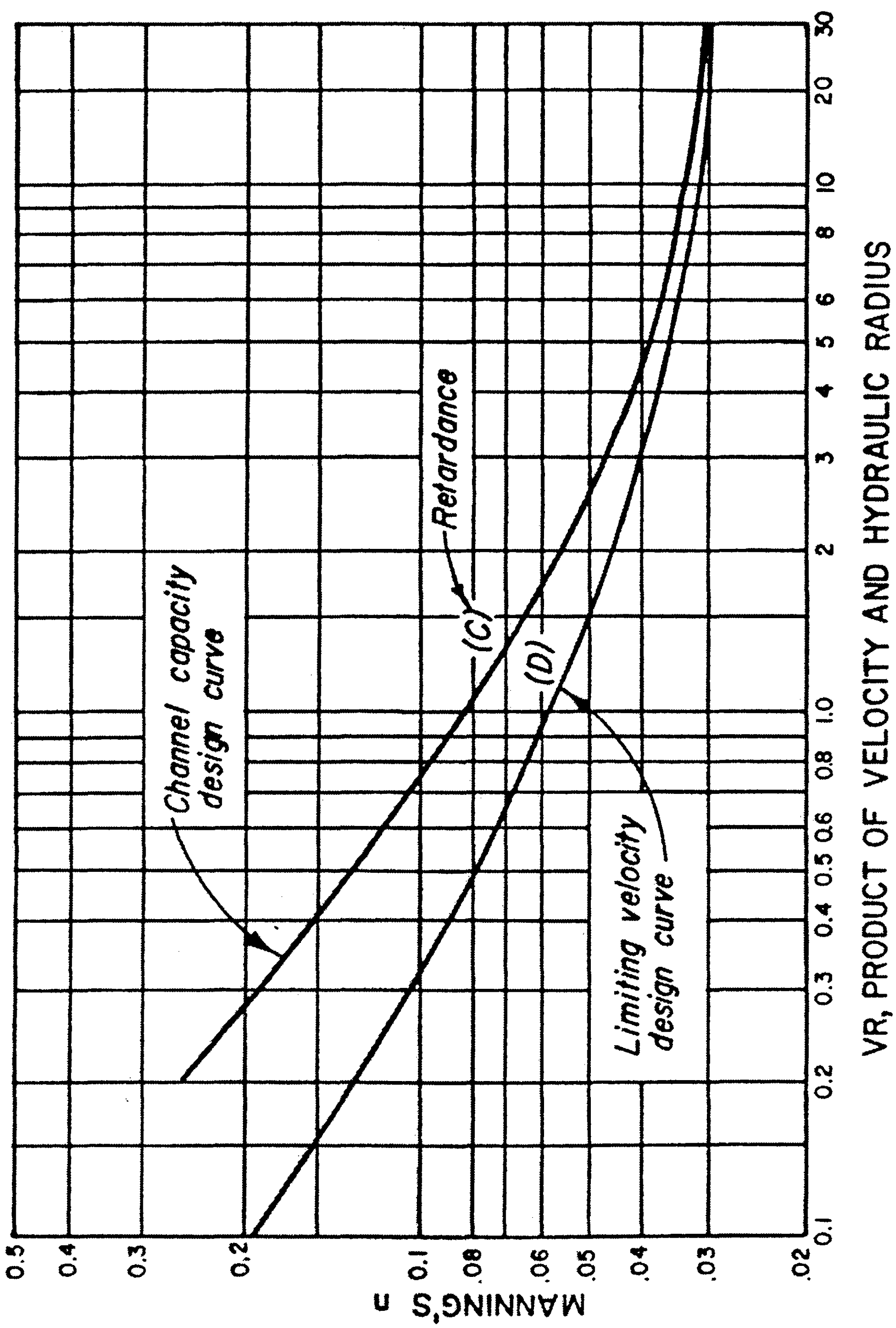
The benefits of trickle channels are: (1) protect edges of channel from erosion, (2) provide for easier maintenance of channel vegetation by confining the flow and maintaining grade, (3) provide for a positive drainage path to lower the local groundwater table, and (4) control sedimentation.

(b) Main Channel

A main channel is required for sandy soils as shown in Figure III-18. Riprap side slopes can be from 2:1 to 3:1. The depth of the main channel is not included in the normal depth limitation. A main channel can also be used for non-sandy soils, subject to the conditions shown in Figure III-16.

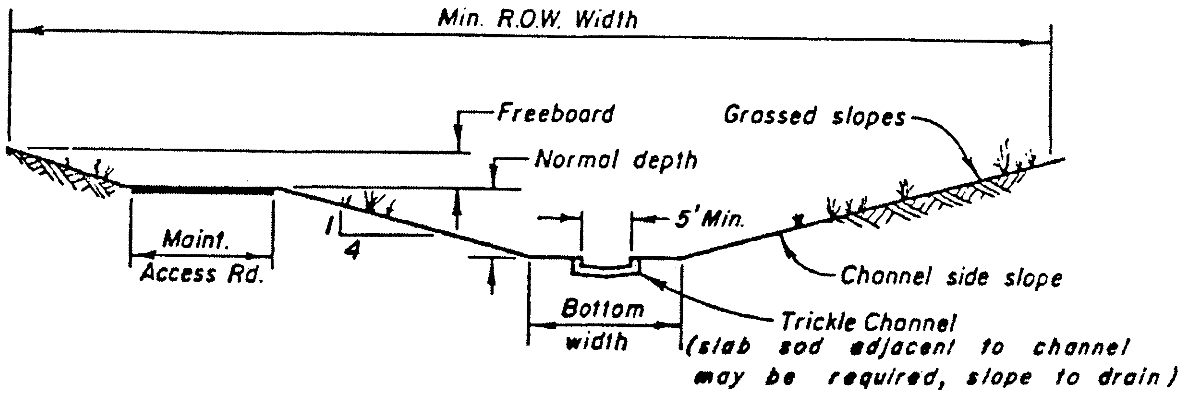
(c) Bottom Width

The minimum bottom width shall be consistent with the maximum depth and velocity criteria.



Source: **From "Handbook of Channel Design for Soil and Water Conservation," U.S. Department of Agriculture, Soils Conservation Service, No. SCS-TP-61 March, 1947, Rev. June, 1954.**

City of **RUSSELLVILLE** Arkansas	**ROUGHNESS COEFFECIENTS FOR GRASSED CHANNELS**
	FIGURE III-15



Notes:

1. Bottom Width: Consistent with maximum allowable depth and velocity requirements, shall not be less than the trickle channel width.

2. Trickle Channel: Minimum capacity to be 1% to 3% of the 100-year flow but not less than 1 cfs. Channel to be constructed of concrete or other materials approved in writing by the CITY ENGINEER.

3. Normal Depth: Normal depth at the 100-year flow shall be such that the product of hydraulic depth (A/T) and the 100-year velocity (V_{max} = 7 fps) be less than 35 cfs/ft.

4. Freeboard: Freeboard to be a minimum of 1 foot

5. Maintenance/Access Road: Minimum width to be 20 feet

6. ROW Width: Minimum width to include freeboard and maintenance access road.

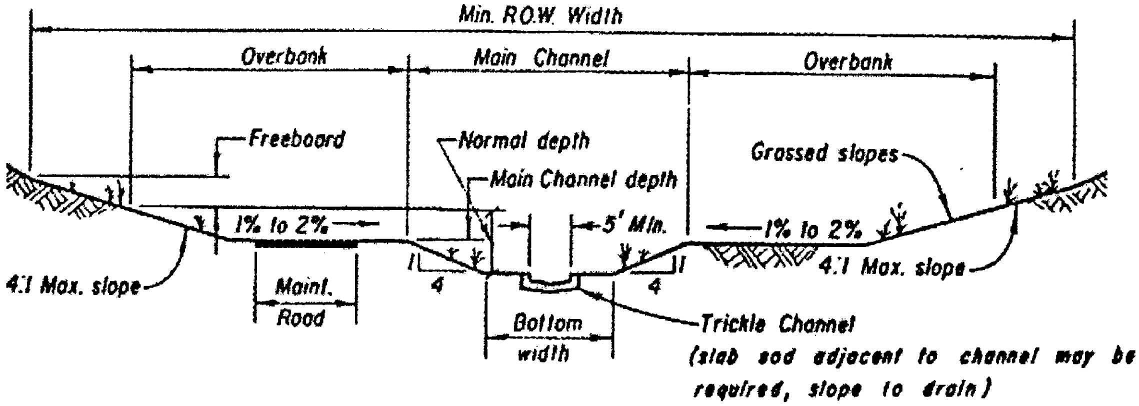
7. Channel Side Slope: Maximum side slope for grassed channels to be 4:1.

8. Froude Number: Maximum value shall not exceed 0.8 for the 100-year flood or less.

9. The maximum flow velocity to be 7 fps for erosion resistant soils or 5 fps for sandy soils.

Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**TYPICAL GRASS LINED CHANNEL SECTION** **TYPE A** **FIGURE III-16**



Notes:

1. Main Channel: Capacity to be less than 20% of the 100-year flow at Main Channel depth. Maximum 100-year flow velocity is 7 fps.

2. Trickle Channel: Minimum capacity to be 1% to 3% of 100-year flow but not less than 1 cfs. Channel to be constructed of concrete or other materials approved in writing by the CITY ENGINEER.

3. Normal Depth: Flow depth for the 100-year flow shall be such that the product of hydraulic depth (A/T) and the 100-year flow velocity (V_{max} = 6 fps) be less than 35 cfs/ft.

4. Freeboard: Freeboard to be a minimum of 1 foot.

5. Maintenance/Access Road: Minimum width to be 20 feet.

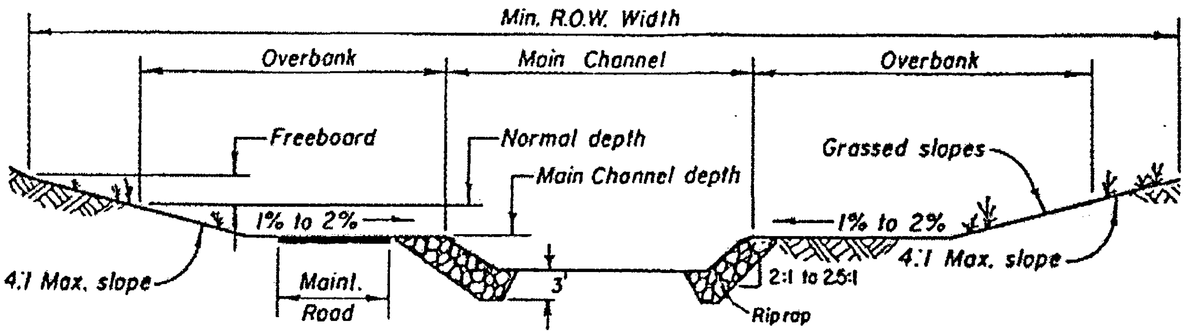
6. ROW Width: Minimum width to include freeboard and maintenance access road.

7. Overbank: Flow in excess of main channel to be carried in this area. Area may be used for recreation purposes.

8. Froude Number: Maximum value shall not exceed 0.8 for the 100-year flood or less.

Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**TYPICAL GRASS-LINED CHANNEL SECTION** **TYPE B** **FIGURE III-17**



Notes:

1. This section is required for channels in sandy soils.

2. Main Channel: Capacity to be the 25-year flow. Maximum 100-year flow velocity is 5 fps. Protect slopes with riprap. Use a Mannings "n"-value of 0.03 for hydraulic calculations.

3. Normal Depth: Flow depth for the 100-year flow, shall not exceed 5 feet, not including the main channel depth.

4. Freeboard: Freeboard to be a minimum of 1 foot.

5. Maintenance/Access Road: Minimum width to be 20 feet. City may require all or part of the road to be surfaced.

6. ROW Width: Minimum width to include freeboard and maintenance access road.

7. Overbank: Flow in excess of main channel to be carried in this area. Area may be used for recreation purposes.

Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**TYPICAL GRASS-LINED CHANNEL SECTION** **TYPE C** **FIGURE III-18**

(d) **Right-of-Way Width**

The minimum right-of-way width shall include freeboard and a 20-foot wide maintenance access.

(e) **Flow Depth**

The maximum design hydraulic depth of flow (flow area divided by the top width, outside the trickle channel area) for the flood shall be limited to 5.0 feet in grass-lined channels.

(f) **Maintenance/Access Road**

A minimum of a 20-foot wide maintenance access shall be provided for all major drainageways.

(g) **Side Slopes**

Side slopes shall be 4:1 or flatter. Slopes as steep as 3:1 may be used in existing developed areas subject to additional erosion protection (Section III-4.3.1.4.7) and approval from the CITY ENGINEER.

4.3.1.4.7 Seeding and Mulching

The requirements for seeding and mulching shall be in accordance with the Arkansas Highway and Transportation Department "Standard Specifications for Highway Construction", 1996 edition.

Seed mixture, bulk seed rate per acre, watering requirements, and fertilizer grade shall be indicated on the plans and will be subject to approval by the CITY ENGINEER.

4.3.1.4.8 Water Surface Profiles

Computation of the water surface profile shall be presented for all open channels utilizing standard backwater methods, taking into consideration losses due to changes in velocity of channel cross sections, drops, waterway openings, or obstructions. The hydraulic and energy gradients shall be shown on preliminary and construction drawings. Refer to Section III-4. 3. 1. 2 for additional information.

4.3.1.5 Concrete-Lined Channels

The criteria for the design and construction of concrete-lined channels are presented below.

4.3.1.5.1 Hydraulics

(a) Freeboard

Adequate channel freeboard above the designed water surface shall be provided and shall not be less than that determined by the following:

$$H_{FB} = 2.0 + 0.025V(d)^{1/3} \qquad \textbf{(III-18)}$$

Where:

H_{FB} = Freeboard height (ft)
V = Velocity (fps)
d = Depth (ft)

Freeboard shall be in addition to super elevation, standing waves, and/or other water surface disturbances. These special situations should be addressed in the Stormwater Management Plan submitted with the construction drawings and specifications.

Concrete side slopes shall be extended to provide freeboard. Freeboard shall not be obtained by the construction of levees.

(b) Super Elevation

Super elevation of the water surface shall be determined at all horizontal curves and design of the channel section adjusted accordingly (see Section III-4.3.1.4.3).

(c) Velocity

Flow velocities shall not exceed 15 fps or result in a Froude Number greater than 0.9 (see Section III-4.3.1.2.2 and Table III-6) during the flood for non-reinforced concrete linings. Flow velocities shall not exceed 5 fps for reinforced linings during the 100-year flood.

(d) "n" Values

Refer to Table III-7 for range of Manning's "n" values.

4.3.1.5.2 Concrete Materials

1. Cement Type: II, IIA, or III
2. Minimum cement content: 550 lbs/cy
3. Maximum water-cement ratio; 0.50
4. Maximum aggregate size: 1½ inches
5. Air content range: 5 to 7%
6. Slump: 1 to 4 inches
7. Minimum compressive strength (f_c): 3250 psi at 28 days

Material	Maximum Velocity	Comments
Grass	5 fps	For Sandy Soils
Grass	6 fps	For Cohesive Soils
Rock Riprap	15 fps	Based on Safety Considerations
Rock Riprap (grouted)	15 fps	Based on Safety Considerations
Concrete	15 fps	Based on Safety Considerations
Gabion	15 fps	Based on Safety Considerations

City of
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VELOCITY LIMITATIONS OF OPEN CHANNELS

TABLE III-6

CHANNEL TYPE	n-VALUE RANGE	RECOMMENDED VALUE
A. Earth-Lined (ditches/canals)		
1. Clean, Weathered	.018 to .025	.022
2. Clean, Gravel	.022 to .030	.025
3. Some Weeds	.022 to .033	.027
4. Non-Maintained	.030 to .040	.035
B. Grass-Lined (man-made)		
1. RV[4] > 10 (see Figure III-15)	.029 to .034	.032[1]
2. RV[4] < 10	.032 to .100	See Figure III-15
C. Natural Streams	.025 to 0.100	Note [2]
D. Riprap-Lined		
1. Ordinary Riprap	$n = .0395\, d_{50}^{\;0.17}$	Section III-4.3.1.6
2. Gabions		0.035
3. Grouted Riprap	.023 to .030	0.027
4. Slope Mattress	.025 to .033	0.028
E. Concrete-Lined		
1. Float Finished/Wood Forms	.013 to .016	Note [3]
2. Slip Formed	.013 to .016	Note [3]
3. Gunite	.016 to .023	Note [3]

Notes:
(1) Use as starting value to estimate channel capacity (see Section III-4.3.1.2)
(2) Refer to Chow (1959), Table 5-6.
(3) High value used for capacity determination and low value used for velocity consideration (refer to Section III-4.3.1.2)
(4) RV is the product of hydraulic radius and velocity.

City of **RUSSELLVILLE** Arkansas	**MANNING'S n-VALUE FOR OPEN CHANNELS** **TABLE III-7**

4.3.1.5.3 Concrete Lining Section

1. Reinforced linings shall have a minimum thickness of 6 inches.

2. The side slopes shall be a maximum of 2 horizontal to 1 vertical or be a structurally reinforced retaining wall if steeper.

3. Weep holes shall be provided as shown on Figures III-19 and III-20.

4.3.1.5.4 Concrete Joints

1. Channels shall be continuously reinforced without transverse joint. Expansion joints shall be installed where new concrete lining is connected to a rigid structure or to existing concrete lining which is not continuously reinforced.

2. Longitudinal joints, where required, shall be constructed on the side walls at least 1-foot vertically above channel invert.

3. All joints shall be designed to prevent differential movement.

4. Construction joints are required for all cold joints and where the lining thickness changes.

4.3.1.5.5 Concrete Finish

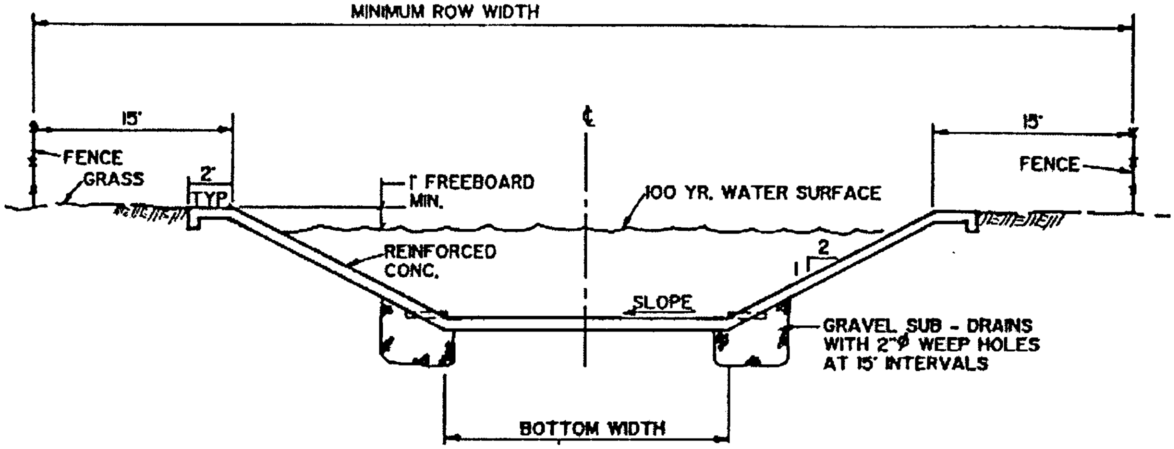
The surface of the concrete lining shall be provided with a roughened finish, such as obtained by a wood float. Excessive working or wetting of the finish resulting in a smooth surface shall be avoided.

4.3.1.5.6 Concrete Curing

All concrete shall be cured in accordance with ASTM C 94.

4.3.1.5.7 Reinforcement Steel

1. Steel reinforcement shall be in accordance with Arkansas Highway and Transportation Department (AHTD) Standard Specifications, Section 503.
2. Ratio of longitudinal steel area to concrete cross sectional area shall be greater than 0.005.
3. Ratio of transverse steel area to concrete cross sectional area shall be greater than 0.0025.
4. For slabs on grade, reinforcing steel shall be placed at the center of the section with a minimum clear cover of 3 inches adjacent to earth.
5. Additional steel, as needed, if a retaining wall structure is used.

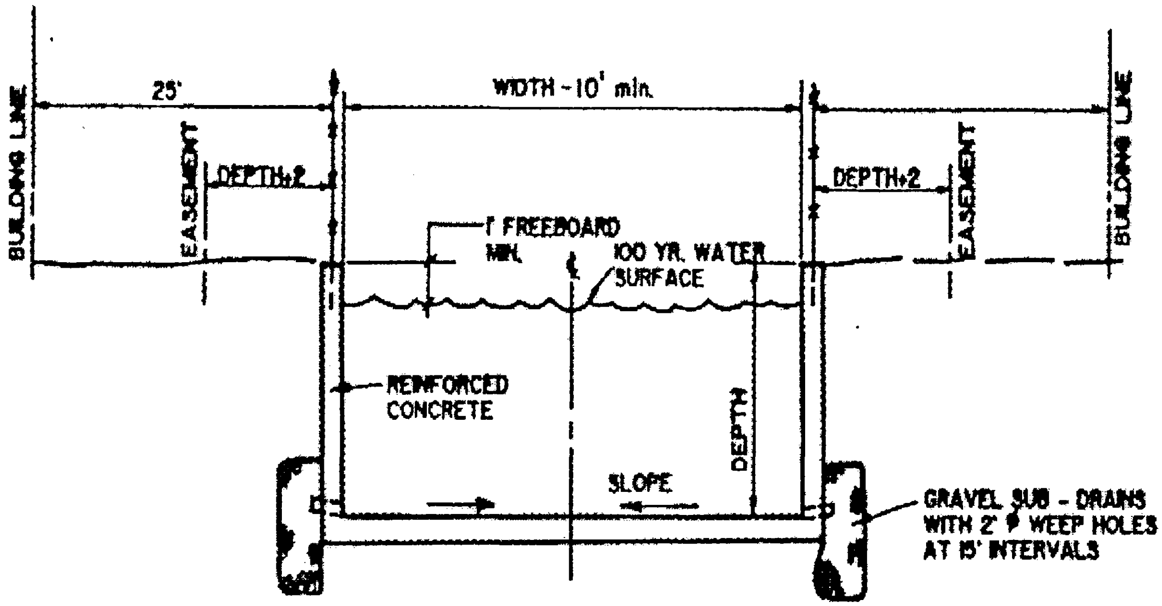


Notes:

1. No above ground trees, shrubs, utilities, or structures will be constructed in the easement. Easement to remain clear for access.
2. Channel bottom to be designed for maintenance equipment loading.
3. Access ramps to be provided to channel bottom at 600-foot intervals.
4. Steps to be located at 250-foot intervals on alternate sides of the channel.
5. Minimum ROW width will be equal to the channel top width plus 30 feet.
6. Minimum bottom width of channel will be 10 feet unless otherwise approved in writing by the CITY ENGINEER.

Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**TYPICAL CONCRETE LINED CHANNEL TRAPEZOIDAL CROSS SECTION** **FIGURE III-19**



Notes:

1. No above ground trees, shrubs, utilities, or structures will be constructed in the easement. Easement to remain clear for access.

2. Channel bottom to be designed for maintenance equipment loading. Channel bottom will be sloped to the center.

3. Walls to be reinforced for maintenance equipment surcharge.

4. Access ramps to be provided to channel bottom at 600-foot intervals.

5. Steps to be located at 250-foot intervals on alternating sides of the channel.

Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**TYPICAL CONCRETE LINED CHANNEL RECTANGULAR CROSS SECTION** **FIGURE III-20**

4.3.1.5.8 Earthwork

The following areas shall be compacted to at least 95% of maximum density as determined by ASTM D 698 (Standard Proctor):

1. The 12 inches of sub grade immediately beneath concrete or other hard lining (both channel bottom and side slopes).
2. Top 12 inches of maintenance road.
3. Top 12 inches of earth subgrade (but not topsoil for grass) within 10 feet of concrete channel lip.
4. All fill material.

4.3.1.5.9 Bedding

Provide 6 inches of granular bedding in accordance with design procedures in Section III-4.3.1.11.5.

4.3.1.5.10 Access Ramps

Access ramps shall be provided to the channel bottom at a maximum interval of 600 feet.

4.3.1.5.11 Safety Requirements

1. A chain link fence shall be installed to prevent access wherever the 100-year channel lining depths exceeds 3 feet. The fence shall be 5-foot high for vertical walls and 4-foot high elsewhere. Gates, with top latch, shall be placed at 250-foot intervals and staggered where fence is required on both sides of the channel.
2. Steps shall be installed not more than 250 feet apart on alternating sides of the channel. Bottom rung shall be placed approximately 12 inches vertically above channel invert.

4.3.1.5.12 Cross Section

The cross section of the channel shall be either trapezoidal (Figure III-19) or rectangular (Figure III-20).

4.3.1.6 Rock-Lined Channel Design Standards

Channel linings constructed from ordinary riprap, grouted riprap, or wire encased rock to control channel erosion have been found to be cost effective where channel reaches are relatively short. Situations for which riprap linings might be appropriate are: (1) where major flows, such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values stated in Section III-4.3.1.5.1; (2) where channel side slopes must be steeper than 3:1; (3) for low flow channels; and (4) where rapid changes in channel geometry occur such as channel bends and transitions. Design criteria applicable to these situations are presented in Section III-4.3.1.11. Section III-4.3.1.11.1 emphasizes requirements associated with ordinary riprap; while Section III-4.3.11.4 contains additional design considerations specifically related to wire enclosed rock. Both Sections III-4.3.1.11.1 and III-4.3.11.4 are valid only for subcritical flow conditions where the Froude Number is 0.8 or less.

4.3.1.7 Composite-Lined Channel Design Standards

The design standards for composite-lined channels will vary in accordance with the materials utilized for construction. For grass-, concrete-, or rock-lined portions of the channel, refer to Sections III-4.3.1.4, III-4.3.1.5 and III-4.3.1.6, respectively. For other lining types, the Engineer will be required to submit the appropriate documentation in support of the use of the materials proposed. Refer to Figure III-21 for typical examples of composite channels and to Table III-8 for limitations on velocity for specific materials.

4.3.1.8 Roadside Ditch Design Standards

The criteria for the design of roadside ditches is similar to the criteria for grass-lined channels. Utilities will not be allowed within the ditch section unless approved by the ENGINEERING DEPARTMENT. Refer to Figure III-22 for definition of terms.

4.3.1.8.1 Capacity

Roadside ditches in combination with the capacity in the pavement area shall have adequate capacity for the 100-year storm runoff peaks. Where the storm runoff exceeds the capacity of the ditch, a storm sewer system shall be required.

4.3.1.8.2 Flow Velocity

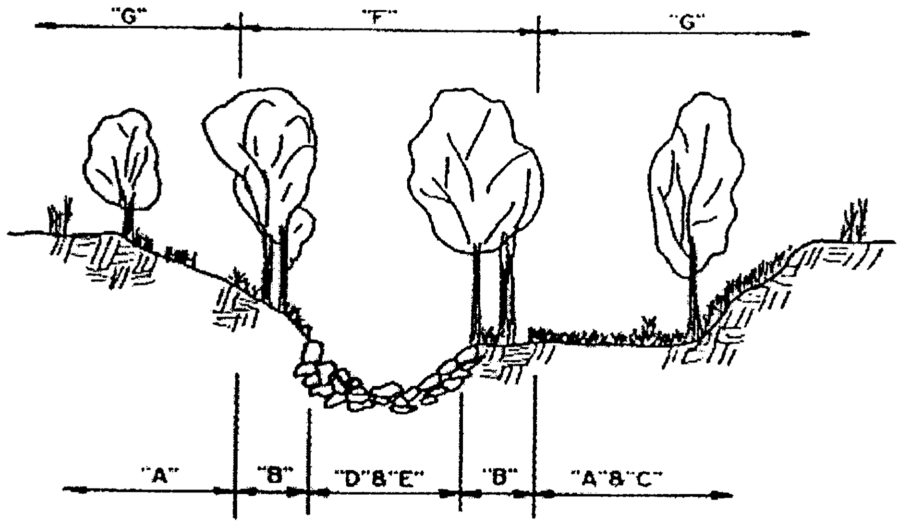
The maximum velocity for the 100-year flood peak shall not exceed 5.0 fps for Type I ditches, and 7.0 fps for Type II or III ditches. The capacity limitations are based on a maximum Froude Number of 0.8 for Types I and II, and 0.9 for Type III.

4.3.1.8.3 Longitudinal Slope

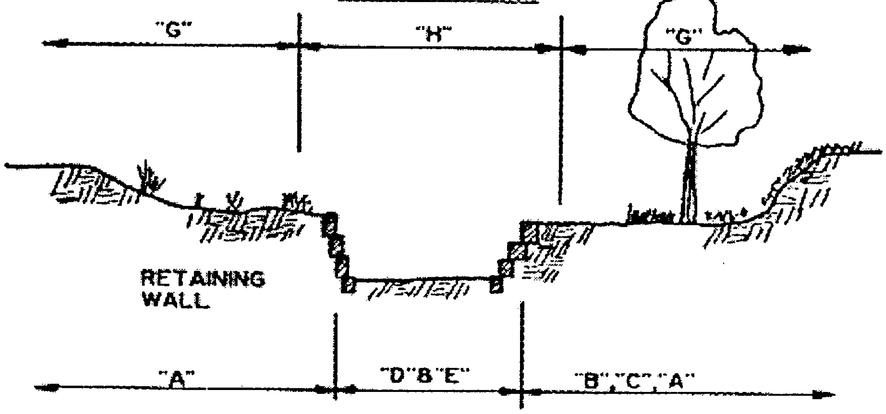
The slope shall be limited by the average velocity of the 100-year flood peaks. Check drops may be required where street slopes are in excess of 2.5%.

4.3.1.8.4 Freeboard

No freeboard is required.



SECTION 3



SECTION 4

Legend:

"A"	Floodplain regulation
"B"	Preserve vegetation
"C"	Provide maintenance access
"D"	Remove vegetation and provide capacity from 2-25 yr.
"E"	Provide stabilization measures (i.e. check drops, bank protection, etc)
"F"	Environmentally sensitive area, choked channel
"G"	Vegetation and encroachment into floodplain
"H"	Low environmental sensitivity

Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**TYPICAL COMPOSITE-LINED CHANNELS** **FIGURE III-21**

REQUIREMENTS FOR CHANNEL LININGS**

$VS^{0.17}/(S_S\text{-}1)^{0.66}$*

(VELOCITY, V fps)	RIPRAP MEAN PARTICLE SIZE, d_{50}
1.4 to 3.2	3”
3.3 to 3.9	9”
4.0 to 4.5	12”
4.6 to 5.5	18”
5.6 to 6.4	24”

*Use S_s = 2.5 unless the source of rock and the density are known at the time of design. V in fps and S in ft/ft.

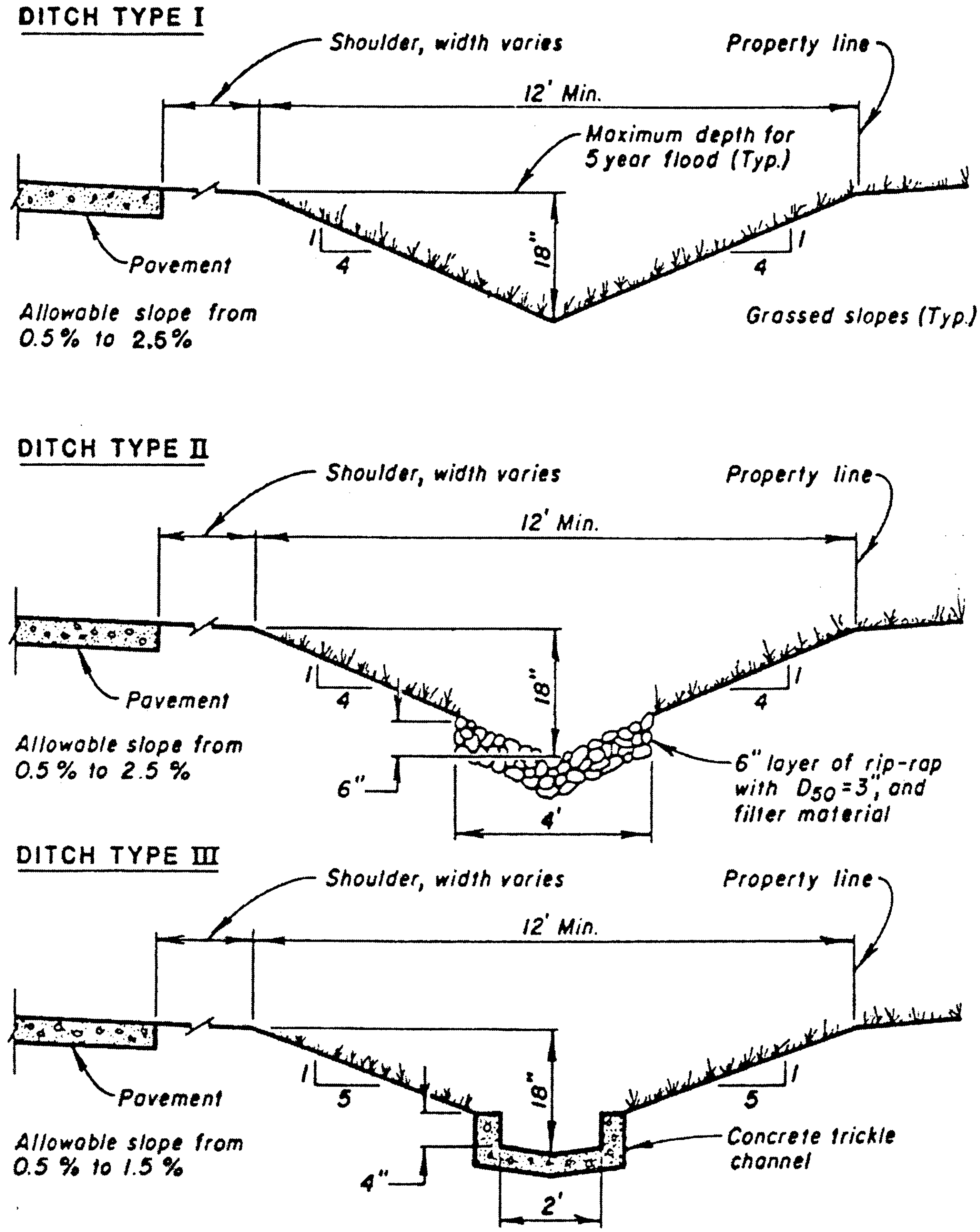
**Table valid only for Froude number of 0.8 or less and side slopes no steeper than 2h:1v.

***Riprap with a mean particle size, d_{50}, of 9 inches or less shall be buried after placement to reduce vandalism.

SM9 slope mattress with toe protection may be substituted for riprap with a mean particle size, d_{50}, of 9 inches or less. (See Table III-10).

G12 gabion with toe protection may be substituted for riprap with a mean particle size, d_{50}, of 12 – 18 inches. (See Table III-10).

City of **RUSSELLVILLE** Arkansas	**RIPRAP REQUIREMENTS FOR CHANNEL LININGS** **TABLE III-8**



Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**ROADSIDE DITCH SECTIONS** **FIGURE III-22**

4.3.1.8.5 Curvature

The minimum radius of curvature shall be 25 feet.

4.3.1.8.6 Roughness Coefficient

Manning's "n" values presented in Figure III-15 will be used in the capacity computation for roadside ditches.

4.3.1.8.7 Grass Lining

The grass lining shall be in accordance with Section III-4.3.1.4.7.

4.3.1.8.8 Trickle Channel

The base flow (i.e., runoff from lawn watering, low intensity rain showers, and snow melt) shall be carried in a trickle channel. The minimum capacity shall be 1.0% to 3.0% of the 100-year flow, but not less than 1 cfs. Trickle channels shall be constructed of concrete or other approved materials to minimize erosion, to facilitate maintenance, and to aesthetically blend with the adjacent vegetation and soils. Substitution of shape and materials may be made if approved by the CITY ENGINEER.

4.3.1.9 Driveway Culvert Design Standards

Driveway culverts shall be sized to pass the 10 year ditch flow capacity without overtopping the driveway. The minimum size culvert shall be an 18-inch RCP round pipe (or equivalent) for collector or higher classified streets, otherwise the minimum size pipe shall be 15-inch diameter. End treatments for the culverts shall be concrete headwalls for arterial or higher classified streets, or flared end sections for residential streets. Use of corrugated metal pipe (CMP) with flared end sections is subject to approval by the CITY ENGINEER. More than one culvert may be required.

4.3.1.10 Other Channel Linings

The use of other channel lining materials may be permitted if approved by the ENGINEERING DEPARTMENT. The ENGINEERING DEPARTMENT will consider linings other than grass, rock, or concrete depending on (1) manufacturers recommendations for the specific product, (2) requirements for local erosion protection of steep side slopes (i.e., steeper than 3:1) and areas of local turbulence in grass-lined channels, and (3) the experience and recommendations of the ENGINEERING DEPARTMENT.

The Engineer will be required to submit the technical data in support of the proposed material. Additional information or calculations may be requested by the CITY ENGINEER to verify assumptions or design criteria. The following minimum criteria will also apply.

4.3.1.10.1 Flow Velocity

The maximum normal depth velocity will be dependent on the construction material utilized. The Froude number shall be less than 0.8.

4.3.1.10.2 Freeboard

Freeboard is defined by Equation III-16.

4.3.1.10.3 Curvature

The center line curvature shall have a minimum radius twice the top width of the design flow but not less than 100 feet.

4.3.1.10.4 Roughness Coefficient

A Manning's "n" value range shall be established by the manufacturers data with the high value used to determine depth/capacity requirements and the low value used to determine Froude Number and velocity restrictions.

4.3.1.10.5 Cross Section

Same as for grass-lined channels, Section III-4.3.1.4.6.

4.3.1.11 Riprap Design Standards

Riprap has proven to be an effective means to deter erosion along channel banks, in channel bottoms, upstream and downstream from hydraulic structures, at bends, at bridges, and in other areas where erosive tendencies exist. The Engineer needs to bear in mind that there are additional costs associated with riprap erosion protection, since riprap is not readily available and installations require frequent inspection and maintenance. Also, wire enclosed riprap (gabion) may require complete renovation every 10 to 15 years if improperly used.

4.3.1.11.1 Ordinary Riprap

Ordinary riprap, or simply riprap, refers to a protective blanket of large loose stones, which are usually placed by machine to achieve a desired configuration. The term ordinary riprap has been introduced to differentiate loose stones from grouted riprap and wire enclosed rock, which are discussed later.

Many factors govern the size of the rock necessary to resist the forces tending to move the riprap. For the riprap itself, this includes the size and weight of the individual rocks, the shape of the stones, the gradation of the particles, the blanket of thickness, the type of bedding under the riprap, and the slope of the riprap layer. Hydraulic factors affecting riprap include the velocity, current direction, eddy action and waves.

Experience has shown that riprap failures result from undersized individual rocks in the maximum size range; improper gradation of the rock, which reduces the interlocking of individual particles; and improper bedding for the riprap, which allows leaching of channel particles through the riprap blanket.

The requirements for lining a channel with riprap for protection from the erosive forces of high velocity flow are presented in Table III-8. The primary limitation on the use of ordinary riprap is to keep the Froude number at 0.8 or less.

4.3.1.11.2 Rock Properties

Rock used for riprap or wire enclosed riprap should be hard, durable, angular in shape, and free from cracks, overburden, shale and organic matter. Neither breadth nor thickness of a single stone should be less than 1/3 the length and rounded stone should be avoided.

The rock should sustain a loss of not more than 40% after 500 revolutions in an abrasion test (Los Angeles machine - ASTM C-535 69) and should sustain a loss of not more than 10% after 12 cycles of freezing and thawing (AASHTO test 103 for ledge rock, procedure A). Rock having a minimum specific gravity of 2.65 is preferred; however, in no case should rock have a specific gravity less than 2.50. In lieu of testing requirements, rock obtained from City approved quarries may be used.

Classification and gradation for riprap are shown in Table III-9 and are based on minimum specific gravity of 2.50 for the rock. Because of the relatively small size and weight, riprap types VL and L should be buried with native top soil and revegetated to protect the rock from vandalism.

4.3.1.11.3 Grouted Riprap

Grouted riprap provides a relatively impervious channel lining which is less subject to vandalism than dumped riprap. Grouted riprap requires less routine maintenance by reducing silt and trash accumulation and is particularly useful for lining low flow channels and steep banks. The appearance of grouted riprap is enhanced by exposing the tops of individual stones and by cleaning the projecting rocks with a wet broom. Grouted riprap should meet all the requirements for ordinary riprap except that the smallest rock fraction (smaller than the 10% size) should be eliminated from the gradation.

As with ordinary riprap, grouted riprap should be placed on an adequate bedding. The recommended minimum grout specifications include entrained air, a 28-day strength of at least 2400 psi, and a high slump (5 to 7 inches) in order to penetrate either the full depth of the riprap layer or at least 2 feet where the riprap layer is thicker than 2 feet. Concrete having maximum aggregate size of 3/4 inches may be substituted for grout when using riprap with d_{50} of 12 inches or larger. Weep holes should be provided at least every 4 to 6 feet at the toe of channel slopes and channel drops to reduce uplift forces on the grouted channel lining.

DRAINAGE MANUAL DESIGNATION	LETTER CODE OF SIZE	LENGTH	WIDTH	DEPTH	NUMBER OF DIAPHRAGMS	CAPACITY CUBIC YARD	MINIMUM ROCK DIMENSION
G36	A	6'	3'	3'	1	2	4"
	B	9'	3'	3'	2	3	4"
	C	12'	3'	3'	3	4	4"
G18	D	6'	3'	1' - 6"	1	1	4"
	E	9'	3'	1' - 6"	2	1.5	4"
	F	12'	3'	1' - 6"	3	2	4"
G12	G	6'	3'	1'	1	0.66	4"
	H	9'	3'	1'	2	1	4"
	I	12'	3'	1'	3	1.33	4"
SLOPE MATTRESS							
SM9	T	10'	6' - 6"	0' - 9"	5	1.80	3"
	U	12'	6' - 6"	0' - 9"	6	2.16	3"

Source: Maccaferri Gabions, Maccaferri Gabions, Inc.

City of
RUSSELLVILLE
Arkansas

STANDARD GABION BASKETS
(ENGLISH SIZES)

TABLE III-9

The grout shall be delivered to the place of final deposit by means that will insure uniformity and prevent segregation of the grout. Placing of grout shall be obtained by pumping under pressure through a 2 inch maximum diameter hose to insure complete penetration of the grout into the rock layer. A vibrator is to be employed near that nozzle during placement to aid the flow of the grout. The excess grout shall be removed by a thorough washing to leave a clean rock face exposed. The grouted riprap should resemble a hand placed stone wall or fireplace rock. Grout shall fill the voids to within approximately 4 inches of the riprap surface.

4.3.1.11.4 Wire Enclosed Rock

Wire enclosed rock refers to rocks that are bound together in a wire basket so that they act as a single unit, usually referred to as a gabion. One of the major advantages of wire enclosed rock is that it provides an alternative in situations where available rock sizes are too small for ordinary riprap. Another advantage is the versatility that results from the regular geometric shapes of wire enclosed rock. The rectangular blocks and mats can be fashioned into almost any shape that can be formed with concrete. The durability of wire enclosed rock is generally limited by the service life of the galvanized binding wire which, under normal conditions, is considered to be about 15 years. Water carrying silt, sand, or gravel can reduce the service life of the wire; also water which rolls or otherwise moves cobbles and large stones breaks the wire with a hammer and anvil action and considerably shortens the life of the wire. The wire has been found to be susceptible to corrosion by various chemical agents and is particularly affected by high sulfate soils. If corrosive agents are known to be in the water or soil, a plastic coated wire should be specified.

Wire enclosed rock is not maintenance free and must be periodically inspected to determine whether the wire is sound. If breaks are found while they are still relatively small, they may be patched by weaving new strands of wire into the wire cage. Wire enclosed rock installations have been found to attract vandalism. Flat mattress surfaces seem to be particularly susceptible to having wires cut and stones removed. Where possible, mattress surfaces should be buried, as it has been found that wire enclosed rock buried under a few inches of soil is less prone to vandalism. Wire enclosed rock installations require inspection at least once a year under the best circumstances and may require inspection every 3 months in vandalism prone areas. Mattresses on sloping surfaces must be securely anchored to the surface of the soil.

Rock filler for the wire baskets should meet the rock property requirements for ordinary riprap. Minimum rock sizes and basket dimensions are shown in Table III-9. The maximum stone size should not exceed 2/3 the basket depth or 12 inches, whichever is smaller.

4.3.1.11.5 Bedding Requirements

Long term stability of riprap and gabion erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures is directly attributable to bedding failures. A properly designed bedding provides a buffer of intermediate sized material between the channel bed and the riprap to prevent piping of channel particles through the voids in the riprap.

The gradation and thickness requirements for the granular bedding, and the placement thereof, will comply with specifications of the Arkansas Highway and Transportation or as otherwise directed by the CITY ENGINEER in writing.

4.3.1.12 Channel Drop Design Standards

4.3.1.12.1 Criteria for Use of Drops

The most common use of channel drops is to control the longitudinal slope of grass-lined channels to keep design velocities within acceptable limits. Sloping and vertical riprap drop structures, Figure III-23, are two possible types of drop structures. Only vertical drop structures of riprap are recommended. Sloping drops with the chute constructed of concrete are acceptable. Riprap drop structures have been studied by other agencies who have developed criteria and charts to aid in the design of these types of structures. All check drops with a unit discharge, q, of 35 cfs/ft or less shall be designed and constructed in accordance with this section. For unit discharges greater than 35 cfs/ft, other types of structures will be required, such as sloping concrete structures with baffle blocks or other appropriate jump control appurtenances.

4.3.1.12.2 Vertical Riprap Drops

The design chart for the vertical channel drop (Table III-10) is based upon the height of the drop and the normal depth and velocity of the approach and exit channels. The channel must be prismatic throughout, from the upstream channel through the drop to the downstream channel. An example of a vertical drop structure is shown on Figure III-23.

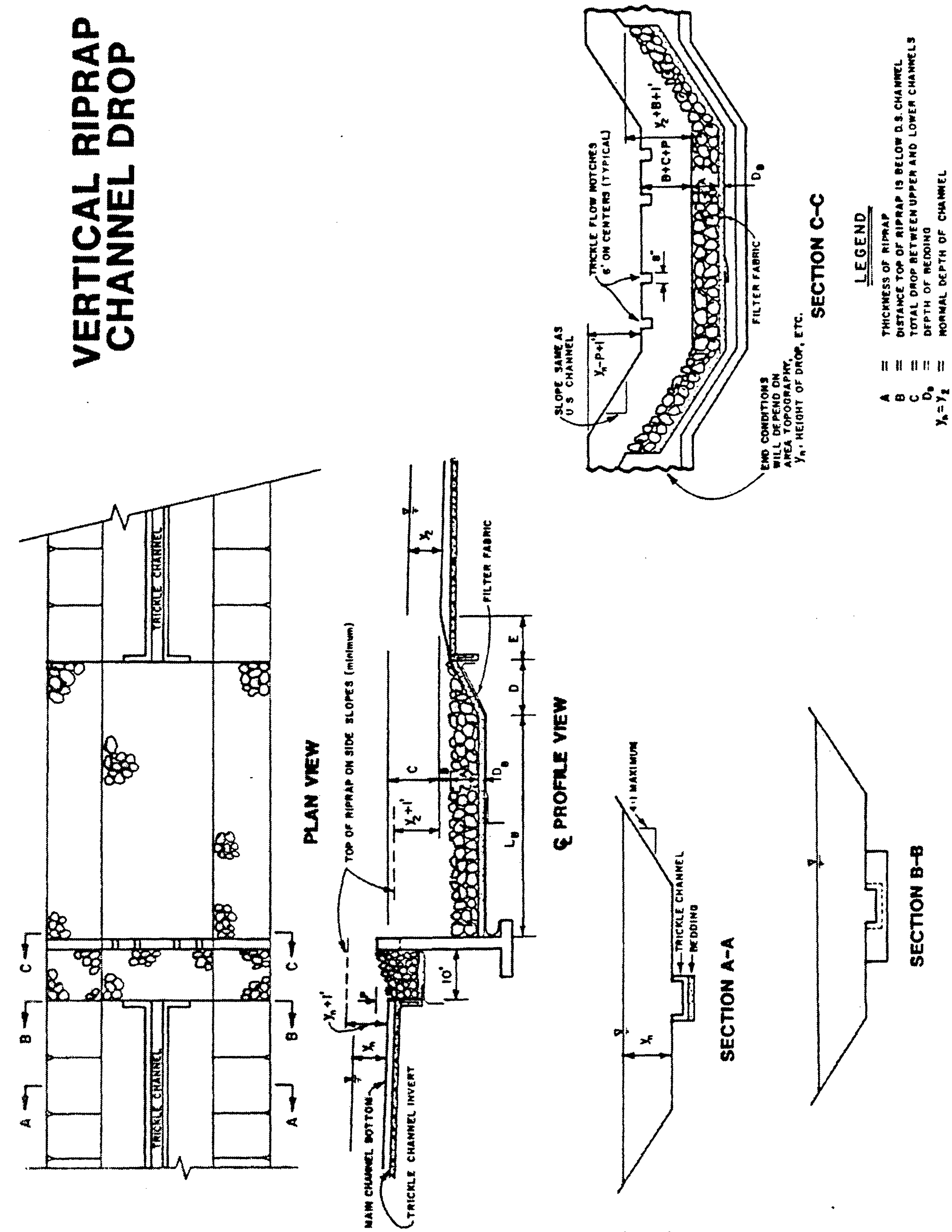
The maximum (steepest) recommended side slope for the riprap stilling basin is 4:1. Flatter side slopes are allowable and encouraged when available right-of-way permits. The riprap should extend up the side slopes to a depth equal to 1 foot above the normal depth projected upstream from the downstream channel. The maximum fall allowed at any one drop structure is 4 feet from the upper channel bottom to the lower channel bottom, excluding the trickle channel.

When the riprap is grouted to form a solid mass of rock and concrete, the side slopes in the stilling basin area can be steepened to 3:1. The grout must be pumped into the rock voids, be vibrated to fill the void volumes, and result in the tips of the rocks projecting from the concrete mass. For additional information refer to Sections III-4.3.1.13.2 and III-4.3.1.11.3.

4.3.1.12.3 Criteria

(a) Approach Depth

The upstream and downstream channels will normally be grass-lined trapezoidal channels with trickle channels to convey normal low water flows. The maximum normal depth Y_n is 5 feet and the maximum normal velocity V_n is 6 fps for erosion resistant soils and 5 fps for easily eroded soils.



Source: Urban Storm Drainage Criteria Manual, Denver, Colorado

City of **RUSSELLVILLE** Arkansas	**VERTICAL RIPRAP CHANNEL DROP**
	FIGURE III-23

C (ft)	Vn (fps)	Yn & Yz (ft)	P (ft)	B (ft)	A (ft)	L_B (ft)	D (ft)	E (ft)	RIPRAP CLASS.
2	5	4	0.1	0.6	2.0	20	4	3	M
2	5	5	*	0.8	2.5	25	5	4	"5"
2	5;7	4	0.1	0.8	2.5	20	5	4	"5"
2	5;7	5	*	0.8	2.5	25	5	4	"5"
3	5	4	0.1	1.0	2.5	20	5	4	"5"
3	5	5	*	1.0	2.5	25	5	4	"5"
3	5;7	4	0.1	1.0	2.5	20	5	4	"5"
3	5;7	5	*	1.0	2.5	25	5	4	"5"
4	5	4	0.1	1.2	3.5	20	7	5	"5"
4	5	5	*	1.2	3.5	25	7	5	"5"
4	5;7	4	0.1	1.4	3.5	20	7	6	"5"
4	5;7	5	*	1.4	3.5	25	7	6	"5"

***See Table below to calculate P**

Notes:
1. See Figure III-23 for definition of symbols
2. See Section III-4.3.1.11 for riprap gradation, classification and bedding requirements.
3. Maximum Allowable C = 4.0'
4. This chart is for ordinary riprap structures only. Other types of drop structures require their own hydraulic analysis. (See Section III-4.3.1.6).
5. Use grouted Type M.

CREST WALL ELEVATION CHART

BOTTOM WIDTH* (ft)	P@Vn = 5 fps (ft)	P@Vn = 7 fps (ft)
5	0.2'	0.2'
40	0.4'	0.2'
100	0.5'	0.3'

*Bottom Width of Approach Channel

Source: Urban Storm Drainage Criteria Manual, 1990
Urban Drainage & Flood Control District, Denver, Colorado

City of
RUSSELLVILLE
Arkansas

VERTICAL RIPRAP CHANNEL DROP DESIGN CHART

TABLE III-10

(b) Trickle Channel

The trickle channel shown in this case (Figure III-23) is a rectangular concrete channel. The concrete channel ends at the upstream end of the upstream riprap apron. A combination cutoff wall and foundation wall is provided to give the end of the trickle channel additional support. The water is allowed to "trickle" through the upstream apron and through the vertical wall. Riprap trickle channels would simply feather into the upstream apron.

(c) Approach Apron

A 10-foot long apron is provided upstream of the cutoff wall to protect against the increasing velocities and turbulence which result as the water approaches the vertical drop. Type M riprap can be used for this apron.

(d) Crest Wall

The vertical wall should have the same trapezoidal shape as the approach channel. The wall distributes the flow evenly over the entire width of the drop structure. This is important to prevent flow concentrations that would adversely affect the riprap basin.

The trickle channel is ended at the upstream end of the approach apron to prevent the trickle channel from concentrating additional water at a point during high flows, thus exceeding the design assumptions. The apron and the vertical wall combine to disperse the flow concentrated in the trickle channel. The trickle flows are allowed through the wall through a series of notches in order to prevent ponding. The voids in the riprap below the notch inverts are expected to silt in or they can be filled at the time of construction.

The wall must be designed as a structural retaining wall. The top of the wall should be placed a distance P above the upstream channel bottom. This is done to create a higher water surface elevation upstream, thus reducing the draw a sudden drop. P can be determined from Table III-10.

(e) Chute Apron I

The riprap stilling basin is designed to force the hydraulic jump to occur within the basin, and is designed for essentially zero scour. The floor of the basin is depressed an amount B below the downstream channel bottom, excluding the trickle channel. This is done to create a deeper downstream sequent depth, which helps keep the hydraulic jump in the basin. This arrangement will cause ponding in the basin. The trickle channel can, depending on the depth, relieve all or some of the ponding. The riprap can also be buried and vegetated to reduce the pond area to a smaller size.

The riprap basin can be sized using Table III-10. The way to use the table is to determine the required height of the drop C (Figure III-23), the normal velocity of the approach channel V_n, and the upstream and downstream normal depths Y_n and Y_2. Both channels must have the same geometry and Y_2 must be equal to Y_n in order to use the table. Enter the row which contains the correct C, V_n, and Y_n and Y_2 and select the riprap classification and all necessary dimensions from that row.

The riprap must be placed on bedding and filter fabric as shown in Figure III-23. The riprap should extend up the channel side slopes a distance of $Y_2 + 1$ foot as projected from the downstream channel. The basin side slopes should be the same as those in the downstream channel (4:1 or flatter) up to the $Y_2 + 1$ foot location, above which riprap slopes as steep as 2:1 are allowable.

(f) Exit Depth

The downstream channel should be the same as the upstream channel, including a trickle channel. For concrete trickle channels a cutoff wall similar to the one used for the upstream trickle channel should be used. In some instances the wall may also be used to control seepage and piping.

Example: Vertical Riprap Channel Drop

Given: Q = 1,600 cfs

Upstream and downstream channel dimensions

Bottom width = 50 ft
S = .0043 ft/ft
Side slopes = 4:1
Yc = 2.9 ft
Y_n = 4.0 ft
V_n = 6.0 fps

Erosion resistant soils
Concrete trickle channel
Drop required = 3.0 ft

Procedure:

Step 1 From Table III-10, for C = 3.0 ft, V_n = 6.0 fps and Y_n and Y_2 = 4.0 ft

Select the riprap designation and the riprap basin dimensions.

Riprap = Type H
B = 1.0 ft
A = 2.5 ft
I-b =20 ft
D = 5.0 ft
E = 4.0 ft

Step 2: Determine P = 0.1 from Table III-10

Step 3: Design retaining wall and finalize dimension

4.3.1.13 Other Channel Drop Design Standards

The criteria in Sections III-4.3.1.5 and III-4.3.1.6 are for ordinary riprap (not grouted), and trapezoidal sections only. For other types of channel drops, additional analysis or special care during construction will be required.

4.3.1.13.1 Grouted Riprap

When riprap is grouted, the tendency is for the concrete to completely cover the rock, resulting in a wavy but smooth surface. This smooth surface would require a larger basin than specified in Table III-10. If the grout is placed in strict accordance with the specifications of Section III-4.3.1.11.3, then the dimensions in Table III-10 can be used for grouted riprap channel drops. The rock size requirement can be reduced by one size from that specified in Table III-10 (i.e., from 18 inch to 12 inch) except 12 inch shall be the smallest size allowed for channel drops and 12 inch shall be substituted for 24 inch.

4.3.1.13.2 Concrete

When the unit discharge in the channel exceeds 35 cfs/ft (see Section III-4.3.1.12.1) or the drop height exceeds 4 feet, riprap drop structures will not be permitted. A different type of channel drop and extensive channel transition will be required. The trapezoidal section must first be transitioned into a vertical concrete channel. The flows are then accelerated by a sloping concrete chute and a concrete stilling basin is constructed to dissipate the energy. The channel is then transitioned back into the trapezoidal grass-lined section.

The detailed hydraulic evaluation of the channel transitions, chute, and energy dissipators are beyond the scope of this MANUAL. The user is referred to Chow (1959) for this information.

4.4 Culverts

4.4.1 General

The function of a drainage culvert is to pass the design storm flow under a roadway, railroad, or other embankment without causing excessive backwater and without creating excessive downstream velocities. The Engineer shall keep energy losses and discharge velocities within reasonable limits when selecting a culvert for a stormwater management structure.

4.4.2 Hydrologic Criteria

Specific hydrologic criteria to be used for developing discharges for the design of culverts associated with a MAJOR DEVELOPMENT shall be as follows:

1. The design discharge for all cross-drain culverts and drainage facility designs shall be the 4% annual chance storm runoff.

2. The design discharge for drainage design for all interior culverts shall be the 10% annual chance storm runoff.

3. All culvert designs shall be checked with the 1% annual chance storm runoff.

4.4.3 Hydraulics of Culverts

Approaching the entrance to a culvert as at point 1 of Figure III-24, the flow is essentially uniform and the hydraulic grade line and energy grade lines are almost the same. As water enters the culvert at the inlet, the flow is first contracted and then expanded by the inlet geometry causing a loss of energy at point 2. As normal turbulent velocity distribution is reestablished downstream of the entrance at point 3, a loss of energy is incurred through friction or form resistance. In short culverts, the entrance losses are likely to be high relative to the friction loss. At the exit, point 4, an additional loss is incurred through turbulence as the flow expands and is retarded by the water in the downstream channel. At point 5 of Figure III-25 open channel flow is established and the hydraulic grade line is the same as the water surface. Culvert flow may be generalized into two major types of flow, inlet and outlet control.

4.4.3.1 Inlet Control

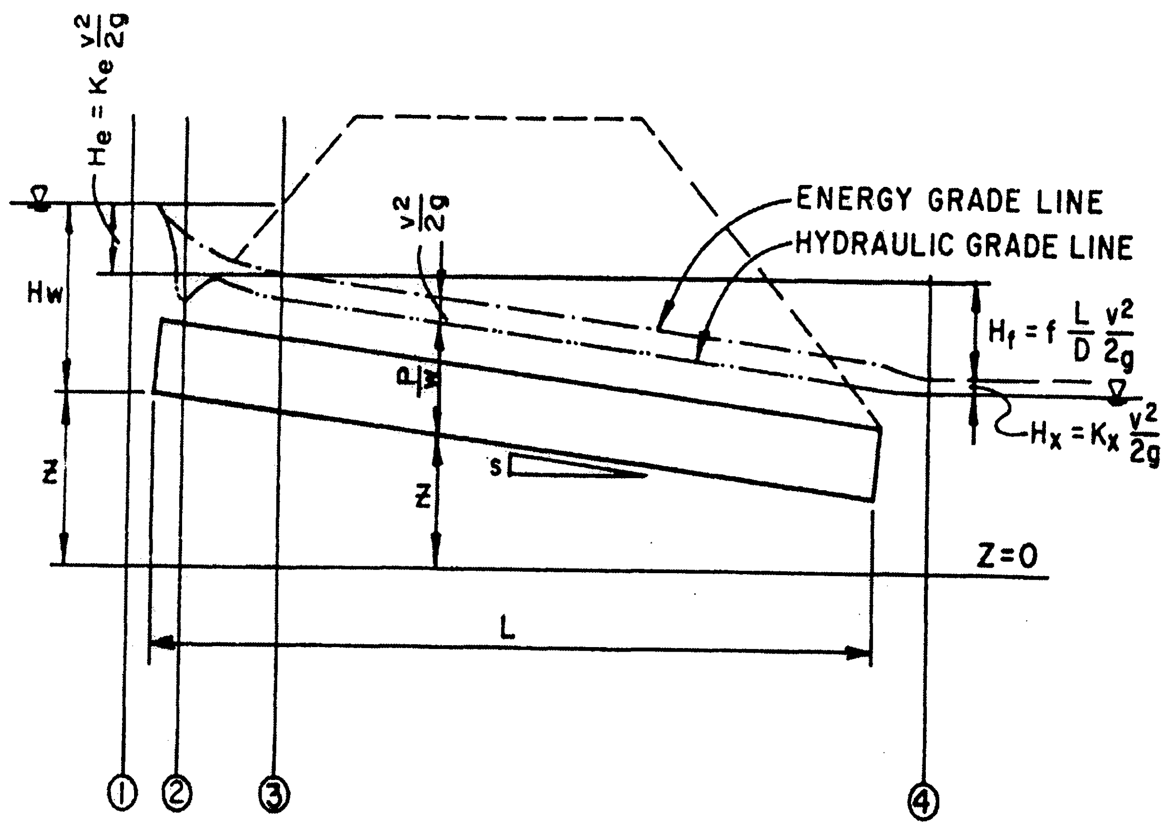
A culvert cannot carry any more water than can enter the inlet. Frequently culverts and open channels are carefully designed with full consideration given to slope, cross section, and hydraulic roughness, but without regard to the inlet limitations. Culvert designs using uniform flow equations rarely carry their design capacity due to limitations imposed by the inlet.

Under inlet control, the cross sectional area of the barrel, the shape of the inlet, and the amount of ponding (headwater) at the inlet are primary design considerations. The size of a culvert operating with inlet control is determined by the size and shape of the inlet and the depth of ponding allowable (headwater) between the flowline elevation of a culvert and the elevation of a finished grade surface or surrounding buildings and facilities. Factors not affecting inlet control design are the barrel roughness, slope, and length and depth of the tailwater.

The headwater (HW) depth for a culvert of a given diameter or height (D) where a discharge is given can be determined by obtaining the HW/D value from Hydraulic Design of Highway Culverts, (FWHA 1985). Maximum headwater depth shall be 2 feet lower than the top of road/curb. The elevation of adjacent facilities (i.e., buildings, etc.) must be reviewed for flooding.

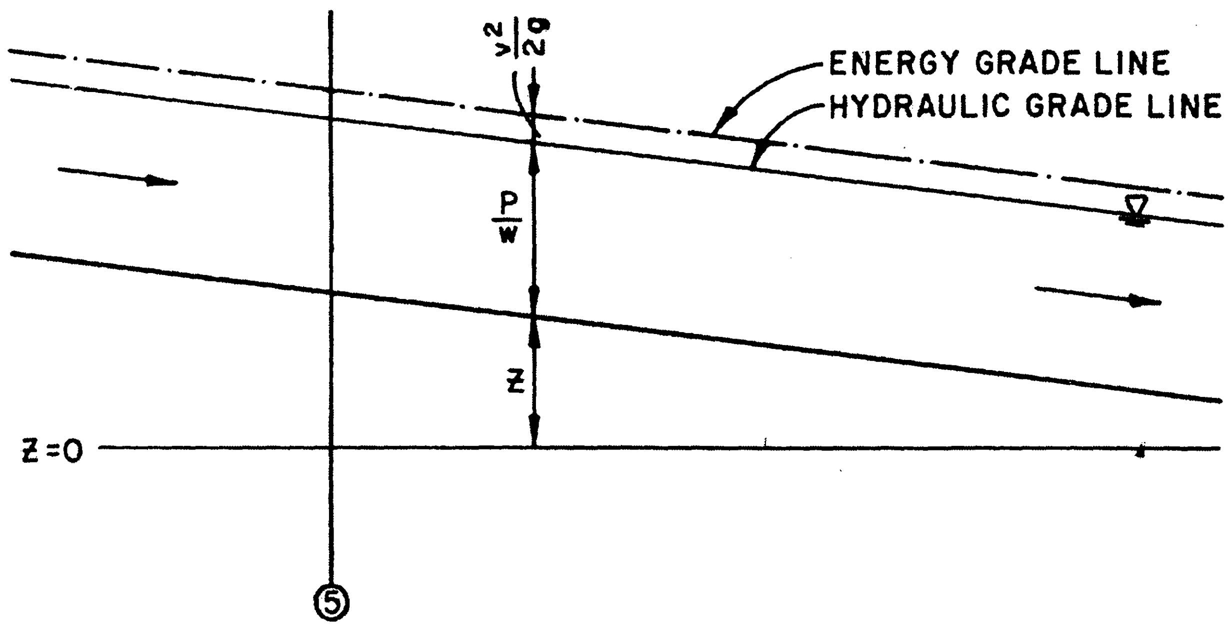
Culvert inlets of various geometric shapes and characteristics can be designed for specific purposes. The advantages and disadvantages of various types of inlets must be weighed carefully prior to selection, and the final choice must include consideration of hydraulics, topography, and overall cost of the installation.

Inlet control for culverts may occur in two ways. The least common occurs when the headwater depth is not sufficient to submerge the top of the culvert and the culvert invert slope is supercritical as shown in Figure III-26.



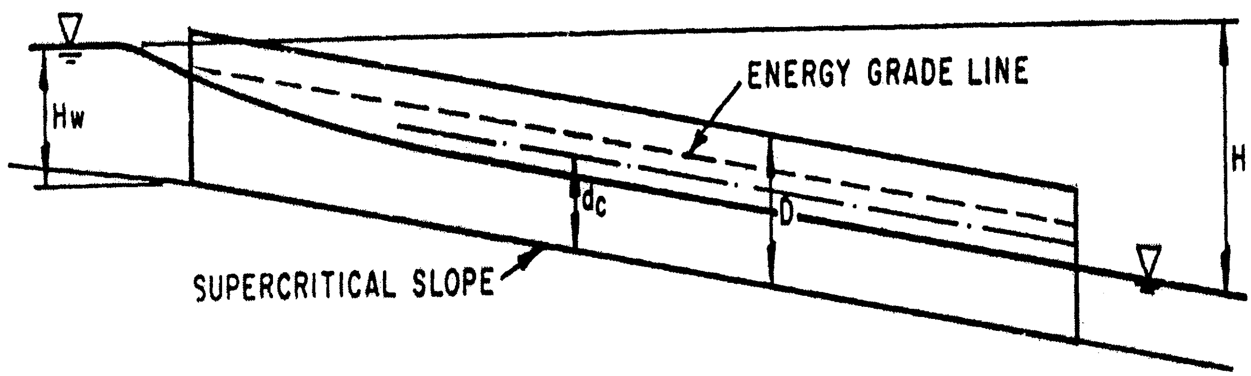
Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**DEFINITIONS OF TERMS FOR CONDUIT FLOW** **FIGURE III-24**



Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**DEFINITIONS OF TERMS FOR OPEN CHANNEL FLOW** **FIGURE III-25**



Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**INLET CONTOL – UNSUBMERGED INLET** **FIGURE III-26**

The most common occurrence of inlet control is when the headwater submerges the top of the culvert, Figure III-27, and the pipe does not flow full. A culvert flowing under inlet control is defined as a hydraulically short culvert.

4.4.3.2 Outlet Control

If the headwater is high enough, the culvert slope sufficiently flat, and the culvert sufficiently long, the control will shift to the outlet. Outlet control will exist under two conditions. The first and least common is that where the headwater is insufficient to submerge the top of the culvert, and the culvert slope is subcritical, Figure III-28. The most common condition exists when the culvert is flowing full, Figure III-28. A culvert flowing under outlet control is defined as a hydraulically long culvert.

Outlet control is dependent upon the depth of water in the outlet channel (TW), slope of the barrel, type of culvert material, and length of the barrel. A culvert will operate under outlet control when the depth of the tailwater, the length, the slope, or the roughness of the culvert barrel act as the control on the quantity of water able to pass through a given culvert. Energy head required for a culvert to operate under outlet control consists of velocity head (Hy), entrance loss (He), and friction loss (Hf). This energy head (H) is obtained from (FWHA 1985), and entrance loss coefficients from Table III-11. The headwater depth (HW) at the culvert entrance is calculated by means of the following formula:

$$\mathbf{HW = H + h_0 - LS_0} \qquad \textbf{(III-19)}$$

Where:

L = Length of culvert (ft)
H = Energy head (ft)
S_0 = Slope of barrel (ft/ft)
h_0 = hydraulic depth at outlet (ft)

$$\mathbf{h_0 = (d_c + D)/2} \qquad \textbf{(III-20)}$$

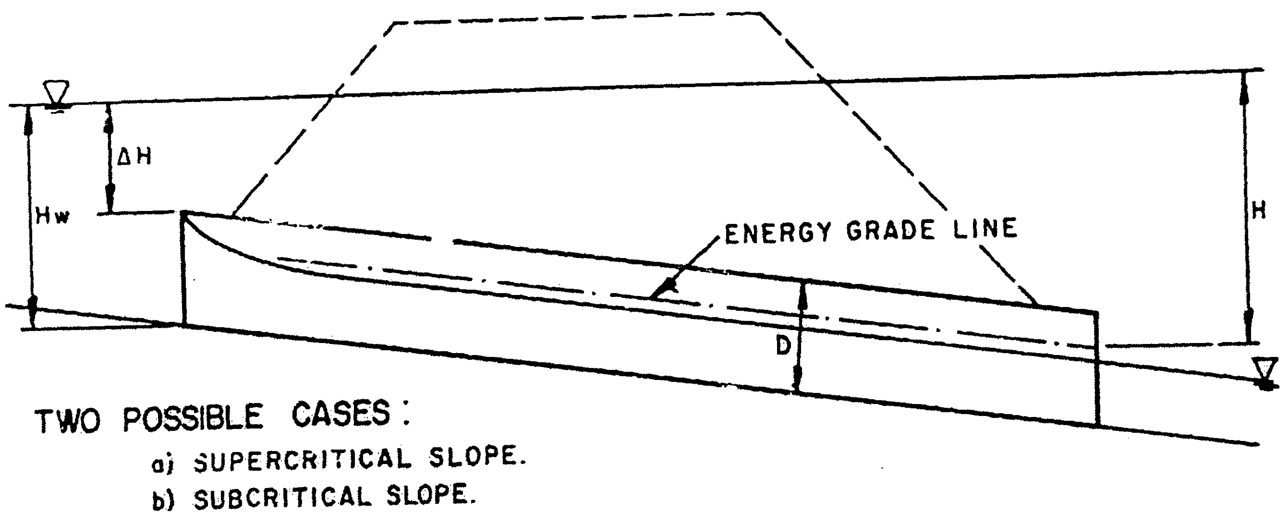
or **TW**, whichever is greater

Where:

d_c = Critical depth of flow in the barrel. Critical depth may be determined by using FWHA. (REF)
D = Height of pipe or box (ft)
TW = Tailwater depth (ft)

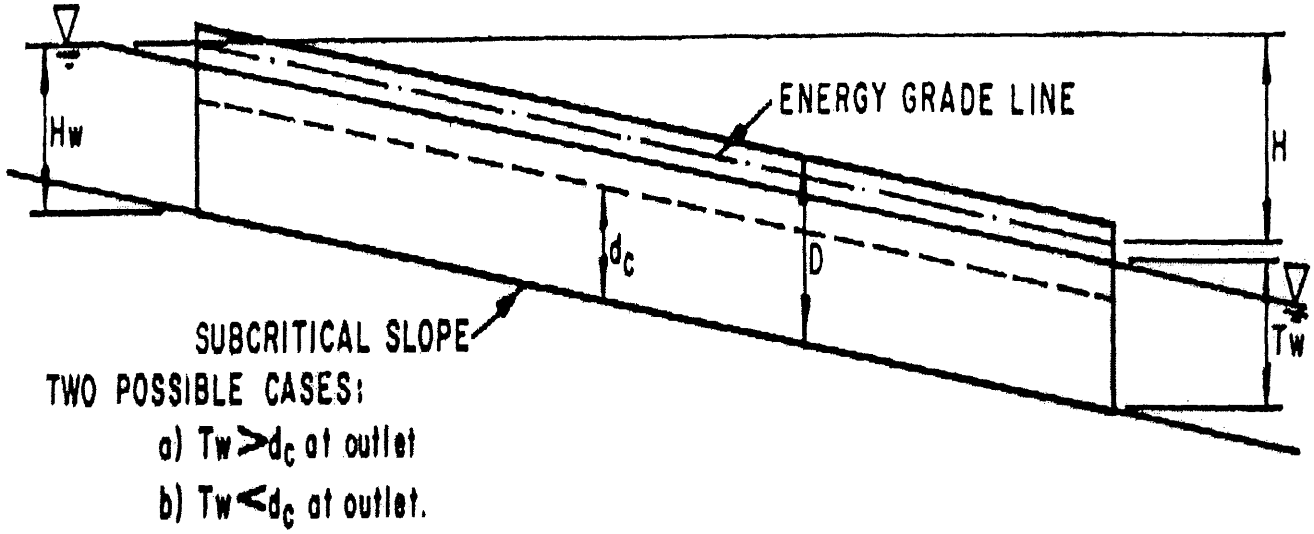
The maximum desirable headwater depth for culverts operating under outlet control shall be the same as described in Section III-4.4.3.1.

See Section III-4.4.9 for culvert design procedures.

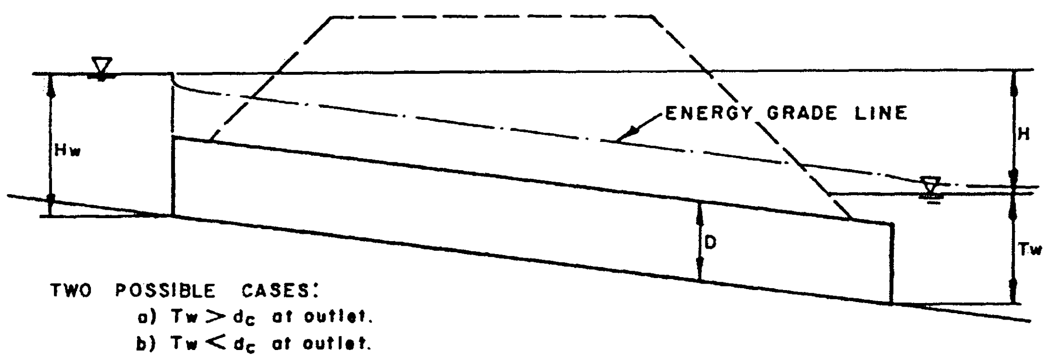


Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**INLET CONTOL – SUBMERGED INLET** **FIGURE III-27**



Partially Full Conduit



Full Conduit

Source: City of Tulsa, S.M.C.M.

City of RUSSELLVILLE Arkansas	OUTLET CONTROL – PARTIALLY FULL AND FULL CONDUITS **FIGURE III-28**

STRUCTURE AND ENTRANCE TYPE	COEFFICIENT k
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Metered to conform to fill slope	0.7
End-Section conforming to fill slope*	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Metered to conform to fill slope, paved or unpaved slope	0.7
End-Section conforming to fill slope*	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End-section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance.

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CULVERT ENTRANCE COEFFICIENTS

TABLE III-11

4.4.3.3 Drainage Inlets

4.4.3.3.1 Importance of Inlets

The importance of inlets can best be illustrated by reviewing the hydraulic considerations which are necessary to design culverts. For purposes of the following review, it is assumed that the reader has a basic working knowledge of hydraulics, and that the user of this manual is familiar with the following equations:

Manning	$Q = (1.486/n)AR^{2/3}S^{1/2}$	**(III-21)**
	$V = (1.486/n)R^{2/3}S^{1/2} = Q/A$	**(III-22)**
Continuity	$Q = v_1A_1 = v_2A_2$	**(III-23)**
Energy	$v^2/2g + P/W + Z + \text{losses} = \text{constant}$	**(III-24)**

4.4.3.3.2 Energy Losses

In short conduits, such as culverts, the form losses due to the entrance can be as important as the friction losses through the conduit. The losses that must be evaluated to determine the carrying capacity of the culverts consist of inlet losses, friction losses, and exit losses.

4.4.3.3.3 Inlet Losses

For inlet losses the governing equations are:

$$Q = CA\,(2gH_e)^{1/2} \qquad \text{(III-25)}$$

$$H_e = K_e(V^2/2g) \qquad \text{(III-26)}$$

Where:

A = Area of Cross Section of flow (ft^2)
Q = Discharge through the pipe (cfs)
C = Constant of discharge of the pipe
A = Area occupied by the water flow (ft^2)
V = Velocity of flow in the pipe (fps)
H_e = Head Loss due to entrance conditions (ft)
K_e = Entrance loss coefficient
$V^2/2g$ = Velocity head of the flow in the pipe
g = Gravitational constant = 32.2 (ft/sec^2)

4.4.3.3.4 Outlet Losses

For outlet losses, the governing equations are related to the difference in velocity head between the pipe flow and that in the downstream channel at the end of the pipe.

4.4.3.3.5 Friction Losses

Friction loss for pipes flowing full can be determined from

$$H_f = f(L/D)(V^2/2g) \qquad \textbf{(III-27)}$$

Where:

f = Friction factor
L = Length of culvert (ft)
D = Diameter of culvert barrel (ft)
V = Velocity of flow in the pipe (fps)
$V^2/2g$ = Velocity head of the flow in the pipe
g = Gravitational constant (32.2 ft/sec^2)

The friction factor has been determined empirically and is dependent on relative roughness, velocity, and barrel diameter. Tables are available in fluid mechanics texts for determination of the friction factor; however, prepared tables or curves are usually used to solve directly for the friction loss.

4.4.3.3.6 Energy Gradient and Hydraulic Gradeline

Figures III-24 and III-25 illustrate the energy grade line and hydraulic grade line and related terms.

The energy grade line (EGL), also known as the line of total head, is the sum of velocity head $V^2/2g$, the depth of flow or pressure head P/W, and elevation above an arbitrary datum represented by the distance Z. The EGL slopes downward in the direction of flow by an amount equal to the energy gradient HL/L, where HL equals the total energy loss over the distance L.

The hydraulic grade line (HGL), also known as the line of piezometric head, is the sum of the elevation Z and the depth of flow or pressure head P/W.

For open channel flow, the term P/W is equivalent to the depth of flow and the hydraulic grade line is the same as the water surface. For pressure flow in conduits, P/W is the pressure head and the hydraulic grade line falls above the top of the conduit as long as the pressure relative to atmospheric pressure is positive.

The design of a culvert, including the inlet and the outlet, requires a balance between cost, hydraulic efficiency, purpose, and topography at the proposed culvert site. Where there is sufficient allowable headwater depth, a choice of inlets may not be critical, but where headwater depth is limited, where erosion is a problem, or where sedimentation is likely, a more efficient inlet may be required to obtain the necessary discharge for the culvert.

While the primary purpose of a culvert is to convey water, a culvert may also be used to restrict flow, that is, to discharge a controlled amount of water while the upstream basin of the stream channel is used for detention storage to reduce a storm runoff peak. For this case, an inefficient inlet may be the most desirable choice.

The inlet types described in this section may be selected to fulfill either of the above requirements depending on the topography or conditions imposed by the Engineer. The entrance coefficient, K_e as used by Equation III-26, is a measure of the hydraulic efficiency at the inlet type, with lower values indicating greater efficiency.

Inlet coefficients recommended for use are given in Table III-11 of this MANUAL.

4.4.3.3.7 Projecting Inlets

Projecting inlets vary greatly in hydraulic efficiency and adaptability to requirements with the type of pipe material used. Figure III-29 illustrates this type of inlet. The primary advantage of projecting inlets is relatively low cost. Because projecting inlets are susceptible to damage due to maintenance of embankment and roadways and due to accidents, the adaptability of this type of entrance to meet the engineering and topographical demands varies with the type of material used.

CMP projecting inlets have limitations which include low efficiency, damage which may result from maintenance of the channel and the area adjacent to the inlet, and restrictions on the ability of maintenance crews to work around the inlet. The hydraulic efficiency of concrete grooved or bell-end pipe is good and, therefore, the only restrictions placed on the use of concrete pipe for projecting inlets is the requirement for maintenance of the channel and the embankment surrounding the inlet. Where equipment will be used to maintain the embankment around the inlet, it is not recommended that a projecting inlet of any type be used.

4.4.3.3.8 Concrete Pipe

Bell and spigot concrete pipe or tongue and groove concrete pipe with the bell end, or with the grooved end, used as the inlet section are quite efficient hydraulically, having an entrance coefficient of about 0.25. For concrete pipe which has been cut, the entrance is square edged, and the entrance coefficient is about 0.5.

4.4.3.3.9 Corrugated Metal Pipe

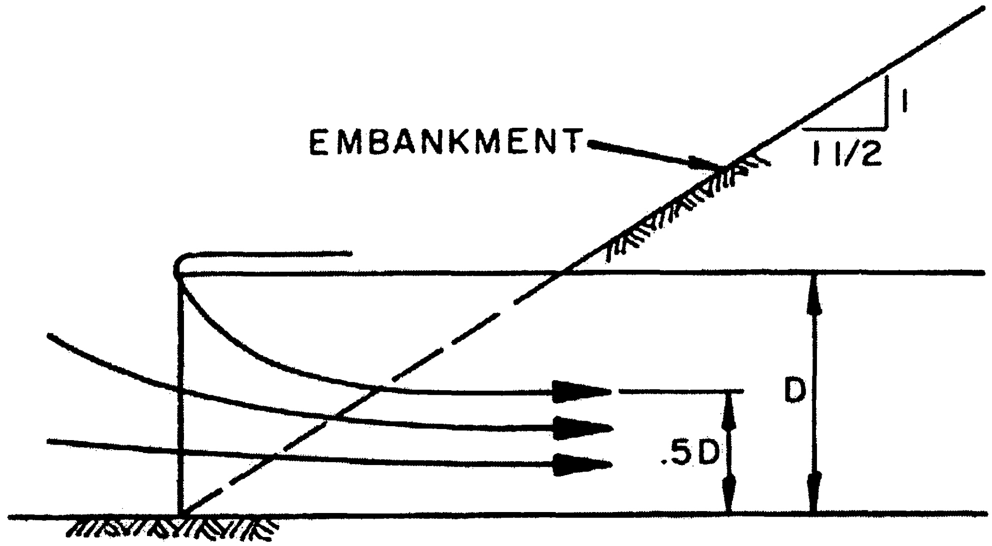
A projecting entrance of corrugated metal pipe (CMP) is equivalent to a sharp-edged entrance with a thin wall and has an entrance coefficient of about 0.9.

4.4.3.4 Inlets with Headwalls

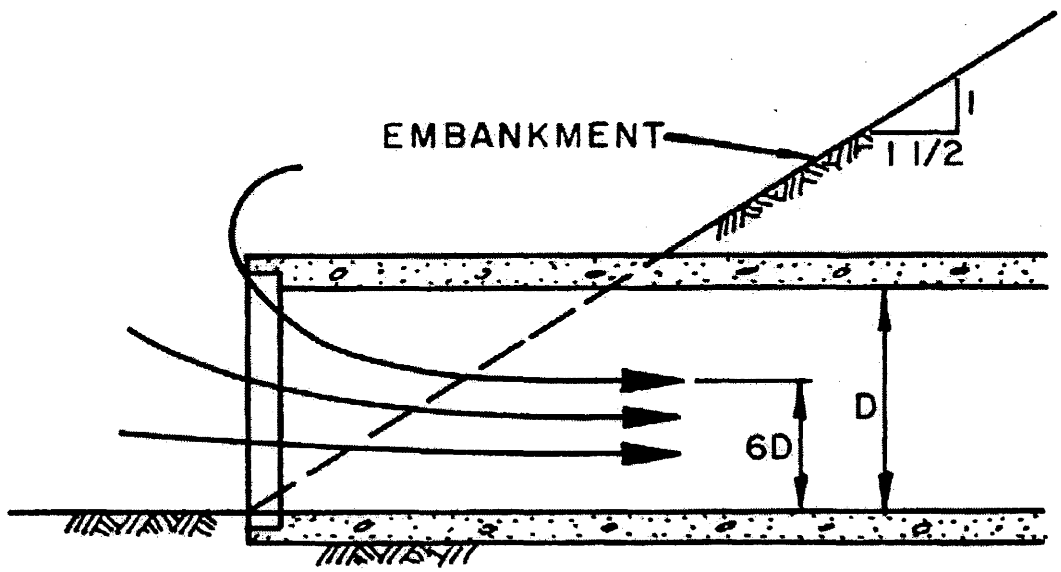
Headwalls may be used for a variety of reasons: increasing the efficiency of the inlet, providing embankment stability, and providing embankment protection against erosion. The relative efficiency of the inlet varies with the pipe material used. Figure III-30 illustrates a headwall with wingwalls.

4.4.3.4.1 Wingwalls

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable and where the culvert is skewed to the normal channel flow. Little increase in hydraulic



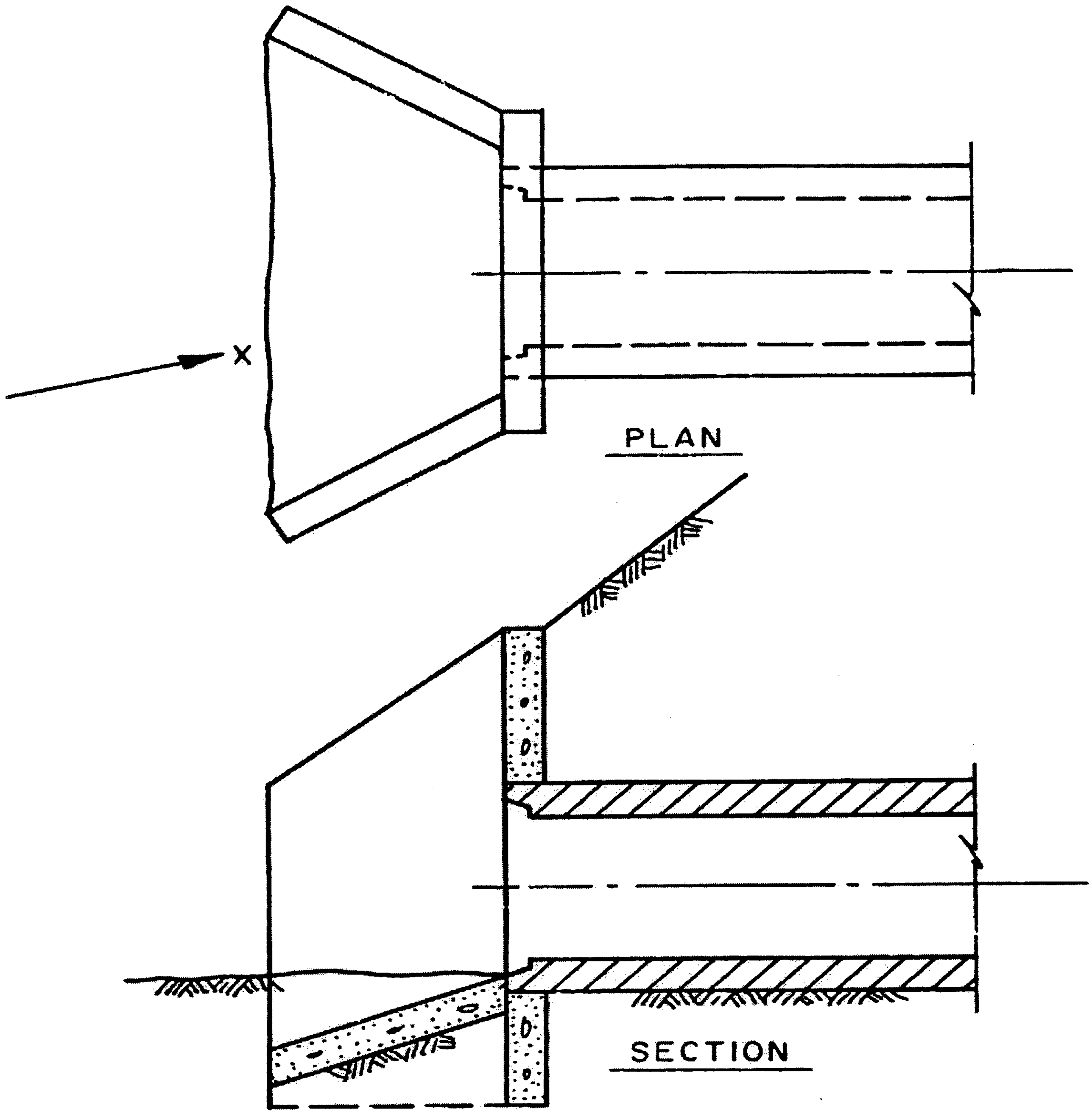
PROJECTING THIN-WALL PIPE ENTRANCE



PROJECTING CONCRETE GROOVED END PIPE ENTRANCE

Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**COMMON PROJECTING CULVERT INLETS** **FIGURE III-29**



Source: City of Tulsa, S.M.C.M.

City of **RUSSELLVILLE** Arkansas	**INLETS WITH HEADWALL AND WINGWALLS** **FIGURE III-30**

efficiency is realized with the use of wingwalls regardless of the pipe material used. Therefore, the use should be justified for reasons other than an increase in hydraulic efficiency. Figure III-31 illustrates several cases where wingwalls are used. For parallel wingwalls, the minimum distance between wingwalls should be at least 1.25 times the diameter of the culvert pipe.

4.4.3.4.2 Aprons

If high headwater depths are to be encountered, or if the approach velocity of the channel will cause scour, a short channel apron should be provided at the toe of the headwall, This apron should extend at least one pipe diameter upstream from the entrance and the top of the apron should not protrude above the normal streambed elevation.

Culverts with wingwalls should be designed with a concrete apron extending between the walls. Aprons must be reinforced to control cracking. As illustrated in Figure III-31, the actual configuration of the wingwalls varies according to the direction of flow and will also vary according to the topographical requirement placed upon theme

For conditions where scour may be a problem due to high approach velocities and special soil conditions, such as alluvial soils, a toe wall is often desirable for apron construction.

4.4.3.4.3 Concrete Pipe

For tongue and grooved or bell end concrete pipe, little increase in hydraulic efficiency is realized by adding a headwall. The primary reason for using headwalls is for embankment protection and for ease of maintenance. The entrance coefficient is equal to about 0.2 for grooved and bell-end pipe, and equal to 0.4 for cut concrete pipe.

4.4.3.4.4 Corrugated Metal Pipe

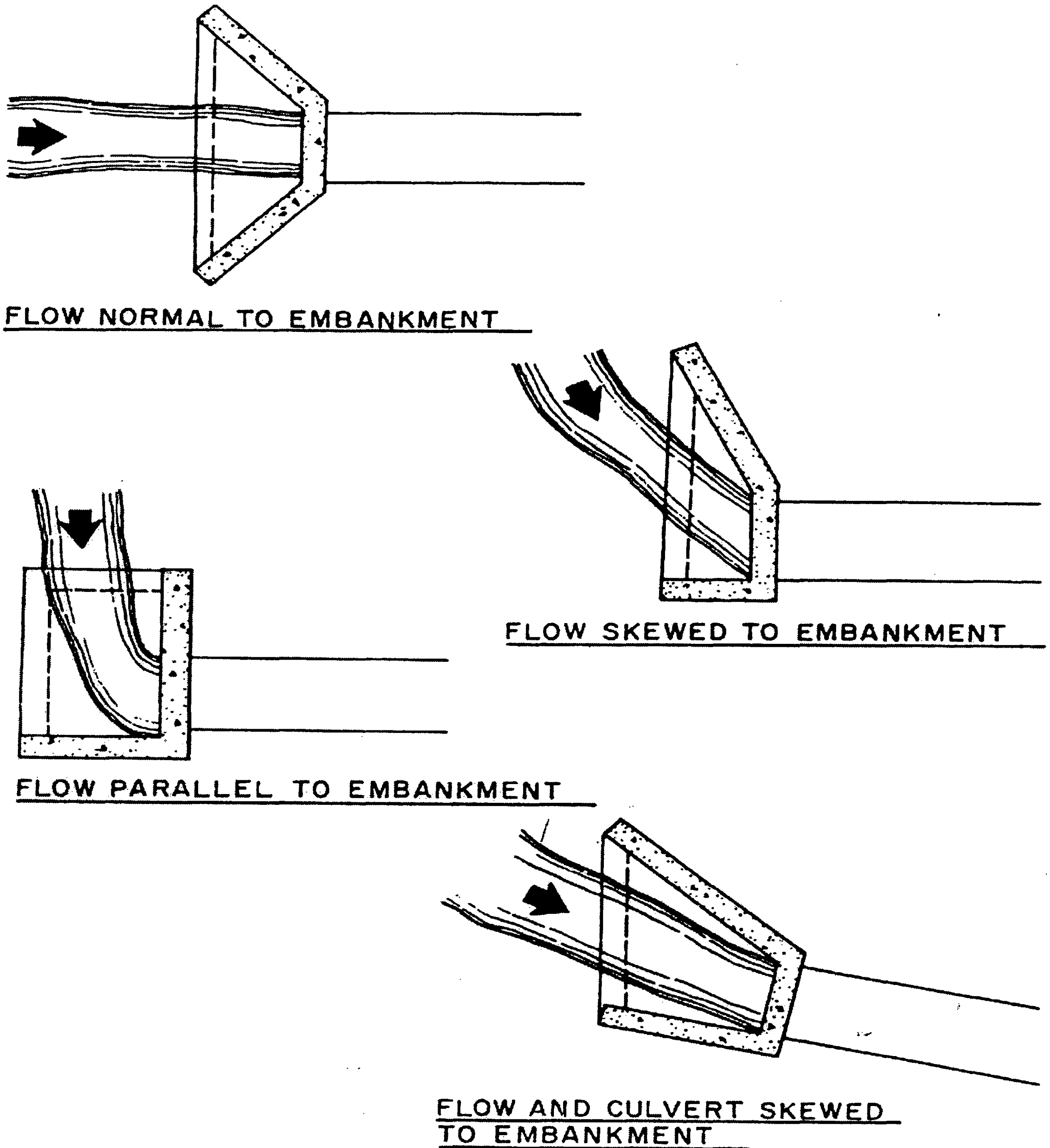
CMP in a headwall is essentially a square-edged entrance with an entrance coefficient of about 0.4. The entrance losses may be reduced by rounding the entrance. The entrance coefficient may be reduced to 0.15 for a rounded edge with a radius equal to 0.15 times the culvert diameter, and to 0.10 for rounded edge with a radius equal to 0.25 times the diameter of the culvert.

4.4.3.5 Special Inlets

There are a great variety of inlets other than the common ones described above. Among these are special end-sections that serve as both outlets and inlets and are available for both CMP and concrete pipe. Because of the difference in requirements due to pipe materials, the special end-sections will be discussed independently according to pipe material and mitered inlets will also be considered.

4.4.3.5.1 Mitered Inlets

The use of this entrance type is predominantly with CMP and its hydraulic efficiency is dependent on the construction procedure used. If the embankment is not paved, the entrance in



Source: City of Tulsa, Stormwater Management Criteria Manual

City of **RUSSELLVILLE** Arkansas	**TYPICAL HEADWALL-WINGWALL CONFIGURATIONS** **FIGURE III-31**

practice usually does not conform with the side slopes, giving essentially a projecting entrance (K_e = 0.9). If the embankment is paved, a sloping headwall is obtained and K_e = 0.60 and, by beveling the edges, K_e = 0.50.

Uplift is an important factor for this type entrance. It is not good practice to use unpaved embankment slopes where a mitered entrance may be submerged to an elevation one-half the diameter of the culvert above the top of the pipe.

4.4.3.5.2 Concrete Pipe

As in the case of CMP, these special end-sections may aid in increasing the embankment stability or in retarding erosion at the inlet. They should be used where maintenance equipment must be used near the inlet or where, for aesthetic reasons, a projecting entrance is considered too unsightly.
The hydraulic efficiency of this type of inlet is dependent on the geometry of the end-section to be used. Where the full contraction to the culvert diameter takes place at the first pipe section, the entrance coefficient (K_e) is equal to 0.5, and where the full contraction to the culvert diameter takes place in the throat of the end-section, the entrance coefficient (K_e) is equal to 0.25.

4.4.3.5.3 Corrugated Metal Pipe

Special end-sections for corrugated metal pipe (CMP) add little to the overall cost of the culvert and have the following advantages:

1. Less maintenance around the inlet.
2. Less damage from maintenance work and from accidents compared to a projecting entrance.
3. An increase in hydraulic efficiency is realized.

When using design charts, charts for square-edged opening for CMP with a headwall may be used.

4.4.4 Headwalls And Endwalls

The normal functions of properly designed headwalls and end walls are to anchor the culvert, to prevent movement due to the lateral pressures, to control erosion and scour resulting from excessive velocities and turbulence, and to prevent adjacent soil from sloughing into the waterway opening. Headwalls shall be constructed of reinforced concrete and may either be straight parallel headwalls, flared headwalls, or warped headwalls with or without aprons, as may be required by site conditions. Multi-barrel culvert crossings of roadways at an angle of 15° or greater shall be accompanied by adequate inlet and outlet control sections.

4.4.4.1 Conditions at the Entrance

It is important to recognize that the operational characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Design of culverts involves consideration of energy losses that occur at the entrance. The entrance head losses may be determined by the following equation:

$$\mathbf{h_e = K_e\,(V_2^2 - V_1^2)/2g} \qquad \textbf{(III-28)}$$

Where:

h_e = Entrance head loss (ft)
K_e = Entrance loss coefficient as shown in Table III-11
V_2 = Velocity of flow in culvert (fps)
V_1 = Velocity of approach (fps)

4.4.4.2 Selection of Headwall or Endwall

In general, the following guidelines should be used in the selection of the type of headwalls or endwalls:

1. Approach velocities are low (less than 6 fps).
2. Backwater pools may be permitted.
3. Approach channel is undefined.
4. Ample right-of-way or easement is available.
5. Downstream channel protection is not required.

Flared Headwall and Endwall:

1. Channel is well defined.
2. Approach velocities are between 6 and 10 fps.
3. Medium amounts of debris exist.

The wings of flared walls should be located with respect to the direction of the approaching flow instead of the culvert axis.

Warped Headwall and Endwall:

1. Channel is well defined and concrete-lined.
2. Approach velocities are between 8 and 20 fps.
3. Medium amounts of debris exist.

These headwalls are effective with drop down aprons to accelerate flow through the culvert and are effective for transitioning flow from closed conduit flow to open channel flow. This type of headwall should be used only where the drainage structure is large and right-of-way or easement is limited.

4.4.5 Culvert Discharge Velocities

The velocity of discharge from culverts should be limited as shown in the table below. Consideration must be given to the effect of high velocities, eddies, or other turbulence on the natural channel, downstream property, and roadway embankment.

Culvert Discharge - Velocity Limitations	
Downstream Channel Condition	**Maximum Allowable Discharge Velocity (fps)**
Earth, consolidated cohesive	6
Sodded	8
Shale	10
Paved or Riprap Apron	15
Rock	15

Energy dissipators will be required at channel drops when the unit discharge exceeds 35 cfs/ft and at culvert outlets when the discharge velocity exceeds those recommended above for a given downstream channel condition.

4.4.6 Energy Dissipators

Energy dissipators are used to dissipate excessive kinetic energy in flowing water that could promote erosion. An effective energy dissipator must be able to retard the flow of fast moving water without damage to the structure or to the channel below the structure.

Impact-type energy dissipators direct the water into an obstruction that diverts the flow in many directions and in this manner dissipates the energy in the flow. Baffled outlets and baffled aprons are two impact-type energy dissipators.

Other energy dissipators use the hydraulic jump to dissipate energy. In this type of structure, water flowing at a higher than critical velocity is forced into a hydraulic jump, and energy is dissipated in the resulting turbulence. Stilling basins are an example of a dissipator where energy is diffused as flow plunges into a pool of water.

The design procedure for energy dissipators can be found in the book "Hydraulic Design of Energy Dissipators for Culverts and Channels," HEC-14, USDOT, FHWA (1975).

4.4.7 Culvert Types and Sizes

The permissible types of culverts under all roadways and embankments are reinforced concrete box, round, or elliptical concrete pipe or pipe arch.

The minimum size of pipe for all culverts shall be 15 inches or the equivalent sized pipe or arch pipe, and 18 inches or the equivalent sized elliptical pipe or arch pipe, for collector and higher classified street, except as approved by the CITY ENGINEER. Box culverts may be constructed in sizes equal to or larger than 4 ft x 3 ft (width versus height), except as approved by the CITY ENGINEER.

If material other than reinforced concrete pipe is to be used, it shall be approved by the CITY ENGINEER.

Flared, pre-cast concrete and metal pipe aprons may be used in lieu of headwalls to improve the hydraulic capabilities of the culverts.

4.4.8 Fill Heights and Bedding

Where possible, the minimum cover over any culvert or box culvert shall be 18 inches, or a minimum of 6 inches from the bottom of the pavement sub-base. Minimum cover less than these values shall be fully justified in writing and approved by the CITY ENGINEER prior to proceeding with final plans. Maximum fill heights and construction procedures shall comply with standards and specifications of the Arkansas Highway and Transportation Department. Box culverts shall be structurally designed to accommodate earth and live load to be imposed upon the culvert. Refer to the Arkansas Highway and Transportation Department's Standard Plans for Typical Box Culvert Designs. When installed within public right-of-ways, all culverts shall be capable of withstanding minimum HS20 loading.

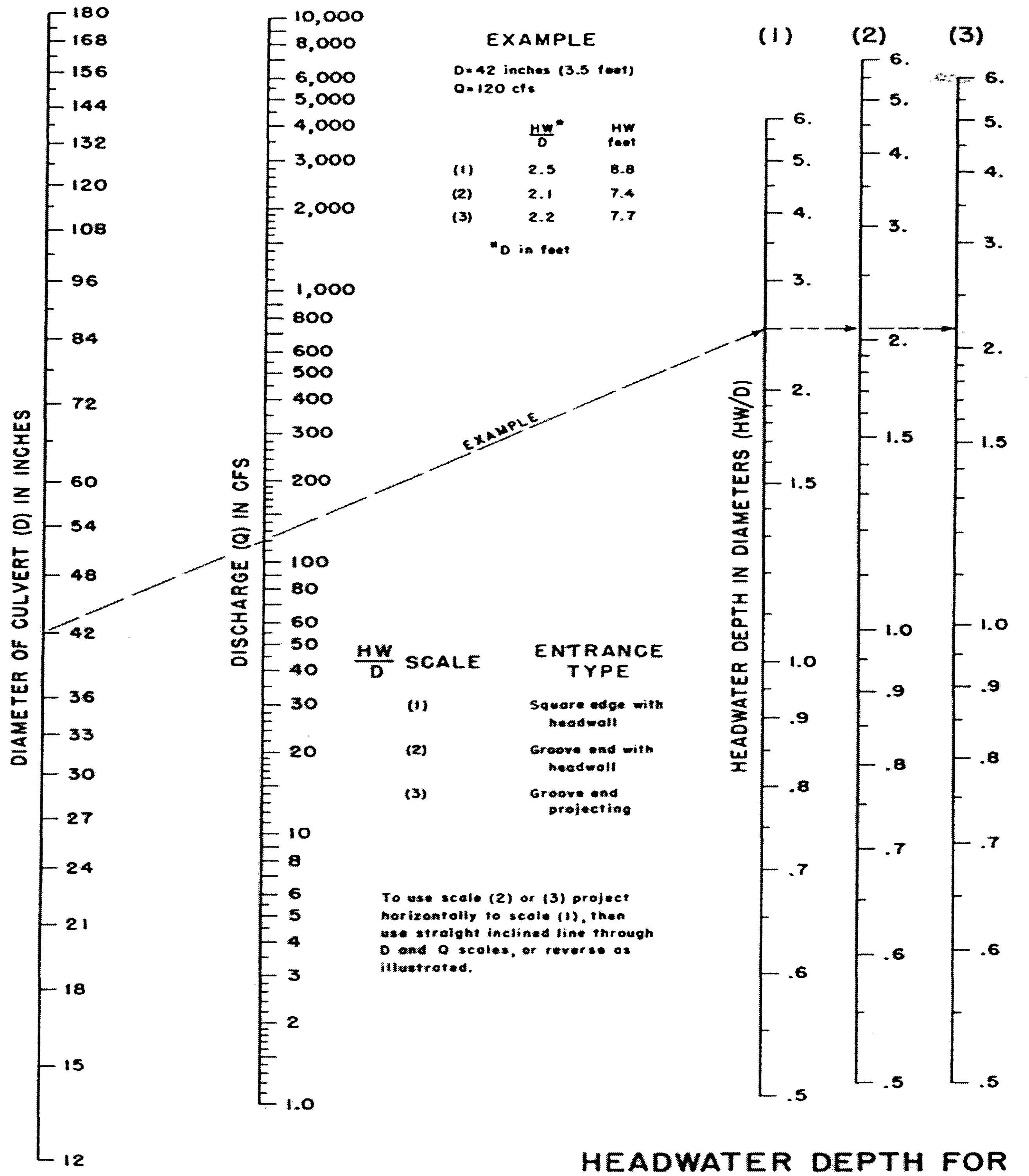
For culverts proposed under railroad facilities, the Engineer shall obtain prior approval from the affected railroad.

4.4.9 Culvert Design Procedures

Culvert design can be performed by several procedures. For culvert design within the City of Russellville and its Planning Area, three procedures are discussed in this MANUAL and should be used in preparation of Stormwater Management Plans in conjunction with MAJOR DEVELOPMENTS. Hydrologic design criteria are specified in the Russellville SDMO and in Section III-4.4.2 of this MANUAL. All culvert designs shall adhere to the specified criteria unless written approval is given in the CITY ENGINEER'S response to the Development Classification Form submitted as specified in DIVISION I of this MANUAL.

4.4.9.1 Nomographs

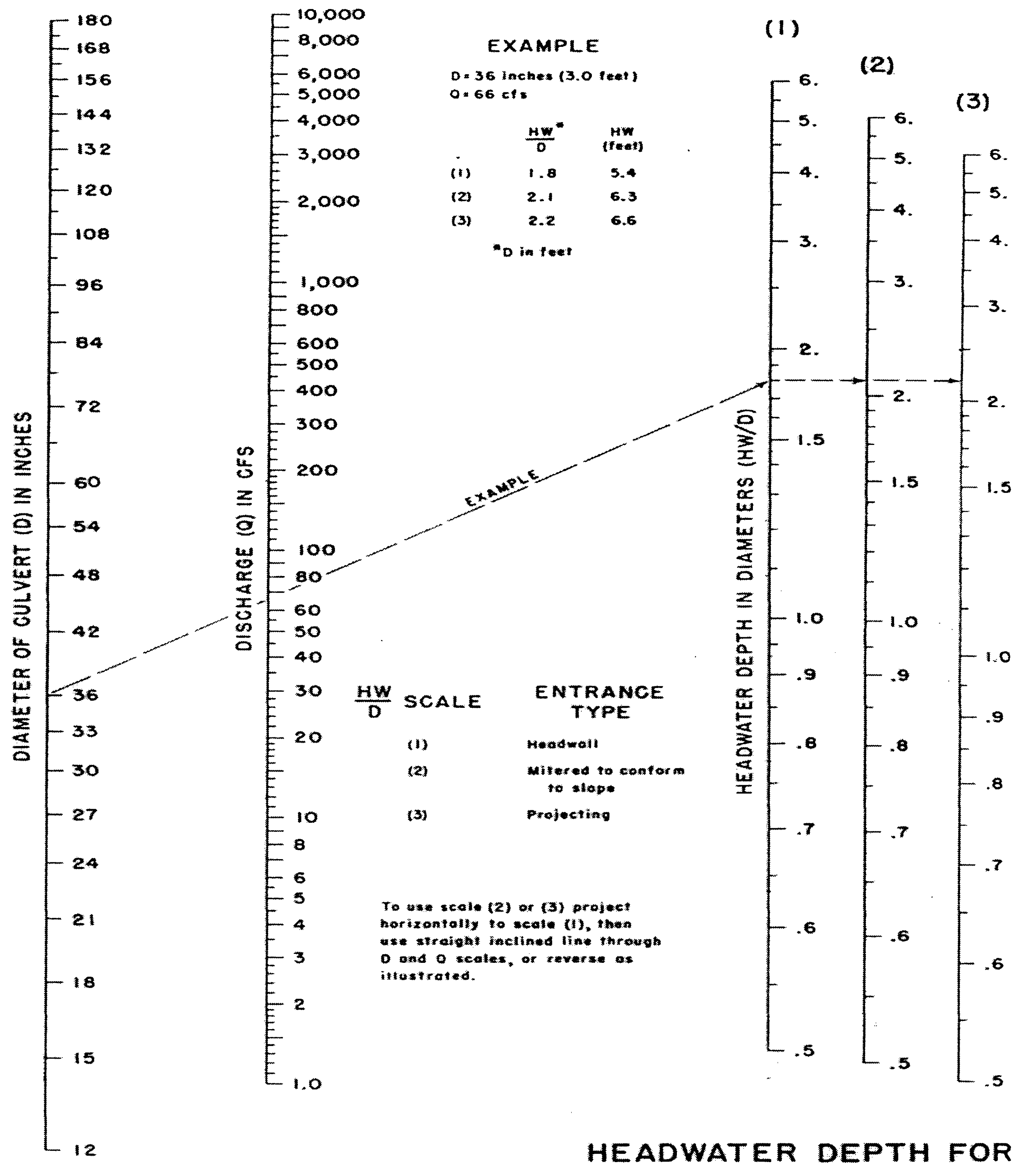
Culvert design may be performed by use of nomographs. Examples of four nomographs for designing culverts are presented on Figures III-32 through III-35. The use of these nomographs is limited to cases where tailwater depth is higher than the critical depth in the culvert. The disadvantage of the nomographs is that they are trial and error. The procedure for design requires the use of both nomographs and is as follows:



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

Source: Hydraulic Manual, Texas Highway Department

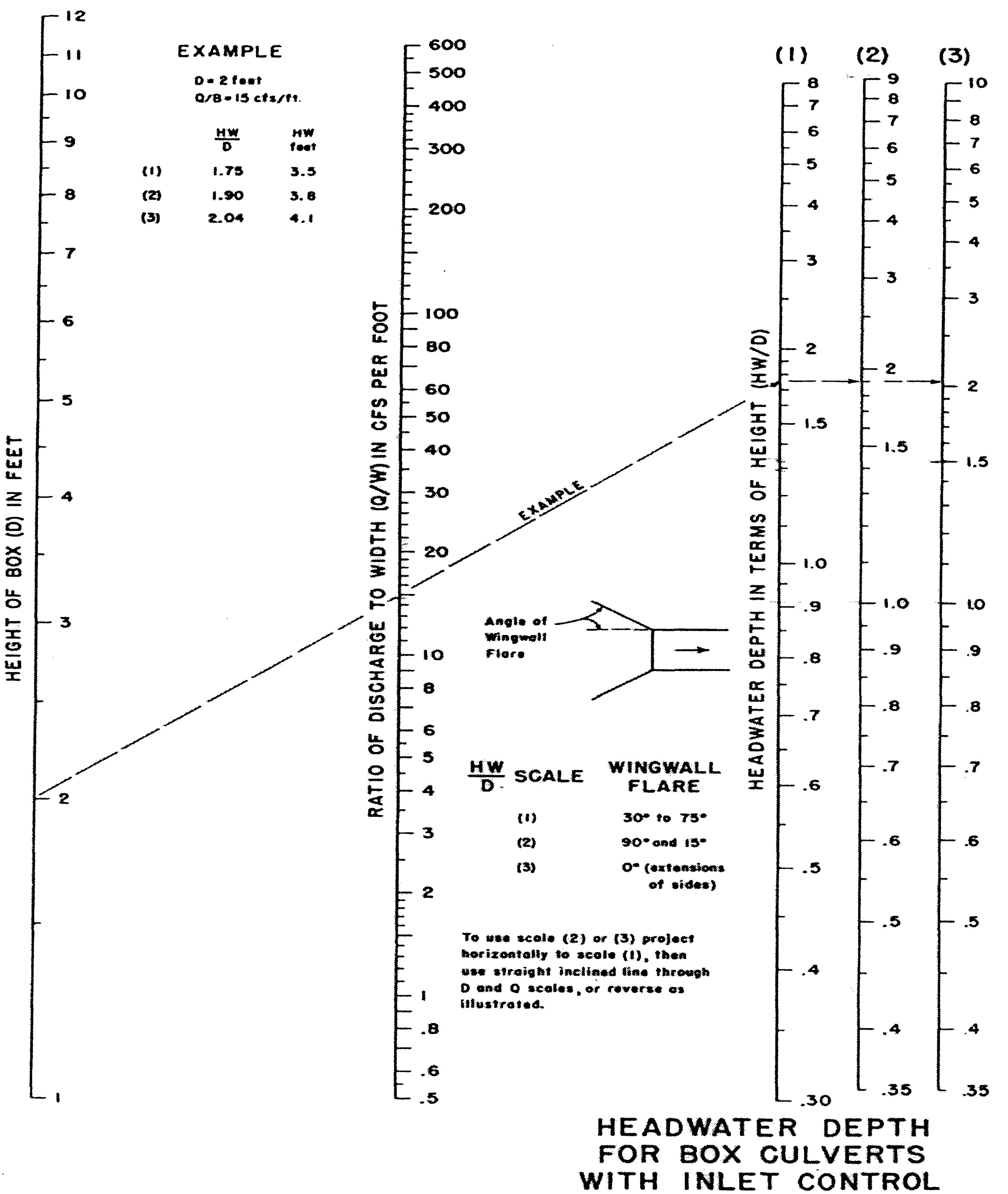
City of **RUSSELLVILLE** Arkansas	**HEADWATER DEPTH FOR CONCRETE PIPE CULVERT WITH INLET CONTROL** **FIGURE III-32**



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

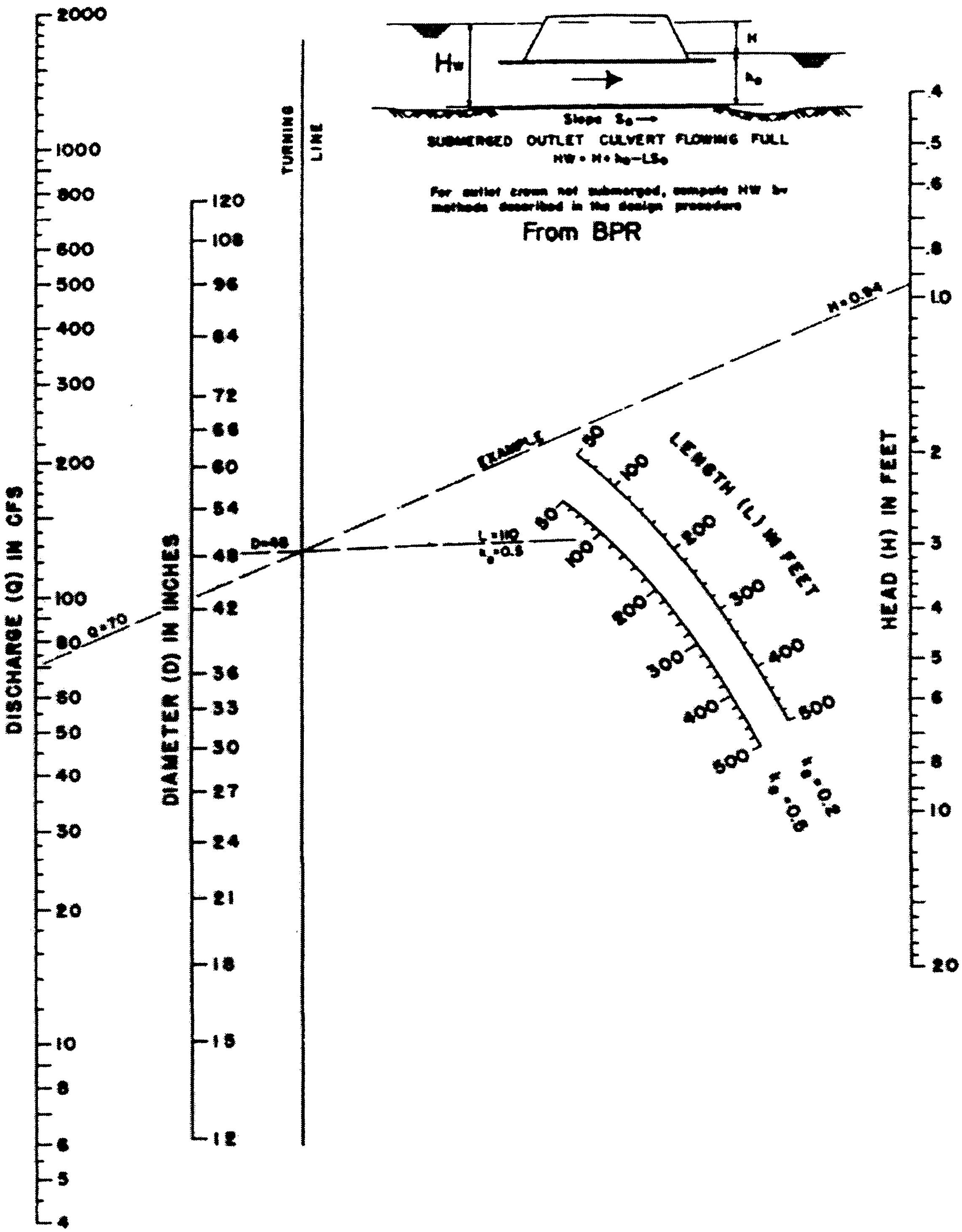
Source: Hydraulic Manual, Texas Highway Department

City of **RUSSELLVILLE** Arkansas	**HEADWATER DEPTH FOR C. M. PIPE CULVERT WITH INLET CONTROL** **FIGURE III-33**



Source: Hydraulic Manual, Texas Highway Department

City of **RUSSELLVILLE** Arkansas	**HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL** **FIGURE III-34**



Source: Bureau of Public Roads, 1963

City of **RUSSELLVILLE** Arkansas	**OUTLET CONTROL NOMOGRAPH** **CIRCULAR RCP CULVERT** **FIGURE III-35**

1. List design data:

   A. Design discharge, Q (cfs)
   B. Length of culvert, L (ft)
   C. Invert elevations in and out (ft)
   D. Allowable headwater depth, HW (ft)
   E. Mean and maximum flood velocities in natural stream (fps)
   F. Culvert type
   G. Entrance type for first selection

2. Determine a trial size by assuming a maximum average velocity based on channel considerations to compute the area, $A = Q/V$.

3. Find Headwater, HW, for trial size culvert for inlet control and outlet control. For inlet control, Figures III-32 through III-34 can be used for most culvert designs. Additional nomographs can be found in various FWHA and AHTD hydraulic design publications.

   A. Connect a straight line through D and Q to scale (1) of HW/D scales and project horizontally to the proper scale.

   B. Compute HW and, if too large or too small, try another size before computing HW for outlet control.

   C. Compute the HW for outlet control; Figure III-35 is an example of a nomograph of a concrete circular pipe with outlet control. Additional nomographs can be found in various FWHA and AHTD hydraulic design publications.

   D. Enter the graph with the length, the entrance coefficient for the entrance type, and the trial size.

   E. Connect the length scale and the culvert size scale with a straight line.

   F. Pivot on the turning line, and draw a straight line from the design discharge on the discharge scale through the turning point to the head scale (head loss, H).

   G. Compute HW from Equation III-19:

   $$HW = H + ho - LS$$

   H. For tailwater depth (TW) greater than or equal to the top of the culvert, ho = TW, and for TW less than the top of the culvert Equation (III-20):

   $$ho = d_c + D/2, \text{ or } TW,$$

   whichever is the greater.

   (If TW is less than d_c, the nomographs cannot be used. See Hydraulic Charts for the Selection of Highway Culverts, Bureau of Public Roads Circular #5 for critical depth charts.)

4. Compare the computed headwaters and use the higher HW to determine if the culvert is under inlet or outlet control.
5. If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet control nomographs.

If a smaller size culvert is selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

4.4.9.2 Manual Computations

Culvert design can also be performed through manual computations. The following is a step-by-step format for manually designing culverts within the City of Russellville and its Planning Area.

The computations involved in selecting the smallest feasible barrel that can be used without exceeding the design headwater elevation are summarized in the tabulation sheet shown on Figure III-36, "Culvert Design Worksheet".

Enter initial data and complete required information for the first approximation. The square feet of opening for the initial trial size may be estimated by the ratio of design discharge divided by 10.

Discharges, Q, to be used in the design of culverts in the City of Russellville, Arkansas and its Planning Area shall be as specified in Section III-4.4.2 unless previously approved in writing by the CITY ENGINEER. Discharges for Detention Outlet Design shall be determined as defined in Section 4.5. Proper values shall be recorded where provided on Figure III-36. The values of discharge used in the design/analysis of culverts may be entered in the "Hydrologic and Channel Information" area on Figure III-36.

Site information will usually provide some of the initial data for beginning culvert design. Entry positions are provided in the "Sketch" area of Figure III-36. Height of the embankment through which the culvert is intended to convey storm runoff may be entered in the space denoted as El.

Maximum HW depth or elevation and TW may be entered as noted on the worksheet. Tailwater is discussed more in depth below.

Culvert length (L), slope (S), and inlet/outlet invert elevations are usually determined by the slope of the associated channel and the geometric characteristics of the embankment through which it will convey storm runoff. These values may change during the design procedure based on such factors as the configuration of the inlet and outlet. These values may also be controlled by "design purpose" such as a culvert outlet for Detention Design. Each of these values may likewise be entered on the worksheet in the "Sketch" area. The value of S should be entered as a "percent" if this worksheet is used. The term L/(100S) computes the vertical distance of the culvert inverts if the slope is entered as a "percent".

Tailwater elevations for the frequency discharges being used for design may be entered off Figure III-36 in the "Hydrologic and Channel Information" area with the associated discharge. The tailwater depth is influenced by conditions downstream of the culvert outlet. If the culvert

City of
RUSSELLVILLE
Arkansas

CULVERT DESIGN WORKSHEET

FIGURE III-36

Source: Bureau of Public Roads

PROJECT:

DESIGNER: ________

DATE: ________

HYDROLOGIC AND CHANNEL INFORMATION

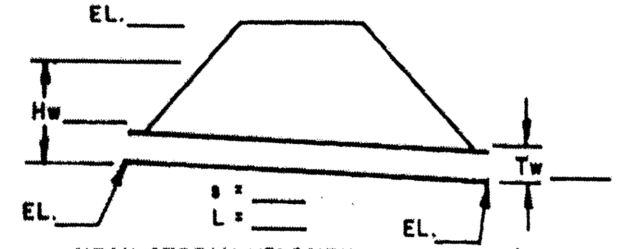
Q_1 = ______ TAILWATER ELEVATION = ____

Q_2 = ______ TAILWATER ELEVATION = ____

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ________



MEAN STREAM VELOCITY = ______ $\frac{L}{100s}$ = ______

MAX. STREAM VELOCITY = ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT.		OUTLET CONTROL $HW = H + h_o - Ls$								CHART		CONTROLLING Hw	OUTLET VELOCITY	COST	COMMENTS
			$\frac{Hw}{D}$	Hw	K_e	H	d_c	$\frac{d_c+D}{2}$	Tw	h_o	Ls	Hw	No.	Hw				

SUMMARY & RECOMENDATIONS:

outlet is located near the inlet of a downstream culvert, then the headwater elevation of the downstream culvert may define the tailwater depth for the upstream culvert. If the culvert outlet is operating in a free outfall condition, then the tailwater is taken as 0.0. If the culvert discharges into an open channel, then tailwater conditions should be determined by either backwater conditions, normal depth (subcritical flow), or critical depth (supercritical flow). Figures III-37, III-38, and III-39 provide a graphic solution for normal depth flow that may be calculated by Manning's Equation (Equation III-21):

The TW is defined as the depth of water measured from the flow line of the culvert (invert) at the outlet to the water surface elevation at the outlet.

Enter TW in Column 10 and applicable stream data in the upper right-hand portion of the Culvert Design Worksheet. Tailwater elevations are entered in the upper left-hand portion of the worksheet.

4.4.9.2.1 Outlet Control Calculation

These calculations are performed before inlet control calculations in order to select the smallest feasible barrel that can be used without the required headwater elevation in outlet control exceeding the allowable headwater elevation.

Nomograph solutions for outlet control of various types and materials of storm sewer culverts are provided in most pipe manufacturing catalogs as well as design publications of the FHWA and AHTD.

The following precedures are typical for the performance at Outlet control calculations. The "Column" referred to corresponds to the columns on the Culvert Design Worksheet presented in Figure III-36.

Column 1: Enter the culvert description, type of structure, and design of entrance.

Column 2: Divide the discharge, Q, by the number of culvert barrels in this trial and enter in this column.

Column 3: Enter the span times height dimensions (or diameter of pipe) of culvert.

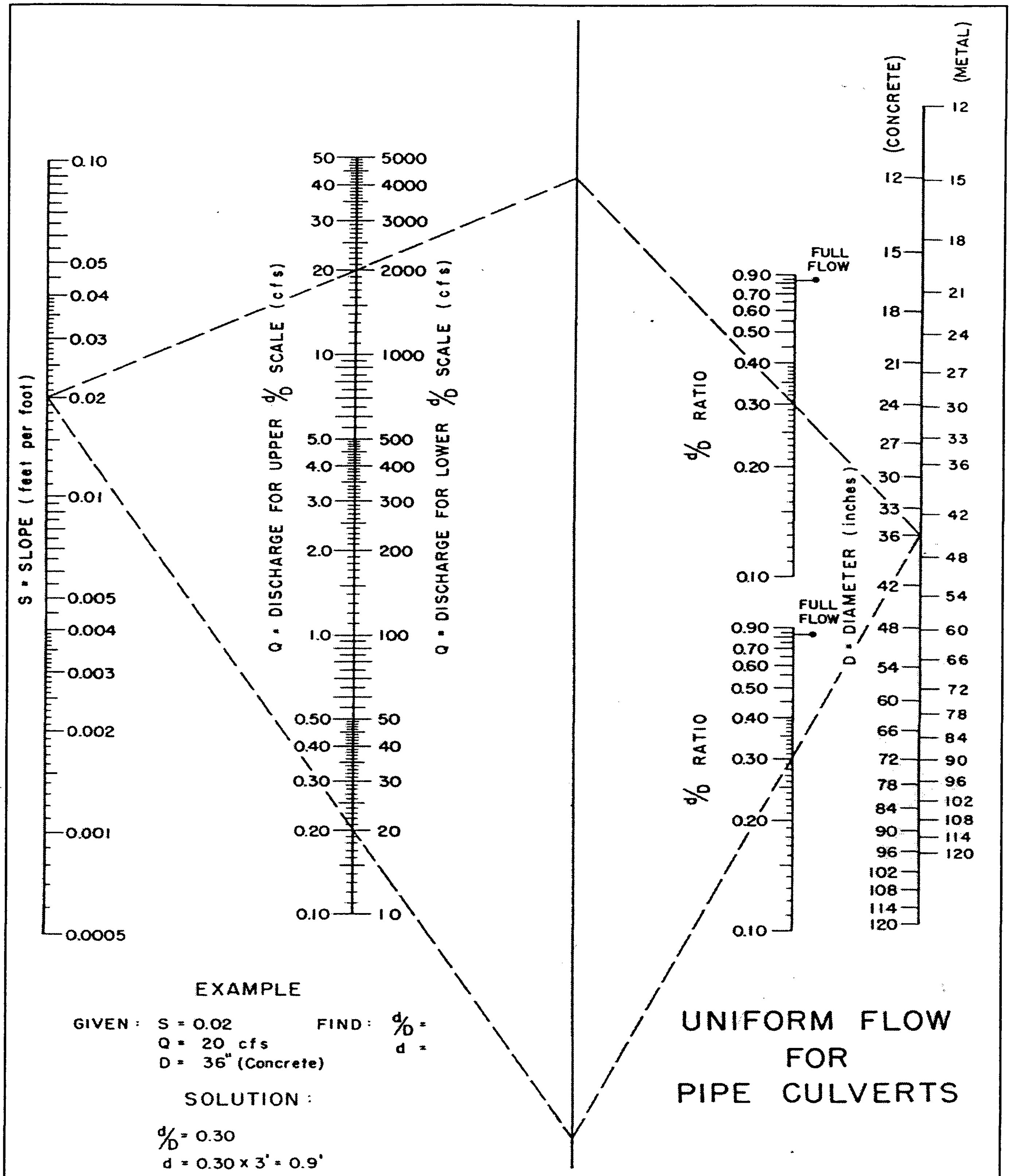
Column 4 & 5 are included in the Procedures for inlet control calculations.

Column 6: Enter the entrance loss coefficient from Table III-11.

Column 7: Enter the head from the applicable outlet control nomograph.

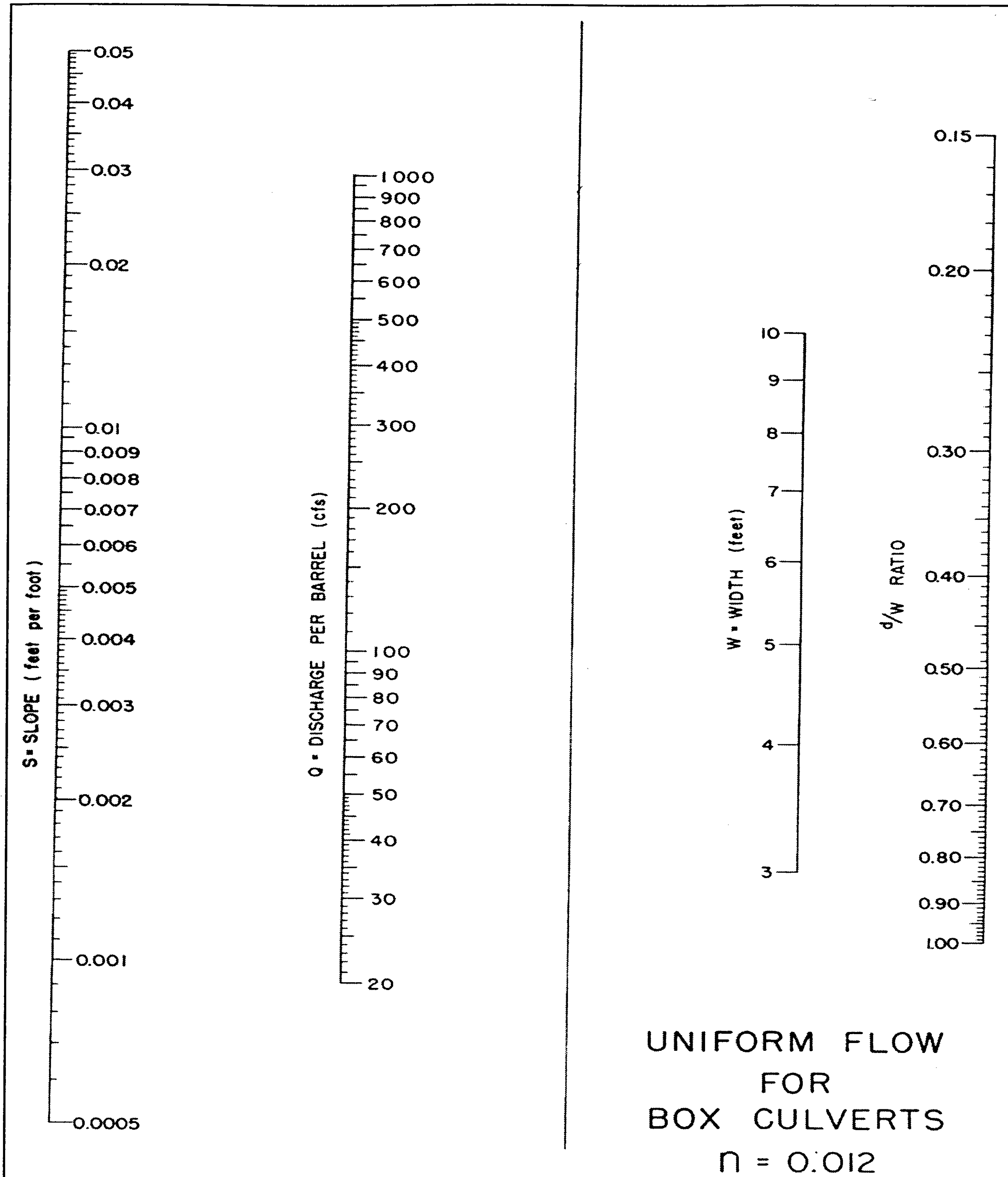
Column 8: Enter the critical depth from appropriate nomograph. Critical depth cannot exceed the height of the culvert opening.

Column 9: For tailwater elevations <u>less</u> than the top of the culvert at the outlet, the hydraulic grade line is found by solving for housing Equation III-20:



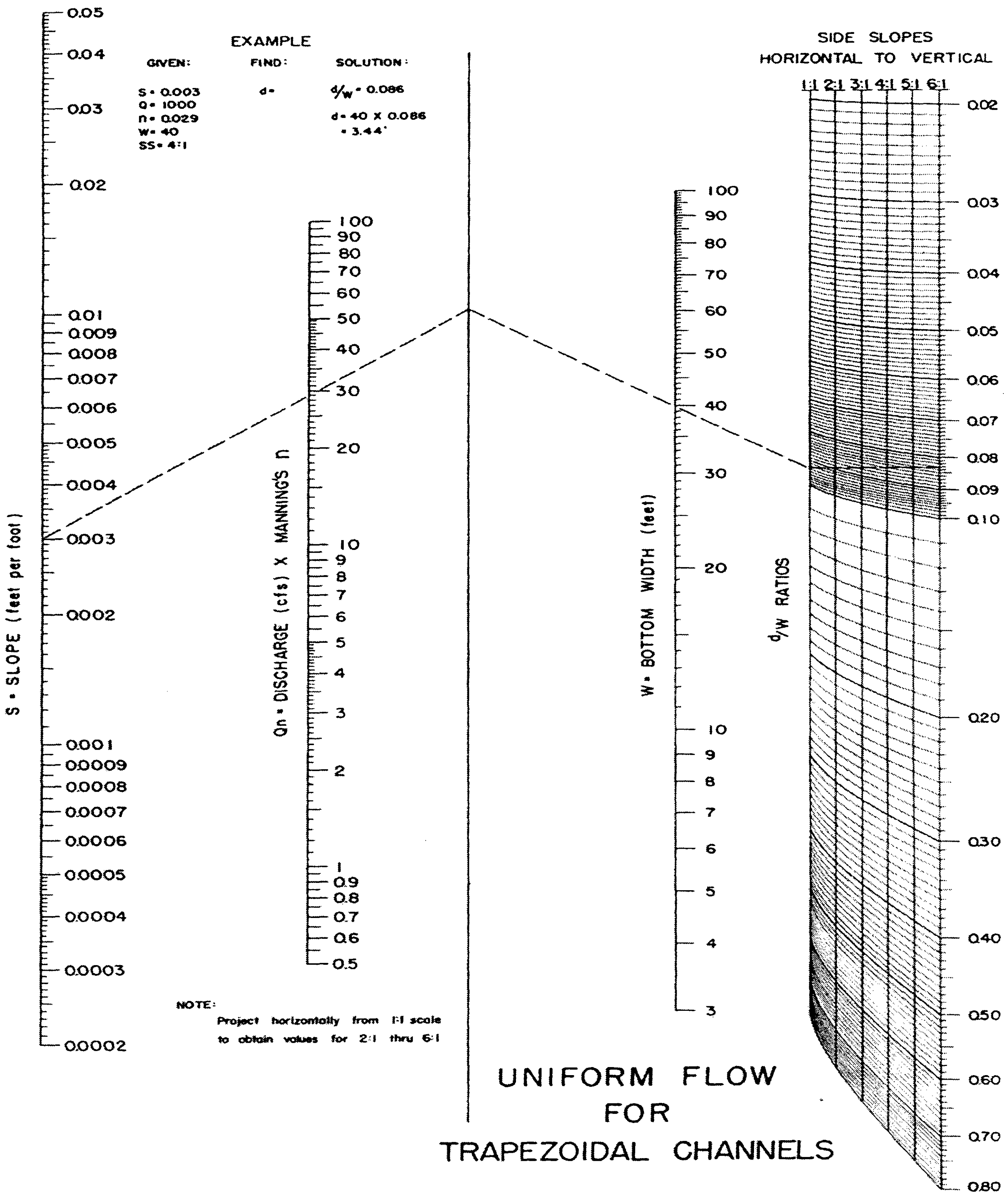
Source: Hydraulic Manual, Texas Highway Department

City of **RUSSELLVILLE** Arkansas	**UNIFORM FLOW FOR PIPE CULVERTS** **FIGURE III-37**



Source: Hydraulic Manual, Texas Highway Department

City of **RUSSELLVILLE** Arkansas	**UNIFORM FLOW FOR BOX CULVERTS** **FIGURE III-38**



Source: Hydraulic Manual, Texas Highway Department

City of **RUSSELLVILLE** Arkansas	**UNIFORM FLOW FOR TRAPEZOIDAL CHANNELS** **FIGURE III-39**

$$H_0 = (d_c + D)/2$$

Column 10: Enter the tailwater depth from initial data shown at the top of form. Refer to tailwater comments under Step 1 for additional guidelines.

Column 11: Enter the larger of Column 9 and Column 10.

Column 12: Enter the product of culvert length times the slope.

Column 13: Headwater elevation required for culvert to pass flow in outlet control (HW_o) is computed by Equation III-19:

$$HW_o = H + h_0 - LS$$

Additional trials may be required. Space for additional trials is provided on Culvert Design Worksheet (Figure III-36).

4.4.9.2.2 Inlet Control Calcuations for Convential and Beveled Edge Culverts

After the minimum barrel size has been determined under Section III-4.4.9.2, the next procedure is similar to that used in FHWA (1985). Nomograph solutions for inlet control conditions of various types and materials of storm sewer culverts are provided in most pipe manufacturing catalogs as well as design publications of the FHWA and the AHTD.

The computations involved in computing inlet headwater elevation are discussed in Section III-4.4.9.1.

Column 4: Enter ratio of headwater depth to height of structure from an appropriate inlet control nomograph.

Column 5: HW is derived by multiplying Column 4 by the height (or diameter) of culvert.

Column 16: Enter greater of two headwater depths (Column 5 or 13).

Column 17: If inlet control governs, outlet velocity equals Q/A, where A is defined by the cross-sectional area of normal depth of flow in the culvert barrel at "S". Graphic solutions for estimating normal depth of flow and velocity are provided in most pipe manufacturing catalogs as well as design publication of the FHWA and AHTD. Equation III-22 may also be used:

$$V = (1.486/n)\, R^{2/3}\, S^{1/2}$$

Column 17: If outlet control governs, outlet velocity equals Q/A, where A is the cross-sectional area of flow in the culvert barrel at the outlet.

Column 18: Record estimated cost if cost considerations are to be considered.

Column 19: "Comments" area to note the results of that particular trial.

4.4.9.2.3 Improved Inlets

Figures shown in this column should be self-explanatory.

See the AHTD's Drainage Design Manual (1991) for improved inlet or side tapered inlet design and broken back culvert design.

4.4.9.3 Culvert Design and Analysis Software

In addition to the analysis previously described, the City will allow the design and analysis of trapezoidal and circular channels, and pipe and box culvert analysis using the FHWA Culvert Analysis (HY8), CulvertMaster, or an acceptable equal method approved by the CITY ENGINEER.

4.4.9.3.1 FHWA Culvert Analysis (HY8)

FHWA Culvert Analysis (HY8) software was developed by the Pennsylvania State University in cooperation with the Bridge Division (HNG-31). The HY8 software is sponsored by the Rural Technical Assistance Program (RTAP) of the National Highway Institute under Project 18B administered by the Pennsylvania Department of Transportation.

The HY8 software automates the design methods described in HDS5, Hydraulic Design of Highway Culverts, FHWA-IP-85-15 (FHWA 1985). HDS5 is available from the Government Printing Office, Washington, DC 20402.

4.4.9.3.2 CulvertMaster

CulvertMaster, developed by Haestad Methods, Inc., is an easy-to-use, Windows-based, culvert simulation and design program. The program can analyze pressure or free surface flow conditions, and in subcritical, critical, and supercritical flow conditions, based on drawdown and backwater. A variety of common culvert shapes and section types are available. Tailwater effects are considered and the user can enter a constant tailwater elevation, a rating curve, or specify an outlet channel section. Culvert hydraulics are solved using FHWA methodology for inlet and outlet control computations. Roadway and weir overtopping are checked in the solution of the culvert.

CulvertMaster does have a hydrologic analysis component to determine peak flow using the Rational Method, SCS Graphical Peak Method, and quick TR-55. The user also has the option of entering a known peak flow rate. The user must enter all rainfall and runoff information (e.g., IDF data, rainfall depths, curve numbers, C coefficients, etc).

4.5 Storage

4.5.1 General

If hydrologic and hydraulic studies reveal that the proposed development would cause increased flood stages that would increase the flood damages to existing developments or property, or increase flood elevations beyond the vertical limits set for the floodplain districts, then the proposed land disturbing activity shall not be permitted unless one or more of the following mitigations measures are used: (1) Onsite detention/retention, (2) offsite or regional detention, or (3) improvements to the existing drainage system.

Stormwater runoff and the velocity of discharge are considerably increased through development and growth of the City. Prior to the development of land, surface conditions provide a high percentage of permeability and a longer time of concentration. With the construction of buildings, parking lots, etc., permeability and the time of concentration are significantly decreased. These modifications may create harmful effects on properties downstream.

Criteria for differential runoff and detention guidelines are set out in the following in an attempt to decrease the possible effects of development on downstream properties due to increased runoff. The criteria presented in this section shall be used in the design and evaluation of all storm runoff storage facilities for the City of Russellville. The review of all proposed development submittals will be based on the criteria presented in this section.

4.5.2 Policies and Standards

In the City of Russellville and its Planning Area, the preferable method of storage of runoff in excess of amounts to be expected from existing conditions is on-site detention. The main purpose of an on-site detention facility is to store the excess storm runoff associated with an increased watershed imperviousness and discharge this excess at a rate similar to the runoff rate from the watershed with existing land uses. The general requirements of such detention facilities are established in Ordinance #1675:

> *Where on-site detention is deemed inappropriate due to local topographical or other physical conditions, alternate methods for accommodating increases in stormwater runoff shall be permitted. Any alternate method will be approved in writing by the CITY ENGINEER prior to its consideration. The methods may include:*
>
> 1. *Off-site or Regional Detention.*
> 2. *In-lieu monetary contributions for drainage system improvements by the City. Channel improvements shall only be used if they are an integral part of a detailed watershed management plan. No in-lieu contributions are allowed when:*
>   a. *Existing flooding occurs downstream from the development, or*
>   b. *If the proposed development will cause downstream flooding.*

4.5.3 On-Site and Regional Detention

Depending upon ownership of the site and the area tributary to a detention site, two types of detention are defined: (1) on-site, and (2) off-site or regional. On-site detention is defined as the privately owned and generally privately maintained facility which serves the developing area in question. Regional detention, also generally referred to as off-site detention, is publicly owned and maintained and generally is part of a planned open space park system or greenbelt area serving a larger portion of the basin. The importance of regional detention is the ability to assure that the facility will be maintained and will function as designed.

The ability of on-site or regional detention to reduce flood peaks in the local drainageways has been recognized by the City of Russellville and detention is therefore required in the watershed, unless the proposed system will connect to an existing system with 100-year flood capacity.

4.5.3.1 General Requirements

The following are some of the requirements of detention storage:

1. Detention designed and built for city agencies may be either wet or dry depending upon multiple-use considerations. On-site detention designed, owned, and maintained by the private sector shall also meet the performance standards of these DESIGN CRITERIA and the SMDO. Wet detention shall have adequate flow through to maintain water levels. Mosquito control shall be incorporated into the maintenance plan.
2. An owner may contribute to the cost of a regional detention site(s) or improvements to downstream conveyances in lieu of constructing on-site detention. However, the basin master plan must include downstream storage identified for "in lieu of" payments in place of on-site detention, or the Developer must adequately demonstrate that "in lieu of" downstream storage will mitigate the increased runoff from the development. In addition, there cannot be any direct identifiable adverse impacts on downstream properties. The "in-lieu fee" contribution shall be based upon an amount of $15,000 per acre-foot of stormwater storage. The acceptance of an "in lieu fee" is totally at the discretion of the CITY ENGINEER.
3. Subject to requirements for this DESIGN CRITERIA and the SMDO, downstream conveyance may be improved to compensate for increased flows if the improvements comply with Ordinance #1675.
4. Filling or development which diminishes the flood storage capacity of a regulatory floodplain area or in any way, increases the potential of flooding to other property owners, shall be compensated for as specified by these DESIGN CRITERIA by providing compensatory storage.

4.5.3.2 Detention Objectives and Functions

The following are four design objectives for detention facility designs:

4.5.3.2.1 Hydraulic Function

The primary function of detention storage is to reduce stormwater runoff from new development to the rate of runoff prior to development. This includes inflow-outflow hydrographs, return frequencies, storage volumes, depths required, and outlet features necessary to achieve the design performance objectives.

The storage volume of detention storage shall be that volume necessary to control the peak stormwater drainage release rate from a development so as not to exceed the pre-developed runoff that occurred before development for all frequencies of runoff up to and including the 100-year runoff.

To determine peak release rates, a minimum of 25-year and 100-year runoff frequencies under pre-developed site conditions shall be investigated to insure reasonable considerations of all frequencies.

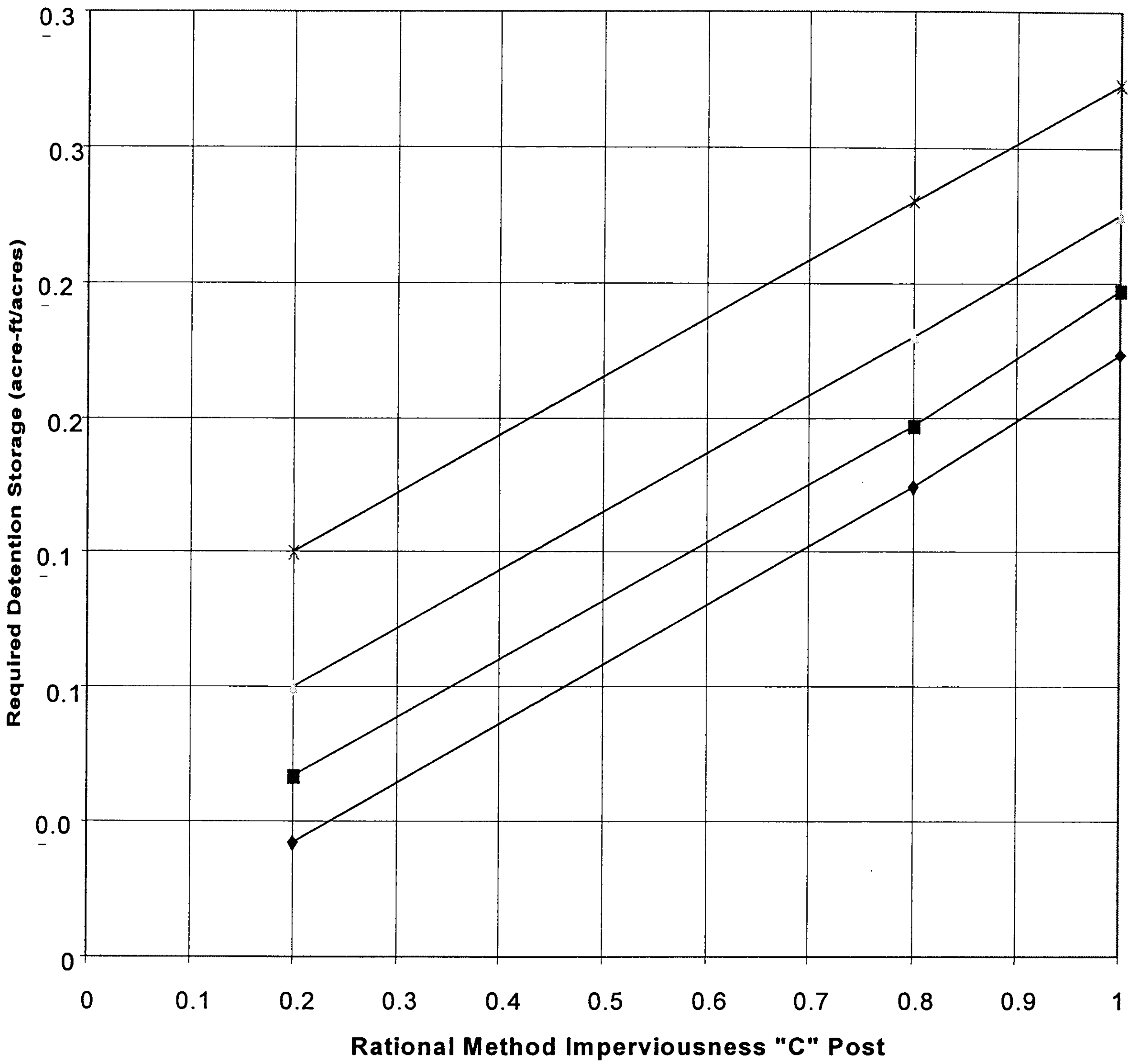
Volumes of detention shall be evaluated according to the following methods:

A. Volume of detention for basins with total drainage areas of less than 2.5 acres may be evaluated by the "Unit Volume Relationship" presented on Figure III-40 or the Modified Rational Method.

B. Volume of detention for basins with total drainage areas less than 100 acres may be evaluated by the Modified Rational Method.

C. For basins with total drainage areas larger than 100 acres, the OWNER'S Engineer shall submit the proposed method of evaluation for the sizing of the detention or retention basin to the CITY ENGINEER as a part of the Development Classification Form. The method will be evaluated for professional acceptance, applicability, and reliability by the CITY ENGINEER. No detailed review for projects larger than 100 acres will be rendered before the method of evaluation of the detention or retention basin is approved. Typical approved computer hydrologic analysis methods for designing detention basins for drainage areas greater than 100 acres in size include PondPack and HEC-1.

The SNYDER procedure, or the SCS procedure (see DIVISION II) shall be used for the design of all detention facilities for basins having tributary areas of 100 acres or more (See table below). The Modified Rational Method may be used for design of detention storage facilities having tributary areas of up to 100 acres. An approved hydrograph method may be selected for design of the detention basins having smaller tributary watersheds. For parcels of land up to 2 acres, a simple curve relationship (Figure III-40) is satisfactory for sizing detention storage volume. The



NOTE: **Use for watersheds of 2.5 acres or less.**

City of **RUSSELLVILLE** Arkansas	**DETENTION VOLUME CURVES** **FIGURE III-40**

CITY ENGINEER may require a more detailed analysis where the more simplified methods are judged inadequate.

The following table defines the acceptable methods for detention basin evaluation:

Acceptable Methods For Detention Basin Evaluation

Acceptable Method	Tributary Watershed Area (Acres)		
	<2.5 Acres	2.5 to 100 Acres	>100 Acres
Unit Volume Relationship (Figure III-40)	YES	NO	No
Modified Rational Method	YES	YES	No
Snyder or SCS Hydrograph	NO	YES	YES

All calculations for detention facilities shall be presented to the CITY ENGINEER for review and approval. Information shall include mass curves, hydrographs, outflow structures analysis, and a storage routing analysis through the facility.

The computed hydraulic detention volume, described by a method presented in this section, will be increased by 25% as a safety factor and to provide for sediment storage of undeveloped and channel erosion of upstream tributary areas. The CITY ENGINEER may reduce this requirement depending on the development characteristics and stream stability of upstream tributary areas.

4.5.3.2.2 Safety

Safety of the detention pond and outlet works must be addressed in the design, which includes embankment stability and the consequences of embankment failure.

4.5.3.2.3 Maintenance

A detention pond and its outlet works should be relatively maintenance free. Maintenance is facilitated by good equipment access for inspection, cleaning, repair, and reconstruction. Maintenance considerations must be part of the design process.

4.5.3.2.4 Multiple Use

A visual amenity consistent with the neighborhood and coupled with multi-use potentials related to recreation can create benefits beyond those related specifically to the hydraulic function. Detention facilities in Russellville must be environmentally sound and compatible with the neighborhood and, where feasible, multi-use should be included.

4.5.3.3 Storage Facility Location

The following conditions and limitations shall be observed in selection and use of the method of detention.

4.5.3.3.1 General Location

Detention facilities shall be located within the parcel limits of the project under consideration. No detention or ponding will be permitted within public road rights-of-way. Location of detention facilities immediately upstream or downstream of the project will be considered by special request if proper documentation is submitted with reference to practicality, feasibility, and proof of ownership or right-of-use of the area proposed. Conditions for general location of detention facilities are identified in the following sections.

4.5.3.3.2 Dry Reservoirs

Wet weather ponds or dry reservoirs shall be designed with proper safety, stability, and ease of maintenance facilities. Maximum side slopes for grass reservoirs shall not exceed 1-foot vertical for 3-foot horizontal (3:1) unless adequate measures are included to provide for the above-noted features. In no case shall the limits of maximum ponding elevation be closer than 20 feet horizontally from any building and less than 1-foot vertically below the lowest sill or floor elevation. The entire reservoir area shall be sodded as required prior to final plat approval or issuance of certificate of occupancy unless bond is posted for completion of said work. Any area susceptible to, or designed as, overflow by higher design intensity rainfall shall be sodded or paved depending upon the outflow velocity.

4.5.3.3.3 Permanent Lakes

Permanent lakes with fluctuating volume controls may be used as detention areas provided that the limits of maximum ponding elevations are no closer than 100-foot horizontal from any building and greater than 2 feet below the lowest sill or floor elevation of any building.

Maximum side slopes for the fluctuating area of permanent lakes shall be 1-foot vertical to 3-feet horizontal (3:1) unless provisions are included for safety, stability, and ease of maintenance.

Special consideration is suggested regarding safety and accessibility of small children in design of permanent lakes in residential areas. Allowances for silting during construction for a period of no less than 1 year is also recommended.

The entire fluctuating area of the permanent reservoir shall be sodded. Also, calculations must be provided to ensure adequate "live storage" is provided for the 1% annual chance storm event. Any area susceptible to or designed as overflow by higher design intensity rainfall (100-year frequency) shall be sodded or paved, depending on the design velocities. An analysis shall be furnished of any proposed earthen dam construction soil. A boring of the foundation for the earthen dam may be requested by the CITY ENGINEER. Earthen dam structures shall be designed by a Professional Engineer and shall comply with the criteria of the Arkansas Soil and Water Conservation Commission regarding the design and operation of dams (ASWCC 1993).

4.5.3.3.4 Parking Lots

Detention is permitted in parking lots to maximum depths of 6 inches. In no case should the maximum limits of ponding be designed closer than 10 feet from a building unless waterproofing of the building and pedestrian accessibility are properly documented and approved.

The minimum freeboard and the maximum ponding elevation to the lowest sill or floor elevation shall be 2 feet.

4.5.3.3.5 Other Methods

Other methods of detention such as seepage pits, French drains, etc., are discouraged. If other methods are proposed, proper documentation of soil data, percolation, geological features, etc., will be needed for review and consideration.

4.5.3.3.6 Verification of Adequacy

Projects shall provide documented verification of adequacy according to the scope and complexity of design signed originally and certified as-built by the same Arkansas registered Professional Engineer, if feasible.

4.5.3.3.7 Outlet Works

Detention facilities shall be provided with effective outlet works. Safety considerations shall be an integral part of the design of all outlet works. Plan view and sections of the structure with adequate details shall be included in the Plans.

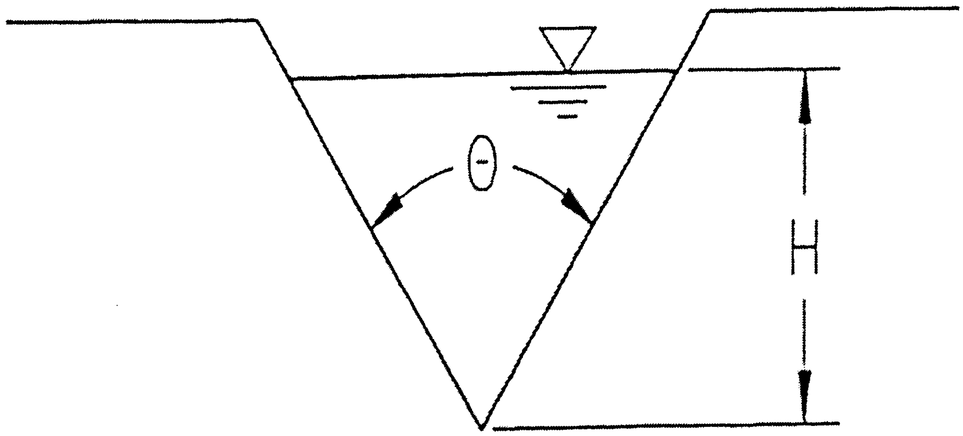
The structure selected shall have documented evidence that it will control the 10-, 25-, 50-, and 100-year floods. Generally, the full range of frequency control is achieved by selecting the 100-year flood and an intermediate frequency, such as the 25-year flood.

The overflow opening or spillway shall be designed to accept the total peak runoff of the improved tributary area or acceptable percentages of blockage (50%) shall be incorporated into the design of the outlet structure.

Figures III-41 through III-43 illustrate design equations for office, V-notch weir, rectangular weir, and multi-stage release outlet structure.

4.5.3.3.8 Discharge Systems

For site-specific runoff, the effectiveness of local detention structures can be acknowledged in the design of any on-site downstream drainage facilities, assuming that the detention facilities comply with all criteria and that they are properly constructed and maintained.



V-Notch Weir Flow Equation

$$Q = C \tan(\Theta/2)H^{5/2}$$

where

Q = Weir discharge in CFS

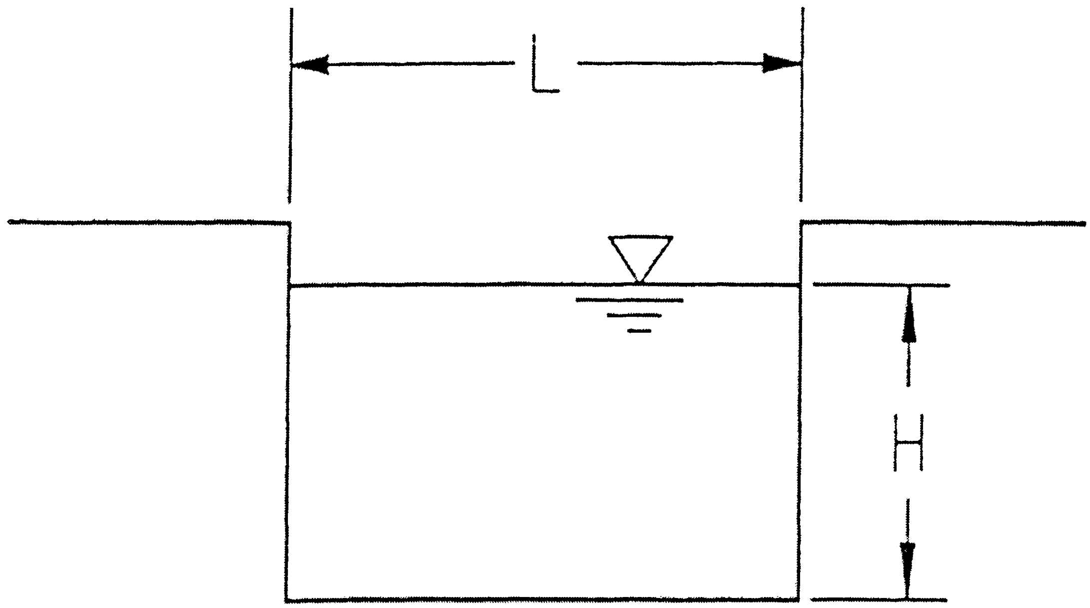
C = Weir coefficient

Θ = Angle of the weir notch in degrees

H = Head on the weir in feet

Source: City of Fayetteville, Drainage Criteria Manual

City of **RUSSELLVILLE** Arkansas	**V-NOTCH WEIR RELEASE** **FIGURE III-41**



Rectangular Weir Flow Equation

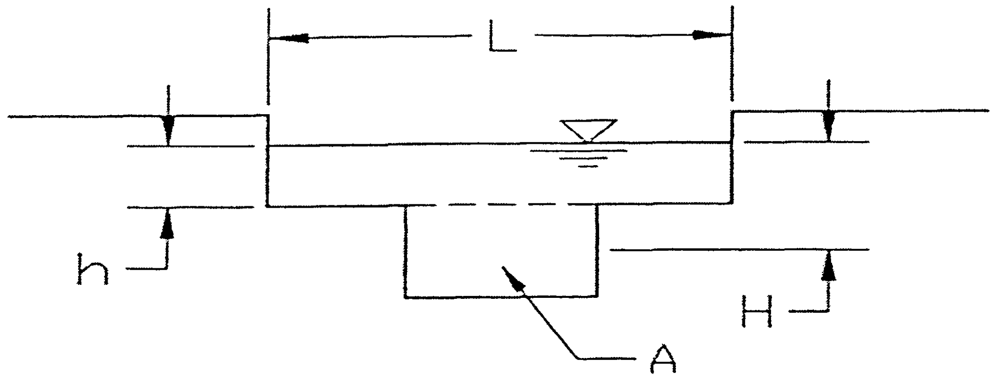
$$Q = CLH^{3/2}$$

where

Q = Weir discharge in CFS
C = Weir coefficient
L = Horizontal length of the weir on feet
H = Head on the weir in feet

Source: City of Fayetteville, Drainage Criteria Manual

City of **RUSSELLVILLE** Arkansas	**RECTANGULAR WEIR RELEASE** **FIGURE III-42**

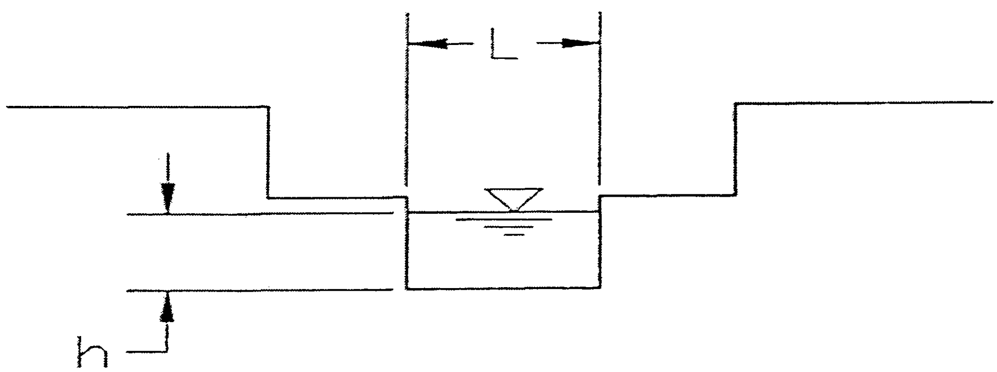


Rectangular Weir and Orifice Flow

$$Q_v = C_v L h^{3/2}$$
$$Q_o = C_o A (2gH)^{1/2}$$

where

Q_v = Weir discharge in CFS
Q_o = Orifice discharge in CFS
C_v = Weir coefficient
C_o = Orifice Coefficient
L = Horizontal length of the weir on feet
h = Head on the weir in feet
H = Head on the orifice in feet



Rectangular Weir Flow Only

$$Q_v = C_v L h^{3/2}$$

where

Q_v = Weir discharge in CFS
C_v = Weir coefficient
L = Horizontal length of the weir on feet
h = Head on the weir in feet

Source: City of Fayetteville, Drainage Criteria Manual

City of **RUSSELLVILLE** Arkansas	**MULTISTAGE RELEASE STRUCTURE** **FIGURE III-43**

In the case of regional detention basins, sizing of the system below the control structure shall be for the total improved peak runoff tributary to the structure, with no allowance for detention unless approved in writing by the CITY ENGINEER.

In the event the Engineer desires to incorporate the flow reduction benefits of existing upstream detention ponds, the following field investigations and hydrologic analysis will be required. (Please note that under no circumstances will the previously approved construction plans of the upstream pond suffice as an adequate analysis. While the responsibility of the individual site or subdivision plans rests with the Engineer of Record, any subsequent engineering analysis must assure that all the incorporated ponds work collectively.)

1. A field survey of the existing physical characteristics of both the outlet structure and ponding volume. Any departure from the original engineer's design must be accounted for. If a dual use for the detention pond exists (e.g., storage of equipment), then this too should be accounted for.

2. A comprehensive hydrologic analysis that simulates the attenuation of the contributing area ponds. This should not be limited to a linear additive analysis, but rather should consist of a network of hydrographs that considers incremental timing of discharge and potential coincidence of outlet peaks.

4.5.3.3.9 Ownership of Stormwater Detention Ponds

Ownership of stormwater detention ponds in residential subdivisions accepted by the City shall be vested in the City of Russellville within 30 days after filing the final plat. The Developer must warrant the operation of the drainage system for a 1-year period after the acceptance by the City by an acceptable Maintenance Bond or equal provided by the Developer's Contractor or the Developer. The bond shall be required to be extended until 1 year after all phases of the subdivision that substantially drain into the basin are completed.

Ownership of stormwater detention ponds in commercial, industrial, and non-residential areas not accepted by the City of Russellville shall be vested in the property owner.

4.5.3.3.10 Easements

No alternation of the drainage system will be allowed without the approval of the CITY ENGINEER. If construction of the basin is not complete, a cash bond or Letter of Credit from an acceptable financial institution shall be posted in addition to the Performance/Payment Bond.

Easements shall be provided in the Plans for detention facilities if the basin is not to be deeded to the City of Russellville.

A minimum 10-foot wide drainage easement shall be provided within the reservoir area, connecting the tributary pipes and the discharge system along the most passable routing of piping system.

4.5.3.3.11 Maintenance

Detention facilities, when required, are to be built in conjunction with storm sewer installation and/or grading. Since these facilities are intended to control increased runoff, they must be partially or fully operational soon after the clearing of the vegetation. Silt and debris connected with early construction shall be removed periodically from the detention area and control structure to maintain the facility's storage capacity.

Maintenance of detention facilities is divided into two components. The first is long-term maintenance that involves removal of sediment from the basin and outlet control structure. Maintenance to an outlet structure is minimum due to the initial design of permanent concrete or pipe structures. Studies indicate that in developing areas, basin cleaning by front-end loader or grader is estimated to be needed once every 5 to 10 years. In residential subdivisions where the detention basin has been accepted by the City, the City is responsible for long-term maintenance. The residential developer and all non-residential property OWNERS are responsible for long-term maintenance in basins not accepted by the City.

Short-term maintenance or annual maintenance is the second component and is the responsibility of the Developer or association for 1 year after acceptance of the final plat or filing of the last subdivision phase that substantially adds stormwater to a detention basin. The items considered short-term maintenance are as follows:

1. Major dirt and mud removal
2. Outlet cleaning
3. Mowing
4. Herbicide spraying (in strict conformance with the City's policies and procedures)
5. Litter control

The responsibility of maintenance of the detention facilities and single-lot development projects shall remain with the general contractor until final inspection of the development is performed and approved, and a legal occupancy permit is issued. After legal occupancy of the project, the maintenance of detention facilities shall be vested with the owner of the detention pond.

4.5.4 Detention Basin Design

1. Compute existing (pre-development) and proposed (developed) site characteristics:

   a. Drainage Area.

   b. Runoff/Composite Runoff Coefficient (Table III-1/Figure III-1). For pre-development unimproved areas, use a runoff coefficient for "Lawns, Sandy Soils".

   c. Time of Concentration (use Figures III-3 and/or III-4).

2. Determine rainfall intensity for existing conditions (2- through 100-year storm) from the City of Russellville Rainfall Intensity Table (Table III-2) or the Rainfall Intensity-Duration-Frequency Curves (Figure III-2).

3. Compute existing peak runoff rates using Rational Formula, Q = CIA. These will also be the maximum allowable release rates from the detention basin.

4. Determine inflow hydrograph using Modified Rational Method (see Section III-4.5.4.1).

5. Find estimated detention volume using Modified Rational Method.

6. Size detention basin based on estimated required volume. Develop stage-storage curve for the detention basin.

7. Size release structure based on allowable release flow. Develop stage-discharge curve for the release structure.

8. Route the inflow hydrographs (developed using Modified Rational Method for the 2- through 100-year storms) through the detention basin using the Modified Puls Method of PondPack, HEC-1, or other method approved by the CITY ENGINEER. See the following sections for a discussion of pre-approved software for detention analysis.

9. Check routed hydrographs to ensure flows do not exceed predevelopment peaks. Adjust detention basin and release structure, if necessary.

4.5.4.1 Modified Rational Method

The following example describes the general procedure to complete a design of a detention basin using the Modified Rational Method. The values and information provided in this example are not real but are only provided to illustrate the procedure.

Example:

Given: A 10-acre site currently agricultural use is to be developed for townhouses. The entire area is the drainage area of the proposed detention basin.

Determine: Maximum release rate and required detention storage.

Solution:

Step 1: Determine 100-year peak runoff rate prior to site development. This is the maximum release rate from site after development.

NOTE: Where a basin is being designed to provide detention for both its drainage area and a bypass area, the maximum release rate is equal to the peak runoff rate prior to site development for the total of the areas minus the peak runoff rate after development for the bypass area. This rate for the bypass area will vary with the duration being considered.

Present Conditions $Q = CIA$

$C = 0.30$
$T_c = 20$ minutes
$I_{100} = 7.0$ in/hr
$Q_{100} = 0.30\ (7.0)\ 10 = 21.0$ cfs (Maximum release rate)

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

NOTE: Incrementally increase durations by 10 minutes to determine maximum required volume. The duration with a peak inflow less than the maximum release rate or where required storage is less than storage for the prior duration is the last increment.

Future Conditions (Townhouses)

$C = 0.80$
$T_c = 15$ minutes
$I_{100} = 7.0$ in/hr
$Q_{100} = 0.80\ (7.7)\ 10 = 61.6$ cfs

Check various duration storms.

20 min	I = 7.0	Q = 0.80	*	(7.0)	*	10 = 56.0 cfs
30 min	I = 5.8	Q = 0.80	*	(5.8)	*	10 = 46.4 cfs
40 min	I = 5.0	Q = 0.80	*	(5.0)	*	10 = 40.0 cfs
50 min	I = 4.4	Q = 0.80	*	(4.4)	*	10 = 35.2 cfs
60 min	I = 4.0	Q = 0.80	*	(4.0)	*	10 = 32.0 cfs
70 min	I = 3.7	Q = 0.80	*	(3.7)	*	10 = 29.6 cfs
80 min	I = 3.4	Q = 0.80	*	(3.4)	*	10 = 27.2 cfs
90 min	I = 3.1	Q = 0.80	*	(3.1)	*	10 = 24.8 cfs

NOTE: Rainfall intensities are for illustrative purposes only and do not represent actual values for the City of Russellville.

The Maximum Storage Volume in cubic feet (cf) is determined by deducting the volume of runoff released during the time of inflow from the total outflow for each storm duration.

$$\mathbf{V = time \times Q_{in} \times 60\ sec/min - 0.5 \times Q_{out} \times (Time + T_c) \times 60\ sec/min}$$

15 min Storm Inflow 15 (61.6) 60 sec/min = 55,440 cf
Outflow (0.5) 30 (21.0) 60 sec/min = 18,900 cf
Storage 36,540 cf

20 min Storm Inflow 20 (56.0) 60 sec/min = 67,200 cf
Outflow (0.5) 35 (21.0) 60 sec/min = 22,050 cf
Storage 45,150 cf

30 min Storm Inflow 30 (46.4) 60 sec/min = 83,520 cf
Outflow (0.5) 45 (21.0) 60 sec/min = 28,350 cf
Storage 55,170 cf

40 min Storm Inflow 40 (40.0) 60 sec/min = 96,000 cf
Outflow (0.5) 55 (21.0) 60 sec/min = 34,650 cf
Storage 61,350 cf

50 min Storm Inflow 50 (35.2) 60 sec/min = 105,600 cf
Outlfow (0.5) 65 (21.0) 60 sec/min = 40,950 cf
Storage 64,650 cf

60 min Storm Inflow 60 (32.0) 60 sec/min = 115,200 cf
Outflow (0.5) 75 (21.0) 60 sec/min = 47,250 cf
Storage 67,950 cf

70 min Storm Inflow 70 (29.6) 60 sec/min = 124,320 cf
Outflow (0.5) 85 (21.0) 60 sec/min = 53,550 cf
Storage 70,770 cf

80 min Storm Inflow 80 (27.2) 60 sec/min = 130,560 cf
Outflow (0.5) 95 (21.0) 60 sec/min = 59,850 cf
Storage 70,710 cf

90 min Storm Inflow 90 (24.8) 60 sec/min = 133,920 cf
Outflow (0.5) 105 (21.0) 60 sec/min = 66,150 cf
Storage 67,770 cf

Step 3: Route design storm hydrograph through the detention basin using the Modified Puls Routing Method or another approved method, based on final detention basin and release structure design. Computer programs to accomplish this operation are readily available. The methods approved by the City include Pond-2 and HEC-1. Other programs using the Modified Puls Method may be used if approved by the CITY ENGINEER.

4.5.4.2 Stormwater Detention Analysis Software

The City will allow the use of the following software or an acceptable equal approved by the City for the analysis of stormwater detention facilities.

4.5.4.2.1 Pond-2

Pond-2 is a program for detention pond design. Pond-2 is included in PondPack, a package developed by Haestad Methods, where it is integrated with Quick TR-55. It estimates detention storage requirements, computes a volume rating table for any pond configuration, routes hydrographs for different return frequencies through alternative ponds, and plots the resulting inflow and outflow hydrographs.

Pond-2 automatically computes outflow rating curves for single or multi-stage outlet structures, and computes the controlling flow rate for outlets operating in series. Pond-2 handles orifices, weirs, box culverts, circular culverts, and more.

4.5.4.2.2 HEC-1

HEC-1 generates hydrographs from rainfall or snow melt, adds or diverts them, then routes them through reaches and reservoirs. HEC-1 models multiple stream and reservoir networks, and has dam failure simulation capabilities. It handles level-pool routing for reservoirs and detention ponds, and routes through stream reaches using Kinematic Wave, Muskingum, Muskingum-Cunge, Modified Puls, and other methods. HEC-1 supports five methods for computing infiltration and abstraction losses, and computes unit hydrographs using the Clark methods, Snyder method, and SCS dimensionless hydrographs.

4.5.4.3 Hydraulic Design Data

Stormwater detention pond outlets shall be designed to limit the peak stormwater discharge rate of the 2-, 10-, 25-, 50-, and 100-year storm frequencies after development to pre-development rates. The principal outlet will be designed to safely convey the runoff resulting from a 4% annual chance storm event. A second outlet, the emergency outlet, will be designed to safely convey the runoff resulting from a 1% annual chance storm event.

Methods for designing two-stage risers are often based on hydraulic equations for weir and orifice flow. In two-stage riser design, the lower-stage orifice controls the more frequent event while the larger, less frequent event, is controlled jointly by both the high-stage weir and the low-stage orifice. The outlet conduit should be sufficiently large to carry the high-stage peak with the water surface in the riser being below the crest of the high-stage weir. Optimally, physical separation of multi-stage outlets will be sufficient to prevent hydraulic interference at all foreseeable operating stages.

4.5.4.4 Outlet Works Features

Figure III-44 presents a schematic diagram of the relationship of the outlet works to the detention storage facility. Integration of hydraulic control elements can be achieved by using characteristics of types of outlets as shown on Figure III-45.

4.5.4.4.1 On-Site Detention

Special attention must be given to the design of outlet structures for controlling runoff from rooftops, parking lots, and small on-site swales. Because runoff volumes from such areas are small, the required outlets are also small, which increases the potential for plugging by debris. Also, because of the multi-purpose nature of these small on-site control facilities, the outlet must release temporarily stored water in a reasonable amount of time. As an example, parking lots should drain relatively fast and not be a nuisance.

Since the detained water may occasionally cause an inconvenience to those using the land for its intended purposes, the temptation may exist to modify or eliminate the on-site detention by changing the outlet. Design of parking lot grading to minimize people/water conflicts and to recognize pedestrian movement needs and patterns can reduce unauthorized facility modifications.

Proper location and protection of outlets can reduce these problems. Manufactured outlets that are more difficult to alter can be used to protect roof drains. Manufactured grates located on the surface discourage tampering of parking lot and swale outlets placed above ground. These grates also keep the outlets clear of debris and preserve their hydraulic capability even when the grates are partially plugged. Grated inlets are also debris catchers and may be plugged when they are most needed; accordingly, the maintenance of grates should be planned where grate usage is unavoidable. The outlet facilities should be accessible and located where their maintenance needs are easily noticed, so that the public will be involved in the maintenance process. Regular maintenance of parking lots and other normally dry public detention areas should assure debris removal, especially removal of paper, plastics, and gravel or rocks.

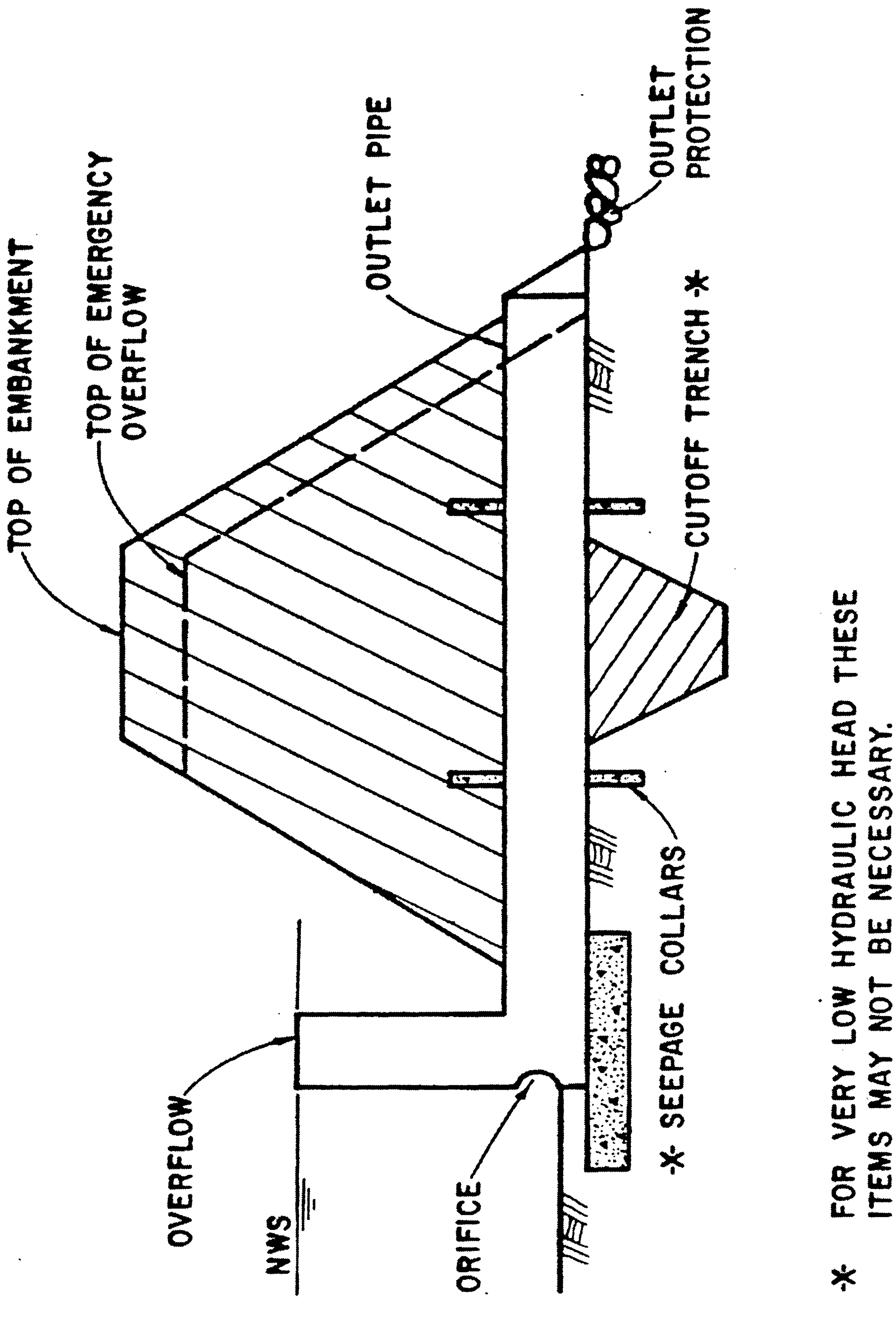
4.5.4.4.2 Retention Basins

In certain instances, such as when the existing development conditions runoff from a watershed would exceed the capacity of the existing downstream facilities, retention basins (i.e., no outlet or with a release at the capacity of the downstream facilities) for the storm runoff may be required by the CITY ENGINEER.

4.5.4.5 Safety Function

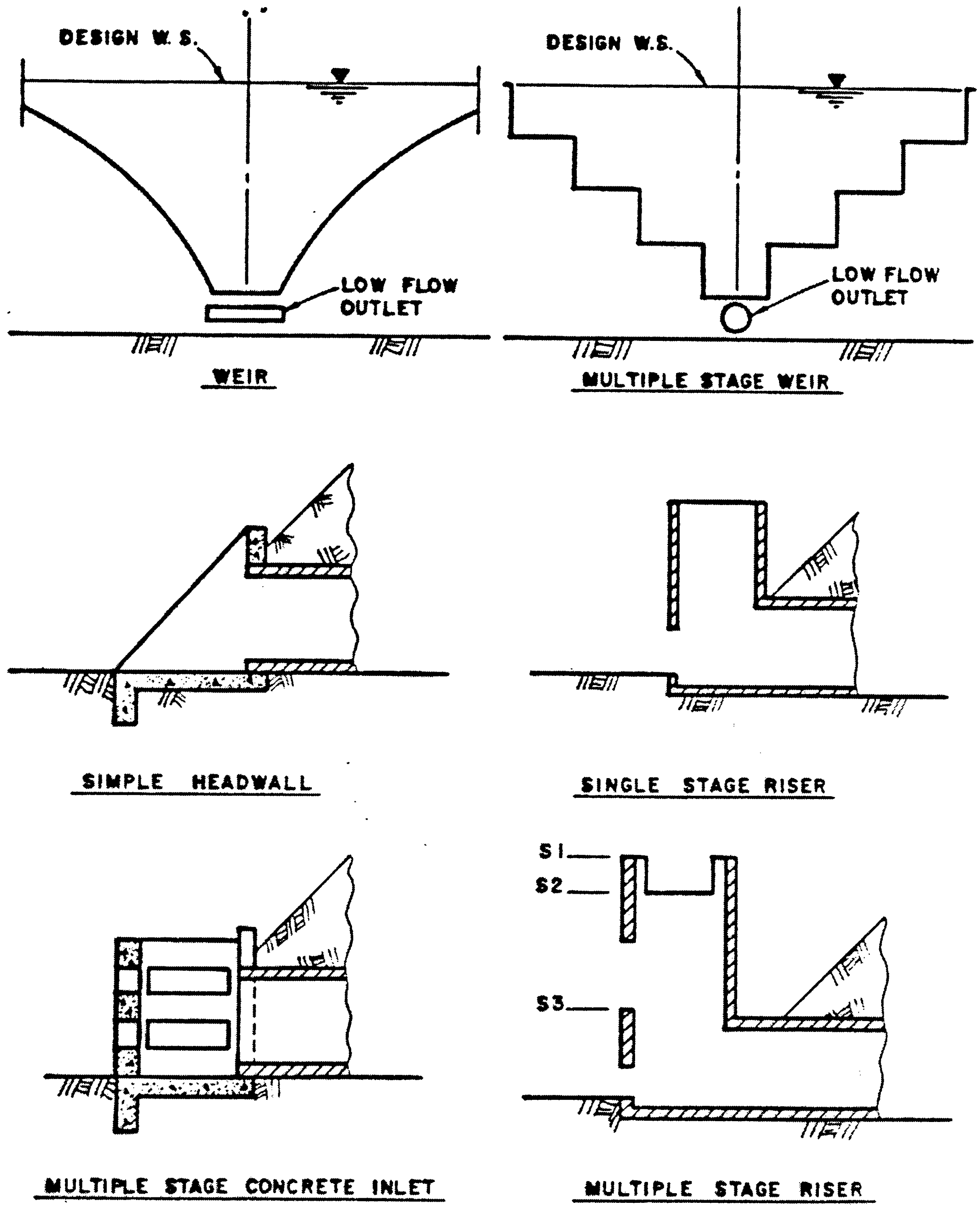
4.5.4.5.1 Protection of the Structure

The stormwater detention dams within the City of Russellville will most likely be classified as small structures by the ASWCC. All emergency spillways and dams features shall be designed, at a minimum, in full compliance with the ASWCC Rules Governing the Design and Operation of Dams (1993).



Source: City of Tulsa, Storm Management Criteria Manual

City of **RUSSELLVILLE** Arkansas	**SCHLEMATIC OF OUTLET WORKS** **FIGURE III-44**



Source: City of Tulsa, Stormwater Management Criteria Manual

City of **RUSSELLVILLE** Arkansas	**EXAMPLES OF OUTLET STRUCTURES** **FIGURE III-45**

If an emergency spillway is sized for less than a probable maximum flood, the embankment of the pond, if any, will be subject to potential overtopping. Consequences of unplanned embankment overtopping range from erosion of the downstream face of the dam to total failure of the embankment with release of the water in storage. If the failure is rapid, it may cause a downstream flood greater than would result if the embankment had not been built.

In some instances, the smaller size and economics of a detention pond virtually preclude a probable maximum flood spillway. In those cases, it may be possible to mitigate the adverse effects of overtopping of the embankment such as by:

1. Flattening of the downstream embankment face.
2. Armoring the dam crest and downstream embankment face.
3. Using regulated floodplain delineation and occupancy restrictions downstream that are representative of conditions without the detention storage.
4. Use of a wide embankment crest, such as is common with urban roads and streets where rapid failure seldom occurs due to modest overtopping depths.
5. Use of non-eroding dam material, such as rolled soil cement or concrete.
6. Design and construction of off-stream detention facilities.
7. Use of a totally excavated pond.
8. Use of an erosion-resistant core inside the embankment.
9. Constructing the detention within a small tributary watershed where the rate and discharges involved are limited and result in short duration overtopping flows of modest non-hazardous proportions.

A substantial percentage of embankment failures are due to inadequate outlet works design and construction, classified as "structural failures". Consequently, the Design Engineer is reminded to direct special attention to the following:

1. Avoid potential piping of water along the outside of the outlet conduits by using cutoff collars, careful material selection, and good compaction around the conduit.
2. Minimize the number of conduits through the embankment.
3. Minimize the potential for leaky joints within the embankment.
4. Do not use thin walled conduit through the embankment without a protective exterior encasement.
5. Where reasonable, design the pipe to operate under little or no internal water pressure.

6. Provide a safety factor in outlet work openings to account for debris collection and design to minimize debris migration to the outlet.

7. Do not depend upon human intervention to operate gates or other controls during a storm runoff event.

4.5.4.5.2 Protection of the Public

Outlet safety considerations include safety to the public at the facility. The outlet works create a potential hazard when in operation due to the possibility of a person being carried into the opening. Grating or trash racks are often used; however, a person can be forced against the grate or trash rack with substantial pressure, preventing escape.

Low entrance velocities at the trash rack are recommended. Fencing or other effective measures should be provided to exclude people from potentially hazardous areas. Alternative measures include education, site grading, signing, planting of thorny shrubs, and grading for "safety ledges" along the pond perimeter.

Outlet works can be designed to reduce the hazard to the public where heavy recreational use is anticipated. For instance, a vertical riser of concrete, timber, or steel can have a series of openings of 12 inches or less from top to bottom with sufficient total area to cause low velocity at the entrances, if compatible with hydraulic requirements. The top of such risers can be grated, or even closed. In some instances the outlet works can be fenced. Appropriate signing is sometimes used to warn the public of the safety hazards involved at the outlet works.

4.5.4.5.3 Liability

Litigation following dam failures often involves a rule of common law that is referred to as "doctrine of strict liability". This rule generally assigns all or most liability for the consequences of a dam failure to the OWNER.

4.5.4.6 Maintenance Function

Maintenance of the detention facility is an integral part of a program for the entire drainage system. The responsibility for maintenance rests with the OWNER of the facility (except where the ENGINEERING DEPARTMENT has agreed to maintain a facility), whether it is a rooftop, parking lot, or surface storage facility. The ENGINEERING DEPARTMENT has the authority to inspect or review private maintenance actions to ensure that private maintenance is being provided. Funding for maintenance, whether public or private, must be assured.

Maintainability of detention ponds should receive particular attention during design. Recognizing the life cycle costs of these facilities, long-term maintenance costs are inevitable and can be minimized only by sensitive consideration and treatment during the design of a detention facility.

4.5.4.6.1 Outlet Works

To reduce maintenance, outlet structures should be designed with no moving parts (i.e., pipes, box culverts, orifices, and weirs). Manually and/or electrically operated gates should be avoided, where possible.

Outlets should be designed with large openings, compatible with the depth-outflow curve desired and with water quality, safety, and aesthetic objectives. Outlets should also be designed to lessen the chances of damage from debris or vandalism.

Outlets that are protected by a trash rack may accumulate trash during and between storm events. To facilitate outlet operation and maintenance, trash racks should be curved or inclined so that debris tends to ride up as the water level rises. Such a design leaves the rack clear and allows for easier cleaning during a storm event.

The discharge end of the outlet structure also requires maintenance. The high velocity outflow can be controlled by riprap on the back slope and downstream channel to reduce erosion potential. Deep toe walls to resist scour (undercutting) should also be provided.

All portions of the outlet structure must be accessible to vehicles, equipment, and personnel both between and during storm events. This includes the floor of the basin as well as ramps to points above the upstream and downstream sides of the outlet structure.

4.5.4.6.2 Embankment

Where maintenance of a grassed side slope by equipment is required, the side slope shall not be steeper than 4:1. Grass shall be used to protect slopes against erosion, however, special cover and treatment of steep slopes may be approved on a case-by-case basis.

4.5.4.6.3 Reservoir Area

Access and maintenance roads for publicly and privately maintained detention facilities shall be provided to remove sediment and debris and for general repair of the facilities. The access shall include a right-of-way at least 20-feet wide, and an all weather maintenance access road for vehicular traffic. The maximum slope of the road is 10%. Road alignment should permit movement of suitable vehicles and equipment to serve the facilities accessed. Where the roadway is subject to erosional tendencies of moving water, adequate protection against scour is required to protect against the 4% annual chance flood event.

Maintenance and access to privately owned and maintained detention areas shall meet performance standards with consideration given to the unique site opportunities and limitations.

Storage reservoirs with a permanent water surface shall be provided with a low level outlet to drain the reservoir for sediment removal and other maintenance requirements. To facilitate sediment removal from all types of reservoirs, access to the bottom shall be provided.

4.5.4.6.4 Multiple Use Function

Detention storage facilities should be compatible with their surroundings. Storage space is less noticeable when a depressed portion of a parking lot or a roof top is used. Detention on a small commercial lot might be a small basin that is shaped and landscaped to compliment the adjacent restaurant buildings, but away from the main entrance.

Detention storage areas, for the most part, are physical storage areas with an embankment, outlet works, and emergency spillway. These typically range anywhere from 1,000 square feet in area to many acres. Opportunities exist to enhance the appearance. Desirable conditions can be achieved through careful design and positioning of the structure, as well as through landscaping. Creative designs, integrated with innovative landscaping, also enhance the appearance of the outlet and pond and often are less expensive.

4.5.4.7 Goals

To change the public perception of drainage facilities and to demonstrate that stormwater management programs do provide multiple benefits, the following aesthetic goals are recommended:

1. To achieve a park-like appearance, as much as possible, without adding appreciably to the maintenance effort.
2. To enhance visual aesthetics by providing planting and placement of trees, evergreens, shrubs, rock gardens, railroad ties, picnic areas, playgrounds.
3. To encourage multi-use with an emphasis on recreational uses.

4.5.4.8 Outlet Structures

An effective mechanism for enhancing interest in detention facility outlet works is to utilize concrete that is "architecturally treated". Outlet structures can be visually enhanced by:

1. Color and surface texture variation of the concrete.
2. Utilizing different shapes and forms of aggregate, and exposing the aggregate to varying extent.
3. Varying the shapes and sizes of the hydraulic structure and its components and using the shapes for benches, climbing play areas, or for other recreational purposes.
4. Providing for continuous flow through a confined portion of the structure while adding ponding and water fall type features.
5. Utilizing rock or wood for portions of the structure.
6. Creating an island or landscaped mound around the outlet riser.

4.5.4.8.1 Spillways

If an emergency spillway will carry water, on the average, less than once every 25 or 100 years, the potential for enhancing the appearance of the spillway by landscaping and/or recreational facilities should be assessed. As long as its flow capacity will not be impeded, aesthetic improvements to the emergency spillway will be appreciated by the public.

4.5.4.8.2 Reservoir Area

Methods for enhancing the aesthetics of the reservoir include:

1. Varying the shape of the reservoir outline, such as providing peninsulas or coves.
2. Providing islands for wildlife refuge.
3. Varying the side slopes and materials along the edge of the reservoir, such as using a combination of wetlands, riprap, grass, walls, or other materials.

4.5.4.9 Open Space Detention

The stormwater detention lakes and dams constructed within the City of Russellville will generally be classified as small structures by the ASWCC. Any such open space detention facility constructed within the City and its Planning Area is to be designed, constructed, and maintained in compliance with the criteria of the SMDO, the regulations of the ASWCC, and all other regulations addressing such facilities.

4.5.4.9.1 ASWCC Regulations

As previously stated, the stormwater detention dams within the City of Russellville will likely be classified as small structures by the ASWCC.

Any dam constructed for the purpose of storing water and under the jurisdiction of the ASWCC shall be designed in accordance with the "Rules Governing the Design and Operation of Dams" published by the ASWCC. Those facilities not subject to the criteria of the state shall be designed and constructed in accordance with the criteria presented herein.

4.5.4.9.2 Grading

Slopes on earthen embankments less than 5 feet in height shall not be steeper than 4:1. For embankment heights between 5 feet and 10 feet, the slopes shall not be steeper than 3:1, but horizontal slope distance shall not be less than 20 feet. For embankments greater than 10 feet in height, the slopes shall be such to maintain slope stability, but horizontal slope distance shall not be less than 30 feet. Contact the CITY ENGINEER for additional requirements. All earthen slopes shall be covered with topsoil and revegetated with grass. Slopes with riprap earthen embankments shall not be steeper than 1½:1. For grassed detention facilities, the minimum bottom slope shall be 0.5% measured perpendicular to the trickle channel.

4.5.4.9.3 Freeboard

The minimum required freeboard for open space detention facilities not under the jurisdiction of the ASWCC is 1.0 feet above the spillway design flood (see Section III-4.5.4.9.5) water surface elevation.

4.5.4.9.4 Trickle Channel

All grassed bottom detention ponds shall include a concrete trickle channel or equivalent performing materials design. Longitudinal slopes shall be selected to achieve a minimum flow velocity of 2 fps.

The benefits of trickle channels are: (1) to protect edges of channel from erosion, (2) to provide for easier maintenance of channel vegetation by confining the flow and by maintaining grade, (3) by providing for a positive drainage path to lower the local groundwater table, and (4) to control sedimentation.

4.5.4.9.5 Outlet Configuration

Presented on Figure III-46 are two examples for detention pond outlet configuration. A Type 1 outlet consists of a grated drop inlet, outlet pipe, and an overflow weir in the pond embankment. The outlet is to control flood frequencies from the 2-year to the 100-year.

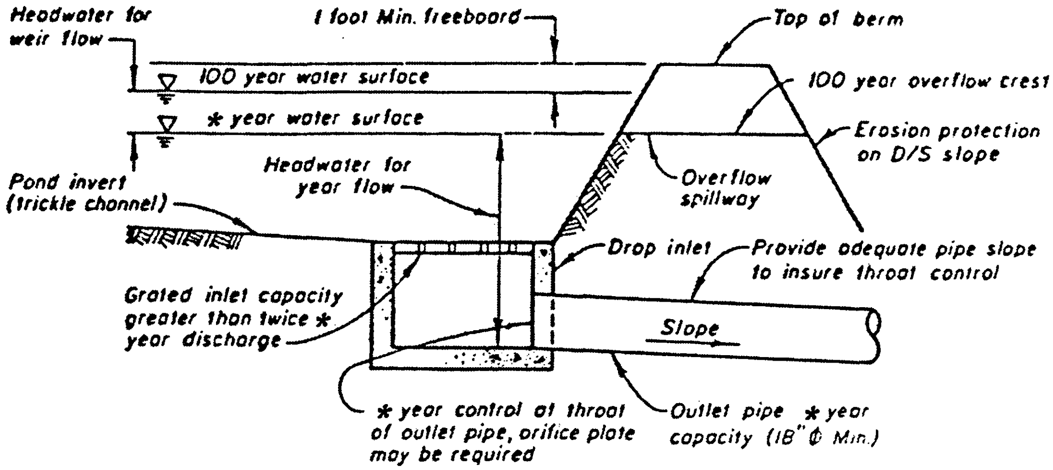
By selecting an intermediate frequency, such as the 25-year flood, the full range of frequency control is generally achieved. It is the responsibility of the Engineer to verify the required frequency control.

The control for the 25-year discharge should generally be at the throat of the outlet pipe under the head of water as defined on Figure III-46. The grate must be designed to pass the 25-year flow with a minimum of 50% blockage (i.e., twice the 25-year flow). Since the minimum size of the outlet pipe is 18 inches, then a control orifice plate at the entrance of the pipe may be required to control the discharge of the design flow. An example orifice plate is shown on Figure III-47. Other outlet configurations will be allowed provided they meet the requirements of the permitted release rates at the required volume and include proper provisions for maintenance and reliability. The outlet shall be designed to minimize unauthorized modifications which affect proper function.

The difference between the 100-year discharge and the surcharged discharge on the 25-year outlet is released by the overflow weir or spillway. If sufficient pond depth is available, the drop inlet and the grate can be replaced by a depressed inlet with a headwall and trash rack. Depression of the inlet is required to reduce nuisance backup of flow into the pond during trickle flows. The maximum trash rack opening dimension shall be equal to the minimum opening in the orifice plate.

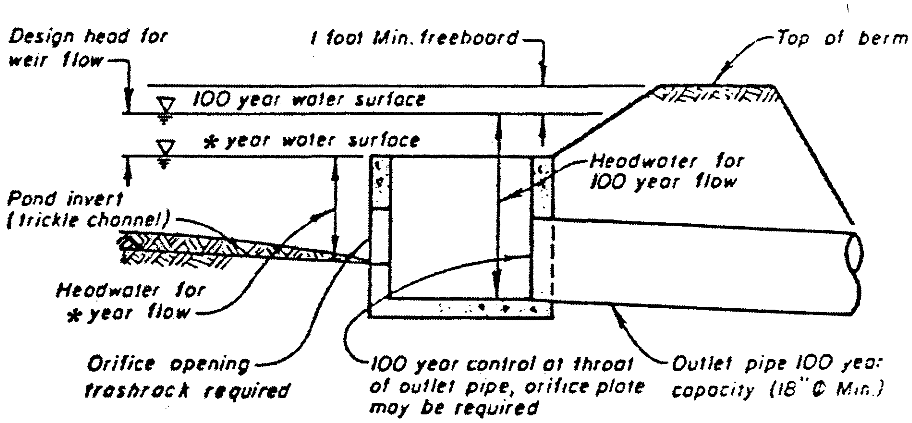
A Type 2 outlet consists of a drop inlet with an orifice controlled inlet for the 25-year discharge and a crest overflow and pipe inlet control for the 100-year discharge. The control for the 25-year discharge generally occurs at the orifice opening for the head as shown on Figure III-47.

DETENTION POND OUTLET CONFIGURATIONS



TYPE 1 OUTLET

No Scale



TYPE 2 OUTLET

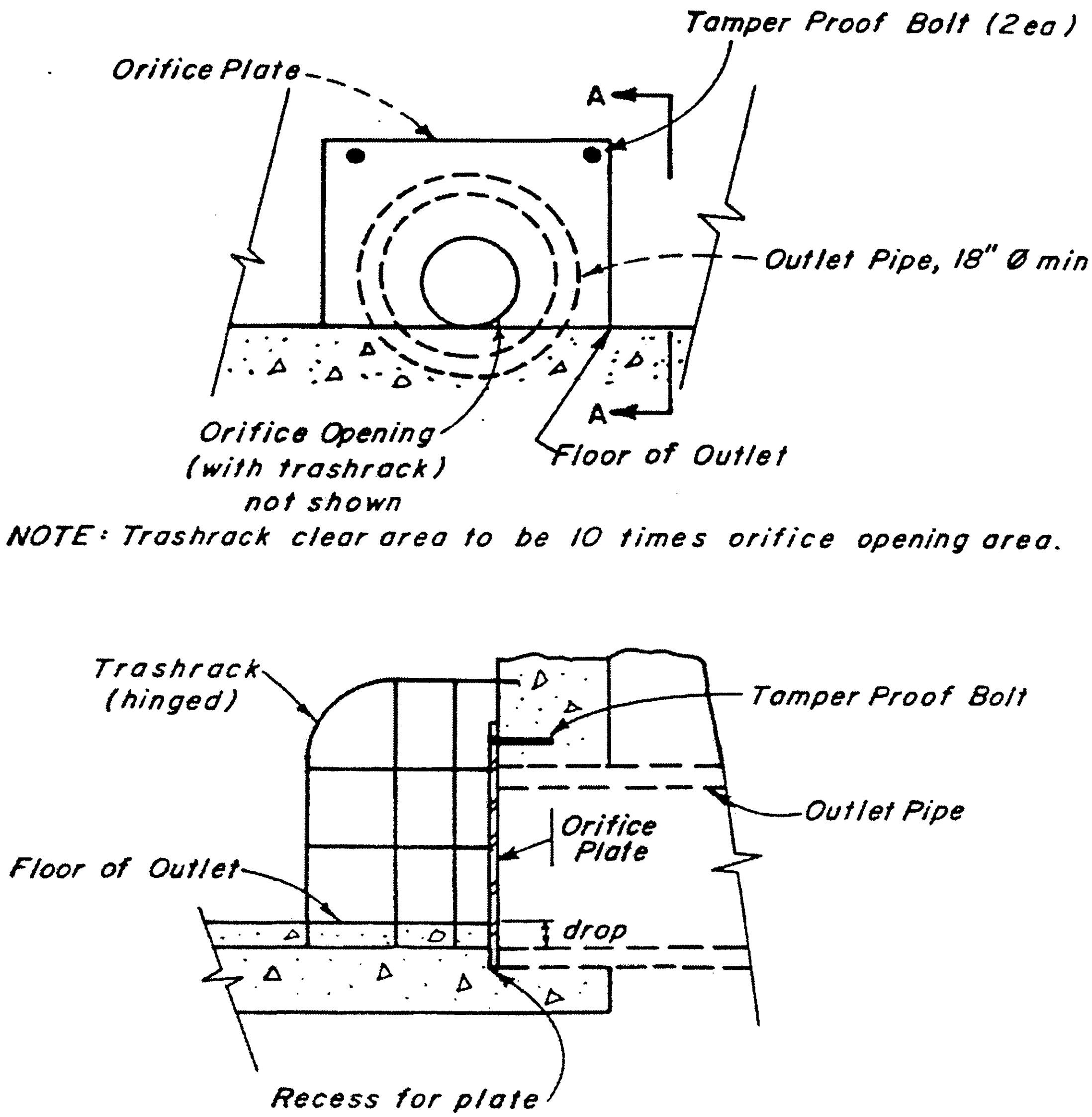
No Scale

NOTE:

*** THE MINIMUM CONTROLLING FREQUENCY FOR THE CITY OF RUSSELLVILLE SHALL BE THE 25-YEAR EVENT.**

Source: City of Tulsa, Stormwater Management Criteria Manual

City of **RUSSELLVILLE** Arkansas	**DETENTION POND OUTLET CONFIGURATIONS** **FIGURE III-46**



Source: City of Tulsa, Stormwater Management Criteria Manual

City of **RUSSELLVILLE** Arkansas	**ORIFICE PLATE DETAILS** **FIGURE III-47**

The control for the 100-year discharge occurs at the throat of the outlet pipe as shown on Figure III-47. However, the difference between the 100-year and 25-year discharge must pass over the weir and therefore the weir must be of adequate length. The effective weir length (L) occurs for three sides of the box. To ensure the 100-year control occurs at the throat of the outlet pipe, a 50% increase in the required weir length is recommended. In addition, the outlet pipe must have an adequate slope to ensure throat control in the pipe.

4.5.4.9.6 Embankment Protection

Whenever a detention pond uses an embankment to contain water, the embankment shall be protected from catastrophic failure due to overtopping. Overtopping can occur when the pond outlets become obstructed or when a larger flood than the design condition occurs. Failure protection for the embankment may be provided in the form of a buried heavy riprap layer on the entire downstream face of the embankment or a separate emergency spillway having a minimum capacity of the required spillway design flood (i.e., 50% of the Probable Maximum Flood (PMF) or the 500-year flood, whichever is greater). Structures shall not be permitted in the path of the emergency spillway or overflow. The invert of the emergency spillway should be at or above the 100-year water surface.

4.5.4.9.7 Vegetation

All open space detention areas shall be revegetated by either irrigated Bermuda sod or natural dry-land grasses.

4.5.4.9.8 Maintenance Access

All open space detention areas shall include an easement or dedicated right-of-way for the purpose of obtaining access from a public ROW and for maintenance activities.

4.5.4.10 Parking Lot Detention

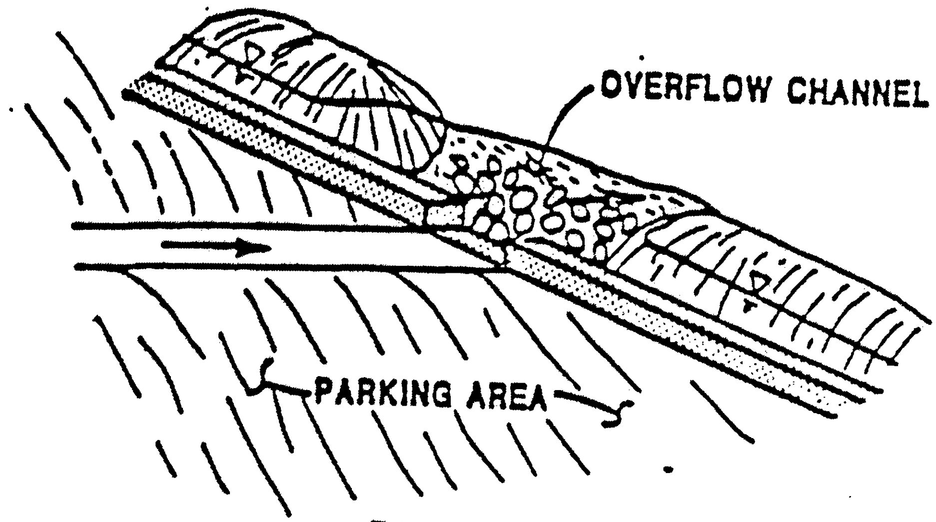
The requirements for parking lot detention are described in the following sections.

4.5.4.10.1 Depth

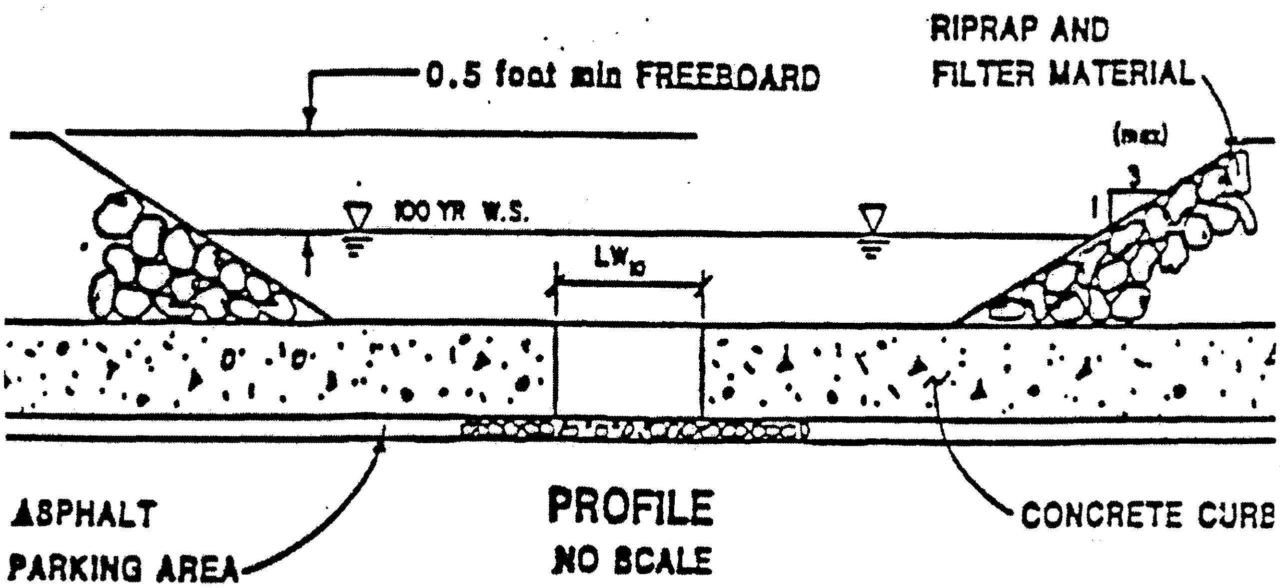
The maximum allowable design depth of the ponding is 6 inches. In no case shall ponding be designed within 10 feet of a building unless waterproofing and pedestrian accessibility are properly documented and approved by the CITY ENGINEER.

4.5.4.10.2 Outlet Configuration

The minimum pipe size for the outlet is 18-inch diameter where a drop inlet is used to discharge to a storm sewer or drainageway. Where a weir and a small diameter outlet through a curb are used, the size and shape are dependent on the discharge/storage requirements. A minimum pipe size of 3-inch diameter is recommended. Refer to Figure III-48 for sample outlet in a parking lot.



PLAN



PROFILE
NO SCALE

Source: City of Tulsa, Stormwater Management Criteria Manual

City of **RUSSELLVILLE** Arkansas	**DETENTION POND OUTLET CONFIGURATION** **PARKING LOT** **FIGURE III-48**

4.5.4.10.3 Performance

To assure that the detention facility performs as designed, maintenance access shall be provided. The outlet shall be designed to minimize unauthorized modifications which effect function. Any repaving of the parking lot shall be evaluated for impact on volume and release rates and are subject to approval by the CITY ENGINEER prior to issuance.

4.5.4.10.4 Flood Hazard Warning

All parking lot detention areas shall have a minimum of two signs posted identifying the detention pond area. The signs shall have a minimum area of 1.5 square feet and contain the following message:

"WARNING"

> **"This area is a stormwater detention pond and is subject to periodic flooding to a depth of (provide design depth for 25-year or 100-year storm, whichever is the design flood.)"**

Any suitable materials and geometry of the sign are permissible, subject to approval by the CITY ENGINEER.

4.5.4.11 Underground Detention

The use of underground detention is generally prohibited unless prior written approval is obtained from the CITY ENGINEER. Should underground detention be approved, the requirements are described in the following sections.

4.5.4.11.1 Materials

Underground detention shall be constructed using corrugated aluminum pipe (CAP), reinforced concrete pipe (RCP), reinforced concrete box culvert (RCBC), or concrete vaults. The material thickness, cover, bedding, and backfill shall be designed to withstand HS20 loading.

4.5.4.11.2 Configuration

Pipe segments shall be sufficient in number, diameter, and length to provide the required minimum storage volume for the 100-year design. As an option, the 10-year design can be stored in the pipe segments and the difference for the 100-year stored above the pipe in an open space detention (Section III-4.5.4.9) or in a parking lot detention (Section III-4.5.4.10). The minimum diameter of the pipe segments shall be 36 inches.

The pipe segments shall be placed side by side and connected at both ends by elbow tee fittings and across the fitting at the outlet (see Figure III-49). The pipe segments shall be continuously sloped at a minimum of 0.25% to the outlet. Manholes for maintenance access (see Section III-4.5.4.11.4) shall be placed in the tee fittings and in the straight segments of the pipe, when required.

Permanent buildings or structures shall not be placed above the underground detention.

4.5.4.11.3 Inlet and Outlet Works

The outlet from the underground detention shall consist of a short (maximum 25 feet) length(s) of CAP or RCP with an 18-inch minimum diameter. A two-pipe outlet may be required to control both design frequencies. The invert of the lowest outlet pipe shall be set at the lowest point in the detention pipes. The outlet pipe(s) shall discharge into a standard manhole or into a drainageway with erosion protection provided. If an orifice plate is required to control the release rates, the plate(s) shall be hinged to open into the detention pipes to facilitate back flushing of the outlet pipe(s).

Inlet to the detention pipes can be by way of surface inlets and/or by a local private storm sewer system.

4.5.4.11.4 Maintenance Access

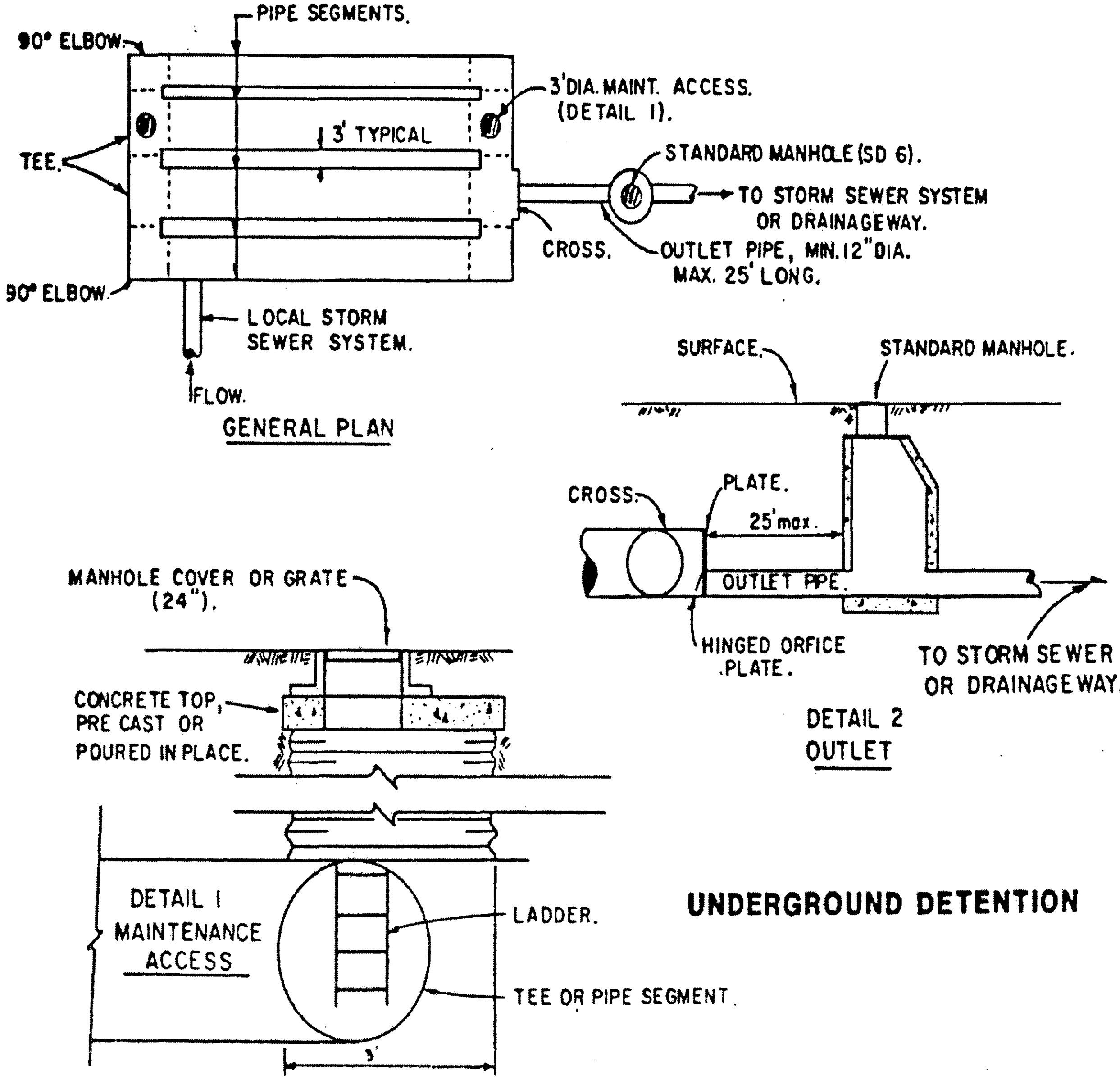
Access easements to the detention site shall be provided. To facilitate cleaning of the pipe segments, 3-foot diameter maintenance access ports shall be placed according to the following schedule:

Maintenance Access Requirements		
Detention Pipe Size	**Maximum Spacing**	**Minimum Frequency**
36 to 54 inches	150 feet	Every pipe segment
60 to 66 inches	200 feet	Every other pipe segment
>66 inches	200 feet	One at each end of the battery of pipes

The manholes shall be constructed in accordance with the detail on Figure III-49.

4.5.4.12 Retention

Retention storage is stormwater runoff that is retained in an impoundment until such time that it can be safely released into a receiving stream. When the CITY ENGINEER determines that stormwater retention must be employed for a specific development, the facility shall be designed using the following criteria:



Source: City of Tulsa, Stormwater Management Criteria Manual

City of **RUSSELLVILLE** Arkansas	**UNDERGROUND DETENTION** **FIGURE III-49**

1. The minimum retention volume shall equal the runoff from a 100-year storm of 24-hour duration. No credit shall be taken for infiltration in establishing the minimum volume.

2. An overflow section shall be provided for the detention facility that will protect embankments from overflow resulting from a 100-year storm when the pond is full and the tributary area is fully developed.

3. Side slopes shall not be steeper than 4:1.

4. A minimum freeboard of 1 foot shall be provided above the maximum retention volume water surface.

5. The applicant must evaluate or assess the impacts of the retention facility on local groundwater levels and the potential for damage to nearby properties.

6. A slow release will be permitted of 0.25 cfs/acre or less if the small flows will be conveyed to a major drainageway and will not cause nuisance conditions such as icing on highways.

7. The use of retention does not relieve the land developer from making permanent detention improvements on his property as a condition of subdivision approval.

8. A drainage maintenance easement shall be granted to the City to ensure that emergency maintenance and access can be legally provided to keep the facility operable. This easement may be vacated when the retention pond function is no longer needed.

9. A maintenance plan must be provided which details mowing, removal of sediment, debris and other requirements.

10. Acceptable alternatives to these requirements may include:

    a. Agreement among landowners wherein historic flow rates are exceeded by upstream landowners and will be accepted by downstream landowners. Such agreements are subject to review by the CITY ENGINEER.

    b. The developer providing off-site drainage improvements to convey stormwater at historic rates to an acceptable outfall point.

4.5.4.12.1 Compensatory Storage

Compensatory storage is defined as the storage provided to compensate for filling or development within the regulatory floodplain.

The volume of the compensatory storage required shall be equal to the fill material volume placed above the natural ground up to the regulatory flood datum (i.e., 100-year water surface plus 1 foot). The volume shall be provided by excavating the natural ground ABOVE the channel thalweg and shall be contiguous with the main channel. A separate storage area outside the floodplain or a sump area within the floodplain is not acceptable unless approved in writing by the CITY ENGINEER prior to the design and any related construction.

5.0 FLOODPLAIN/FLOODWAY POLICES

5.1 General

Generally a proposed development situated partially or totally within a delineated "100-year Flood Boundary" (1% annual chance flood event) area in a FIS will be classified as a MAJOR DEVELOPMENT. As such, the developer shall consider and comply with all applicable floodplain/floodway regulations of the Federal Emergency Management Agency (FEMA) and the City of Russellville, including the SMDO. **In no case** will a proposed development be approved that is situated partially or totally within a delineated ***Floodway***, unless specifically approved in writing by the CITY ENGINEER and acknowledged by a FEMA representative.

Further information regarding Floodplain Management is presented in Appendix B.

5.2 Evaluation Requirements

All analyses, designs, and reports regarding a land disturbing activity in an FIS stream reach shall be performed/prepared by a Professional Engineer, licensed in the State of Arkansas, and must demonstrate competence in floodplain analysis. All field surveys that form a basis for such designs and/or analyses shall be "tied" to a Reference Mark located and defined in the FIS report.

5.3 Documentation Requirements

A copy of all calculations performed during the floodplain/floodway analyses shall be submitted to the CITY ENGINEER and retained on file until such time that it has been reviewed and approved by FEMA. All reports and design plans shall be properly sealed and signed by the Registered Professional Engineer performing the designs/analyses. Electronic files of all models and input files used in the floodplain analyses shall be provided to the CITY ENGINEER on a media approved by the CITY ENGINEER.

5.4 Disclaimer

No action, or lack thereof, by the CITY ENGINEER shall relieve the OWNER of the responsibility to comply with all applicable floodplain/floodway requirements of proposed location of the land disturbing activity.

6.0 EROSION AND SEDIMENTATION CONTROL

6.1 General

The DEVELOPER of a proposed development classified as a MAJOR DEVELOPMENT shall be required to submit a detailed Erosion and Sedimentation Control Plan. The developer shall submit a detailed Erosion and Sedimentation Control Plan that will be instituted during the construction of the proposed development as outlined in the SMDO. The CITY ENGINEER may impose additional requirements as conditions for the approval of the Development Classification Form.

6.2 Disclaimer

No action, or lack thereof, by the CITY ENGINEER shall relieve the OWNER of the responsibility to minimize erosion/sedimentation problems that may develop during and following the proposed land disturbing activities.

7.0 MAINTENANCE

7.1 General

The DEVELOPER of a proposed development classified as a MAJOR DEVELOPMENT will be required to submit a detailed Maintenance Plan as outlined in the SMDO to insure that the stormwater management features will be properly maintained.

The CITY ENGINEER may impose additional requirements as conditions for the approval of the Development Classification Form. The Maintenance Plan will include an assurance in writing submitted by the OWNER to the CITY ENGINEER that maintenance of the Stormwater Management facilities will be performed. The OWNER shall grant the right of the CITY ENGINEER and/or his representative to inspect the stormwater management facilities at his discretion.

No action, or lack thereof, by the CITY ENGINEER shall relieve the OWNER of the responsibility to provide regular maintenance to the stormwater management facility serving the proposed development.

7.2 Privately Owned Stormwater Management Facilities

All stormwater management facilities shall be privately owned and maintained unless the City accepts the facility for City ownership and maintenance. The owner of all private facilities shall grant to the City a perpetual, non-exclusive easement which allows for public inspection and emergency repair. Private maintenance requirements shall be a part of the deed to the affected property.

7.3 Publicly Owned Stormwater Management Facilities

1. Stormwater Management Plans shall include designation of all easements needed for inspection and maintenance of the drainage system and stormwater management facilities. As a minimum, easements shall have the following characteristics:

   a. Provide adequate access to all portions of the drainage system and structures.
   b. Provide sufficient land area for maintenance equipment and personnel to adequately and efficiently maintain the system with a minimum of 10 feet along both sides of all drainageways, streams, channels, etc., and around the perimeter of all detention and retention facilities, or sufficient land area for equipment access for maintenance of all stormwater management facilities. This distance shall be measured from the top of the bank or toe of the downstream side of the dam, whichever is applicable.
   c. Restriction on easements shall include prohibiting all fences and structures which would interfere with access to the easement areas and/or the maintenance function of the drainage system.

2. To improve the aesthetic aspects of the drainage system, a landscape plan for all portions of the drainage system shall be part of the Stormwater Management Plan. This landscape plan shall address the following:

   a. Tree saving and planting plan.

   b. Types of vegetation that will be used for stream bank, stabilization, erosion control, sediment control, aesthetics, and water quality improvement.

   c. Any special requirements related to the landscaping of the drainage system and efforts necessary to preserve the natural aspects of the drainage system.

3. Regional stormwater management facilities will be publicly owned and/or maintained.

DIVISION IV. ASSOCIATED DRAINAGE

1.0 GENERAL

For the City of Russellville, Arkansas and its Planning Area, associated drainage facilities may be required due to planned MINOR and MAJOR DEVELOPMENTS that are not specifically addressed in those DIVISIONS of this MANUAL. In addition, improvements to existing infrastructure are often needed to reduce flooding conditions affecting the City. This DIVISION of the MANUAL attempts to address those associated drainage facilities, specifically Flow in Streets, Flow into Inlets, and Flow in Underground Storm Sewers. Where this DIVISION of the MANUAL differs from requirements of previously adopted ordinances of the City, the Arkansas Highway and Transportation Department (AHTD), and the Federal Highway Administration (FHWA), it will yield to those previously adopted requirements. However, any deviation from the requirements of this MANUAL shall be approved in writing from the CITY ENGINEER prior to the preparation of any plans and subsequent construction.

2.0 FLOW IN STREETS

2.1 Overview

Provision for the proper drainage of a roadway traversing a built-up municipality generally is a more difficult and more important problem of design than for roadways traversing sparsely settled rural areas. This is attributable to the (1) wide roadway sections and flat grades, both in longitudinal and transverse directions; (2) shallow water courses; (3) absence of side ditches and concentration of all flow; (4) large property damages which may occur from ponding of water or from flow of water through built-up areas; and (5) fact that the roadway section must act not only as a medium to carry traffic, but also as a channel to carry the water to some disposal point. Unless proper precautions are taken, this flow of water along the roadway will interfere, with or possibly halt the passage of roadway traffic.

An inadequate street drainage system may cause damage to surrounding property resulting from water overflowing the roadway curbs; hazard and delay to traffic caused from excessive ponding, sags, or excessive flow along roadway grades; and subsequent weakening of base and subgrade due to saturation from ponding of long duration.

The limit of flow of water in the streets should be related to the extent and frequency of interference to traffic and the likelihood of flood damage to surrounding property. Interference to traffic is regulated by design limits on the spread of water into traffic lanes. Flooding of surrounding property from streets is controlled by limiting the buildup of runoff to the top of the curb for the design storm.

2.2 Flow Problems

2.2.1 Interference Due to Flow in Streets

Water that flows in a street, whether from rainfall directly onto the pavement surface or overland flow entering from adjacent land areas, will flow in the gutters of the street until it reaches an overflow point or some other outlet, such as a storm sewer inlet. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively encroach into a traffic lane. On streets where parking is not permitted, as with many arterial streets, flow width exceeding a few feet becomes a traffic hazard.

As the width of flow increases further, it becomes impossible for vehicles to operate without moving through water, and they must use the now-inundated lane. Splash from vehicles traveling in the inundated lane obscures the vision of drivers of vehicles moving in the open lane. Eventually, if width and depth of flow become great enough, the street loses its effectiveness as a traffic-carrier. During these periods, it is imperative that emergency vehicles such as fire trucks, ambulances, and police cars be able to traverse the streets by moving along the crown of the roadway.

2.2.2 Interference Due to Ponding

Storm runoff ponded on the street surface because of grade changes or the crown slope of intersecting streets has a substantial effect on the street's traffic carrying capacity. The manner in which ponded water affects traffic is essentially the same as for curb flow, that is, the width of spread into the traffic lane is critical. Ponded water will often completely halt all traffic. Ponding in streets has the added hazard of surprise to drivers of moving vehicles, often producing erratic and dangerous responses.

2.2.3 Interference Due to Water Flowing Across Traffic Lane

Whenever stormwater runoff, other than limited sheet flow, moves across the traffic lane, a serious and dangerous impediment to traffic flow occurs. The cross-flow may be caused by super-elevation of the curb, a street intersection, overflow from the higher gutter on a street with cross fall, or simply poor street design. The problem associated with this type of flow is the same as for ponding in that it is localized in nature. Vehicles may be traveling at high speed when they reach the location. If vehicular movement is slow and the street is lightly traveled, as on residential streets, limited cross flows may not cause significant interference and may be permitted. However, no intentional cross flow of street pavements will be permitted without written permission from the CITY ENGINEER prior to the development of plans.

The depth and velocity of cross flows, if permitted, shall be maintained within such limits that they do not have sufficient force to threaten moving traffic.

2.2.4 Interference Due to Water Flowing Through Intersections

As the stormwater flow approaches an arterial street or tee intersection, an inlet is required. Concrete swales may be used to convey water across residential streets at the intersection of a residential street and a larger capacity street at the approval of the CITY ENGINEER. Swales are not allowed across larger capacity streets without written approval from the CITY ENGINEER prior to the development of plans.

2.2.5 Effects on Pedestrians

In areas with heavily used sidewalks, splash due to vehicles moving through water adjacent to the curb is a serious problem.

Streets should be classified with respect to pedestrian traffic as well as vehicular traffic. As an example, streets that are classified as residential and located adjacent to a school should be considered as arterial for pedestrian traffic. The allowable width of gutter flow and extent of ponding should reflect this fact.

2.3 Prevention of Flow Problems

The prevention of the flow problems discussed above may be achieved by a properly designed storm drainage system.

Design Procedures – The design of storm drainage systems is generally divided into the following steps:

1. Determine inlet location and spacing to limit the width of spread of water flowing in the streets to a maximum specified in Section IV-2.4 for the type of roadway proposed or as otherwise specified by the city engineer.

2. Determine gutter flow conveyed to each inlet, described in Section IV-2.5, and then determine the size and type of inlet required to limit the spread determined in Step 1, described in Section IV-3.

3. Prepare a plan layout of the storm drainage system defining the following design data:

    a. Location of required inlets.

    b. Flow intercepted by each inlet.

    c. Location of storm drainage pipes.

    d. Direction of flow.

e. Location of manholes and/or junction boxes.

f. Location of existing facilities such as water, sanitary sewer, gas, telephone, underground cable, and other utilities.

4. Design storm drainage pipes based on the procedure described in Section IV-4.0.

5. Perform final hydraulic check of the system described in Section IV-5.0.

Design Criteria – The criteria discussed in the following paragraphs is to be used in the storm drainage system designs within the City of Russellville and its Planning Area.

Street Cross Section – All streets within the City of Russellville, Arkansas and its Planning Area shall comply with the requirements of Section IV-5.0, STREETS, of Russellville Ordinance #1401 or any successor. A 2-foot curb and gutter with a 6-inch high curb shall typically be used. Cross sections of curbs and gutters are subject to the approval of the CITY ENGINEER.

Hydrology – The following hydrologic design criteria shall be used in the computation of allowable flow in the gutters of streets within the City of Russellville, Arkansas and its Planning Area:

For gutter flow in all roadways, a minimum rainfall intensity of 4.0 inches per hour will be used. The rainfall intensity for a 25-year storm event associated with the proposed inlet location shall be used if it is greater than 4.0 inches per hour.

Flow bypassing each inlet must be included in the total gutter flow to the next inlet downstream. A bypass of 10 to 20% per inlet will result in a more economical drainage system. Refer to Section 3.0 of this DIVISION of the MANUAL for inlet design.

The 100-year frequency storm and associated depth of flow shall be determined and plotted on the construction drawings. The streets, sidewalks, and driveway cuts shall be designed such that flow from the 25-year storm event or all storms with a rainfall intensity of up to 4.0 inches per hour does not convey through driveway cuts and across private property but remains generally within street right-of-way and/or drainage easements.

2.3.1 Longitudinal Slope

A minimum longitudinal gradient is more important for a curbed pavement, since it is susceptible to stormwater spread. Flat gradients on uncurbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge.

Curb and gutter grades that are equal to pavement slopes shall not fall below 0.5%. Minimum grades can be maintained in very flat terrain by use of a sawtooth profile. For long vertical curves, cross slope may be varied slightly to achieve 0.5% minimum gutter grade.

2.3.2 Minimum and Maximum Velocity

To ensure cleaning velocities at very low flows, the gutter shall have a minimum slope of 0.005 ft/ft (0.5%). The maximum velocity of curb flow shall be 10 fps. Along sharp horizontal curves, peak flows tend to jump behind the curb line at driveways and other curb breaks. Water running behind the curb line can result in considerable damage due to erosion and flooding. In a gutter where the slope is greater than 0.10 ft/ft (10%) and the radius is 400 feet or less, design depth of flow shall not exceed 4 inches at the curb.

2.4 Allowable Spread of Water

The limiting widths of spread, T_{Max}, defined below, shall be used for the design of street drainage within the City of Russellville, Arkansas and its Planning Area. Inlets shall be located at low points and wherever the flow exceeds the limit of spread defined. Inlet design, including gutter depression, shall conform to Section 3.0 of this DIVISION of the MANUAL.

The allowable spread of water for all classifications of streets defined above applies to all configurations and layouts including the case of streets built on hillsides parallel to the contours. The maximum allowable spread of depth of water defined above applies even if the downhill lane is theoretically clear.

Interstate and Controlled Access Highways – Limit T_{Max} to one-half the width of the outer lane.

Major Highways – Limit T_{Max} to the width of the outer lane.

Minor Highways – Limit T_{Max} to a width and depth that will allow passage of one lane of traffic with safety.

2.4.1 Principal and Minor Arterial Streets

Limit the width of spread, T_{Max}, to provide one clear traffic lane in each direction during the peak flows of the design storm as defined below:

1. Local commercial/boulevard streets shall be designed so that a minimum 12-foot traffic lane, independent of the curb and gutter, is provided in each direction during the peak flows of the design storm.

2. Four and five lane arterial streets (nonboulevard) shall be designed so that a minimum of one 12-foot traffic lane is provided in each direction during the peak flow of the design storm.

Example

Street width – 52 feet back-to-back; standard 2-foot curb and gutter with 6-inch curb; two 12-foot lanes to remain clear.

> Street flow in each gutter shall not exceed T_{Max},
> ((52 – (.5 + 12 + .5 + 12))/2 = 13.5 feet.

2.4.2 Collector Streets

The flow of water in gutters of a collector street shall be limited so that one standard 12-foot lane will remain clear during the peak runoff from the design storm.

Example

Street width – 40 feet back-to-back; standard 2-foot curb and gutter with 6-inch curb; one 12-foot traffic lane to remain clear.

> Width of street flow spread in each gutter shall not exceed T_{Max}
> ((40 – (.5 + .5 + 12))/2 = 13.5 feet

2.4.3 Local and Residential Streets

The flow of water in gutters of local and residential streets, including alleys, shall be limited such that a minimum 8-foot lane in the center of the street will remain clear during a storm event with an intensity of 4.0 inches per hour.

The requirement to keep a minimum clear lane of 8 feet shall apply to Local Residential Streets (27 feet back to back) and Alleys varies – contact the CITY ENGINEER.

Example

Street width – 27 feet back-to-back; standard 2-foot curb and gutter with 6-inch curb; one 8-foot traffic lane to remain clear.

> Street flow in each gutter shall not exceed T_{Max}
> ((27 – (.5 + .5 + 8))/2 = 9 feet

2.5 Gutter Flow Determination

2.5.1 Straight Crowns

Flow in gutters associated with straight crown pavement is normally calculated by using Manning's Equation for various hydraulic properties for uniform flow in pavement gutters and triangular channels. The equation is:

$$Q_G = 0.56\ z/n\ S_G^{0.5}\ Y_G^{2.6667} \qquad \textbf{(IV-1)}$$

Where:

Q_G = Gutter discharge (cfs)
z = Reciprocal of the crown or transverse slope of the pavement (ft/ft)
n = Roughness coefficient
S_G = Street grade or gutter slope (ft/ft)
Y_G = Depth of flow in gutter (ft)

The nomograph in Figure IV-1 provides for direct solution of flow conditions for triangular channels most frequently encountered in urban street drainage design. For a standard concrete gutter, a value of 0.016 for "n" is recommended.

2.5.2 Parabolic Crowns

Flow in gutters that are on parabolic crown pavements is calculated from a variation of Manning's Equation for steady flow in a prismatic open channel.

$$\log Q_G = K_0 + K_1 \log S_G + K_2 \log Y_G \qquad \textbf{(IV-2)}$$

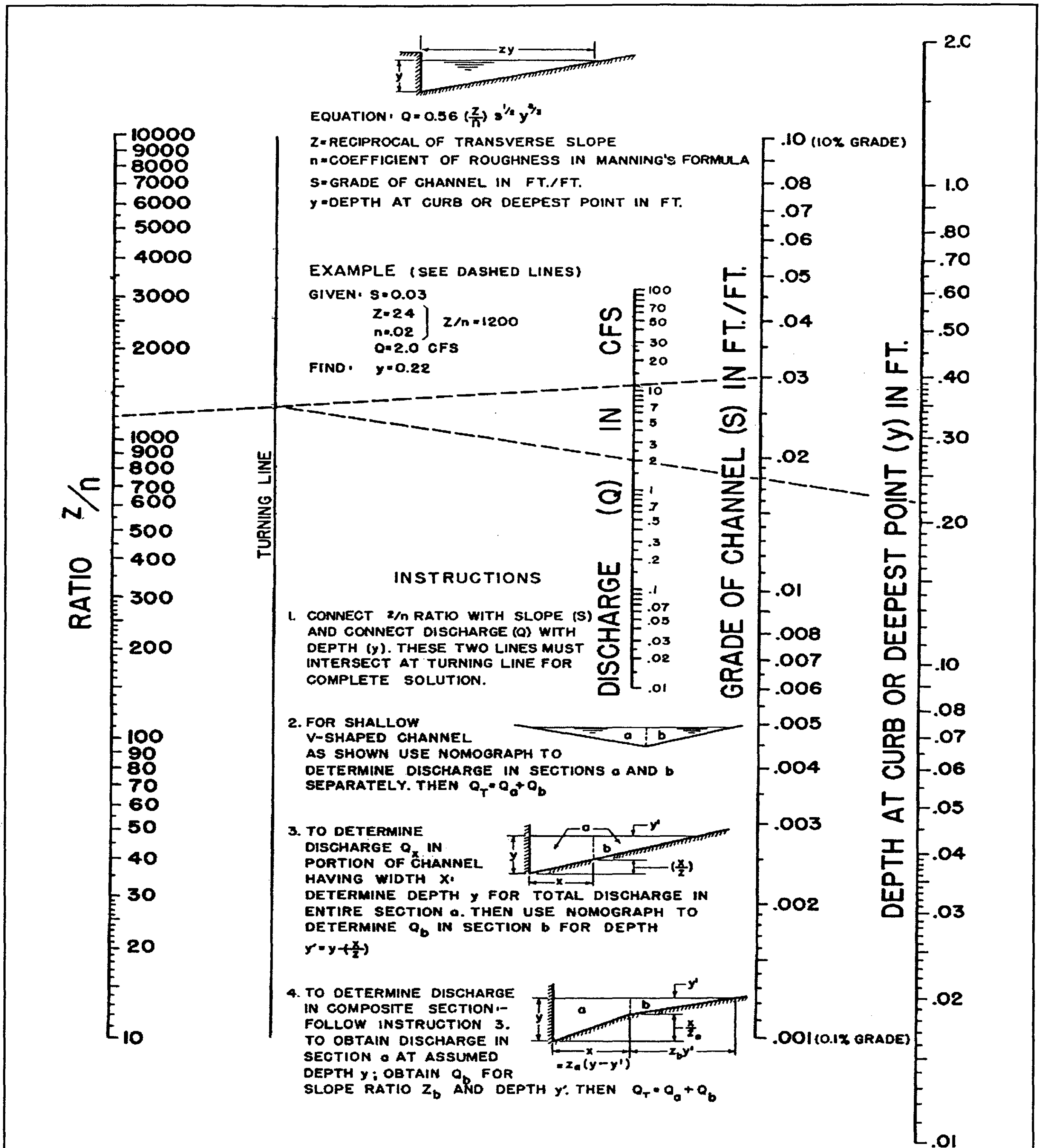
Where:

Q_G = Gutter flow (cfs)
S_G = Street grade (ft/ft)
Y_G = Water depth in the gutter (ft)
K_0, K_1, K_2 = Constant coefficients shown in the table below for different street widths

COEFFICIENTS FOR PARABOLIC STREETS			
Street Width* (ft)	COEFFICIENTS		
	K_0	K_1	K_2
30	2.85	0.50	3.03
36	2.89	0.50	2.99
40	2.85	0.50	2.89
44	2.84	0.50	2.83
48	2.83	0.50	2.78
60	2.85	0.50	2.74

*Note: Based on the Transportation Criteria Manual, the street width is measured from face of curb to face of curb (FOC-FOC).

Source: City of Austin, Watershed Management Division



Source: AASHTO Model Drainage Manual, 1998

City of **RUSSELLVILLE** Arkansas	**NOMOGRAPH FOR CURBED GUTTER** **FIGURE IV-1**

3.0 FLOW INTO INLETS

3.1 General

Limiting the amount of water flowing along the gutters or ponding at the sags along the streets to quantities, which will not interfere with the passage of traffic, can be accomplished by placing inlets at such points and at such intervals to prevent very large accumulations of drainage water. Inlets serve as discharge points for flow being conveyed in the gutters. Thus, when flow in the gutter exceeds the limit of spread, an inlet needs to be designed at that point along the street. Table IV-1 is included to help in the sizing and placement of inlets to control the width of spread and to assist in the design effort.

3.2 Inlet Design Procedure

The basic steps in the design of storm drainage inlets are described in the following sections.

3.2.1 Location of Inlets

Identifying known inlet locations is the first step in the design process. Inlets will be located at all low points along the street and at all intersections where flow across the intersection is not allowed. It can usually be assumed an inlet will be located at each corner of an intersection of major roadways. See Section IV-2.4.

3.2.2 Watershed Map Development

A watershed map should be developed to define the watershed draining to the proposed outlet of the development. Subwatersheds should be delineated that drain to each known inlet location.

3.2.3 Determination of Runoff

Preliminary runoff computations should be performed to determine the approximate flow to be expected in the gutter at each inlet, assuming each inlet will intercept the total flow draining to it. The Rational Method as described in DIVISIONS II and III shall compute those flows. The rainfall intensity to be used in the design process is defined in Section III-3.3.1.4. As mentioned earlier, the flow of water in the gutter should be restricted to a depth, and corresponding width, that will not cause the water to spread out over the traveled portion of the roadway in such amount and depth as to obstruct or cause a definite hazard to traffic. The depth of flow naturally depends upon the quantity of water involved, the gutter gradient, the coefficient of roughness or frictional coefficient of the gutter and paving material, and the cross slope of the roadway.

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q_G = [0.56/n]\ S_T^{1.6667}\ S_G^{0.5}\ T^{2.6667} \qquad \text{(IV-3)}$$

INLET COMPUTATION FORM

Project: ____________________

Date________ Sp______ Route________
Computed By____________ Sheet____ of____

Inlet		Drainage Area				Runoff			Roadway			Gutter Properties			Inlet Sizing						Remarks
Inlet No.	Location	Number	Area (acres)	Time of Concentration "T_c" min	Intensity "I" (4.0 In/Hr Min.) in/hr	Q_{sub} From D.A. At Inlet	Previous Carry Over (Col. 21)	Total Q At Inlet (cfs)	Transverse Slope, S_T (ft/ft)	"Z" – $1/S_T$	"Z/n" Ratio	Grade "S_G" ft/ft	Depth at curb "d" or "y" (feet)	Spread Width "T" Feet	Type	"K"	Length "L_i" (feet)	Depression, a (in)	Intercepted "Q_i" (cfs)	Carry Over "Q_i – Q_i/I" (cfs)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	

City of
RUSSELLVILLE
Arkansas

INLET CALCULATION FORM

TABLE IV-1

Where:

Q_G = Gutter flow rate (cfs)
n = Manning's roughness coefficient
S_T = Pavement cross slope (ft/ft)
S_G = Gutter gradient or longitudinal slope (ft/ft)
T = Width of flow or spread (ft)

Note: Manning's n value for concrete curb and gutter is 0.016.

Knowing the allowable width of spread, T_{Max}, from Section 2.4 of this DIVISION, the associated gutter flow can be calculated knowing the other parameters of Equation IV-3. A comparison of the maximum gutter flow for the allowable width of spread and the runoff computed for that inlet will determine if additional inlets are required. If the computed gutter flow expected at the inlet is greater than maximum flow for the allowable flow width, additional inlets will be required. Additional inlets should be located starting from the high point of the longitudinal slope of the pavement in order to intercept as much gutter flow as possible before it reaches a low point inlet. This is important since the longitudinal slope flattens as the low point is approached. A ratio of the maximum allowable gutter flow to the total runoff computed to the low-point inlet will provide an approximate subwatershed area to be served by the additional inlet(s). Iterative trial locations will need to be evaluated to determine the location of the needed inlet(s). It is important to remember that some bypass flow is permissible, especially for the upper inlets. This procedure is continued until an adequate number of inlet locations are determined. While a low point or sag inlet will be required for each low point, a majority of the expected runoff should be intercepted before it reaches the low point. Similar procedures are performed for each longitudinal slope with a preliminary inlet located.

Upon the preliminary location of needed inlets, use Table IV-1 to complete the analysis of the inlets and the design of the opening(s) required. The following data should be developed and entered as follows:

Column 1: Inlet No. – Assigned number (or label) of drainage structure.

Column 2: Location – Stationing, if available, or general street location.

Column 3: Drainage Area No. – Assigned number (or label) of drainage sub-area.

Column 4: Drainage Area – Area contributing runoff to the inlet (acres).

Column 5: Time of Concentration – Runoff time of the sub-area to this inlet (minutes).

Column 6: Intensity "I" – Intensity of rainfall for the sub-area and design storm or 4.0 in/hr, whichever is greater. (See Section III-3.3.1.4)

Column 7: Runoff, "Q_{sub}", from Sub-Area to Inlet – Compute using Rational Formula:

$$Q = CIA \qquad \textbf{(IV-4)}$$

Where:

C = Runoff coefficient for the sub-drainage area
I = Intensity of rainfall = 4.0 in/hr or I_{25} if greater
A = Area (acres) determined in #4 above

Column 8: Carry Over from Column 21 – Bypass or carry over from previous inlet.

Column 9: Total Flow – Surface 'Q' Total – $Q_{total} = Q_{sub} + Q_{bypass}$

Column 10: Transverse Slope S_T – Transverse slope at "Inlet #" (in ft/ft). This is equivalent to the roadway cross-slope.

Column 11: Z – Equal to the reciprocal of the Transverse Slope, S_T.

Column 12: Z/n – The value of Z divided by the Manning's coefficient of the material of the pipe.

Column 13: Gutter Slope, S_G – Longitudinal gutter slope at "Inlet #" (in ft/ft). This is equivalent to the roadway centerline profile.

Column 14: Depth at Curb – Depth of flow at the curb approaching the inlet, noted as "d" or "y". Refer to Figure IV-1.

Column 15: Spread Width (T) – Width of flow (feet) into the roadway. T = "d" or "y"/S_T (For a "normal crown" street, S_T = 0.0208 ft/ft or 1/4 inch per 1 foot.)

Column 16: Type – Type of inlet, curb inlet (CI), Grate Inlet (DI), or Combination (CDI).

Column 17: K – This coefficient is used to determine the inlet capacity of a grate inlet on grade. Refer to Figure IV-2.

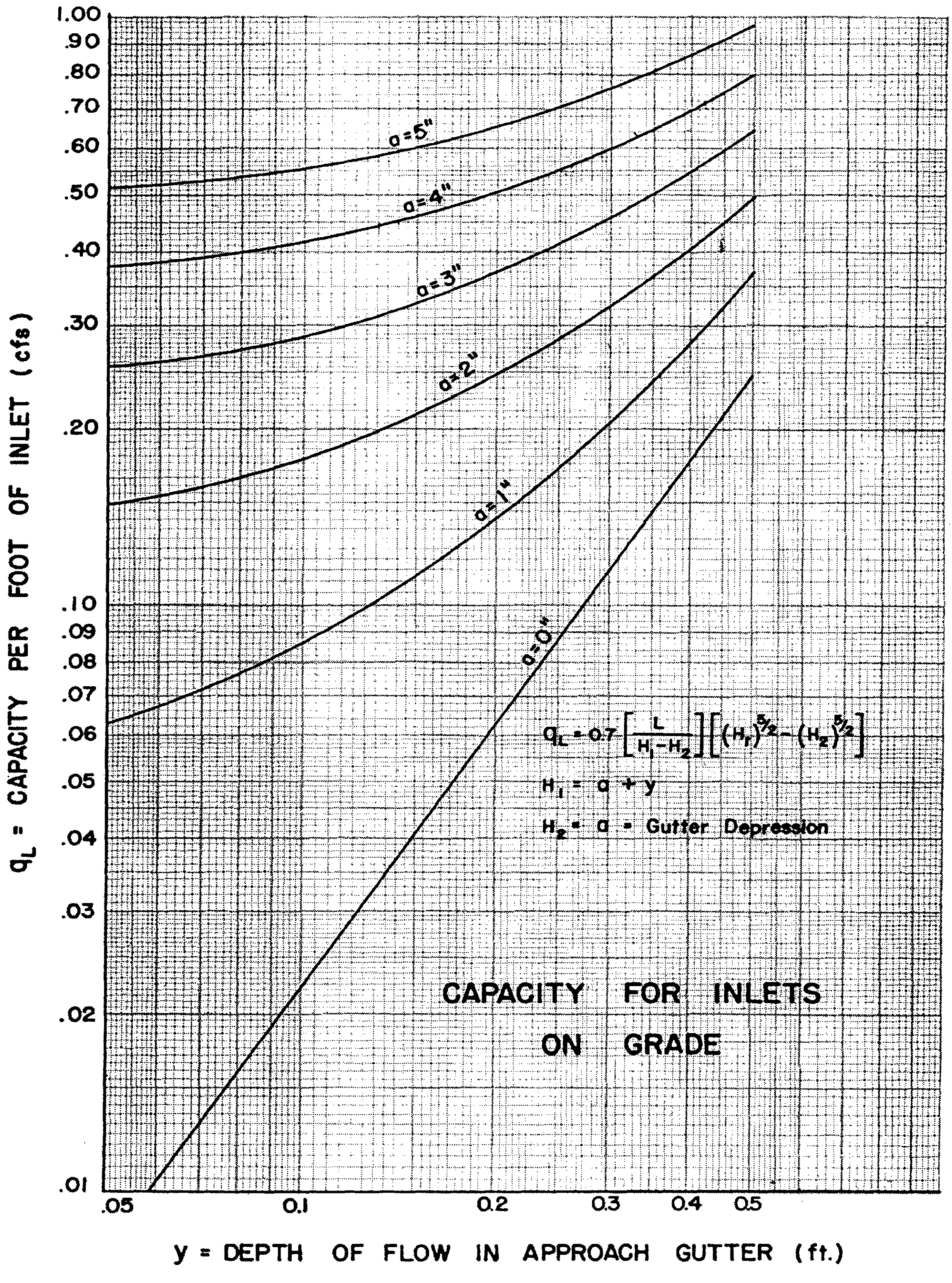
Column 18: Length – Length of opening of curb inlet or grate inlet (ft).

Column 19: Depression, "a" – Depth in inches that the throat of a curb or top of grate inlet is depressed below the normal gutter line, normally zero.

Column 20: Intercepted Flow – Capacity of the inlet or the flow the inlet will intercept (cfs).

Column 21: Carry Over – Flow that bypasses the inlet and proceeds to the next inlet (cfs).

Proceed to the next inlet, Column 1.



City of **RUSSELLVILLE** Arkansas	**CAPACITY OF CURB INLETS ON GRADE** **FIGURE IV-2**

Note: Computer solutions for gutter flow analysis are acceptable. The computer printout should contain the same information that is shown in Table IV-1.

3.2.4 Type and Spacing of Inlets

Both curb opening inlets, grate inlets, or a combination of curb opening and grate inlets may be used for intercepting runoff. Curb opening inlets are preferred because of their self-cleansing ability, as grate inlets clog easily. In some instances, however, the use of grates will be found necessary either with or without curb openings in combination. Where grates are used, the design and placing of the grates should be such that the grate bars will be parallel to the direction of flow of the water rather than perpendicular to the flow. Experiments have shown that this minimizes the clogging by small debris and increases the capacity of the grate. Construction details for various standard inlets are presented in the Arkansas Highway and Transportation Department (AHTD) Storm Drainage Manual. In the interest of standardization and resultant reduction in cost, it is recommended that these details be closely followed on all projects unless there is some particular reason for deviating therefrom and then only with prior written approval from the CITY ENGINEER.

As a general rule inlets should be placed at all low points in the roadway surface and at suitable intervals along long slopes as necessary to prevent excessive ponding or spread on any section of the roadway.

3.2.4.1 Curb Opening Inlets on Grade

The amount of water that will be intercepted by a curb inlet of given length, given depression, "a", and on a given gradient may be determined directly from Figure IV-2. It should be noted that it is not always necessary or desirable to intercept all the water in the gutter at any given point. It will often be found satisfactory and economical to allow a portion of the water to flow past an inlet to succeeding inlets. This quantity of water that passes an inlet is called "Carry-over" or "Bypass" and the proper handling thereof is covered in the illustrative problem included herein.

The following is an outline of the procedure for the design of curb inlets on grade:

1. Determine the following input data:

   a. Gutter flow rate – Q_G (cfs)
   b. Gutter (longitudinal) slope – S_G (ft/ft)
   c. Transverse slope – S_T (ft/ft)
   d. Roughness coefficient – n

2. D is the depth of water (or head) in the gutter immediately upstream of the curb inlet opening (feet). However, before this depth can be calculated, certain parameters must be set. In the case of street design, it is undesirable to have the street inundated and impassable due to the amount of runoff drainage down a

given street. Therefore, the maximum allowable top width of water flow, or spread, T, in the gutter and street must be regulated such that flooding does not occur. Maximum spread limits are presented in Section 2.4.

3. With the gutter discharge rate, Q_G, known, T can be solved by applying the modified Manning Equation:

$$\mathbf{Q_G = (0.56/n)ZD^{2.6667}} \qquad \textbf{(IV-5)}$$

Since $D = T(S_G)$

And $S_G = 1/Z$

T can be derived by:

$$\mathbf{T = [Q_G n(Z^{1.6667})/0.56(S_G^{\ 0.5})]^{2.6667}} \qquad \textbf{(IV-6)}$$

Where:

T = Top width, or spread, of water flow (ft)
Q_G = Gutter discharge to the inlet structure (cfs)
Z = Reciprocal of the transverse slope (ft/ft)
n = Roughness coefficient
S_G = Gutter (longitudinal street) slope (ft/ft)
D = Depth of water in the gutter (ft)

4. Once T is calculated and determined to be within its imposed limits, D can be calculated as follows:

$$D = T/Z$$

Where:

D = Depth of water in the gutter, upstream from the grate (ft)

5. The inlet capacity of the curb inlet, Q_I, can then be determined by using Figure IV-2. If 100% interception is not achieved, the overflow must be included in the next system.

3.2.4.1.1 Grate Inlets on Grade

The flow of water through grate openings may be treated in the same manner as flow of water through rectangular orifices. The formula in most general use for flow through orifices is stated as follows:

$$\mathbf{Q_G = CA/(gh)^{0.5}} \qquad \textbf{(IV-7)}$$

Where:

Q_G = Gutter discharge (cfs)
C = Coefficient of discharge (approximately 0.7)
A = Area of orifice (the net area of the openings in the grates, square feet)
g = Acceleration due to gravity (32.2 ft/sec^2)
h = Head on grate (ft)

This formula gives the theoretical capacity of the grate inlet. Since grate inlets are subject to considerable clogging it is recommended that for practical purposes the capacity of the grate inlet be taken as ½ of the value given by this formula, or conversely that the net area of the grate be twice as large as the theoretical area required when calculated by the above formula.

The following is an outline of the procedure for the design of grate inlets on grade:

1. Determine the following input data:

   a. Gutter flow rate – Q_G (cfs)
   b. Gutter (longitudinal) slope – S_G (ft/ft)
   c. Gutter (transverse) slope – S_T (ft/ft)
   d. Roughness coefficient – n

2. D is the depth of water (or head) in the gutter immediately upstream of the grate (feet). However, before this depth can be calculated, certain parameters must be set. In the case of street design, it is undesirable to have the street inundated and impassable due to the amount of runoff drainage down a given street. Therefore, the maximum allowable top width of water flow, or spread, T, in the gutter and street must be regulated such that flooding does not occur. Maximum spread limits are presented in Section 2.4.

3. With the discharge rate, Q_G, known, T can be solved by applying the modified Manning Equation IV-5:

   $$Q_G = (0.56/n)ZD^{2.667}$$

   Since $D = T(S_L)$

   and $S_T = 1/Z$

   T can be derived:

   $$\mathbf{T = [Q_G n(Z^{1.6667})/0.56(S_G^{0.5})]^{2.6667}} \qquad \textbf{(IV-8)}$$

Where:

T = Top width, or spread, of water flow (ft)
Q_G = Gutter discharge to the inlet structure (cfs)
Z = Reciprocal of the transverse slope (ft/ft)
n = Roughness coefficient
S_G = Gutter (longitudinal street) slope (ft/ft)

4. Once T is calculated and determined to be within its imposed limits, D can be calculated as follows:

   $D = T/Z$

   Where:

   D = depth of water in the gutter, upstream from the grate (ft)

5. The inlet capacity of the grate, Q_I, can then be determined by using Figure IV-3. If 100% interception is not achieved, the overflow must be included in the next system.

3.2.4.1.2 Combination Inlets on Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. The "1/2 capacity" rule should be ignored for this type of inlet configuration. Thus, capacity is computed by neglecting the curb-opening inlet and the design procedures should be followed based on Section IV-3.2.4.1.1.

3.2.4.1.3 Curb Opening Inlets in Sags

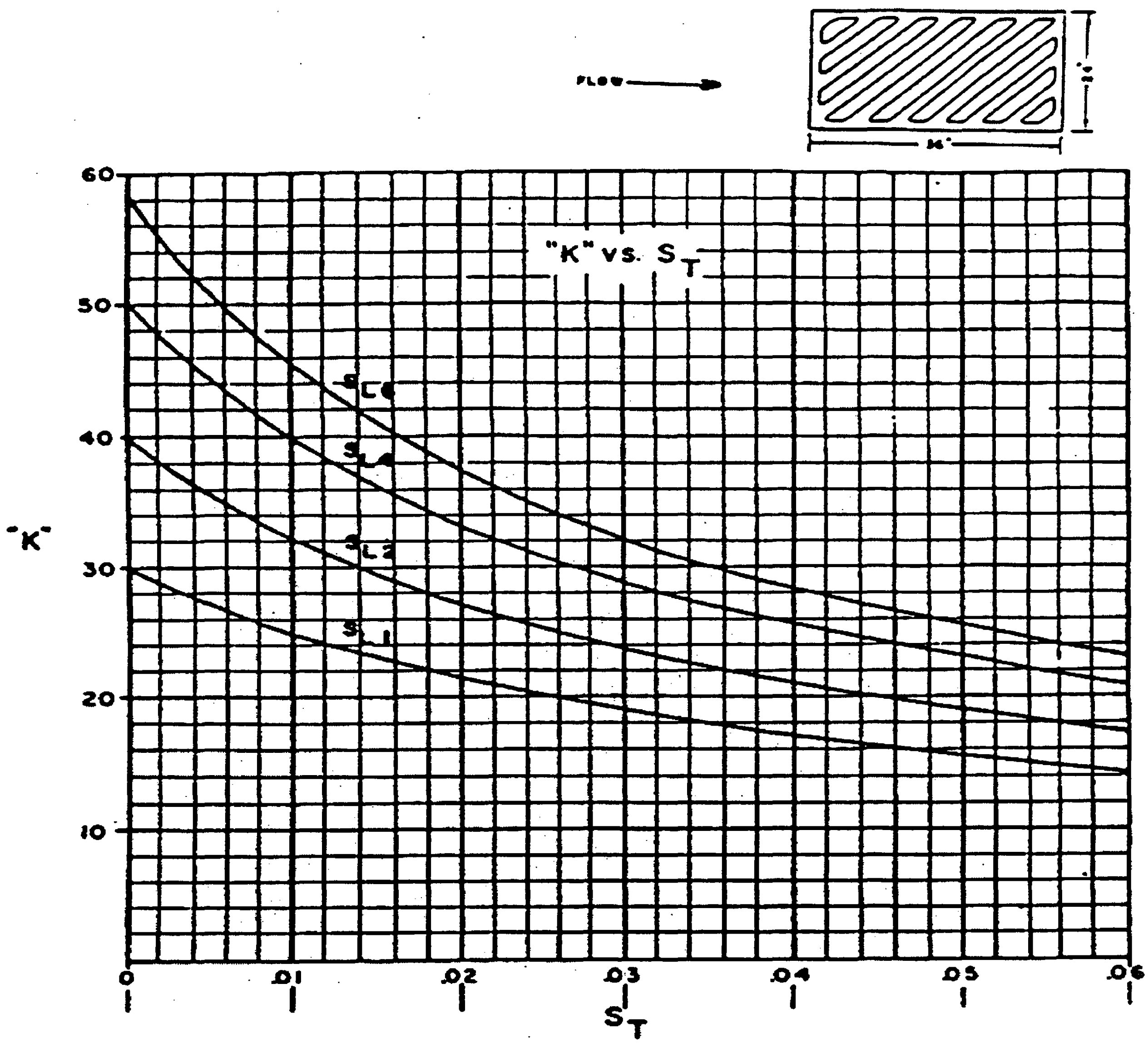
Under all ordinary conditions the flow of water through a curb-opening inlet located at a sag or low point in the grade may be computed by the weir formula:

$$\mathbf{Q_G = 3.087\ Lh^{1.5}} \qquad \textbf{(IV-9)}$$

Where:

Q_G = Gutter discharge (cfs)
L = Length of opening (ft)
h = Head or depth of water at the opening (ft)

The proper orifice formula should be used in computing the discharge rather than the weir formula where the depth of water is such that the curb inlet is completely submerged. This is a very rare condition, as the proportioning of inlets should be such as to preclude ponding in



S_T = TRANSVERSE GUTTER SLOPE

S_L = LONGITUDINAL GUTTER SLOPE

K = GRATE INLET COEFFICIENT

$Q = KD^{5/3}$

Where: D = depth of water in gutter, upstream from the grate (ft)
Q = discharge intercepted by grate (cfs)

Note: For $S_L > 6\%$, use the curve for $S_L = 6\%$

City of RUSSELLVILLE Arkansas	GRATE INLET COEFFICIENTS ON GRADE FIGURE IV-3

sufficient depth to submerge the inlet. Figure IV-4 presents a nomograph for determining the capacity of curb inlets in a sag.

3.2.4.1.4 Grate Inlets in Sags

Because a grate inlet in a sag or sump condition is subject to clogging, a curb opening is required as a supplemental inlet. The capacity of a grate in a sump depends upon the area of the openings and the depth of water at the grate; however, the capacity should be computed as described in Section IV-3.2.4.1.3. Figure IV-5 presents curves for the sizing of standard drop inlet grates.

3.2.4.1.5 Combination Inlets in Sags

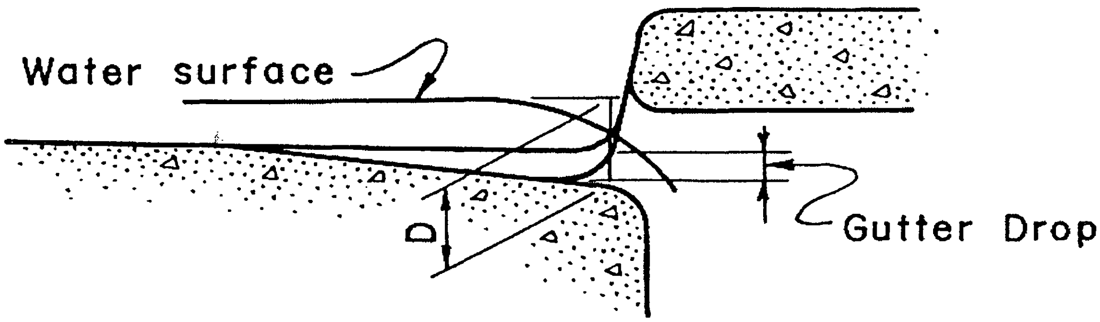
Combinations of curb slots and grate inlets may be used to advantage under certain special conditions, but such combinations are not recommended for general use. Wherever the curb opening type of inlet can be used it will generally prove to be more economical and more desirable than the grate type of inlet, and the use of grates should ordinarily be confined to those instances where it is impracticable to provide any curb opening at all or where the length of opening which can be provided is not sufficient to intercept all the water which must be cared for. The theory commonly advanced by proponents of combination curb slot-grate inlets is that floating debris will be carried on the surface of the water through the curb slot leaving the grate unclogged and hence operating at maximum efficiency. Investigations conducted by various hydraulic research laboratories, however, have not revealed any great gain resulting from such combinations. Authentic data on the true capacities of such combinations are insufficient to allow the establishment of any very accurate factors for determining the true capacity of a combination inlet.

For design purposes, however, it is believed reasonable to assume that the capacity of the combination inlet will run about 50% of the sum of the individual capacities of the grate and the curb slot, computed in the manner described in the preceding paragraphs. In other words, it is recommended that the capacity of the curb slot inlet and the capacity of the grate inlet (without reduction) be computed separately, added together, and the working capacity of the combination be taken as 50% of the total computed capacities.

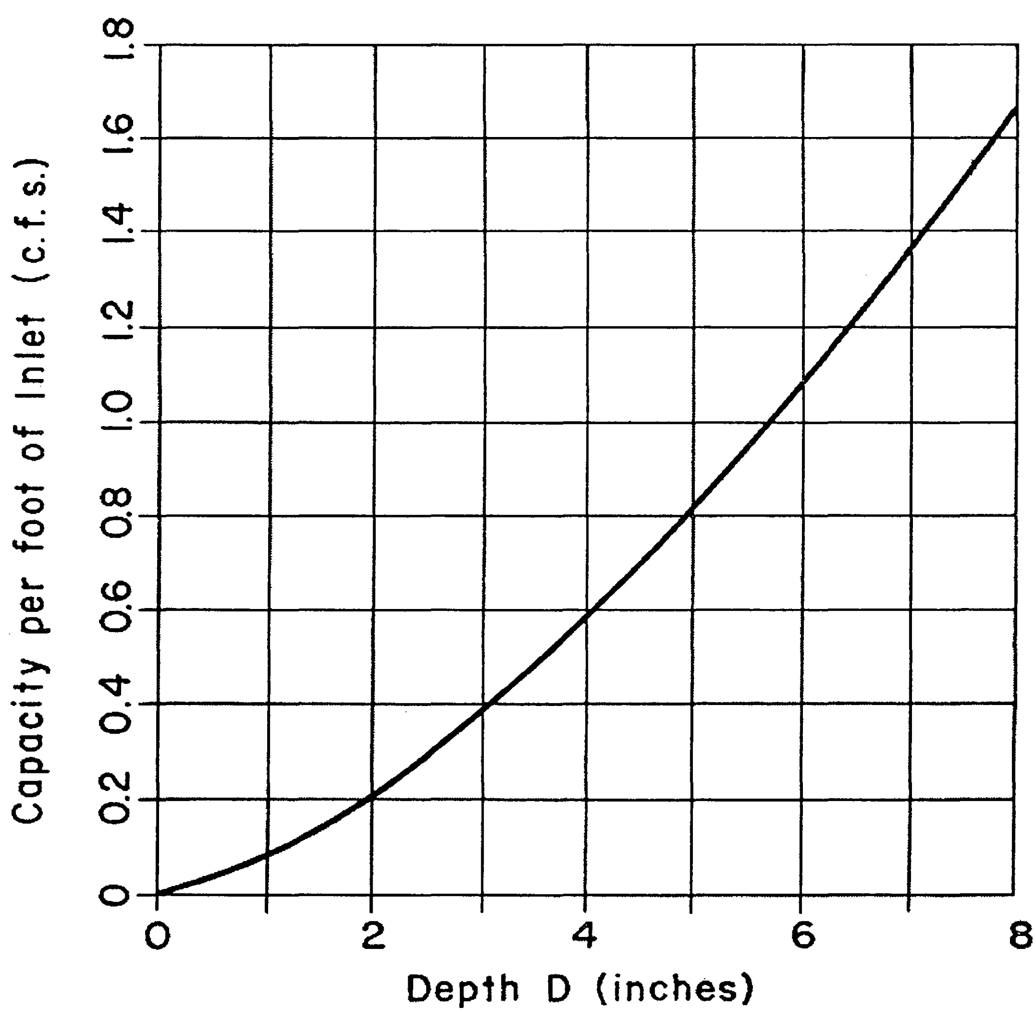
4.0 FLOW IN UNDERGROUND STORM SEWERS

4.1 General

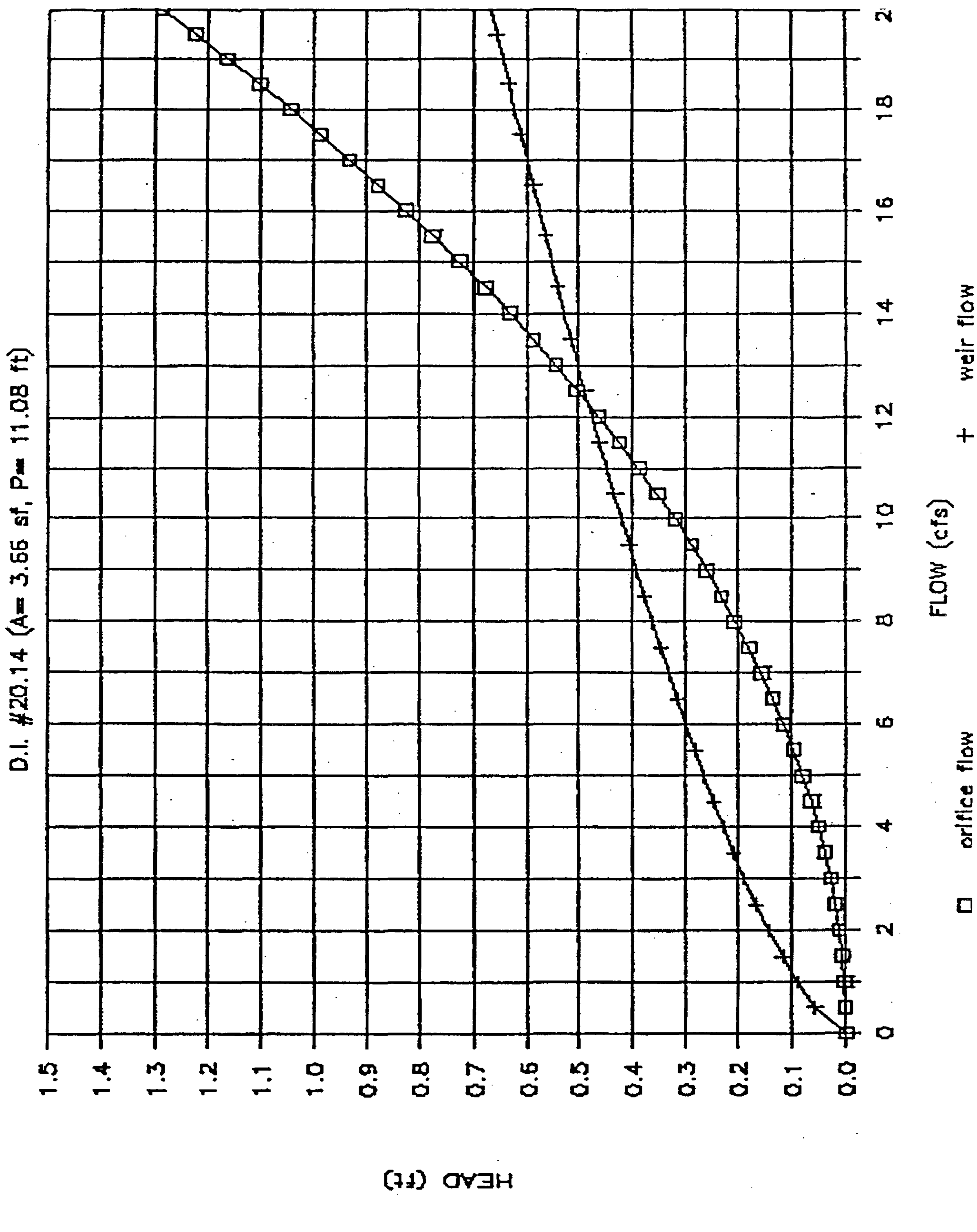
After the tentative locations of inlets, drain pipes, and outfalls with tailwaters have been determined and the inlets have been sized, the next logical step is the computation of the rate of discharge to be carried by each drain pipe and the determination of the size and gradient of pipe required to convey this discharge. This is done by proceeding in steps from upstream of a line to downstream to the point at which the line connects with other lines or the outfall, whichever is



SECTION



City of **RUSSELLVILLE** Arkansas	**CAPACITY OF LOW POINT INLETS** **FIGURE IV-4**



City of **RUSSELLVILLE** Arkansas	**WEIR AND ORIFICE FLOW CURVES STANDARD GRATE** **FIGURE IV-5**

applicable. The discharge for a run is calculated, the drainpipe serving that discharge is sized, and the process is repeated for the next run downstream. It should be recognized that the rate of discharge to be carried by any particular section of drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

The time of concentration is the time required for water to flow from the most distant point of the area drained to the point of the sewer system under consideration, and is generally made up of two components: the time required for water to flow from the most distant point of the drainage area to the inlet, which is known as the inlet time, and the time required for the water to flow through the pipe from that inlet to the point of the sewer line under consideration. In other words the time of concentration for any point on a sewer line is the inlet time for the inlet at the upper end of the line plus the time of flow through the pipe from the upper end of the sewer to the point in question, unless the time for another branch or inlet at that point is greater. In municipal areas the time of concentration is seldom less than 5 minutes or more than 20 minutes. A minimum time of concentration of 5 minutes is recommended for general use.

4.2 Storm Drain Pipes

4.2.1 Introduction

For ordinary conditions, storm drain pipes should be sized on the assumption that they will flow full or practically full (85% full) under the design discharge but will not be placed under pressure head. Manning's Equation is recommended for capacity calculations.

Storm sewer systems should generally be designed for non-pressure conditions. When hydraulic calculations do not consider minor energy losses such as expansion, contraction, bend, junction, and manhole losses, the elevation of the hydraulic gradient for design flood conditions should be at least 1.0 foot below surface inlet elevation. As a general rule, minor losses, should be considered when the velocity exceeds 6 fps (lower if flooding could cause critical problems). If all minor energy losses are accounted for, it is usually acceptable for the hydraulic gradient to reach 6 inches below the grate elevation.

4.2.2 Capacity

4.2.2.1 Formulas for Gravity and Pressure Flow

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is Manning's Equation and it is expressed by the following equation:

$$\mathbf{V = [1.486\ R^{0.6667}\ S^{0.5}]\ /n} \qquad \mathbf{(IV\text{-}10)}$$

Where:

V = Mean velocity of flow (fps)
R = Hydraulic radius (ft) - defined as the area of flow divided by the wetted flow surface or wetted perimeter
S = Slope of hydraulic grade line (ft/ft)
n = Manning's roughness coefficient

In terms of discharge, the above equation becomes:

$$\mathbf{Q = [1.486\ AR^{0.6667}\ S^{0.5}]/n} \qquad \textbf{(IV-11)}$$

Q = Rate of flow (cfs)
A = Cross sectional area of flow (ft^2)

For pipes flowing full, the above equations become:

$$\mathbf{V = [0.590\ D^{0.6667}\ S^{1/2}]/n} \qquad \textbf{(IV-12)}$$

$$\mathbf{Q = [0.463\ D^{.6667}\ S^{0.5}]/n} \qquad \textbf{(IV-13)}$$

D = diameter of pipe (ft)

The Mannings equation can be written to determine friction losses for storm drain pipes as:

$$\mathbf{H_f = (2.87\ n^2V^2L)/S^{1.333}} \qquad \textbf{(IV-14)}$$

$$\mathbf{H_f = (29\ n^2LV^2)/(R^{1.333})(2g)} \qquad \textbf{(IV-15)}$$

Where:

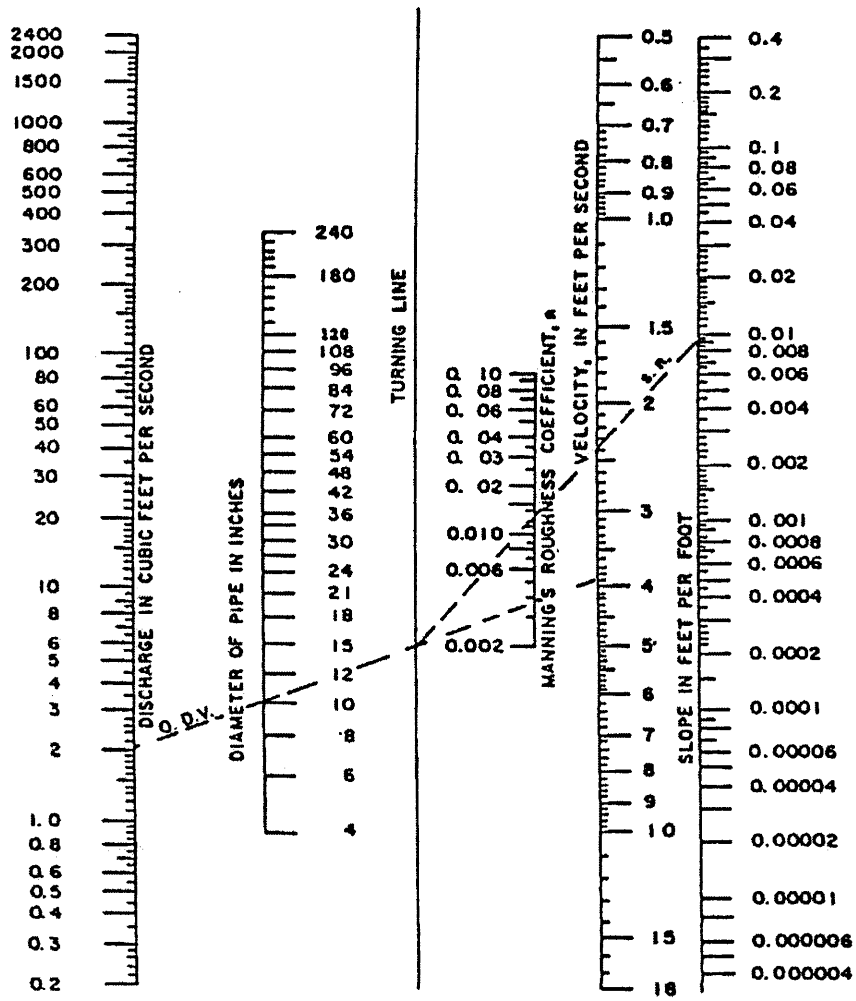
H_f = Total head loss due to friction (ft)
L = Length of pipe (ft)
g = Acceleration due to gravity = 32.2 ft/sec^2

4.2.2.2 Nomographs and Table

The nomograph solution of Manning's formula for full flow in circular storm drain pipes is shown on Figures IV-6, IV-7, and IV-8. Figure IV-9 has been provided to solve the Manning's equation for part full flow in storm drains.

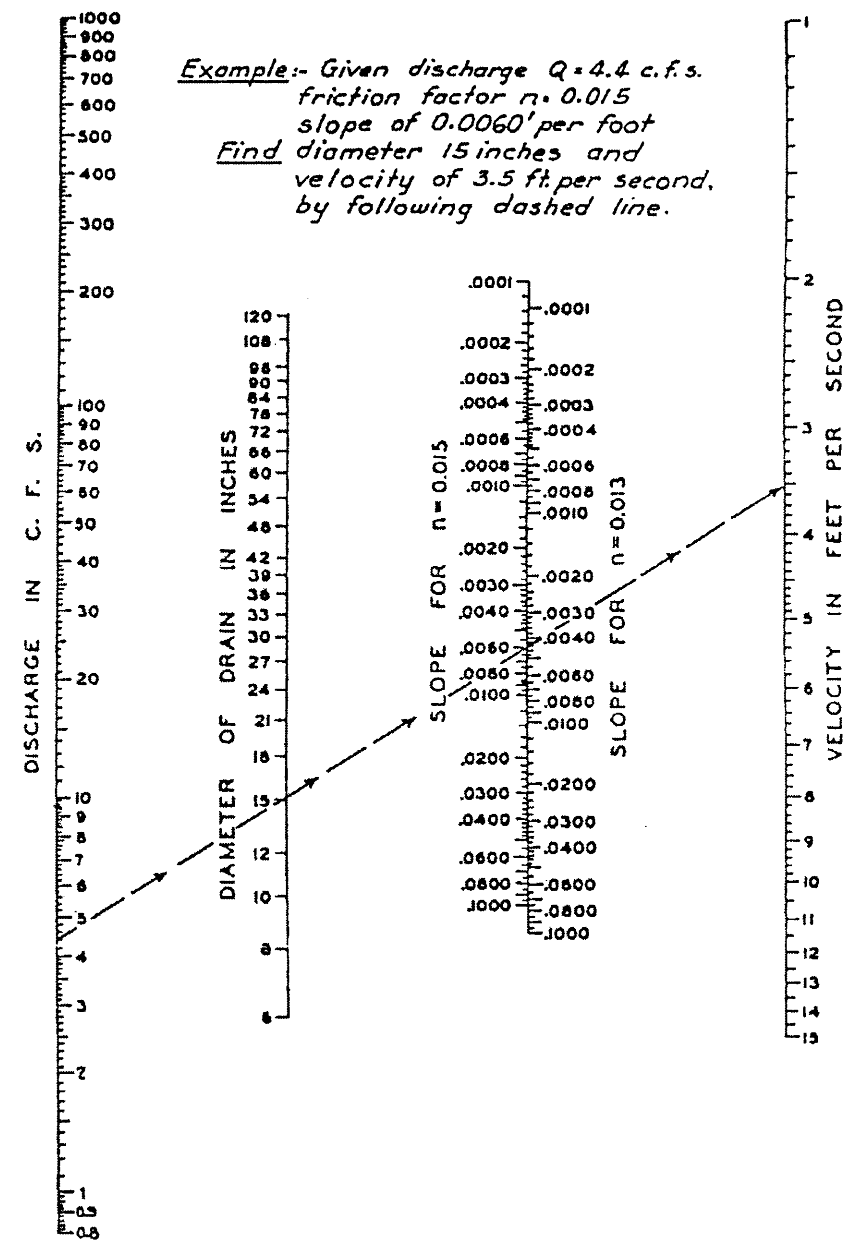
4.2.3 Determination of Pipe Sizes

After the locations of inlets, pipe runs, and outfalls have been determined and the inlets proportioned, the next step will be the computation of the quantity of water to be carried by each



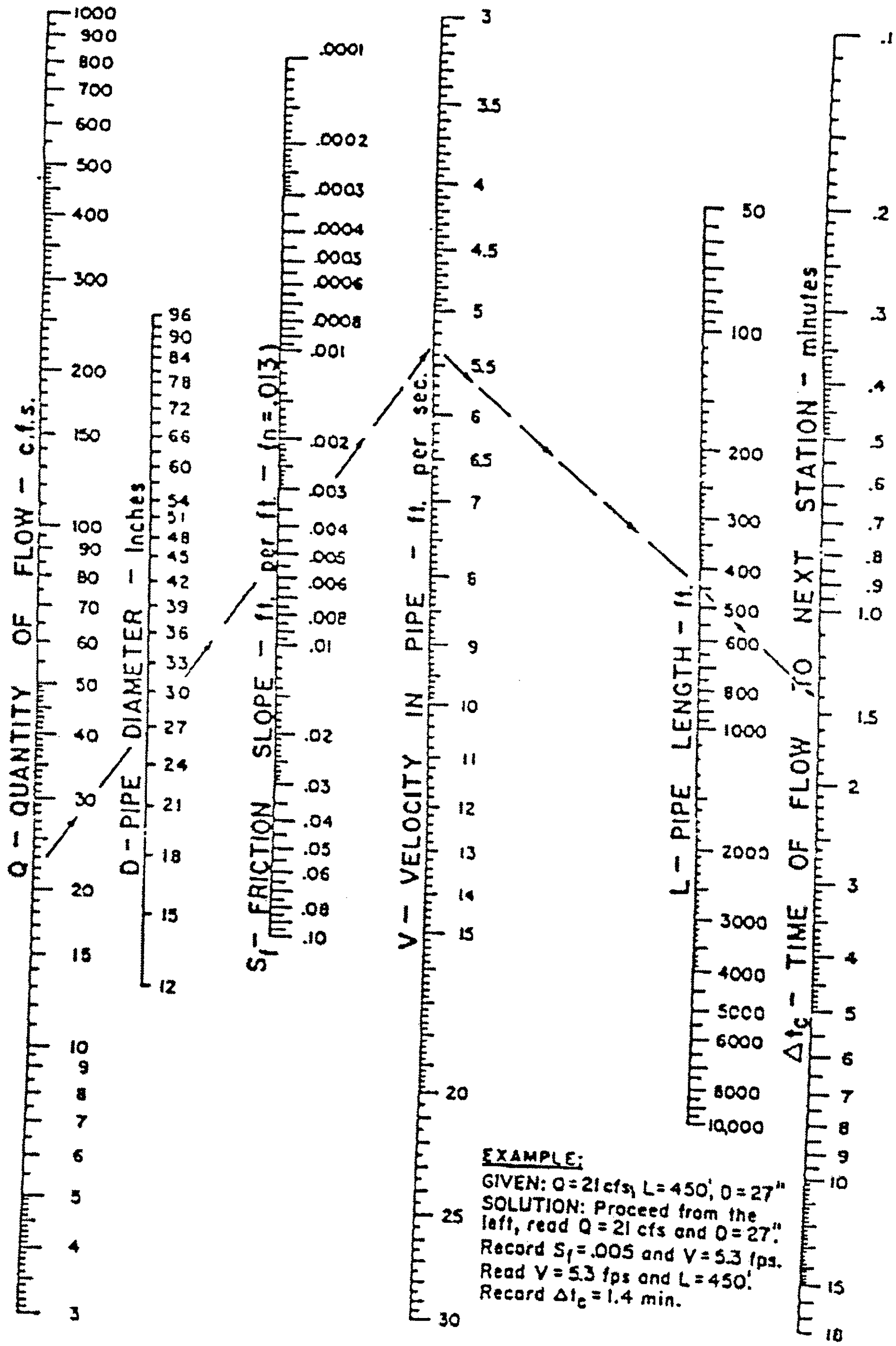
Source: AASHTO Model Drainage Manual, 1998

City of **RUSSELLVILLE** Arkansas	**NOMOGRAPH FOR SOLUTION TO MANNING'S COMPUTATIONS FOR STORM SEWER FLOW** **FIGURE IV-6**



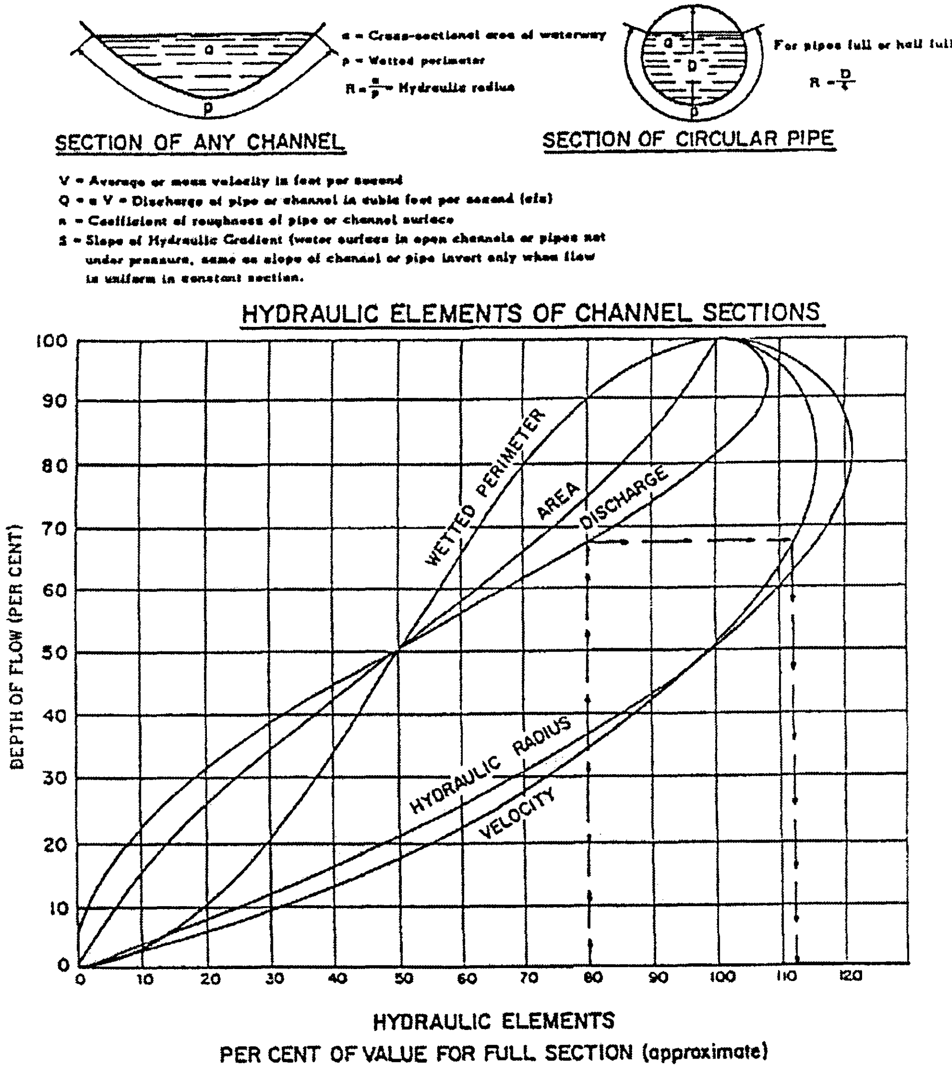
Source: AASHTO Model Drainage Manual, 1998

City of **RUSSELLVILLE** Arkansas	**NOMOGRAPH FOR COMPUTING CIRCULAR PIPE SIZE, FLOWING FULL – n= 0.015** **FIGURE IV-7**



Source: AASHTO Model Drainage Manual, 1998

City of **RUSSELLVILLE** Arkansas	**CONCRETE PIPE NOMOGRAPH**
	FIGURE IV-8



V = Average of mean velocity in feet per second
Q = Discharge of pipe or channel in cubic feet per second
S = Slope of hydraulic grade line

Source: AASHTO Model Drainage Manual, 1998

City of **RUSSELLVILLE** Arkansas	**HYDRAULIC ELEMENTS FOR CIRCULAR SECTIONS** **VARIOUS DEPTHS OF FLOW** **FIGURE IV-9**

pipe run and the determination of the size and gradient of pipe required to care for this water. It should be borne in mind that the quantity of water to be carried by any particular section of pipe is not necessarily the sum of the inlet design quantities of all inlets above that section of pipe, but as a general rule is somewhat less in amount than this total. This is due to the fact that the time of concentration enters into the picture and as the time of concentration grows larger the proper rainfall intensity to be used in the design grows smaller. In determining the quantity of flow in the design of any particular run of pipe, the time required for the water to flow from the most remote point on the drainage area should be computed and the corresponding value of rainfall intensity derived. The quantity is then calculated from the Rational Formula Q = CIA.

For all ordinary conditions pipes should be designed on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. Manning's Equation is recommended.

The following general rules should be observed in the proportioning of sewer pipes:

1. Do not use pipe sizes less than 18-inch diameter for main storm sewers or for long laterals and not less than 15-inch diameter for short laterals. Use only pipe material types approved by the CITY ENGINEER in determining Manning's "n" factor.

2. Place all pipes on such slope that the velocity of flow when full will not be less than 2 fps. Where slopes are comparatively flat it is desirable that the storm sewer sections and slopes be designed so that the velocity of flow will increase progressively, or at least will not appreciably decrease, in passing from the first inlet to the outlet of the sewer so that solids washed into the sewer and transported by the flowing stream will be carried on through and out of the sewer and will not be dropped at some point due to a sharp decrease in velocity.

3. Do not use non-standard sizes of pipe.

4. Do not discharge the contents of a larger pipe into a smaller one even though the capacity of the smaller pipe may be greater due to steeper slope.

5. At changes in size of pipe always place the soffits or top inside surfaces of the two pipes at the same level rather than placing the flow lines at the same level. Where flow lines are placed at the same level the smaller pipe must discharge against head and it will be necessary to plot hydraulic grade lines in order to determine the true discharge. Naturally this rule cannot be followed in every instance, but it should be adhered to wherever it is practicable to do so.

6. Where closed storm sewers cross between lots to continue into other facilities, a swale or approved equal shall be constructed over the storm sewer to provide the combined capacity of the 100-year peak flow. When storm sewers are designed for the 100-year peak flow and are not within a public street, an overflow easement shall be provided that prohibits structures from blocking the flow. The

easement shall be sized to pass the 100-year flow. The 100-year water surface elevation shall be plotted on the design drawings.

There will be no deviations from the rules stated above unless approved in writing by the CITY ENGINEER.

4.2.4 Design Procedure

The design of the underground portion of the storm drainage system may proceed as outlined in this section. The computations involved in the proportioning or sizing of the various runs of sewer pipe are summarized in the tabulation sheet in Table IV-2 titled “Storm Drainage Pipe Computation Sheet”. Data recorded in Table IV-1, “Inlet Computation Form”, will be needed to perform the following steps:

Columns 1 through 5: The values shown in these columns are believed to be self-explanatory.

Column 6: The length of each run as shown in this column is the length center to center of inlets or manholes. This length is used in determining the time of flow from one inlet or manhole to another. (Note that these lengths are not to be used as pay lengths of pipe since our standard specifications provide that pay lengths shall include only the actual net length of pipe and shall not include the distance across inlets or manholes where no pipe actually is placed.)

Columns 7, 8, and 9: The time of concentration is the time required for water to flow from the most remote part of the drainage area or areas involved to the upper end of the pipe run under consideration. For the first run the time of concentration is the inlet time for the first inlet. For all succeeding runs, the time of concentration may be either the time as computed along the sewer line or the inlet time of the inlet at the beginning of the run under consideration, depending upon which of these two periods is the longer. Accordingly, both times are shown in the tabulation for purposes of comparison and the larger of the two is used in determining I and Q, unless this larger value is less than 5 minutes in which case the established minimum time of 5 minutes is used.

The time of concentration shown in Column 7 is computed by taking the time of concentration for the preceding run and adding to it the time required for water to flow through the preceding run to the beginning of the run under consideration. At junctions of lines, the larger value of the time of concentration is used.

Columns 10 and 11: The value of I and Q are computed in the same manner as explained under “Inlet Computations”.

Columns 12, 13, and 14: The size and gradient of pipe as shown in columns 12 and 13 must be chosen in such manner that the pipe when flowing full, but not under head, will carry an amount of water approximately equal to or greater than the computed discharge, Q. In other words, the

STORM DRAINAGE PIPE COMPUTATION SHEET

From	To	Drainage Area No.	Total D.A. Ac.	Total C.A.	Length ft.	Time of Concentration - Minutes: Along Sewer Line	Time of Concentration - Minutes: Inlet Time	Time of Concentration - Minutes: Used in Design	I In./Hr.	Q cfs	Design: Dia. In.	Design: Slope %	Design: Cap. cfs	Design: Vel. Ft/sec	Remarks
1	2	3	4	5	6	7	8	9	10	11	15	16	17	18	20

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STORM DRAINAGE PIPE COMPUTATION SHEET

TABLE IV-2

"Capacity" shown in Column 14 must be approximately equal to or greater than the value Q shown in Column 11.

The capacity may be calculated by Manning's formula, $Q = 1.486/nAR^{0.667}\ S^{0.5}$. For circular concrete pipes, the capacity may be taken directly from Figure IV-6.

Column 15: The velocities shown in this column are taken from Figure IV-8 and are used in Column 7 in determining the time of flow through each run of pipe.

4.2.4.1 Manholes

Manholes or combination manholes and inlets should be placed wherever necessary for clean out and inspection purposes. It is good engineering practice to place manholes at changes in pipe gradients, changes in pipe sizes, changes in direction, and at junctions of pipe runs. Manholes should not be constructed further than 500 feet apart in long runs where the size or direction is not changed. The invert of the manhole section should be rounded to match the inverts of the pipes entering the manhole in order to reduce eddying and resultant head losses. At junctions of sewer lines, right angle intersections should be avoided if possible and the two lines should be brought together at an acute angle to minimize head losses.

5.0 HYDRAULIC GRADE LINE (HGL)

5.1 General

The final step in designing a storm drain system is to check the hydraulic grade line (HGL) as described below. Computing the HGL will determine the elevation, under design conditions, to which water will rise in various inlets, manholes, junctions, and etc.

On Figure IV-10 on the next page is a summary of energy losses which should be considered. Following this on Figure IV-11 is a sketch showing the proper and improper use of energy losses in developing a storm drain system.

5.2 Procedure

In calculating the hydraulic grade line within a closed storm sewer system, all head losses shall be computed to determine the water surface elevation within various structures.

The calculations are begun at the upstream or downstream opening, dependent upon whether the pipe is in inlet or outlet control. If it is inlet control the hydraulic grade line is the headwater elevation minus the entrance loss and the difference velocity head. If the outlet controls, the tailwater surface elevation or 0.8 times the diameter of the pipe, whichever is higher, is the outlet hydraulic grade line. Hydraulic grade lines will be required only as requested on a case by case basis.

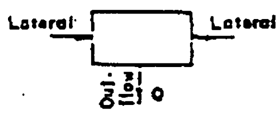
$$H_{tm} = \frac{V^2}{2g}$$

TERMINAL JUNCTION LOSSES
(at beginning of run)

Where g = gravitational constant, 32.2 feet per second per second.

$$H_e = 0.5\,\frac{V^2}{2g}$$

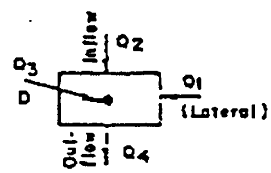
ENTRANCE LOSSES
(for structure at beginning of run)
Assuming square-edge



$$H_{j1} = \frac{V^2 \text{(Outflow)}}{2g}$$

JUNCTION LOSSES

Use only where flows are identical to above, otherwise use H_{j2} Equation.

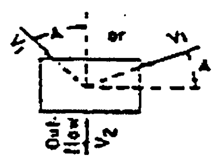


$$H_{j2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ_1V_1^2}{2gQ_4}$$

JUNCTION LOSSES
(After FHWA)

Total losses to include H_{j2} plus losses for changes in direction of less than 90° (H_b).

Where K = Bend loss factor (Figure 5-6 page 5-38)
Q_3 = Vertical dropped-in flow from on inlet
V_3 = Assumed to be zero



$$H_b = \frac{KV_1^2}{2g}$$

BEND LOSSES
(changes in direction of flow)

Where K	Degree of Turn in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSS (H_f)

$H_f = S_f \times L$

Where H_f = friction head
S_f = friction slope
L = length of conduit

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

Where Q = discharge of conduit
n = Mannings coefficient of roughness (use 0.013 for R.C. Pipes)
A = area of conduit
R = hydraulic radius of conduit (D/4 for round pipe)

TOTAL ENERGY LOSSES AT EACH JUNCTION

$H_T = H_{tm} + H_e + H_{j1}$ or $H_{j2} + H_b + H_f$

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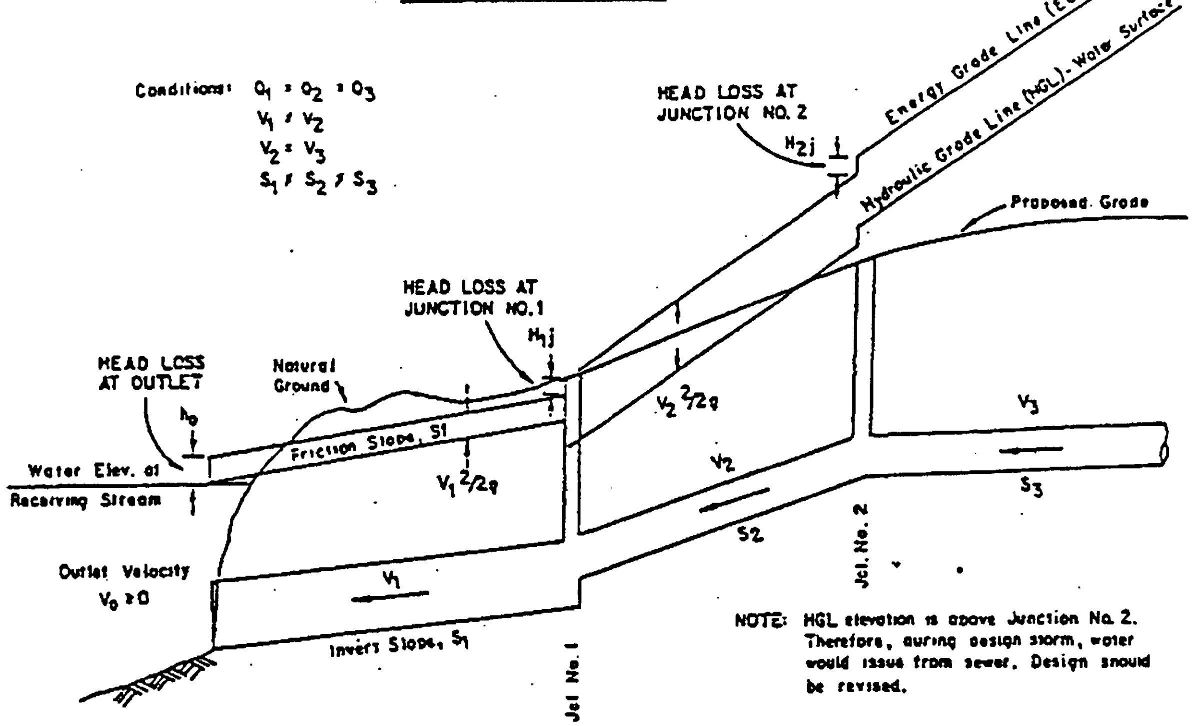
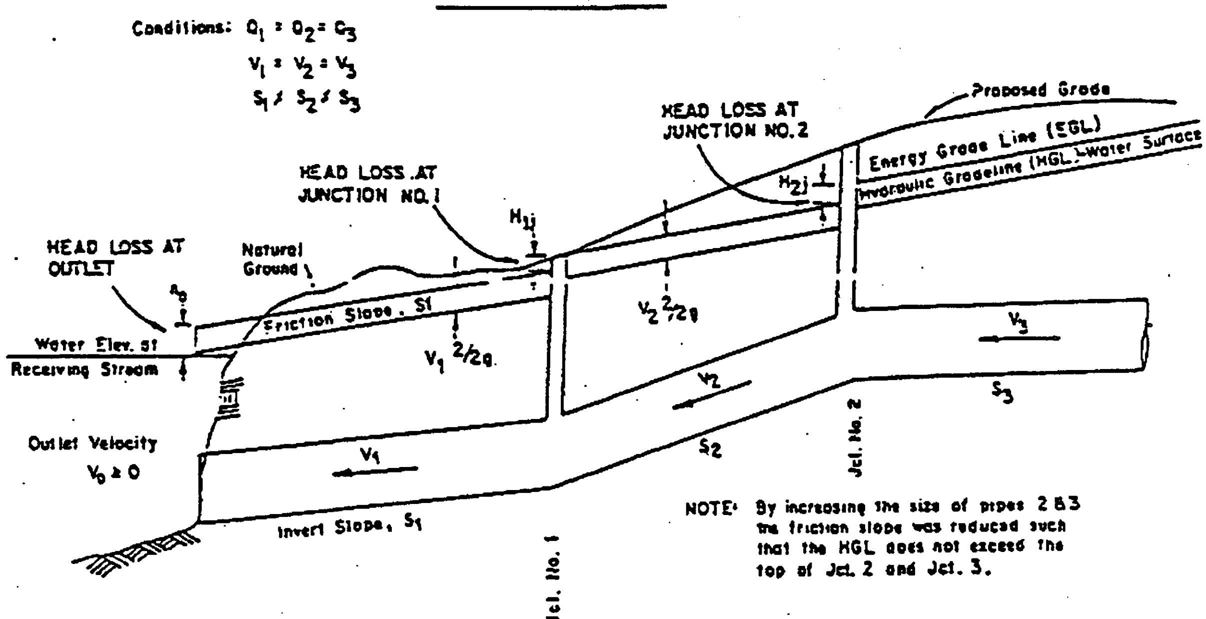
SUMMARY OF ENERGY LOSSES

FIGURE IV-10

IMPROPER DESIGN



PROPER DESIGN



City of **RUSSELLVILLE** Arkansas	**PROPER vs. IMPROPER DESIGN** **FIGURE IV-11**

The head losses are calculated beginning from the control point to the first junction and the, procedure is repeated for the next junction. The computation for the hydraulic grade line with outlet control may be tabulated using Table IV-3 and the following procedure:

Column 1: Enter the station for the junction immediately upstream of the outflow pipe. Hydraulic grade line computations begin at the outfall and are worked upstream taking each junction into consideration.

Column 2: Enter the outlet water surface elevation or 0.8 diameter plus invert outlet elevation of the outflow pipe whichever is greater.

Column 3: Enter the diameter (D_0) of the outflow pipe.

Column 4: Enter the design discharge (Q_0) for the outflow pipe.

Column 5: Enter the length (L_O) of the outflow pipe.

Column 6: Enter the friction slope (S_f) in ft/ft of the outflow pipe. This can be determined from the following formula:

$$S_f = (Qn/(1.486AR^{2/3}))^2 \qquad \textbf{(IV-16)}$$

Multiply the friction slope (S_f) in Column 6 by the length (L_0) in Column 5 and enter the friction loss (H_f) in Column 7.

$$H_f = S_f L_O \qquad \textbf{(IV-17)}$$

Column 8: Enter the velocity of the flow (V_O) of the outflow pipe.

Column 9: Enter the contraction loss (H_C) by using the formula:

$$H_C = 0.25(V_O^2)/2g \qquad \textbf{(IV-18)}$$

Column 10: Enter the design discharge (Q_1, Q_2, Q_3 .....) in cfs for each pipe flowing into the junction. Neglect lateral pipes with inflows of less than 10% of the mainline outflow.

Column 11: Enter the velocity of flow (V_1, V_2, V_3 .....) in fps for each pipe flowing into the junction (for exceptions see Step 10).

Column 12: Enter the product of (Q_1 V_1, Q_2 V_2, Q_3 V_3) for each incoming pipe. When several pipes inflow into a junction, the line producing the greatest (QV) product is the line which will produce the greatest expansion loss (H_C). (For exceptions, see Step 10).

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HYDRAULIC GRADE LINE COMPUTATION SHEET

TABLE IV-3

Hydraulic Grade Line

Project________________________________

Station	Outlet Water Surface Elev	D_o	Q_o	L_o	S_f	H_f	Junction Loss												Final H	Inlet Water Surface Elev	Rim Elev
							V_o	H_o	Q_I	V_i	$Q_I\ V_i$	$\frac{V_i^2}{2g}$	H_i	Angle	$H_\blacktriangle$	H_t	$1.3\ H_t$	$0.5\ H_t$			
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

Note: Final $H = H_f + H_t$ $\quad H_i = 0.35\ (V_i^2/2g)$ $\quad H_o = 0.25\ (V_o^2/2g)$ $\quad H_\blacktriangle = K\ (V_i^2/2g)$ $\quad H_t - H_i + H_o + H_\blacktriangle$

90 ° K = 0.70 45 ° K = 0.47
70 ° K = 0.64 30 ° K = 0.35
60 ° K = 0.56 15 ° K = 0.19

Column 13: Enter the product of ($V_1^2/2g$, $V_2^2/2g$, $V_3^2/2g$ …) for each incoming pipe.

Column 14: Enter the controlling expansion loss (H_E) using the formula:

$$H_E = 0.35(V_1^2)/2g. \qquad \textbf{(IV-19)}$$

Column 15: Enter the angle of skew of each incoming pipe to the outflow pipe (for exception, see Step 10).

Column 16: Enter the greatest bend loss (H_B) calculated by using the formula:

$$H_B = KV_1^2/2g \qquad \textbf{(IV-20)}$$

Where:

K = the bend loss coefficient corresponding to the various angles of skew of the incoming pipes.

Values of K for Change in Direction of Flow in Lateral	
K	**DEGREES OF BEND (IN JUNCTION)**
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90 and greater

K values for other degree of bends can be obtained by interpolating between values.

Column 17: Enter the total head loss (H_t) by summing the values in Column 9 (H_0), Column13 (H_1), and Column 15 (H_B).

Column 18: If the junction incorporates adjusted surface inflow of 10% or more of the mainline outflow, i.e., drop inlet, increase H_t by 30% and enter the adjusted H_t.

Column 19: If the junction incorporates full diameter inlet shaping, such as standard manholes, reduce the value of H_t by 50% and enter the adjusted value.

Column 20: Enter the FINAL H, the sum of H_f and H_t, where H_t is the final adjusted value of the H_t.

Column 21: Enter the sum of the elevation in Column 2 and the final H in Column 20. This elevation is the potential water surface elevation for the junction under design conditions.

Column 22: Enter the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Column 21. If the potential water surface elevation exceeds the rim elevation or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the HGL.

Repeat the procedure starting with Step 1 for the next junction upstream.

The above method of checking the hydraulic gradient does not take into account all items such as head losses in inlets, head losses or gains due to changes in velocity, etc., but it is considered accurate enough for all practical purposes so long as the hydraulic gradient is kept well below the throats of inlets.

6.0 RELATED DRAINAGE FEATURES

6.1 General

For street drainage there are often other features that must be considered in the overall design of a properly designed system. The following sections present a brief discussion of additional features that must be considered.

6.1.1 Median Ditches

Large median areas and inside shoulders should be sloped to a center swale, preventing drainage from the median area from running across the pavement. This is particularly important for high-speed facilities, and for facilities with more than two lanes of traffic in each direction.

6.1.2 Roadside Ditches

Roadside ditches (when allowed) will be required behind the shoulder of roadways without curb and gutter to convey storm drainage away from the pavement to a discharge point. The ditch shall comply with the criteria presented in DIVISION III.

In addition to the design of roadside ditches, a design shall be provided for driveway culverts for each individual lot on the plan. Driveway culverts shall be designed according to criteria presented in DIVISION III.

6.1.3 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal. Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 1%. When bridges are placed at a vertical curve and the longitudinal slope is less than 1%, the gutter spread should be checked to ensure a safe, reasonable design.

Scuppers are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. However, the use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over embankments, slope protection, navigation channels, driving lanes, or railroad tracks. Runoff collected and transported to the end of the bridge should generally be collected by inlets and down drains.

For situations where traffic under the bridge or environmental concerns prevent the use of scuppers, grated bridge drains should be used.

6.1.4 Median Barriers

Weep holes are often used to prevent ponding of water against median barriers (especially on super elevated curves). In order to minimize flow across traveled lanes, it is preferable to collect the water into a subsurface system connected to the main storm drain system.

DIVISION V. DEFINITIONS, ABREVIATIONS, AND SYMBOLS

1.0 DEFINITIONS

The following is a list of terms and their meaning as used in this MANUAL and as shall be used in the submission of all materials required by this MANUAL and the Russellville Stormwater Management Drainage Ordinance. For the purpose of this Drainage Manual, the following terms, phrases and words, and their derivatives, shall have the meaning given herein. The use of any other terms to represent definitions expressed herein shall be approved by the CITY ENGINEER before their use.

1. **Acre-foot** – the equivalent volume of water 1 foot deep on 1 acre or 43,560 cubic feet.
2. **As-built plan** – a set of engineering or site drawings that delineate the specific permitted stormwater management facility as actually constructed.
3. **Backwater** – water that is "backed" from downstream due to various forces resisting its conveyance in a channel/floodplain.
4. **Base Flood** – the flood that has a 1% chance of being equaled or exceeded in any given year, often referred to as the 100–year frequency flood.
5. **Base Flood Elevation** – the elevation of the water surface produced by the flood that has a 1% chance of being equaled or exceeded in any given year, often referred to as the 100-year frequency flood. Base Flood Elevations, **BFE**'s, are shown on Flood Insurance Rate Maps to denote the elevation of the base flood at that approximate location on the floodplain.
6. **Best Management Practices** – a wide range of management procedures, schedules of activities, prohibitions on practices, and other management practices which have been demonstrated to effectively control the quality and/or quantity of stormwater runoff and which are compatible with the planned land use.
7. **Bypass** – flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade.
8. **Channel** – a watercourse, either earthen, grassed, or concrete , which has definite bed and banks and conveys water either continuously or intermittently.
9. **City Engineer** – the duly designated Head of the Engineering Department or department of public works, or his duly authorized agent.
10. **City Engineering Department** – the department responsible for all stormwater management activities and implementation of the provisions of this ordinance.
11. **Combination Inlet** – A drainage inlet composed of a curb-opening and a grate. This is a Charlotte-Mecklenburg standard catch basin on roadways.
12. **Critical depth** – the depth of flow for which the specific energy is at a minimum.
13. **Cross-drain culvert** – a culvert located under a roadway.
14. **Curb-opening inlet** – A drainage inlet consisting of an opening in the roadway curb.
15. **Curve Number** – a unitless number, ranging from 1 to 100, representing the percentage of precipitation that actually runs off of the drainage area and reaches the point of analysis. It is dependent on location, topography, soil type, vegetative cover, land uses, and surface characteristics of the drainage area. It is used typically in performing runoff computations using NRCS methodologies.

16. **Design report** – the report that accompanies the Stormwater Management Plan and includes data used for engineering analysis, results of all analysis, design and analysis calculations (including input files and results obtained from computer programs), and other engineering data that would assist the City Engineer in evaluating proposed stormwater management facilities.
17. **Designer** – a professional who is permitted to prepare plans and studies required by this ordinance.
18. **Detention** – the collection of water during a runoff event for release at a controlled rate.
19. **Detention structure** – a permanent stormwater management structure whose primary purpose is to temporarily store stormwater runoff and release the stored runoff at controlled rates.
20. **Development** – generally, any of the following actions undertaken by a public or private individual or entity:
    a. the division of a lot, tract or parcel of land into two (2) or more lots, plots, sites, tracts, parcels or other divisions by plat or deed, or
    b. any land change, including, without limitation, clearing, tree removal, grubbing, stripping, dredging, grading, excavating, transporting and filling of land.
21. **Develop land** – to change the runoff characteristics of a parcel of land in conjunction with residential, commercial, industrial, or institutional construction or alteration.
22. **Developed land use conditions** – the land use conditions according to the current City Land Use Map or proposed development plan.
23. **Discharge** – the rate of flow, measured in volume per unit of time (i.e., cubic foot per second) in a channel.
24. **Drop inlet** – A drainage inlet with a horizontal or nearly horizontal grate.
25. **Duration** – the total length of time of a storm event, usually measured in hours.
26. **Easement** – a grant or reservation by the owner of land for the use of such land by others for a specific purpose or purposes, and which must be included in the conveyance of land affected by such easement.
27. **Energy Grade** – the sum of the hydraulic grade, which is the sum of the pressure head (p/γ) and the elevation head (z) at any given point along a channel reach, and the velocity head at that point.
28. **Energy Grade Line** – the "line" representing the total energy change between two points of flow along a channel reach.
29. **Equivalent cross slope** – An imaginary continuous cross slope having conveyance capacity equal to that of the given compound cross slope.
30. **Erosion** – the wearing away of land surface by the action of wind, water, gravity, ice, or any combination of those forces.
31. **Erosion and sediment control** – the control of solid material, both mineral and organic, during a land disturbing activity to prevent its transport out of the disturbed area by means of wind, water, gravity, or ice.
32. **Excess runoff** – the depth of precipitation, usually measured in inches, of the portion of the total precipitation from a storm event that exceeds the initial losses and infiltration/evaporation losses.
33. **Existing land use conditions** – the land use conditions existing at the time of the most recent official aerial photography available from the City.

34. **FHBM** – an official map (Flood Hazard Boundary Map) published by FEMA with a Flood Insurance Study (FIS) that reflects the boundary or limits of the 100– and 500–year floods.
35. **FIRM** – an official map (Flood Insurance Rate Map) published by FEMA with a Flood Insurance Study (FIS) that reflects the areas of special flood hazards and the risk premiums zones applicable to a community or county. It should include the boundary of the 100– and 500–year floods, any floodways designated, survey reference marks (RM), control section locations, and equal elevation lines (BFE) for the base flood.
36. **Flanking Inlets** – Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets is to intercept debris as the slope decreases and to act in relief of the inlet at the low point.
37. **Flood** – the runoff resulting from a meteorological event that can not be conveyed by a natural or manmade channel.
38. **Flood Insurance Study** – an official report provided by the Natural and Technological Hazard Division of the Federal Emergency Management Agency (FEMA). For specified stream reaches within a community or county, it includes:
    a. the results of a detailed study of the flooding problems within those reaches;
    b. the hydrologic analysis of expected discharge magnitudes for the 10–, 50–, 100–, and 500–year flood events for each study stream;
    c. the hydraulic analysis of the expected flood levels for each frequency event for each stream studied;
    d. the floodways determined to be reserved to adequately convey the base flood;
    e. plotted flood profiles for each study stream and frequency flood studied;
    f. a tabular listing of floodway and flood hazard data;
    g. a Flood Hazard Boundary Map (FHBM); and
    h. a Flood Insurance Rate Map (FIRM).

    In newer Flood Insurance Studies, all mapping is combined on the FIRM.
39. **Floodplain** – an area of land that is susceptible to being inundated by a flow that can not be conveyed within the banks of a channel. A common floodplain referenced in this manual is the area inundated by the base flood (100–year) as shown on maps prepared by the Federal Emergency Management Agency (FEMA).
40. **Floodplain Encroachments** – a significant obstruction within the floodplain that would result in an increase in the water surface elevation, particularly of the base flood.
41. **Flood Profile** – a plot of water surface elevation versus stream distance for a given frequency flood.
42. **Flood Stage** – the level or depth of water in a channel at a specific location just prior to overtopping the channel banks.
43. **Flood Storage Area** – the area outside of a designated floodway which permits the storage of water during a flooding event.
44. **Floodway** – the area of the floodplain, including the channel and adjacent overbank areas, that must be reserved to convey the base flood (100–year) without cumulatively increasing the water surface elevation more than 1 foot.
45. **Four percent (4%) annual chance storm** – a storm that is capable of producing rainfall expected to have a 4% chance of being equaled or exceeded in any given year, often referred to as the 25–year storm.

46. **Frequency** – a statistical representation of how often a particular event may be equaled or exceeded.
47. **Friction head** – the energy or force resisting the flow of water in a channel due to the roughness of the channel.
48. **Frontal flow** – the portion of the gutter which passes over the upstream side of a grate.
49. **Froude Number** – a unitless value that represents the relationship of actual fluid velocity to wave celerity – the speed at which waves will travel outward when a rock is tossed into the water.
50. **Grading** – excavating, filling (including hydraulic fill), or stockpiling of earth material, or any combination thereof, including the land in its excavated or filled condition.
51. **Grate perimeter** – the sum of the lengths of all sides of a grate, except that any side adjacent to a curb is not considered a part of the perimeter in weir flow computations.
52. **Gutter** – that portion of the roadway section adjacent to the curb which is utilized to convey storm water runoff.
53. **Head loss** – the difference, denoted in units of feet, between the Energy Grade at two points along a reach of channel flow.
54. **Headwater** – water that is "piled up" at an upstream point along a channel/floodplain that is pushing the water downstream.
55. **Hydraulic Grade** – the sum of the pressure head (p/γ) and the elevation head (z) at any given point along a channel reach.
56. **Hydraulic Grade Line** – the "line" representing the water surface between two points of flow along a channel reach or the set of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run.
57. **Hydraulics** – the branch of science that deals with the mechanics of the movement of fluid (water in this manual) along a surface or channel.
58. **Hydrology** – a science of the water cycle or the interrelationship between water on and under the earth's surface and in the atmosphere
59. **Hydro–35** – a publication of U. S. Weather Service that is a series of isohyetal plots of mass rainfall depths for various rainfall events with frequencies from 1 to 100 years and durations of 5 minutes to 60 minutes for approximately the eastern half of the continental United States.
60. **IDF Curve** – an Intensity–Duration–Frequency curve which is a plot of the relationship of rainfall intensity and storm duration for a specific frequency storm event at or near a specific location. IDF curves can be developed for any location from mass rainfall data derived from TP–40 and Hydro–35 mass rainfall data.
61. **Impervious** – the condition of being impenetrable by water.
62. **Imperviousness** – the degree to which a site is impervious.
63. **Infiltration** – the passage or movement of water through the soil profile.
64. **Infiltration/evaporation losses** – the rate of loss of precipitation, usually measured in inches per hour, that occurs during a runoff event due to infiltratin of water into the earth's subsurface and to evaporation to the earth's atmosphere. For a total storm event, these losses are usually measured in inches.
65. **Initial loss** – the depth of precipitation, usually measured in inches, that is retained in depressions and on surfaces prior to the actual beginning to runoff.
66. **Inlet efficiency** – the ratio of flow intercepted by an inlet to the total flow in the gutter.

67. **Intensity** – the average rate of precipitation occurring during a storm event, usually measured in inches per hour.
68. **Interior culvert** – a culvert that is not located under a roadway.
69. **Interception** – the storage of rainfall due to vegatative foliage and other surfaces that intercepts rain as it falls toward the earth.
70. **Lag time** – the time between the center of the rainfall event and the peak of the runoff hydrograph.
71. **Laminar flow** – flow that is characterized by smooth flow lines, such as the appearance of a heavy liquid being poured.
72. **Land disturbing activity** – any use of the land by any person that results in a change in the natural cover or topography that may cause erosion and contribute to sediment and alter the quantity of stormwater runoff.
73. **Mass rainfall** – the total measured depth of precipitation resulting from a given frequency and duration storm event.
74. **Maintenance** – any action necessary to preserve stormwater management facilities in proper working condition, in order to serve the intended purposes set forth in Article I of this Ordinance and to prevent structural failure of such facilities. Maintenance shall not include actions taken solely for the purpose of enhancing the aesthetics associated with stormwater management facilities.
75. **Manning's "n"** – a value representing the roughness of a surface or its resistance to flow.
76. **Natural waterways** – waterways that are part of the natural topography. They usually maintain a continuous or seasonal flow during the year and are characterized as being irregular in cross–section with a meandering course. Construction channels such as drainage ditches shall not be considered natural waterways.
77. **Nonerodible** – a material, e.g., natural rock, riprap, concrete, plastic, etc., that will not experience surface wear due to natural forces of wind, water, ice, gravity, or a combination of those forces.
78. **On–site stormwater management** – the design and construction of a facility necessary to control stormwater runoff within and for a single development.
79. **One percent (1%) annual chance storm** – a storm that is capable of producing rainfall expected to have a 1% chance of being equaled or exceeded in any given year.
80. **Open channel flow** – flow in a channel that is open to the atmosphere. This term usually has reference to a natural stream, a man–made channel that is not "closed", or a culvert flowing partially full.
81. **Ordinary high water** – the level or depth of water in a channel that resides for a majority of the time. It is normally identified by a distinct line of vegetation along the banks of a channel.
82. **Person responsible for the land disturbing activity** – the person who has or represents having financial or operational control over the land disturbing activity; and/or the landowner or person in possession or control of the land who directly or indirectly allowed the land disturbing activity or has benefited from it or who has failed to comply with any provision of this ordinance.
83. **Post–development conditions** – the conditions which exist following the completion of the land disturbing activity in terms of topography, vegetation, or land use and rate, volume, or direction of stormwater runoff.

84. **Precipitation/Rainfall** – any moisture that falls from the atmosphere, including rain, snow, sleet, hail, or dew. Rainfall is a specific type of precipitation generally referred to in this manual.

85. **Pre–developed conditions** – those land use conditions that existed prior to the initiation of the land disturbing activity in terms of topography, vegetation, or land use and rate, volume, or direction of stormwater runoff.

86. **Preliminary plat** – the preliminary plat of a residential subdivision submitted pursuant to the City's Subdivision Regulations.

87. **Pressure flow** – a type of flow that pushes against the top of a channel, such as a culvert, as well as the bottom and sides.

88. **Pressure head** – the height of a column of water that would exert a unit pressure equal to the pressure of water.

89. **Rainfall excess** – the excess rainfall to reach a point within a watershed after interception, depression storage, and infiltration capacities have been met.

90. **Rational Method** – a method for computing peak discharge for a relatively small drainage area. The discharge is computed as a function of the drainage area in acres, the rainfall intensity for a specific frequency storm event in inches per hour, and a coefficient that represents the runoff characteristics of the drainage area (Q=CIA).

91. **Reach** – a portion of the length of a watercourse that has similar geometric and geomorphic features.

92. **Record survey** – a final field survey which locates the visible surface features of a constructed stormwater facility on the ground, but without locating non–visible or subsurface features such as the actual route and elevation of buried pipe.

93. **Regional stormwater management** – the design and construction of a facility necessary to control stormwater runoff within or outside a development and for one or more developments.

94. **Registered Civil Engineer** – a civil engineer properly registered and licensed to conduct work within the City.

95. **Registered Land Surveyor** – a land surveyor properly registered and licensed to conduct work within the City.

96. **Registered Landscape Architect** – a landscape architect properly registered and licensed to conduct work within the City.

97. **Regression equations** – with respect to storm runoff, the equations developed from regression analysis of long periods of data collection from numerous watersheds to determine the probable discharge for a given frequency in watersheds where actual gage data is not available. The equations developed will be related to various physical features of the watershed.

98. **Responsible personnel** – any foreman, superintendent, or similar individual who is the on–site person in charge of land disturbing activities.

99. **Retention** – water collected and stored after a storm runoff event.

100. **Retention structure** – a permanent structure whose primary purpose is to permanently store a given volume of stormwater runoff. Release of the given volume is by infiltration and/or evaporation.

101. **Runoff Coefficient**– a unitless number, ranging from 0 to 1, representing the percentage of precipitation that actually runs off of the drainage area and reaches the point of analysis. It

is dependent on the topography, soil type, vegetative cover, land uses, and surface characteristics of the drainage area.

102. **Runoff hydrograph** – the plot of the runoff rate versus time for a storm of a specific frequency and duration over the drainage area being analyzed. It can be generated with actual gage data or with synthetic data.

103. **Scupper** – a vertical hole through a bridge deck for the purpose of deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.

104. **Sediment** – solid particulate matter, both mineral and organic, that has been or is being transported by water, air, ice, or gravity from its site of origin.

105. **Side-flow interception** – flow which is intercepted along the side of a grate inlet, as opposed to frontal interception.

106. **Slope** – the vertical change in elevation between two points within a watershed relative to the length of travel of rainfall runoff.

107. **Splash-over** – portion of the frontal flow at a grate which skips or splashes over the grate and is not intercepted.

108. **Spread** – the width of flow measured perpendicularly from the roadway pavement edge or the lip of the gutter.

109. **Stabilization** – the installation of vegetative or structural measures to establish a soil cover to reduce soil erosion by stormwater runoff, wind, ice and gravity.

110. **Stage** – the elevation of flow or water surface in a channel above some elevation datum.

111. **Stage work or stage construction** – a plan for the staged construction of stormwater facilities where portions of the facilities will be constructed as different stages of the proposed development are started or completed.

112. **Starting water surface elevation** – an elevation, known or assumed, that is assigned as starting boundary conditions for a given discharge in backwater analysis.

113. **Statistical analysis** – with respect to flow and stages in a watercourse, the statistical evaluation on long periods of data collection to determine the probable discharge or stage that will occur at a specific location for a given frequency.

114. **Stormwater Concept Plan** – the overall proposal for a storm drainage system, including stormwater management structures, and supporting documentation as specified in the Stormwater Management Design Manual, for each proposed private or public development to the extent permitted by law. Also included are the supporting engineering calculations and results of any computer analysis, if necessary.

115. **Stormwater Management** – the collection, conveyance, storage, treatment, and disposal of stormwater runoff in a manner to minimize accelerated channel erosion and/or increased flood damage, and in a manner to enhance and ensure the public health, safety, and general welfare, which shall include a system of vegetative or structural measures, or both, that control the increased volume and rate of stormwater runoff caused by manmade changes to the land.

116. **Stormwater Management Design Manual (SMDM)** – the manual of design, performance, and review criteria for stormwater management practices, prepared under the direction of the City Engineer. Copies of this manual can be obtained from the City Engineering Department.

117. **Stormwater Management Facilities** – those structures and facilities that are designed for the collection, conveyance, storage, and disposal of stormwater runoff into and through the drainage system.

118. **Stormwater Management Plan (SMP)** – the set of drawings and other documents that comprise all of the information and specifications for the drainage systems, structures, concepts and techniques that will be used to control stormwater as required by this Ordinance and the Stormwater Management Design Manual. Also included are the supporting engineering calculations and results of any computer analysis.
119. **Stormwater Management qualitative control** – a system of vegetative, structural, or other measures that reduce or eliminate pollutants that might otherwise be carried by stormwater runoff.
120. **Stormwater Runoff** – the direct response of a watershed to precipitation and includes the surface and subsurface runoff that enters a ditch, stream, storm drain or other concentrated flow during and following the precipitation.
121. **Stream modeling** – the mathematical representation of flow in a physical channel to provide a means to evaluate how the channel will respond to various flows. It will provide a tool to evaluate how various changes in the watershed development will effect the resulting water surface elevations at various points of interest along a stream.
122. **Subcritical flow** – flow for which the velocity is lower than critical velocity and occurring at a depth at or above critical depth.
123. **Subdivision** – (a) The creation of one or more new streets, alleys or other public ways; or, the changing of any rights–of–way of any existing streets, alleys or other public ways. (b) Any division or re–division of lot, tract, or parcel or land, regardless of its prospective use. Such subdivision may be accomplished by platting or by description of metes and bounds or otherwise into two (2) or more lots or other divisions for sale or improvement. The following are not defined as subdivisions:
    a. The combination or recombination of portions of previously platted lots where the total number of lots is not increased and the resultant lots are in accordance with the rules and regulations contained in the City's Subdivision Regulations and with the City's Zoning Ordinance.
    b. Division or sale of land by judicial decree which shall not be deemed a division for purposes of this Ordinance.
    c. The acquisition of land for the purpose of widening or opening of streets when the acquisition and work is done by the City, State, or other governmental agency.
    d. The division of land into parcels greater than five (5) acres where no street right–of–way dedication is involved.
124. **Supercritical flow** – flow for which the velocity is higher than critical velocity and occurring at a depth below critical depth.
125. **Swale** – a structural measure with a lining of grass, riprap, or other materials which can function as a detention structure and convey stormwater runoff without causing erosion.
126. **Tailwater** – water at a downstream point that is impacting or resisting the flow of water from upstream. It often refers to the depth of water below a hydraulic structure such as a culvert or bridge.
127. **Time of Concentration** – the length of time, usually measured in minutes, required for initial runoff from a storm event to reach the point of analysis from the most remote point of the watershed or drainage area contributing runoff to that point.
128. **TP–40** – a publication of the U. S. Weather Bureau that is a series of isohyetal plots of mass rainfall depths for various rainfall events with frequencies from 1 to 100 years and durations of 30 minutes to 24 hours for the continental United States.

129. **TP–49** – a publication of the U. S. Weather Bureau that is a series of isohyetal plots of mass rainfall depths for various rainfall events with frequencies from 1 to 100 years and durations of 1 day to 7 days for the continental United States.
130. **Turbulent Flow** – flow that is characterized by flow lines that are not smooth but that often form eddies, resulting in a "mixing" in that portion of the flow.
131. **Unit Hydrograph** – the plot of the runoff rate versus time for a storm producing a volume of 1 inch of excess runoff over the drainage area being analyzed.
132. **Variance** – the modification of the minimum stormwater management requirements for specific circumstances where strict adherence of the requirements would result in unnecessary hardship and not fulfill the intent of this ordinance.
133. **Velocity** – the speed or rate of travel of water along a surface or channel. It is often measured in terms of "feet" or "meters" per second and noted as "fps".
134. **Velocity Head** – the force or energy produced by the velocity of a quantity of water being conveyed in a channel/floodplain or quantity proportional to the kinetic energy of flowing water expresed as a height or head of water.
135. **Waiver** – the relinquishment from stormwater management requirements by the City Engineer for a specific land disturbing activity on a case–by–case review basis.
136. **Watercourse** – a natural or manmade channel which conveys water, either continuously or intermittently.
137. **Water quality** – those characteristics of stormwater runoff from a land disturbing activity that relate to the physical, chemical, biological, or radiological integrity of water.
138. **Water quantity** – those characteristics of stormwater runoff that relate to the rate and volume of the stormwater runoff to downstream areas.
139. **Watershed/Drainage Basin** – the drainage area contributing stormwater runoff to a single point.
140. **Watershed Modeling** – the mathematical representation of a physical watershed or channel to provide a means to evaluate how it will respond to various frequency storm events. It will provide a tool to evaluate how various changes in the watershed development will effect how it responds.

2.0 ABBREVIATIONS

The following is a list of abbreviations to be used in the use of this MANUAL and the submission of all materials required by this MANUAL and the Russellville Stormwater Management Drainage Ordinance. Other abbreviations may be found within this MANUAL that are not listed in this section but are defined where used. Variation of some of these abbreviations may also be found within the text of this MANUAL but should be defined in the section where they are used. The use of any other abbreviation in submittal materials required by this MANUAL to represent terms expressed herein shall be approved by the CITY ENGINEER prior to their use.

Abbreviation	**Meaning**
A	Area
ac	acre
ac–ft	acre-foot
AASHTO	American Association of State Highway and Transportation Officials
AHTD	Arkansas Highway and Transportation Department
ASWCC	Arkansas Soil and Water Conservation Commission
BFE	Base Flood Elevation
BW	bottom width
C	coefficient of runoff in Rational Equation for runoff computation
C_w	weir coeffecient in the weir equation
CAP	corrugated aluminum pipe
cfs	cubic feet per second
CI	curb inlet
CMP	Corrugated Metal Pipe
CMPA	Corrugated Metal Pipe Arch
CN	Curve Number
COE or Corps	US Army Corps of Engineers
D or d	depth (of channel, gutter flow) (ft)
D_c	critical depth, ft
D_n	normal depth, ft
D_{50} or d_{50}	Mean Particle Size
DA	drainage area
DI	drop inlet
DRO	direct runoff
El or Elev	elevation
f	friction factor (used if pipe flow computations)
F	Froude Number
FAA	Federal Aviation Administration
FEMA	Federal Emergency Management Agency
FHBM	Flood Hazard Boundary Map
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study

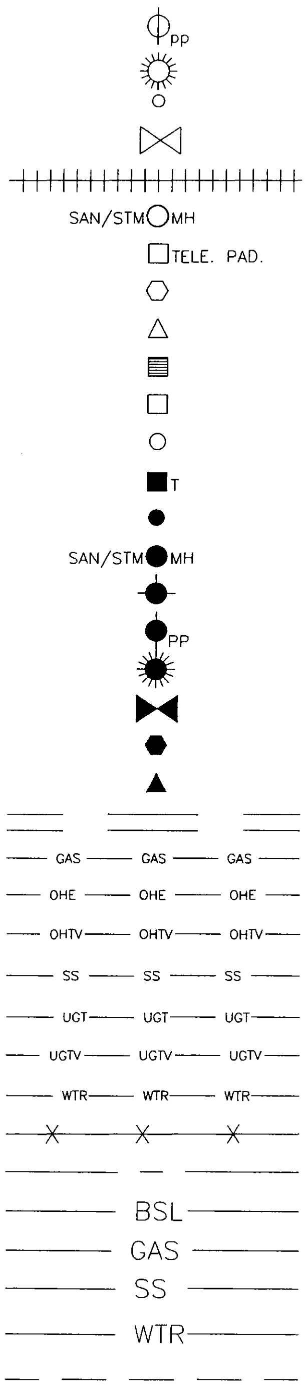
fps	feet per second
ft	feet
g	gravitational constant, 32.2 ft/sec^2
H	head (ft)
H_f - H_{FB}	freeboard height (ft)
I	intensity of rainfall (inches per hour)
in	inches
HEC–1	Hydraulic Engineering Center Watershed Analysis Program
HEC–2	Hydraulic Engineering Center Backwater Analysis Program
HEC–HMS	Hydraulic Engineering Center Hydrologic Modeling System
HEC–RAS	Hydraulic Engineering Center River Analysis System
HGL	hydraulic grade line
HW	headwater depth (ft)
K	loss coefficient
L	length (pipe, channel, etc.) (ft)
MH	manhole
mi	mile
n	Manning's roughness coefficient
NRCS	Natural Resources Conservation Service
Q or q	discharge, volume of flow over a unit of time (cfs)
PMF	probable maximum flood
R	hydraulic radius (A/WP)
RCBC	Reinforced Concrete Box Culvert
RCP	Reinforced Concrete Pipe
RM	FIS Elevation Reference Mark
ROW	right-of-way
RVF	rapidly varied flow
S	slope, unit/unit
SCS	Soil Conservation Service
S_f	friction slope
S_G	street grade or gutter slope (unit/unit)
SS	side slope of an embankment (unit/unit)
SMDM	Stormwater Management Drainage Manual
SMDO	Stormwater Management Drainage Ordinance
T	top width of water surface (spread on pavement)(ft)
T_c	time of concentration (minutes)
t_I	initial or overland flow time (minutes)
T_{max}	maximum allowable spread of water
T_p	time to peak (minutes)
t_T	time of travel (minutes)
TW	tailwater depth (feet)
USGS	US Geological Survey
V	velocity, fps
V_c	critical velocity (fps)
V_n	normal velocity (fps)
WP	wetted perimeter (ft)

WSEL	water surface elevation
Y_G	depth of flow in gutter (ft)
Z	reciprocal of the transverse slope

3.0 SYMBOLS

The following figures illustrate a list of symbols to be used in the use of this MANUAL and the preparation and submission of all materials required by this MANUAL and the Russellville Stormwater Management Drainage Ordinance. The use of any other symbols to represent features of proposed Stormwater Management Plans or proposed drainage facilities for the the City of Russellville and its Planning Area shall be approved by the CITY ENGINEER prior to their use.

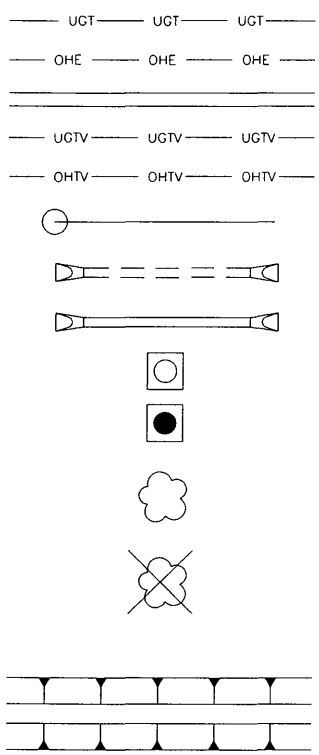
LEGEND



- EXISTING POWER POLE
- EXISTING LIGHT POLE
- EXISTING POST INDICATOR VALVE
- EXISTING WATER VALVE
- RAILROAD TRACKS
- EXISTING SANITARY / STORM MANHOLE
- EXISTING TELEPHONE PEDESTAL
- EXISTING WATER METER
- EXISTING GAS METER
- EXISTING GRATED INLET
- EXISTING DROP INLET
- FOUND IRON PIN
- PROPOSED TELEPHONE PEDESTAL
- SET IRON PIN
- PROPOSED SANITARY STORM SEWER MANHOLE
- PROPOSED FIRE HYDRANT
- PROPOSED POWER POLE
- PROPOSED LIGHT POLE
- PROPOSED WATER VALVE
- PROPOSED WATER METER
- PROPOSED GAS METER
- EXISTING CURB
- EXISTING GAS LINE
- EXISTING OVERHEAD ELECTRIC
- EXISTING OVERHEAD TELEVISION
- EXISTING SANITARY SEWER LINE (SPECIFY SIZE & TYPE)
- EXISTING UNDERGROUND TELEPHONE
- EXISTING UNDERGROUND TELEVISION
- EXISTING WATER LINE (SPECIFY SIZE & TYPE)
- CHAIN LINK OR WIRE FENCE
- PROPERTY LINE
- BUILDING SET BACK LINE
- PROPOSED GAS LINE (SPECIFY SIZE & TYPE)
- PROPOSED SANITARY SEWER LINE (SPECIFY SIZE & TYPE)
- PROPOSED WATER LINE (SPECIFY SIZE & TYPE)
- EASEMENT (TYPE & DIMENSION)

City of **RUSSELLVILLE** Arkansas	**TOPOGRAPHIC SYMBOLS**
	FIGURE V-1

LEGEND



PROPOSED UNDERGROUND TELEPHONE
PROPOSED OVERHEAD ELECTRIC
PROPOSED CURB
PROPOSED UNDERGROUND TELEVISION
PROPOSED OVERHEAD TELEVISION
GUY
EXISTING PIPE (SPECIFY SIZE & TYPE, I.E. 24" CMP)
PROPOSED PIPE (SPECIFY SIZE & TYPE, I.E. 24" CMP)
EXISTING JUNCTION BOX W/MANWAY
PROPOSED JUNCTION BOX W/MANWAY
EXISTING TREE
TREE TO BE REMOVED
DITCH (SPECIFY CHANNEL LINING, I.E., GRASS, RIPRAP, CONCRETE)

City of **RUSSELLVILLE** Arkansas	**TOPOGRAPHIC SYMBOLS** **FIGURE V-2**