URBAN DRAINAGE AND FLOOD CONTROL
CRITERIA MANUAL AND HANDBOOK

FOR

THE CITY OF STILLWATER
OKLAHOMA

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SUMMARY OF POLICY & CRITERIA

This Appendix is a summary of the specific criteria and submittal requirements contained in the main body of this Manual. It is also a reference to the more complete discussions of criteria and methodologies in the main part of the Manual.

For the experienced designers, the following criteria may well be enough to proceed into design. The locations in the main text of the Manual tables and other specific information is noted along with the criteria. The specific methodology referenced to in the main text of the Manual is to be utilized unless specific permission is obtained by the City Engineer for an alternative method. Besides activating specific criteria, a main purpose of this Manual is to provide uniformity in design approach to assist in timely review of submittals by the City Engineer.

For unusual cases or for less experienced designers, the references to the specific text of the Manual which discusses the design methodology will assist the designers to locate the proper information.

The graphs and tables which are required for design are not repeated in this Chapter. It is intended that most designers review the general and specific considerations concerning a specific subject in the main text.

1. <u>MAJOR AND MINOR DRAINAGE</u>. Relative to the purpose served, urban drainage has two separate and distinct drainage systems, the minor drainage system and the major drainage system. The minor drainage system serves a convenient function for people and transportation. Consisting of streets, roadside ditches, storm sewers, and inlets, it is designed to effectively transport the 2-year to 10-year frequency storm runoff. The major drainage system serves a function of protecting lives and property against potential major damages resulting from a 100-year frequency storm runoff as well as preserving major roads for movement

of military and civilian emergency forces. This magnitude of runoff has a one percent chance of occurring in any given year in any single drainage basin. (Part I, Chapter I, page I-31 and Part I, Chapter IV, page IV-3).

2. PREVENTIVE AND CORRECTIVE MEASURES. There are two basic elements of a drainage or flood control program. First, there is the preventive aspect. When this is achieved through comprehensive floodplain management, any increase in the existing flood damage potential will be minimized. Second, there is the corrective element. By affecting the course which floodwaters take, the corrective approach seeks to mitigate flood damages which result from unwise development of floodprone areas. The corrective element of a drainage program is mostly applicable to areas which do not have identifiable drainage patterns. (Part I, Chapter I, page I-5).

3. DRAINAGE AND FLOOD CONTROL PROGRAM. The City of Stillwater shall have a unified program for drainage and flood control. This program will seek to mitigate future flood damages and potential loss of life while systematically reducing existing flood damage and hazard through comprehensive drainage and floodplain management. Where undeveloped floodplains exist, land uses will be controlled to prevent development that would result in increased flood losses. Existing flood problems will be mitigated by applying the proper combination of preventive and corrective measures.

   The City of Stillwater will develop a storm water management system that will prevent frequent nuisance flooding in urban areas outside of floodplains.

   The urban drainage and flood control measures shall be planned and carried out to reduce public and private costs, including the cost of new housing. In addition, the measures shall provide for efficient processing of development request and equitable application of regulations. (Part I, Chapter II, page II-1.)

4. OBJECTIVES. Within the context of the overall development goals of the City of Stillwater, drainage and flood control programs will be governed by the following objectives:

   A. To retain non-urbanized floodplains in a condition that minimizes interference with flood water conveyance, flood water storage, aquatic and terrestrial ecosystems, and ground and surface water interfaces.

   B. To reduce exposure of people and property to the flood hazard.

   C. To systematically reduce the existing level of flood damages.

   D. To ensure that corrective works are consistent with the overall goals of the City.

   E. To minimize erosion and sedimentation problems and enhance water quality.

   F. To protect environmental quality and social well-being, and economic stability.

   G. To plan for both the large flooding events and the smaller, more frequent flooding by providing both major and minor drainage systems.

   H. To minimize future operational and maintenance expenses. To reduce exposure of public investment in utilities, streets, and other public facilities (infrastructure).

   I. To minimize the need for rescue and relief efforts associated with flooding and generally undertaken at the expense of the general public.

   J. To acquire and maintain a combination of recreational and open space systems utilizing floodplain lands.
   (Part I, Chapter II, page II-1).

5. OKLAHOMA STORMWATER LAW. The principals as detailed in Part I, Chapter III, "Oklahoma Stormwater Law" shall be utilized on all stormwater management systems.

6. AREAS OUTSIDE CITY LIMITS. The City regulates land development within 3 miles of the City limits when a rural water meter is requested from a rural water district. These cases of land development must meet the

City Subdivision Regulations. When City utilities are extended beyond the corporate limits, those developments using these utilities must meet the City Codes of the City of Stillwater. (Part I, Chapter II, page II-7).

7. The private engineer's role may vary from the design of a small subdivision and/or small street extensions, to preparation of a master drainage plan for an entire drainage basin, and ultimately, final design. The City staff engineers are involved in the entire range of these studies and designs and seek standardized methodology and criteria to facilitate review and approval.

   The various steps to be utilized in developing and implementing a storm drainage plan are defined as follows:

   - o <u>Get the facts.</u> This is the most important aspect and relates to historic, future, and existing land use, historic, and existing drainage paths, basic hydrology, (including rainfall, runoff, vegetation and infiltration), capacities of the existing facilities, presence of floodplains, impacts on adjacent properties, evaluation of the existing situation, and the presence (or lack of) a master drainage plan for the area and/or basin.

   - o <u>Conceptual Design.</u> Based on the fact situation, develop and analyze all reasonable alternatives. Depending on the size of area being considered, this phase may simply mean that facilities required to meet City standards are determined and shown on the proper sized drawings drawn to a specified scale. Or it may mean an extensive investigation in which the hydrologic, hydraulic, sociological, urban infrastructure, and cost interrelationships are investigated to develop a master plan.

   - o <u>Master Planning.</u> Based on the results obtained from the conceptual design process and upon the concurrence of the City and reviewing agency, a Master Plan is developed. This plan describes in detail the recommended alternative, shows sizes, types, and location for required drainage facilities, and is sufficient in detail for designing new roads, bridges, and other urban utilities. The Master Plan may only be a floodplain information report when structural solutions are not recommened or a more detailed delineation of facilities required to meet City standards for small subdivisions.

- Final Design. Detailed drawings and specifications are prepared. These are suitable for review approval, and construction of all, or segments, of the Master Drainage Plan.
- Construction. Physical placement of drainage facilities according to the final design drawings and specifications. This phase requires onsite supervision by the designer and/or City.
- Maintenance. Maintaining natural or artificial drainage facilities by the City or by others according to a procedure approved by the City. This includes snagging, mowing, silt and debris removal, erosion control, and periodic cleaning of inlets, pipes, ditches, and culverts. (Part I, Chapter IV, page IV-1).

8. FLOODPLAIN MANAGEMENT. This concept has been adopted by the City through this Manual and through its participation in the Flood Insurance Program. It is to be considered an integral part of planning. Modifications using the floodway criteria adopted by the City are acceptable; however, structurally oriented measures will be used only where:
   - Existing conditions warrant their economical use, and
   - The use of structural measures can be demonstrated to have no adverse effects downstream or upstream. (Part I, Chapter IV, page IV-3; Part I, Chapter IV, Page IV-15 to 20).

9. FLOODWAY CRITERIA. No filling or construction will be permitted where the depth of water during the one percent flood is 1.5 feet or greater, or where the percent of encroachment of the floodplain width is greater than 30 percent, whichever is more restrictive. To protect the rights of later applicants, a maximum allowable encroachment up to 30 percent will be split equally between owners on each side of the floodplain. When due to depth limitations the allowable encroachment cannot be evenly divided, the unused percentage may be applied to the other side of the channel so long as the depth of water or the total percentage of encroachment do not exceed 1.5 feet or 30 percent, respectively. (Part I, Chapter II, page II-6).

10. MULTI-OBJECTIVE FLOODPLAIN MANAGEMENT. The City recognizes that multiple-objective floodplain management requires multi-purpose planning. Where multi-purpose benefits will result from the implementation of the drainage policy, funds from other appropriate sources will be sought to supplement the drainage funds. (Part I, Chapter II, page II-5).

11. MAJOR DRAINAGE DESIGN EVALUATION. Proposals for both preventive and corrective drainage and flood control measures will be evaluated on the likely discharge arising from the appropriate critical duration rainfalls of 1 percent plan. (The 1 percent probability runoff--the one in 100-year event--is that which has a 1 percent chance of being equalled or exceeded in any given year). Lesser storms will also be evaluated to arrive at a more complete assessment of effects. Larger storms will be evaluated conceptually to ensure that the best alternative is chosen for reduction of significant life and economic impacts.

    When actual works are being designed, the level of protection will be determined on the basis of economic analyses, availability of funds and physical constriants. Corrective works may be designed to protect against floods with lesser frequency that the 100-year flood, or at a greater flood level if high risk is involved. (Part I, Chapter II, page II-5).

12. MAJOR DRAINAGE FLOODPLAINS. Floodplains will be delineated on the basis of the 1 percent in 100-year flood. (Part I, Chapter II, page II-6).

13. HYDROLOGIC ANALYSIS OF POSSIBLE ALTERNATIVES FOR MAJOR DRAINAGE. The procedures described in the Hydrology Chapter will be followed for each of the possible alternatives. This would include runoff routing and production of runoff hydrographs which will show volumes and peak discharge values in comparison to the existing conditions and future conditions with no modifications to the stream network. Determination of water surface profiles shall be made to determine residual floodplains

remaining, when alternatives being considered do not completely contain the 100-year event.

It is important, at this point, to analyze these results to identify both the negative and positive modifications made to the runoff response characteristics of the basin with various alternatives. (Part I, Chapter IV, page IV-20.

14. PRELIMINARY DESIGN REPORT FOR MAJOR DRAINAGE. A supporting report must be prepared documenting the investigation which cover the topics listed in this Section. This report documents the process and information of the conceptual design process, describes in detail the agreed upon plan, costs, and phasing. (Part I, Chapter IV, page IV-24.)

15. MAPPING FOR PRELIMINARY DESIGN OF MAJOR DRAINAGE. All topographic mapping in the major drainageway will be at a scale of 1" = 100' with 2-foot contours. (Part I, Chapter IV, page IV-24).

16. FINAL DESIGN REQUIREMENTS FOR MAJOR DRAINAGE. When approved by the City Engineer for parcels involving only floodplain management is involved mapping at a scale of 1" = 100' will normally be suitable; however, for all cases involving structural channel modification, the mapping scale will be at a scale of 1" = 50'. A contour interval of 1 foot will be used when, in the opinion of the City Engineer, it is necessary to clearly define the proposed facilities. Contours will be required to illustrate proposed earthwork.

    Profiles will be provided for all facilities and the hydraulic grade lines for both the design runoff event and the 100-year event (if they are different). These will be provided on prints for review by the City.

    A complete list of information required for design submittals is provided at the end of this Chapter. (Part I, Chapter IV, page IV-24).

17. WATER SURFACE ELEVATION IN RECEIVING DRAINAGE FACILITIES-MINOR AND MAJOR DRAINAGE. The first step is to determine the design water surface elevation in 1) the drainage system to which the new facility is tributary, or 2) the downstream water surface (hydraulic grade line) in the facility (channel or conduit) into which the improvements drain. This step for Item 2 is for localized improvement, natural channels, or when the proposed improvements are entirely contained within a segment of the drainage system. (Part I, Chapter IV, page IV-25).

18. TRANSITION FROM MINOR TO MAJOR DRAINAGE. When the total tributary area exceeds 80 acres to any system, the designer should begin rough calculations to see if the 100-year event criteria are being exceeded. When the criteria begins to be exceeded the minor storm system (ditches or pipes) should be increased. When the size of the minor storm system facilities exceeds the 10-year runoff event, then the system is to be analyzed as a major drainage system. (Part I, Chapter IV, page IV-4).

19. PLANNING FOR MINOR DRAINAGE. Planning and design for the minor storm drainage system must be considered from the viewpoint of both the regularly expected storm (the minor storm) and the major storm occurrence. Depending on land use, street classifications, and inundation criteria (see Chapter II, Part II) the minor design storm will have a frequency ranging from once in two years to once in five years. There are criteria similar to that for the minor storm which also must be met for the major storm or 100-year event. The minor storm drainage system must be capable of handling both types of event within the criteria established. (Part I, Chapter IV, page IV-27).

20. DESIGN PROCESS FOR MINOR DRAINAGE.

- o Using methods described in the Hydrology Chapter of Part II of this Manual and as subsequently described, the designer computes the runoff rates for the design storm starting at the uppermost reaches of the basin.
- o The storm sewer system begins when the design storm runoff exceeds the gutter (or roadside ditch) capacity. The design proceeds downstream until the system outfalls into the major drainage facilities.

- o Again, from the upper-most reaches of the basin, the designer computes the runoff from the 100-year storm. When street capacity criteria are exceeded for this major storm, the designer should increase the size of the storm sewer that was sized for the minor storm. This increase in sewer size should increase the flow in the pipe network and reduce the street flow to within the established criteria. The combined total of the allowable street carrying capacity should equal the major design runoff.

- o The previous three steps consitute preliminary design. Up to this point, junction losses in storm sewers are ignored and roughness coefficients are increased by 25 percent. The final design of a storm sewer system must include junction loss computations. This procedure is explained in Chapter IV of Part II. (Part I, Chapter IV, pages IV-28 and 29).

21. MAPPING FOR PRELIMINARY DESIGN OF MINOR DRAINAGE. Mapping at a scale of 1" = 100' is to be used except, when in the opinion of the City Engineer, the area is so large as to be better shown at a scale of 1" = 200'. (Part I, Chapter IV, page IV-30).

22. BASIC DATA FOR PRELIMINARY DESIGN OF MINOR DRAINAGE. Classify probable future type of development within the basin as it affects both hydrology and hydraulic design. Classify streets as to storm water drainage carrying capacity. Determine design frequency for minor drainage design. Develop intensity duration frequency curves for both the minor design frequency and the major 100-year storm (Part I, Chapter IV, Page IV-30).

23. DEVELOP ALTERNATIVE CONCEPTS. In many cases, numerous potential layouts are possible. Here the engineer should review the reasonable alternative concepts, selecting those that appear most practical from an intuitive standpoint. Planning of a storm sewer system should have as its

objective the design of a balanced system in which all portions wil be l used to their full capacity without adversely affecting the drainage of any area. (Part I, Chapter IV, page IV-30).

24. LAYOUT PRELIMINARY CONDUIT ALIGNMENTS FOR DESIGN PURPOSES. Set grades to be used for preliminary design procedures. Several preliminary layouts should be considered. (Part I, Chapter IV, page IV-30).

25. DIVIDE BASIN INTO SUBBASINS FOR DESIGN POINTS. When dividing into subbasins, it should be remembered that at various inlets on a continuous grade only a portion of street flow will be removed to the storm sewer system. At intersections of urban principal and minor arterials, it will be necessary to remove 100 percent of the minor runoff from the road surface to preclude cross street flow. (Part I, Chapter IV, page IV-31).

26. LOCATION OF OUTLET. This point is covered more fully in Chapter IV of Part II, "Storm Sewers", however, certain points need special emphasis. First, the outlet should be located at the historic outfall point. In cases where this point has already been altered, the second point must be adhered to. The second point is that the resulting outflow should not do more harm than would have occurred if the improvement was not built. This second point applies even though the outlet is located at its historic point of outfall. (Part I, Chapter IV, page IV-31).

27. UTILITIES. All above-ground and below-ground utilities are to be located and shown in plan and profile. (Part I, Chapter IV, page IV-32).

28. STREETS. Streets are to meet the criteria as set forth in Chapter II of Part II, "Streets." (Part I, Chapter IV, page IV-32).

29. INLETS. Inlets are to meet the criteria as set forth in Chapter III of Part II, "Inlets." In regard to location, it may be necessary to start the storm sewer earlier than might be required for street capacity when

a street is crossed in which crosspans and cross flows are not permissible.

30. SYSTEM SIZING FOR MINOR DRAINAGE. The frequency of design runoff, or rainfall return period, to be used for the minor storm drainage system would range from once in two years to once in ten years. A summary of the design frequency to be used in Stillwater for storm sewer design is presented below in Table 1.

TABLE 1

STORM DESIGN FREQUENCY - MINOR STORM

Land Use	Return Period (Frequency)
1. Residential	2 years
2. General commercial area	5 years
3. Airports (Does not include major drainages which traverse area)	5 years
4. Business/commercial areas	5 years
5. Special high value areas and transportation corridors (limited application to Stillwater)	10 years

(Part I, Chapter IV, page IV-35)

31. REVIEW ALTERNATIVES. Review alternative plans with all who are involved in the final decision, including the City Engineer's office and the City Planning staff. (Part I, Chapter IV, page IV-37).

32. PRELIMINARY DESIGN REPORT FOR MINOR DRAINAGE. A preliminary design report will be prepared and supported by computations which covers the items listed in the check list and those subjects previously described. Except as may be deemed necessary by the City Engineer, a benefit/cost analysis will not be necessary.

33. SELECTION OF ALTERNATIVE. Based on the Preliminary Design Report, an alternative (with modifications as required) will be selected by the City Engineer to be final designed).

34. FINAL DESIGN OF MINOR DRAINAGE. The following items are to be accomplished in final design:

- Hydrology. Depending on the impact of refinements made in the alternative selection process, the final design hydrology may range from a review of the preliminary design hydrology to additional hydrologic modeling. The same hydrologic techniques (and often the same hydrology) are used for final design as for preliminary design. The type of hydrologic method to be used is defined in Chapter I of Part II, "Hydrology." (Part I, Chapter IV, page IV-39).

- Mapping. For many large minor storm drainage facilities, it will necessary to utilize mapping at a scale of 1" = 20' to 1" = 50' with 2-foot contours along the route, unless the City Engineer determines that 1-foot contours are necessary. While a subjective choice, the scale of mapping is to be approved by the City Engineer, however, the larger scale mapping will generally be necessary where numerous utility conflicts exist. (Part I, Chapter IV, page IV-39).

- Streets and Utilities. Prior to commencing final hydraulic design, it is necessary to obtain detailed information on street grades, utilities, and final grades adjacent to the improvements where the grade is likely to change due to development. This information should be displayed on plan and profile drawings and used as constraints in the final hydraulic design. The location of other utilities which serve a local function only should not be considered as a major constraint. (Part I, Chapter IV, page IV-39).

- Hydraulically Designed Sewer System. The water level in the receiving major drainageway should be determined for the design storm frequency. If this elevation is above the crown of the storm sewer, it is less likely that special outlet control devices will be necessary to prevent erosion. If the major drainageway is flowing at less than the design depth, the outlet should be reviewed for possible erosion tendencies.

The final hydraulic design of a system should be on the basis of procedures set forth in Chapter IV of Part II of the Manual. A realistic "n" value for final design should be used based on actual pipe roughness. The conduits should be treated as either open channels or conduits flowing full, as the case may be. For open channel flow, the energy grade line should be used as a base for calculation. For conduits flowing full, the hydraulic grade line should be calculated. If possible, storm sewers are to be designed flowing full.

The design engineer must review the hydraulic grade line for runoff conditions exceeding the initial design storm. This is to insure that the hydraulic grade line does not rise above the ground surface and thus cause unplanned discharge to the street. Because of the greater opportunity for management of excess runoff, the closed conduit approach to design shall generally be used to prevent transporting a problem to another area with unknown and often damaging results.

The design generally proceeds upstream from the outfall utilizing the hydraulic procedures from determining pipe losses and junction losses as shown in Chapter IV of Part II of this Manual. (Part I, Chapter IV, page IV-39, 40; Part II, Chapter IV, page IV-3).

- <u>Design Inlets.</u> Utilizing City standard inlets, the design of inlets should be carried on simultaneously with the design of the remainder of the storm sewer system. The allowable street carrying capacity should be continuously equated to the design runoff from the Rational Method to determine where inlets will be necessary. The design of inlets should be based on the local tributary basin runoff, which may have a shorter time of concentration (and higher discharge) than for the main storm sewer system. (Part I, Chapter IV, page IV-40).

- <u>Determine Structural Aspects.</u> The structural aspects of pipe and appurtenances to be utilized in the storm sewer system should be designed by thorough methods to insure that they are both adequate and economical. Certain of these decisions must be made prior to hydraulic design of the system since the geometry of junctions, the type of inlets to be utilized, and the pipe material will influence the design. (Part I, Chapter IV, page IV-40, 41).

- <u>Final Construction Plans.</u> Final construction plans and specifications should be of sufficient accuracy and clarity to guarantee that the designer's ideas are carried to completion by field installation. The Final Design check list is included at the end of this Chapter. (Part I, Chapter IV, page IV-41).

35. <u>HYDROLOGY, PART II, CHAPTER I.</u>
    - Table I-1 (page I-2) lists the guideline criteria for which method is to be used at a minimum to determine runoff peaks and quantities (where required). (Part II, Chapter I, page I-1, 2).

    - The data and methods in Part II, Chapter I used for computing rainfall and runoff are specifically compiled and/or adopted for use in Stillwater. Unless specific permission is received by the City Engineer, the provisions of this Chapter shall be used by the designer on work to be approved or completed for the City.

36. <u>STREETS, CURBS, AND GUTTERS, PART I, CHAPTER II, SPECIFIC DESIGN CRITERIA.</u> An overall approach to storm runoff management includes using the street system to transport runoff to inlets during the minor storm and to transport runoff from storms that are greater than the storm sewer capacity. According to the street classification and/or the surrounding land use, certain criteria (set forth herein) are used to determine at what point the minor and major drainage facilities begin, such criteria are being based on encroachment (maintenance of traffic lanes) for the minor storm and on inundation limitations for the major storm. (Part II, Chapter II, page II-1).

A. The typical Stillwater street cross section is shown in Figure II-1, page II-5). In addition to the requirements shown in Figure II-1 the street should have a minimum grade of 0.4 percent. Inlets should be a minimum of 25 feet downstream of any curb cut. In locating curb cuts near inlets in already storm sewered areas the same spacing should be utilized to locate the curb cut. Figure II-2 (Page II-6) illustrates typical standard curb configurations to be used in Stillwater. (Part II, Chapter II, page II-4).

B. Figure II-3 (page II-7) illustrates the typical cross section to be used when cross fall occurs from one gutter to the other. This configuration is important to prevent sheet flow across the street, which reduces the street capacity during frequently occurring rainfall events or ice formation during the winter. Some sheet flow across the centerline of local streets is acceptable during the design minor frequency storm event, but should not occur for the rainfall events which occur more frequently than the one-year event nor during the design minor frequency storm event. Cross flow should not be allowed on streets whose designation is equal to or greater than the collector. (Part II, Chapter II, page II-4).

C. On local streets, where cross fall is necessary due to the existing topography, inlets may be placed in the lower curb, and the street crown removed to allow flow from the upper curb to reach the inlet in the lower curb at specified locations when approved by the City Engineer. (Part II, Chapter II, page II-4).

D. Driveway entrances should be recessed into the curb and not be made by building up in the gutter. The driveway should slope up at an elevation equal to the top of the curb so runoff within the street cannot flow onto adjacent property through the driveway entrance. (Part II, Chapter II, page II-4).

E. Inverted crown or dished streets should not be utilized for local, collector or arterial streets, or for freeways. (Part II, Chapter II, page II-8).

F. When local streets intersect arterial or collector streets, the grades of the arterial or collector street should be continued uninterrupted. (Part II, Chapter II, page II-8).

G. When collector and arterial streets intersect, the grade of the more major street should be maintained insofar as possible. No form of cross pan should be constructed across an arterial street for drainage purposes. (Part II, Chapter II, page II-8).

H. Conventional cross pans may be utilized to transport runoff across local streets when a storm sewer system is not required. The cross pan size and slope should be sufficient to transport the runoff across the intersection with encroachment equivalent to that allowed on the street. Infrequently, pans may be used on collector streets. (Part II, Chapter II, page II-8).

I. Pavement encroachment limits for the minor storm runoff will meet the criteria of Table II-1 of Part II. (Part II, Chapter II, page II-11).

J. When the allowable pavement encroachment has been determined, the theoretical gutter carrying capacity for a particular encroachment shall be computed using the modified Manning's Formula as shown on Figure II-6 (page II-13). (Part II, Chapter II, page IV-11, 12, and 13).

K. The actual flow rate allowable per gutter shall be calculated by multiplying the theoretical capacity by the corresponding factor obtained from Figures II-7 (page II-14). Discharge curves have been developed for standard streets. The designer will be able to develop discharge curves for non-standard streets and for streets with crossfall. (Part II, Chapter II, page II-12 and 14).

1. Determination of the allowable flow for the major storm shall be based upon two considerations:
   - o Theoretical capacity, based upon allowable depth and inundated area.
   - o Reduced allowable flow due to velocity considerations.

The allowable depth and inundated area for the major storm shall be limited as set forth in Table II-2. (page II-15).

Based upon the allowable depth and inundated area as determined from Table II-2, the theoretical street carrying capacity shall be calculated. Manning's formula shall be utilized with an n value applicable to the actual boundary conditions encountered which may include grassed areas and sections with differing geometry.

The actual flow allowable within the street right-of-way shall be calculated by multiplying the theoretical capacity by the corresponding factor obtained from Figure II-7. (page II-14).

Where allowable ponding depth would cause cross street flow, the limitation shall be the minimum allowable of the two criteria set forth in Table II-2 or Table II-3. (page II-16).

When the direction of flow is toward a principal arterial street, the allowable carrying capacity shall be calculated by applying the reduction factor from Figure II-8 (page II-18) to the theoretical gutter capacity. The grade used to determine the reduction factor shall be same effective grade used to calculate the theoretical capacity. (Part II, Chapter II, pages II-12 to II-16).

M. The foregoing provisions are subject to modifications in business areas and heavily-used pedestrian areas. (Part II, Chapter II, page II-19).

37. INLETS, PART II, CHAPTER III, SPECIFIC DESIGN CRITERIA.

A. The following general recommendations are made for the utilization of different types of stormwater inlets.

Sump Conditions

- True Sump. The use of depressed curb opening inlets is recommended. Each true sump should be reviewed to determine if the area affected by ponding is within acceptable limits. (Part II, Chapter III, page III-9).

- Sumps Formed by Crown Slope of Cross Section at Intersection. The use of curb opening inlets is recommended, though combination inlets may be successfully utilized. A small amount of ponding may cause storm runoff to flow over the crown of the cross street and continue down the gutter. (Part II, Chapter III, page III-9).

Continuous Grade Conditions

Except as permitted by the City Engineer, combination inlets should be used on continuous grades. (Part II, Chapter III, page III-9).

Shallow Overland Flow Conditions

Except as permitted by the City Engineer, under certain conditions, slotted drains may be utilized. (Part II, Chapter III, page III-9).

B. The following reduction factors should be applied to the theoretical calculated capacity of inlets based upon their type and function. The reduction factors compensate for effects which decrease the capacity of the inlet such as debris plugging, pavement overlaying, and in variations of design.

TABLE 2

REDUCTION FACTORS TO APPLY TO INLETS

Condition	Inlet Type	% of Theoretical Capacity Allowed
Sump	Curb Opening	80
Sump	Grated	50
Sump	Combination	65
Continuous Grade	Curb Opening	80
Continuous Grade	Deflector	75
Continuous Grade	Longitudinal Bar Grate	75
	incorporating recessed transverse bars	60
Continuous Grade	Combination	110% of that listed for type of grate utilized.
Shallow Overland Flow	Slotted Drains	80

The allowable capacity of an inlet should be determined by applying the applicable factor from Table 2 to the theoretical capacity calculated in accordance with the appropriate design charts. (Part II, Chapter III, pages III-9, 10).

C. The design chart to be used for curb opening inlets in sumps is Figure III-5 (Page III-11). (Part II, Chapter III, pages III-10, 11).

D. The design charts to be used for curb opening inlets on continuous grade, Figure III-6, (i), (ii), and (iii) (pages III-13, 14) for the standard depression configuration as utilized by the City of Stillwater. (Part II, Chapter III, pages III-12, 13, 14).

E. The design method used to compile the capacity charts, Figures III-1 and III-11 (pages III-20, 21), is based on comprehensive research. These figures are based on the conditions illustrated in Figure III-9. (pages III-19). The charts can be used to determine the capacity for the recommended inlet type (Neenah R 3246 and R 3246-17).

38. FINAL HYDRAULIC DESIGN STORM SEWERS, PART II, CHAPTER IV, SPECIFIC DESIGN CRITERIA.

A. Although not always feasible, the recommended procedure is to design storm sewers to flow under pressure. (Part II, Chapter IV, page IV-3).

B. Because of the nature of hydraulic elements in circular conduits, it may be reasonably assumed that open channel flow will occur only when the flow depth is less than 80 percent of the conduit diameter. (Part II, Chapter IV, page IV-3).

C. Provisions for self-cleaning of storm sewers shall be made in the hydraulic design. (Part II, Chapter IV page 9).

D. As shown in Figure IV-9, page IV-19) construction details of manholes for storm sewer system should deviate somewhat from standard manholes for sanitary sewers (Part II, Chapter IV, pages IV-17, 19).

E. Unless higher loss is specifically planned for a straight flow in a manhole the pipes should be positioned vertically so that they are between the limits of inverts aligned or crowns aligned. An offset in the plan and/or profile is allowable provided the projected area of the smaller pipe falls within that of the larger. Aligning the inverts of the pipes is probably the most efficient as the manhole bottom then supports the bottom of th jet issuing from the upstream pipe. (Part II, Chapter IV, page IV-17).

F. The design water surface should be at least 6 inches below the gutter grade at the inlet to allow the inlet to function properly. (Part II, Chapter IV, page IV-21).

G. The following requirements will be met in regards to storm sewers:

- Main Location -- The location of storm sewers shall be cleared with, and approved by the City Engineer. (Part II, Chapter IV, page IV-63).

- Alignment -- Storm sewers shall be straight between manholes insofar as possible. Where long radius curves are necessary to conform to street layout, the radius of curvature divided by the pipe diameter shall be at least 6.0. Radius of curvature specified should coincide with standard curves available in the type material utilized wherever possible. Specially fabricated bends will be permissible as long as their effect is included in the final hydraulic design. (Part II, Chapter IV, page IV-63).

- Crossings -- Crossings with other underground utilities except at intersections shall be avoided. Crossings, if necessary, should be at an angle greater than 45 degrees. (Part II, Chapter IV, page IV-63).

  The storm sewer main and/or the utility must be structurally reinforced if insufficient vertical clearance is available. Standard allowable clearance without reinforcing between storm and sanitary sewers is 24 inches. (Part II, Chapter IV, page IV-63).

- Manhole Spacing -- Spacing of manholes shall conform to the following table.

TABLE 3
MANHOLE SPACING

Pipe Size	Maximum Spacing
15" or less	600 feet
18" to 36"	600 feet
42" or greater	800 feet

(Part II, Chapter IV, page IV-63).

- Direction Changes -- Short radius bends may be used on 24" and larger pipes when flow must undergo a direction change at a junction or bend. Reductions in headloss at manholes may be realized in this way. A manhole shall always be located at the end of such short radius bends. (Part II, Chapter IV, Page IV-63, 64).

- <u>Manhole Geometry</u> -- Except as may be needed to induce head loss, the manhole bases shall be shaped as indicated in Figure IV-9, with the deflector height being equal to the crown of the outlet pipe. Deflections greater than 18-inches in height shall have toe pockets. (Part II, Chapter IV, Page IV-64).

H. Except for slotted drains (see Chapter III, "Inlets"), storm sewer grades should be such that a minimum of 3'-0" cover over the crown of the pipe is maintained. Uniform slopes shall be maintained between manholes unless specifically approved otherwise.

Final grades shall be set with full consideration to capacity required, sedimentation problems, and other design parameters, but the minimum slopes shall be that capable of producing the cleansing velocity as determined from Figure IV-4 (page IV-9). The grade will depend upon the geometry and roughness of the conduit. (Part II, Chapter IV, page IV-64).

I. Storm sewers may be constructed of any suitable material acceptable to the governing body, as long as it is capable of matching requirements set forth in this Manual. Soils tests shall be conducted when there is a possibility that conditions exist which would cause premature failure of certain materials. Structural calculations must be carried out on any material to verify that it is acceptable.

When alternate types of materials are acceptable for bidding purposes, hydraulic designs must be completed for each material to verify that both materials will be acceptable. The minimum line diameter for mains and connectors will be 12 inches. (Part II, Chapter IV, pages IV-64, 65).

J. Unless paralleled by an existing utility easement, the minimum width of easement for installation of a storm sewer should be the pipe diameter plus 18 feet. With a parallel existing utility easement, the minimum width of easement shall be the pipe diameter plus 9 feet. (Part II, Chapter IV, page IV-65).

39. RATIONAL METHOD FOR SIZING STORM SEWER SYSTEM, PART II, CHAPTER IV, APPENDIX IV-A, PRELIMINARY DESIGN. This method is a part of preliminary design and represents the hydrology portion of the final design. That is, it established the estimated flows which need to be carried in the system. An example is also contained in Appendix IV-A of Part II, which develops the discharges used in the hydraulic (final) design example contained in the main body of Chapter IV, Part II. After the preliminary minor system design is completed and checked for its interaction with the major runoff, reviews made of alternatives, hydrological assumptions verified, new computations made and final data obtained on street grades and elevations, the engineer should proceed with final hydraulic design of the system. (Part II, Chapter IV, page IV-A-1).

40. MAJOR DRAINAGE, PART II, CHAPTER V, SPECIFIC DESIGN CRITERIA.

A. The following general criteria and recommendations are made for open channel design.

   - Whenever practical, the channel should have slow flow characteristics, be wide and shallow, and be natural in its appearance and functioning. (Part II, Chapter V, page V-3).

   - Artificial channels (except concrete-lined) should be designed with a Froude number less than 0.8. (Part II, Chapter V, page V-7).

- Roughness coefficients (n) for use in Manning's equation vary considerably according to type of material, depth of flow, and quality of workmanship. Tables V-1 (page V-9) and V-2 (Pages V-11 and 12) list roughness coefficients for pipes and for various artificial channels. (Part II, Chapter V, pages V-9, 11 and 12).

- No utility crossings will be permitted except those which meet the criteria of this Manual for bridges, etc.

B. The following criteria are applicable to concrete lined channels:

- Whether the flow will be supercritical or subcritical, the lining must be designed to withstand the various forces which act on the channel. Supercritical flow offers substantial challenge for the designer, and without prior approval of the City Engineer, supercritical channels will not be used. (Part II, Chapter V, page V-10).

- All channels carrying supercritical flow shall be lined with continuously reinforced concrete, the reinforcing being continuous both longitudinally and laterally. There shall be no diminution of wetted areas cross section at bridges or culverts. Freeboard shall be adequate to provide a suitable safety margin, the safety margin being at least 2 feet or an additional capacity of approximately one-third of the design flow. Bridges or other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load which might be imposed upon the structure in the even of major trash plugging.

  Concrete-lined channels must be protected from hydrostatic uplift forces by the use of underdrains and weepholes, which are often created by a high water table of momentary inflow behind the lining from localized flooding. (Part II, Chapter V, page V-13).

- Because of field construction limitations, the designer should not use a Manning n roughness coefficient any lower than 0.013 for a well-troweled concrete finish. The freeboard should equal the velocity head plus 1.0 feet. (Part II, Chapter V, page V-13).

C. The following criteria are applicable to Grass-Lined Channels (Artificial).

- Because of their similarity to natural channels, a well-designed grass-lined channel is considered to be the most desirable aritifical channel. (Part II, Chapter V, page V-13).

- For an irrigated or non-irrigated Bermuda Grass lining, the maximum velocity for the major storm design runoff of 8.0 feet per second should be used. This permits an economical cross section and yet keeps scour problems within reasonable limits. Without a satisfactory grass cover established, however, the annual flows will cause serious channel cutting and bank cutting at bends. (Part II, Chapter V, page V-14).

- The maximum design depth of flow is 5.0 feet, though 4.0 feet is preferable. Erosion is a function of velocity, depth, and time. Urban runoff peaks are generally short-lived, which makes velocity and depth key design parameters. For channels with design capacities greater than 4,000 cubic feet per second, greater depths can be considered. (Part II, Chapter V, page V-14).

- Grass-lined channels, to function well, normally have slopes of from 0.2 to 0.6 percent. Where the natural topography is steeper than desirable, drops should be utilized. (Part II, Chapter V, page V-14).

- The less sharp the curves, the better the channel functioning will be. In general, centerline curves should not have a radius of less than about twice the design flow top width, but not less than 100 feet. (Part II, Chapter V, page V-14).

- o Bridge deck bottoms and sanitary sewer often control the freeboard along the channel banks in urban areas. Where they do not control, the allowance for freeboard should be equal to the velocity head plus 1 foot. Where appropriate floodplain zoning is used, localized overflow in certain areas may be desirable because of ponding benefits. Except as may be specified by the City Engineer, all channels will be designed for a freeboard of 18-inches for the design storm. (Part II, Chapter V, page V-14).

- o Grasses will meet and be sown according to the requirements of Soil Conservation Service Standard number 443. (Part II, Chapter V, Pages V-17, 18).

- o Unless otherwise specified by the City Engineer, Bermuda Grass will be used for all permanent grass cover. Grass-lined channels will normally not be artificially irrigated.

- o The flatter the side slope, the better. A normal minimum is 4:1. Under special conditions, the slopes may be as steep as 3:1 which is also the practical limit for mowing equipment. (Part II, Chapter V, page V-19).

- o The maximum depth should be limited to 4.0 feet, though 5.0 feet is acceptable where good maintenance can be expected and where durations of peak flows are short-lived. (Part II, Chapter V, page V-20).

- o The bottom width should be at least 6 to 8 times the depth of flow. Twenty to 30 times the depth is common. (Part II, Chapter V, page V-20).

- o Trickle channels or underdrain pipes are required on all urban grassed channels. Trickle channels are preferred because of maintenance. The trickle channel capacity should be 0.5 to 1.0 percent of the design flow, the lower value being more applicable to underdrain pipes. (Part II, Chapter II, page V-20).

- o Typical channel cross sections for grassed channels are shown in Figure V-3. (Part II, Chapter V, page V-21).

- o Drops in excess of 3.0 feet should be avoided. (Part II, Chapter V, page V-22).

D. The following criteria are applicable to European-type channels.

- o This type of channel refers to artificial channels with grassed bottoms and concrete sides. The sides may be cast-in-place or precast and may have several different types of texture. The criteria listed previously for grass-lined channels shall apply. (Part II, Chapter V, page V-23).

E. Earth channels of an artificial character, that is, either constructed channels or heavily modified natural channels, shall not be used for drainage because of the potential erosion and damage to those downstream. (Part II, Chapter V, page V-23).

F. The following criteria are applicable to natural channels:

- o It can be assumed initially that the changed runoff regime will result in new and highly active erosional tendencies. Careful hydraulic analysis must be made of natural channels to counteract these new tendencies. In some cases, slight modification of the channel will be required to create a somewhat better stabilized condition for the channel. (Part II, Chapter V, page V-23).

- The usual rules of freeboard depth, curvature, and other rules applicable to artificial channels do not apply. For instance, there are significant advantages which may accrue if the designer 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 areas during the major runoff peak. Although the usual design criteria for artificial open channels do not apply to natural channels, such criteria can be used to advantage in gaging the adequacy of a natural channel for future changes in runoff regime. (Part II, Chapter V, page V-24).

- Utilization of natural channels requires that primary attention be given to erosive tendencies and carrying capacity adequacy. The floodplain of the waterway must be defined so that adequate zoning can take place to protect the waterway from encroachment to maintain its capacity and storage potential. (Part II, Chapter V, page V-24).

- General criteria for analyzing the effectiveness of natural channels are:

  i Channel and overbank capacity adequate for 100-year runoff.

  ii Velocities in natural channels do not exceed critical velocity for a particular section which is only rarely more than 10 fps.

  iii Define water surface limits so that floodplain can be zoned.

  iv Filling the flood fringe reduces valuable storage capacity and tends to increase downstream runoff peaks. Filling should be discouraged in the urban waterways where hydrographs tend to rise and fall sharply. The specific policies of the City in regard to floodplain fill will be used.

v Use roughness factors (n) which are representative of unmaintained channel conditions.

vi Construct drops or check dams to control water surface profile slope, particularly for the initial storm runoff.

vii Prepare plans and profiles of floodplain. Make appropriate allowances for future bridges which will raise the water surface profile and cause the floodplain to be extended.

viii Use a freeboard of a minimum of 18-inches.

ix For backwater computations, the channel cross-sections must be at no more than 500 feet and more often when channel properties change or if more accurate results are desired. The channel cross sections must be divided into sections with like properties and the appropriate "n" factor applied. The primary difficulty with using the HEC-2 program is its applicability to structures, particularly bridges and culverts. The designer must be sure to check the results for reasonableness for any water surface program used. It may be necessary to compute structure hydraulics by hand. (Part II, Chapter V, pages V-25, 26).

G. The following criteria are applicable to closed conduits:

o Box culverts are often considered to be covered free-flow conduit. They are open channels with a cover. (Part II, Chapter V, page V-27).

o Structural requirements are efficiency for sustaining external loads, rather than hydraulic efficiency, usually control the shape of the box culvert. (Part II, Chapter V, page V-27).

o Computational procedures for flow in closed conduits are essentially the same as for canals and lined channels, except that special consideration is needed in regard to rapidly increasing flow resistance when the conduit reaches full.

Special flow limiting inlets may be used to eliminate this condition. (Part II, Chapter V, page V-27).

- o Structural design must account for internal pressure if pressure will exist. (Part II, Chapter V, page V-27).
- o Because of sediment load normally associated with urban runoff, the bottom of a box culvert should be lined with steel plates when the average velocity exceeds 20 fps. (Part II, Chapter V, page V-27).
- o Roughness coefficients shall be determined from Table V-4. (Part II, Chapter V, page V-28).
- o Ports for air are needed at the entrance to obviate both positive and negative pressures, and to permit released entrained air to readily escape from the conduit. (Part II, Chapter V, page V-29).
- o Where bends must be used, superelevation of the water surface must also be studied and allowances made for a changing hydraulic radius, particularly in high velocity flow.
- o Dynamic loads created by the curves must be analyzed to insure structural integrity for the maximum flows. (Part II, Chapter V, page V-29).
- o Hydraulic design must account for entrained air when high velocities are encountered. (Part II, Chapter V, page V-30).
- o The conduit must be designed to eliminate sediment depositional problems during storm runoffs which have a frequency of occurrence of about twice a year. (Part II, Chapter V, page V-30).

- A long box culvert should be easy to inspect, and therefore, access manholes are desirable at various locations. (Part II, Chapter V, page V-3).

- A large box culvert with a special entrance and an energy dissipator at the exit usually need an access hole for vehicle use in case of major repair work being necessary. (Part II, Chapter V, page V-31).

H. The following criteria are applicable to riprap:

- The City has a preference for gabion-type riprap. (Part II, Chapter V, page V-31).

- The riprap layer shall contain about 40 percent of the rock pieces smaller than the required size is, as stable or more stable than individual rocks of the required size. (Part II, Chapter V, page V-32).

- A riprap layer should be about one and one-half times or more as thick as the dimension of the large rocks and that the riprap should be placed over a gravel layer. (Part II, Chapter V, page V-33).

- For sizing the riprap, see Figures V-4, 5 and 6. (Part II, Chapter V, pages V-34, 35, and 36).

- For sizing and design of gravel layer, see page V-37. (Part II, Chapter V, page V-37).

- For channel drops, the gabions should be keyed into both banks to prevent flanking, and downstream cutting should be considered. Gabion baskets should be laid on a gravel filter. (Part II, Chapter V, page V-38).

I. The following criteria are applicable to hydraulic structures:

- o The graphs and charts contained in this Manual are suitable for planning, preliminary design, and preliminary cost estimating. For final design, the designer must refer to the original publications for more detail.

  In addition to the normal earth, hydrostatic (uplift) and traffic forces on hydraulic structures, the structural engineer must consider:

  i The dynamic forces of water,
  ii Erosion due to high velocity,
  iii Impact from debris lodging in bends or on piers and abutments,
  iv Debris plugging the inlet to conduits and causing the conduit to flow partially full,
  v Vibration, and
  vi Cavitation (mostly in outlet structures and in bends of high velocity conduits (Part I, Chapter IV, page IV-26).

- o The best hydraulic perfomance in a channel above or below a hydraulic strucutre is obtained when the confining sidewalls are parallel and the distribution of flow across the channel is maintained uniform. (Part II, Chapter V, page V-44).

J. The following criteria are applicable to channel drops:

- o Channel drops are to be used to permit adjustment of a thalweg which is too steep for the design conditions. In urban drainage work, it is often desirable to use several low head drops in lieu of a few higher drops. (Part II, Chapter V, page V-55).

- A drop with a sloped face of 2:1 or 4:1 is generally suitable. The face should be roughened so as to dissipate energy, at least for the lower and more frequent flows. (Part II, Chapter V, page V-55).

- The use of vertical drops should generally be avoided because of the cost of the structure and resulting turbulence. However, at times the vertical drop will be used and for that reason the following criteria are presented. (Part II, Chapter V, page V-55).

K. The following criteria are applicable to bridges:

- Bridge openings should be designed to have as little effect on the flow characteristics as is possible, consistent with good bridge design and economics. However, in regard to supercritical flow with a lined channel, the bridge should not affect the flow at all. That is, there should be no projections into the design water prism. (Part II, Chapter V, page V-59).

- The method of planning for bridge openings must include water surface profile and hydraulic gradient analyses of the channel for the major storm runoff. Once this hydraulic gradient is established without the bridge, the maximum reasonable effect on the channel flow by the bridge should be determined. In most cases, this should not exceed a backwater effect of more than 6 to 12 inches, and may require less.

- The bridge opening freeboard criteria to be used in Stillwater is the velocity head plus 2.0 feet. This may require lowering a section(s) of the approach road and raising the bridge to meet this criteria, even though the road would not be overtopped for the design flood.

41. MAN MADE STORAGE, PART II, CHAPTER VI, SPECIFIC DESIGN CRITERIA.

- When provision of storage is being considered, the designer must verify that the attenuation of the peak runoff will not undesirably aggravate any potential downstream peaking conditions for a range of flood frequencies. (Part II, Chapter VI, page VI-2).

All man-made storage should be planned to meet the following general requirements to provide safe facilities that will help to achieve the goals and objectives of the City of Stillwater.

i Facilities should be coordinated with the development goals and objectives and the existing land use.

ii Facilities should be designed to protect against failure that would increase the potential for downstream flood loss and must meet the standards of the Oklahoma Water Resources Board.

iii Facilities should be evaluated with consideration of normal flow conditions, frequent events, less frequent intense events such as the 100-year frequency rainfall event, and maximum probable events. The evaluation of such considerations will ensure that the storage does not worsen downstream flood conditions.

iv Facilities should be designed with careful attention to a particular design event. A design rainfall probability of 1 percent should normally be used unless specific minor facilities are being evaluated.

v Facilities should be planned with respect to the topography, soil, and geology.

vi Facilities should be planned to reduce the degree of operation, maintenance, and administrative needs.

vii Provisions should be made to ensure the maintenance of the facilities over their design life.

viii Floodplains should be regulated downstream of new storage facilities to prevent new encroachment into the area protected by the storage. A storage facility should not encourage creation of new flood hazards or set the stage for larger disasters than formerly. (Part II, Chapter VI, Pages VI-5, 6).

ix Detention ponding may be required to meet provisions of Oklahoma Stormwater Law (See Part I, Chapter III).

o The drain time for roofs, storage areas, and storage parking lots (i.e. parking for a car dealership) is less stringent than are plazas and parking lots frequented by the public. For the former category, a drain time of 2 to 4 hours is reasonable. For the latter category, a drain time from 1 to 2 hours is reasonable. The designer should consider that maximum depths are attained infrequently, usually an average of once each 100 years. Further, the hard surface detention facilities should be designed such that snow melt and storm runoff from events does not pond. (Part II, Chapter VI, Page VI-16).

o Vegetated surface storage can range from open space and passive recreation areas to high intensity recreation areas. In the former case, the detention time can range up to 24 hours or longer if successive use of the storm water is desired. Further, there are greater opportunities to attenuate lower frequency runoff events as well as the design runoff event (usually the 100-year event) and this provision should be incorporated into the design. (Part II, Chapter VI, page VI-16).

o Table VI-2 (Page VI-17) lists the maximum and normal average depths for the various types of uses where hard surface detention ponding is normally located. The maximum depth of ponding refers to the depth of water at a low point, typically for draining and detention pond. Except for roof top ponding, in both instances it is assumed that a particular use area is not fully covered by stormwater thereby allowing movement through any area during and after a runoff event. (Part II, Chapter VI, page VI-17).

- For passive recreational and open space areas, there are no limits as to depths which are more logically determined by topography and the storage volume required.

  For high intensity recreation areas, the maximum allowable average depth is 5 feet. In instances where this criteria requires too much land be acquired to attain the required storage volume, it is recommended that terracing be used. The high intensity recreation activities can be located on the highest level where the maximum depth criteria can be met. Designers should arrange the detention facilities such that the minimum depth in the facility is near to where the public will have the most immediate access. (Part II, Chapter VI, pages VI-16, 17).

- In open space and passive recreation areas, the steepness of side slopes is governed by side slope stability as determined by soils investigation.

  High intensity recreation areas require side slopes to be no steeper than of 4:1 (4 horizontal to 1 vertical) where grass is to be maintained and 3:1 in non-grassed areas. In addition, both types of areas need one area no steeper than 10:1 to allow for the entrance and exit of maintenance vehicles. (Part II, Chapter VI, page VI-18).

42. CULVERT DESIGN, PART II, CHAPTER VIII, SPECIFIC DESIGN CRITERIA.

- Corrugated metal pipe (CMP) culverts are to be used only for minor drainage facilities (i.e., under driveways crossing roadside channels) and for temporary installations for major drainage. In the former situation, the entrance and exits must have headwalls or end sections, or be beveled. CMP culverts are not to be used for permanent installations on major drainages. (Part II, Chapter VIII, page VIII-1).

- Concrete culverts for major drainages must have end-sections, improved entrances (described later), or headwalls. Concrete culverts do not require headwalls or end-sections for driveway crossings of raodside channels. (Part II, Chapter VIII, page VIII-2).

- Inlet coefficients will be obtained from Table VIII-1. (Part II, Chapter VIII, page VIII-9).

- 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. (Part II, Chapter VIII, page VIII-14).

- Culverts with wingwalls should be designed with a concrete apron extending between the walls. Aprons must be reinforced to control cracking. (Part II, Chapter VIII, page VIII-14).

- Table VIII-2 (page VIII-21, 22) lists the culvert design charts and their applications. (Part II, Chapter VIII, page VIII-21, 22).

- Skewed culverts (culverts not parallel to the direction of flow) will not be acceptable, unless in the opinion of the City Engineer that not other alignment is reasonable. (Part II, Chapter VIII, page VIII-75).

- Where upstream detention storage requires headwater depth in excess of 20 feet, reducing the culvert size is recommended rather than using the inefficient projecting inlet to reduce discharge. (Part II, Chapter VIII, page VIII-75).

- Consideration shall be given for the use of trash racks (Part II, Chapter VIII, page VIII-76).

43. CHECK LIST FOR PRELIMINARY DESIGN SUBMITTALS

o Basic Data

Map of total drainage basin
Map of area to be storm-sewered
Characteristics of streets
Street grades and direction of slope
Location and elevation of outfall points for minor and major drainage
Rainfall curves
Character of future development
Degree of imperviousness
Soil and water table data
Utility information

o Hydrology

Design criteria tabulation for minor and major storm runoff
Peak discharge computations for pipe sizing
Peak discharge computations for major storm runoff
Assumptions as to upstream storage

o Layout

Streets and street names
Irrigation ditches
Street drainage flow direction
Drainage basin and subbasins
Storm sewer layout with sizes
Storm inlet locations
Cross pan locations
Open drainageways
Layout of major drainage system showing flows and directions
Scale
North arrow
Signature blocks for review approvals
Location map and subdivision names
Conflicting utilities

- General

  Title Block (lower right-hand corner preferred)
  Scale
  Date and revisions
  Name of professional engineer or firm
  Professional Engineer's seal
  Statement as to specifications
  Approval spaces with data spaces
  Drawing numbers
  Statement as to adherance to drainage policies and criteria in the Drainage Criteria Manual

- Drainage Area Plan

  North Arrow
  Contours (maximum 2-foot intervals)
  Location and elevation of USGS bench marks
  Property lines
  Boundary lines (counties, districts, tributary area, etc.)
  Streets and street names and approximate grades with width
  Subdivision (name and location by section)
  Existing irrigation ditches
  Existing drainageways and structures including flow directions
  Drainage subbasin boundaries
  Easements required
  Proposed curbs and gutters and gutter flow directions
  Proposed cross pans and flow directions
  Proposed inlet locations and inlet sizes
  Proposed piping and open drainageways
  Critical minimum finished floor elevations for protection from major storm runoff.

- Construction Plans

  North Arrow
  Property lines and ownership or subdivision information
  Street names and easements with width dimensions
  Testhole locations and log
  Existing utility lines (buried), location and depth
    Water
    Gas
    Telephone
    Storm drain
    Irrigation ditches
    Sanitary Sewers

- Vertical and horizontal grids with scales
  Ground surface existing and proposed
  Existing utility lines where crossed
  Pipes

Plan showing stationing
Profile
Size, lengths between manholes and type
Grades
Inlet and outlet details
Manhole details (station number and invert elevation)
Typical bedding detail

- Open Channels

  Plan showing stationing
  Profile
  Grades
  Typical cross section
  Lining details

- Special structures (manholes, head walls, trash racks, etc.)

  Plan
  Elevations
  Details

TABLE OF CONTENTS

CHAPTER I
PURPOSE, SCOPE, AND DEFINITIONS

CHAPTER I

PURPOSE, SCOPE, AND DEFINITIONS

This document, later referred to as the Manual or Drainage Manual, represents a combination of aspects related to urban runoff management. The Manual defines the goals, objectives, policies, and principles for urban runoff management in the City of Stillwater. The Manual delineates the planning/design process, discusses Oklahoma stormwater law, and articulates requirements for drainage design submittals. These first two elements are in Part I of the Manual.

Part II of the Manual serves two purposes. The first includes articulation of specific criteria for use in Stillwater and delineation of data specific to Stillwater (i.e. rainfall). The second purpose is to provide the designer most of the tools necessary to design urban drainage works, and, therefore, the Manual is also a handbook. In regard to the latter purpose, it is not intended that standard procedures from basic publications be changed.

For a designer who is familiar with a design procedure and its graphics, it is intended that they be as nearly the same as possible (i.e. culvert design). Then, the designer does not have to read an entire section to verify that it is the same process which he already knows. On the other hand, many engineers do not have all of the various publications necessary to plan and design urban drainage facilities, so the inclusion of basic design procedures and graphics from other publications will assist many designers by assembling most needed information into a single handbook.

The publication of this Manual is the culmination of the first phase of Stillwater's attack on urban runoff problems. Succeeding phases of master planning, which will be based on this Manual, will greatly assist developers, engineers, and local government through the use of drainage management. This will lead to development of further policies, definition of floodplains (future condition), and corrective measures which will:

- Prevent future problems from occurring, and
- Develop a systematic program of eliminating existing problems, as the availability of money allows.

The implementation of the drainage plans integrated with the urban environment will allow the City of Stillwater to obtain a variety of amenities and a reduction of urban runoff damages at a lower total cost than if each were done separately.

BASIC CONCEPTS

Urban planning has long been closely associated with water resources management and planning. The hydrologic cycle, specifically water, has been identified as a valuable planning tool because it has many common interfaces with urban subsystems. These include highways, parks, solid waste, land use, sewerage and general sanitation, utilities, and streets.

Urban drainage and flood control are very important aspects of water resources management because they relate to water supply, sewerage, aquifer recharge, irrigation, and urban layout based on natural drainage patterns. Throughout the world, it is increasingly being recognized that urban planning and the planning for drainage and flood control go hand-in-hand.

Storm runoff will occur no matter how well or how poorly the drainage planning is done. The quality of the planning determines the costs to the developer, to the community, and the effect on the residents and on other urban subsystems.

Planning Objectives

The overall objective of urban drainage and flood control planning is to help achieve an orderly, efficient, pleasant, and diverse urban area which in turn will complement other efforts conducive to public health, safety, and welfare.

Accomplishment of the comprehensive drainage goals and objectives can be assisted by a broad drainage planning process. Such a process should be used for analysis of all drainage and flood control problems. The planning process seeks to achieve the goals and objectives of the drainage problem I-2 within the context of the comprehensive goals and objectives of the City. A given drainage plan may be oriented toward various portions of the drainage network, but should always address important relationships.

The urban design team should think in terms of natural drainage easements and street drainage patterns, and should coordinate its efforts with the drainage engineers to achieve the goals and objectives of the City of Stillwater. Drainage and flood control measures are costly when planning is poor or mediocre. Good planning results in lower cost drainage facilities and a better community.

Definitions

The following is a narrative description of some of the more basic terminology used in the concepts of urban drainage planning. The definitions are not comprehensive and are simply included at this point to provide the reader with some cohesive terminology prior to beginning the following chapters.

Relative to the purpose served, urban drainage has two separate and distinct drainage systems, the minor drainage system and the major drainage system. The minor drainage system serves a convenience function for people and transportation. It is designed to effectively transport the 2-year to 10-year frequency storm runoff. The major drainage system serves a function of protecting lives and property against potential major damages resulting from a 100-year frequency storm runoff as well as preserving major roads for movement of military and civilian emergency forces. This magnitude of runoff has a one percent chance of occurring in any given year in any single drainage basin. Out of 100 drainage basins, a 100-year or more frequency storm runoff can be expected to occur each year.

While separate and distinct as to purpose, the minor and major drainage systems relate to one another. The major drainage system can be further described as that route which runoff follows during a major rainfall event, whether or not the route is planned and designed and whether or not development is wisely situated relative to that route. A well-conceived and well-planned major drainage system can reduce and often eliminate the need for an underground storm sewer system. The major drainage system is analyzed for the 100-year event, even though its facilities may be designed for a lesser frequency for economic reasons.

The minor system consisting of property line swales, streets and gutters, storm sewers, and smaller open drainageways should discharge to a major drainageway at frequent intervals for economy.

Streets and gutters are designed to be part of the minor drainage system. Limitations are imposed so as not to disrupt the main traffic-carrying function. During the 100-year storm runoff, the streets and gutters will carry more than the minor storm runoff design. Planning and design constraints are imposed so that this major drainage function is kept within reasonable limits.

The definition of a Drainage Master Plan is as follows:

> The drainage master plan describes in detail the recommended plan for drainage and the courses of action for implementation in terms of priorities. It shows sizes, types and location of drainage facilities on maps in sufficient detail to allow planning of new roads and bridges. In some cases, the master plan will be a floodplain information report because structural solutions will not be recommended.

The scope of an urban Drainage Master Plan must be broad enough to deal with water resources management. It must adequately relate to urban planning already done or being done concurrently, and it must relate to the river basin.

There are two basic elements of a drainage or flood control program. First, there is the preventive aspect. When this is achieved through comprehensive floodplain management, any increase in the existing flood damage potential will be minimized. Second, there is the corrective element. By affecting the course which floodwaters take, the corrective approach seeks to mitigate flood damages which result from unwise development of flood-prone areas. The corrective element of a drainage program is mostly applicable to areas which do not have identifiable drainage patterns.

The preventive and corrective approaches to drainage and flood control presented in the Master Plan are multipurpose and ideally should be implemented by an array of the possible drainage management alternatives. Floodplain areas, when properly managed, provide opportunities to help improve the quality of life. Urban networks and recreation uses, when properly planned, can be integrated into the floodplain areas. Where appropriate, floodplains in public ownership may be developed as linear parks in which selective recreational facilities, including trails for hiking, cycling, and horseback riding, might be established. Any such developments must be taken into consideration when city policies are developed.

There are many demands on the land and water resources of an urban region. The demands are associated with efforts to achieve a variety of objectives, such as economic development, regional development, transportation, social well-being, and environmental quality. Because these resources are limited and the demands are not, these objectives compete with one another. In terms of drainage, competing objectives must be considered and reconciled through a formal planning strategy. A planning strategy contains:

- Goals. The purpose toward which an endeavor is directed.
- Objectives. The end toward which the actions are directed.
- Principles. The foundation from which one proceeds and which governs the overall endeavor.

- Policies. The means or plan which is employed to achieve the end effort, and
- Criteria. Specified operational requirements.

Within this context, demands for the land and water resources can be ordered in terms of their ability to achieve desired goals and objectives. This is a powerful device that can be used in the decision-making process. Drainage must be viewed as one of many issues affecting the use of land in Stillwater.

Drainage basins are convenient units for water resources management purposes. Within the boundary of each drainage basin, a system of watercourses has evolved which is specifically related to the physical and hydrological conditions. The watercourses and the floodplains developed through periodic inundations are the primary areas of consideration in drainage basin management. However, to mitigate flood losses, control erosion, manage sedimentation, and abate water pollution, it is necessary to formulate management polciies not only for the watercourses and floodplains, but also for all parts of the drainage basin.

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GOALS, OBJECTIVES, POLICIES AND PRINCIPLES

CHAPTER II

GOALS, OBJECTIVES, POLICIES AND PRINCIPLES

Water resources management and planning have long been closely associated with urban planning. Water has been identified as a valuable planning tool because it has many common interfaces with urban subsystems. These include highways, parks, open space, solid waste, land use, sewerage and general sanitation, utilities, and streets.

Urban drainage and flood control management are important aspects of water resources management because they relate to water supply, sewerage, aquifer recharge, irrigation, and urban layout based on natural drainage patterns. Throughout the world, it is increasingly being recognized that urban planning and the planning for drainage and flood control go hand-in-hand.

Storm runoff will occur no matter how well or how poorly the drainage planning is done. The quality of the planning determines the costs to the developer, to the community, and the effect on the residents and on other urban subsystems.

Drainage and flood control planning and engineering must be based on the goals, objectives, and policies of the City of Stillwater, on basic drainage principles, and on Oklahoma Stormwater Law if they are to be acceptable and implemented.

The following statements are the guides and framework for urban drainage and flood control planning and engineering in Stillwater.

GOALS

Drainage and flood control in the City of Stillwater and environs shall be an integral part of the comprehensive planning process. It is a subsystem of a larger and more comprehensive urban system.

The City of Stillwater shall have a unified program for drainage and flood control. This program will seek to mitigate future flood damages and potential loss of life while systematically reducing existing flood damage and hazard through comprehensive drainage and floodplain management. Where undeveloped floodplains exist, land uses will be controlled to prevent development that would result in increased flood losses. Existing flood problems will be mitigated by applying the proper combination of preventive and corrective measures.

The City of Stillwater will develop a storm water management system that will prevent frequent nuisance flooding in urban areas outside of floodplains.

Urban stormwater pollution will be controlled through use of corrective and preventive measures.

The urban drainage and flood control measures shall be planned and carried out to reduce public and private costs, including the cost of new housing. In addition, the measures shall provide for efficient processing of development requests and equitable application of regulations.

OBJECTIVES

Within the context of the overall development goals of the City of Stillwater, drainage and flood control programs will be governed by the following objectives:

1. To retain non-urbanized floodplains in a condition that minimizes interference with flood water conveyance, flood water storage, aquatic and terrestrial ecosystems, and ground and surface water interfaces.
2. To reduce exposure of people and property to the flood hazard.
3. To systematically reduce the existing level of flood damages.

4. To ensure that corrective works are consistent with the overall goals of the City.
5. To minimize erosion and sedimentation problems and enhance water quality.
6. To protect environmental quality and social well-being, and economic stability.
7. To plan for both the large flooding events and the smaller, more frequent, flooding by providing both major and minor drainage systems.
8. To minimize future operational and maintenance expenses.
9. To reduce exposure of public investment in utilities, streets, and other public facilities (infrastructure).
10. To minimize the need for rescue and relief efforts associated with flooding and generally undertaken at the expense of the general public.
11. To acquire and maintain a combination of recreational and open space systems utilizing floodplain lands.

POLICY

The rules which are employed to achieve the objectives of the urban drainage and flood control effort are based on the following policies:

1. The City of Stillwater will establish and publish criteria for drainage and flood control planning and design. Guidance relative to consruction, operation, and maintenance of urban drainage systems will also be provided. The City will adopt criteria relevant to all public and private drainage interests. Such criteria will be periodically reviewed and revised in the light of new knowledge, changing circumstances, and adjustments in overall comprehensive goals and objectives.
2. The City, within the context of regional policies and in conjunction with other governmental agencies and other relevant drainage interests, will prepare reports for each appropriate drainage basin, outlining the proposed methods of managing urban drainage and

floodwaters and associated land use. Inputs will be elicited from appropriate interest groups during the planning process. The plans will be placed on exhibition for public comment prior to their adoption. The basin plans will be prepared in accordance with priorities established by the City Commission. Improvements embodied within an adopted plan will be made consistent with the fiscal capabilities of the City.

3. Plans for drainage basins shall be periodically reviewed and revised in the light of new knowledge, changing circumstances, and adjustments in overall comprehensive goals and objectives. Unless otherwise determined, such reviews will be at intervals of approximately 5 years.

4. The cooperation of governmental agencies and other relevant drainage interests, including the land development industry, will be sought to coordinate individual development and drainage schemes with the plans.
5. Consideration will be given to a full-range of preventive and corrective approaches, including the following:

   - Delineation of floodplains,
   - Control of floodplain land uses,
   - Acquisition of selected floodplains and major drainage routes including use of purchase, dedication, development rights, and use easements,
   - Stormwater quality enhancement,
   - Floodplain information and education,
   - Flood forecasts and emergency measures,
   - Flood proofing,
   - Flood insurance,
   - Restriction of the extension of water and sewer facilities in floodplains,
   - Detention (retardation) and retention of urban stormwater runoff, and

o Construction of flood control and urban drainage works.

The combination of strategies will be tailored to a specific site and will balance engineering, economic, environmental, and social factors in relationship to stated comprehensive goals and objectives.

The City recognizes that multiple-objective floodplain management requires multi-purpose planning. Where multi-purpose benefits will result from the implementation of the drainage policy, funds from other appropriate sources will be sought to supplement the drainage funds.

6. In general, proposals for both preventive and corrective drainage and flood control measures will be evaluated on the likely discharge arising from the appropriate critical duration rainfalls of 1 percent probability and an urbanized basin as defined by the comprehensive plan. (The 1 percent probability runoff--the one in 100-year event--is that which has a 1 percent chance of being equalled or exceeded in any given year). Lesser storms will also be evaluated to arrive at a more complete assessment of effects. Larger storms will be evaluated conceptually to ensure that the best alternative is chosen for reduction of significant life and economic impacts.

   When actual works are being designed, the level of protection will be determined on the basis of economic analyses, availability of funds and physical constraints. Corrective works may be designed to protect against floods with lesser frequency than the 100-year flood.

   However, recognition will be given to the need to protect certain structures against failure from floods arising out of runoff events having recurrence intervals in excess of 100 years. This would apply where the risk of failure would represent a potentially high hazard. Such situations would include the design of the spillways of dams and high road embankments.

7. Floodplains will be delineated on the basis of the one hundred year flood. The 1 percent flood will be computed by using synthetic hydrology based on rainfall-runoff relationships or by statistical analysis of flood records where these are reliable and long term. Hydrologic modelling will be utilized as an aid in the planning and design effort.

8. The City will encourage passive type uses of the floodplains.

9. The City will develop and implement corrective drainage and flood control plans that will mitigate existing drainage problems. Such plans will be coordinated with comprehensive goals and objectives and will consider a combination of structural and nonstructural measures. Improvements will be based on official priorities established by the City Commission and in accordance with the fiscal capabilities of the City.

10. Pollution control programs will be integrated into the drainage and flood control programs.

11. The City will:
    - o Expand and improve its capacity to provide flood warning information,
    - o Continue with a program of hydrologic research and investigations to improve the understanding of the rainfall-runoff process and to develop better methods for estimating the discharges of given storm recurrence intervals.
    - o Compile basic data on rainfall-runoff relationships,
    - o Adopt dependable storm runoff determination methods which would be used on a uniform basis, and
    - o Require that major drainage facilities be planned taking both rate of runoff and volume of runoff into consideration.

12. New development in the floodplains will be discouraged. No filling or construction will be permitted where the depth of water during the 1

percent flood is 1.5 feet or greater, or where the percent of encroachment of the floodplain width is greater than 30-percent, whichever is more restrictive. To protect the rights of later applicants, a maximum allowable encroachment up to 30 percent will be split equally between owners on each side of the floodplain. When due to depth limitations the allowable encroachment cannot be evenly divided, the unused percentage may be applied to the other side of the channel so long as the depth of water or the total percentage of encroachment do not exceed 1.5 feet or 30 percent, respectively.

13. Acquire and maintain combination recreation and open space utilizing floodplain.

14. The City regulates land development within 3 miles of the City limits when a rural water meter is requested from a rural water district. These cases of land development must meet the City Subdivision Regulations. When City utilities are extended beyond the corporate limits, those developments using these utilities must meet the City Codes of the City of Stillwater.

15. Provide for the safe and efficient movement of public and private transportation, and emergency vehicles, during major and minor flooding events.

PRINCIPLES

1. The Drainage System is Part of a Larger Environmental System

The drainage system is a part of a larger interrelated comprehensive urban system. The drainage system can be managed as simply a support system for an urban area or it can be managed in a way that will assist efforts to achieve a broad range of goals and objectives. In the latter sense, it is a means to an end, not an end in itself.

Urbanization has the potential to increase both the volume and rate of stormwater runoff. The influence of planned new development within a drainage basin must be analyzed and adjustments made to minimize the creation of flood problems. Local and regional goals help to define the drainage works prescribed for a watercourse.

2. Primary Natural Function of Floodplains

   The floodplain is nature's prescribed easement along a watercourse. The primary natural function of each watercourse and its associated floodplain is the collection, storage, and transmission of stormwater runoff. This function cannot be subordinated to any other use of the floodplain without costly compensatory control measures. Within these constraints, the floodplains have the potential to help improve water quality and air quality, provide open space, preserve important ecosystems, and accommodate properly planned urban network systems.

3. Stormwaters Require Space

   Stormwater management is a time related, space allocation problem. Water cannot be compressed. If natural storage is reduced by urban or other land use practices without appropriate compensatory measures, additional space will be claimed by the floodwaters at some other location(s).

4. Stormwaters Have Potential Uses

   Stormwater is often a resource out of place. In such cases, storage of stormwater is the first step in a program to make use of the resource. These storage areas can be designed and operated to provide aesthetic amenities and recreational space. The stored water may have the potential to be used for irrigaion, groundwater recharge, low flow augmentation or industrial water supplies.

5. Water Pollution Control Measures are an Essential Feature

   Water pollution control is essential to a realization of the potential benefits to be derived from watercourses and floodplains. Pollution control measures, which deal with both point and nonpoint discharges, are an integral part of a drainage and flood control program.

6. *Preventive Measures Less Costly*

Preventive measures are less costly to the taxpayers than are corrective measures.

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OKLAHOMA STORMWATER LAW

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OKLAHOMA STORMWATER LAW

by Ruth M. Wright*

INTRODUCTION

A storm moves in over a basin. The rain hits the earth -- some of the water percolates into the ground in a diffused manner, collecting in depressions and swales, gathering in gullies, eventually to flow into and become creeks and streams. If the storm is of a great magnitude, the water cannot be contained within the banks of the creeks and streams, so the water spreads out over the floodplain -- the natural path it has created for itself over geologic time.

With the advent of man, a storm still moves in over the basin and the waters move downhill. But now there are changes in the natural topography. Depressions are filled in. The land is made impermeable by streets, parking lots, and rooftops, resulting in less water percolating into the ground. Streets and storm sewers collect the water so that more water with greater velocity may be discharged onto lower lands, or discharged in a different location. Embankments and dikes are built which divert the course of flood waters and reduce natural detention. Roads and bridges constrict the flows, causing waters to back up and flood lands which would not have been flooded, or would have been flooded to a lesser extent. Rivers are straightened and channelized which speeds up the flow and has greater impact downstream. These changed conditions can cause injury greater than formerly, and spawn lawsuits requesting damages for the injury and/or injunctions to prevent injury. In addition, as government steps in to attempt to manage surface waters, watercourses, and floodplains by constructing facilities or by the use of police power (drainage ordinances, subdivision regulations, floodplain zoning and other techniques), a host of other legal confrontations arise.

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This Chapter sets out the legal framework for stormwater planning in Oklahoma. It is essential that municipalities and their planners and engineers have a legal basis for their work so that implementation does not confront legal obstacles at a future date. In addition, potential liability due to injury caused by stormwater facilities should be avoided.

The outline of this Chapter is as follows:

Introduction
Summary and Conclusions
Distinction Between Watercourse Waters and Surface Waters
1. Law of Watercourses
2. Law of Surface Waters
Oklahoma Watercourse and Surface Water Law
1. Interference with a Watercourse
2. Altering Surface Water Runoff
3. Right to Restore Original Bank of Watercourse
4. Limited Right to Repel Unnatural Waters
5. Detention Ponds
6. Ordinary and Extraordinary Floods
7. Municipal Liability
8. Governmental Immunity
9. Remedies
Financing the Project: The Drainage and Flood Control and Utility Fee
Management of Stormwater by Municipalities

All cases cited in the text are listed in alphabetical order at the end of this Chapter in Appendix A. The Attorney General Opinion No. 70-234 is reproduced as Appendix B.

SUMMARY AND CONCLUSIONS

1. A riparian landowner along a watercourse may take measures to protect himself from the harmful effects of flood waters, but it is fundamental that no one may change, divert, obstruct, or otherwise interfere with the natural flow of a watercourse without being liable for damages to persons and properties injured by such actions. The floodplain of the ordinary flood is part of the watercourse.

2. Where an upper landowner collects surface water, sends it down in a different manner or concentrated form, or in unnatural quantities or velocities, or discharges it in a different location, he is liable for any damage caused thereby. Conversely, a lower landowner may not cast surface waters back onto upper land to the detriment of the upper landowner. The basic principle is that one cannot prevent injury to one's own property by transferring that injury to one's neighbor's property. Oklahoma courts call this "the common enemy rule modified by the rule of reason."

3. Where a party interferes with natural detention, either by filling it in or by cutting through its banks, he is liable for injury to lower landowners caused by change in surface water runoff. Artificial ponds which catch surface water are recognized as beneficial for flood and erosion control.

4. Where one party has caused <u>unnatural</u> water to flow onto another's property, the second party has a right to repel such waters; however, this right is strictly limited to placing the parties in the same conditions as prior to any construction. Nor may a party, in repelling such waters, cause injury to innocent third parties.

5. A riparian owner on a watercourse may construct embankments or other structures necessary to <u>maintain</u> his bank of the stream or to restore it to its original course.

6. While a landowner has the right to improve his property, this right is qualified by the "golden maxim" of the common law that one must so use his own property as not to injure the rights of another. This maxim is used by courts in stormwater cases.

7. If injury to persons or property is due <u>solely</u> to an extraordinary flood, there is no liability.
   a. However, if a person's negligence, commingled with an extraordinary flood, was a contributing proximate cause of the harm, such person is liable.

b. It is negligent to build a structure (e.g., inadequate bridge or culvert), which causes damage during an ordinary flood; if such a structure is a proximate cause of injury during an extraordinary flood, liability results.

c. In only a few Oklahoma cases has the defense of "extraordinary flood" been successful against liability.

d. The flood of record on a watercourse is an ordinary flood for all subsequent events. When an even greater flood occurs, it then becomes the new standard, and there is a duty to meet the new conditions.

e. With the technological advances in meteorology and hydrology, and with storm events and floods now being discussed nationwide in terms of their statistical probability, it may become increasingly difficult to convince a court or jury that the flood which caused injury was an "extraordinary flood," i.e., one whose magnitude could not be anticipated or foreseen using ordinary diligence.

8. The overriding rule is that natural watercourse and surface water conditions should be maintained wherever possible. Where they are changed, the changes must be designed so that resulting flow conditions will not cause more harm than under natural conditions.

9. The best approach in planning and designing drainage works is to attempt to retain natural and historic conditions of flow.

10. On-site detention of stormwaters should be encouraged, not only because it decreases the size and therefore the cost of storm sewers, but also because it is a safeguard against potential liability for concentrating or increasing surface water runoff.

11. Any embankments constructed to detain or retain water should be safe from failure in the event of larger floods. The Maximum Probable Flood would be a prudent criterion.

12. Wherever possible, artificial channels should follow natural thalwegs. Transbasin diversions which increase natural flow should be avoided unless the risks are adequately evaluated and such diversion is shown to be prudent.

13. Installation of inadequately sized drainage structures should be avoided, especially if such structures cause development and filling of the natural watercourse so that larger flood flows are altered causing damage to properties which would not have been damaged otherwise.

14. Nonstructural floodplain management provides a basis for master planning which has the least exposure for the city in terms of potential liability. It is a natural approach to solving urban drainage problems before they develop, or before they get worse.

15. Municipalities are treated like private parties in watercourse and surface water cases. Governmental immunity as a defense against liability has rarely been mentioned, and never successfully used, in Oklahoma watercourse and surface water cases. Therefore, it would be foolhardy for a city to depend on governmental immunity to protect it from liability in stormwater cases.

16. Floodplain regulations should be viewed, not as governmental interference with private property rights, but as protection of private property against the unlawful use of other private property, which individually or cumulatively would cause flood injury which would not have occurred prior to the development.

17. The federal insurance program's one-foot rise criterion for floodway delineation appears to be inappropriate in Oklahoma. Since this criterion permits full development of the floodplain to the point where the one-percent floodwaters would be one foot higher than under natural conditions, it is almost by definition stating that a city's regulations will result in cumulatively causing more harm than formerly by raising

flood levels. Under Oklahoma watercourse law, if such changes actually cause injury, liability results.

18. New urban development should be required to not materially increase the amount of storm runoff nor change natural drainage conditions. This will protect lower properties. It will also protect the developer from liability, and not place the city in a potential liability position for having permitted the development to alter drainage conditions which result in injury. On the other hand, if the city requires the developer to maintain natural runoff conditions, by whatever means are suitable, it is only complying with the basic principles of Oklahoma law.

19. Drainage planning should be based on runoff which will result from future urban development which can be reasonably anticipated.

20. It is essential to get the facts before undertaking a drainage plan or design. Before starting engineering computations, the following questions should be addressed:
    a. What is the problem? Would preventive measures aid in limiting the problem?
    b. What causes the drainage problem? Where does the water come from? From what lands?
    c. Who will benefit from the corrective solution? Are the benefits sufficient to justify the use of public funds in the amount required?
    d. Is there an identifiable channel or thalweg where the storm runoff will flow? Is it continuous downstream?
    e. Would the proposed corrective action handle the "ordinary flood", that is, a flood whose magnitude can be anticipated by using ordinary diligence? Would it handle the flood of record on that watercourse? Would it handle the one-percent frequency runoff event? In the case of a much larger flood, such as the Standard Project Flood or the Maximum Probable Flood, would the corrective works cause the excessive floodwaters to flow in a different location or direction than they would naturally?

DISTINCTION BETWEEN WATERCOURSE WATERS AND SURFACE WATERS

Stormwater law developed across the Nation by courts deciding the rights, duties, and liabilities between private landowners. A basic distinction was made between surface waters and watercourse waters. Surface waters were waters which ran in a diffused manner overland, or in depressions and swales, while a watercourse had definite banks and bed. Floodwaters which overflowed the banks of the watercourse and followed the course of the stream to its natural outlet, or which upon subsidence returned to the stream, were also held to be governed by the law of watercourses. Floodwaters which had entirely lost their connection with the stream, however, and spread out over the adjoining countryside never to return to the stream, would probably be governed by surface water law. While a "nice" distinction in the law, an obvious problem is at what point in their flow do surface waters collecting in swales and gullies suddenly become watercourse waters. Where state courts have adopted surface water rules which are incompatible with their watercourse rules, the courts are in a real dilemma. Even though the waters are hydrologically all part of the same system, the decision regarding liability may hinge totally on the category into which the errant waters are placed.

Oklahoma courts also have differentiated between watercourse waters and surface waters. A watercourse has been described in Chicago, R. I. & P. Ry. Co. v. Groves, 20 Okl 101, 93 P. 755 (1908); Chicago, R. I. & P. Ry Co. v. Morton, 57 Okl. 711, 157 P. 917 (1916); Garrett v. Haworth, 183 Okl. 569, 83 P. 2d 822 (1938), as follows:

> "Where the natural confirmation of the surrounding country necessarily collects therein so large a body of water, after heavy rain or the melting of large bodies of snow, as to require an outlet as to some common reservoir, and whether such water is regularly discharged through a well-defined channel with which the force of the water has made for itself, and which is the accustomed channel

> through which it flows or has ever flowed, it constitutes a watercourse or waterway."

In addition, areas covered during normal floods by the floodwaters of a watercourse constitute a portion of that watercourse. Town of Jefferson v. Hicks, 23 Okl. 684, 102 P. 79 (1909); Chicago, R. I. & P. Ry. Co. v. Groves (already cited); Cole v. Missouri, K. & O. R. Co., 20 Okl. 227, 94 P. 540 (1908).

Surface waters, on the other hand are:

> "those which, in their natural state, occur on the surface of the earth and places other than definite streams, lakes or ponds, and they may originate from any source and may be flowing vagrantly over broad lateral areas, or occasionally for brief periods, in natural depressions. The essential characteristics of such waters are that they are short lived flows diffused over the ground, and are not concentrated or confined in bodies of water conforming to the definition of lakes or ponds." Dobbs v. Missouri Pacific R. Co., 416 F. Supp. 5, 9 (E.D. Okl. 1975), a federal case involving floodwaters, quoting this definition from an Oklahoma water resources case.

Fortunately, the rules which the Oklahoma courts have adopted regarding these two categories are totally compatible with each other; therefore the distinction has not been critical and in some cases has not even been made. However, since the theories on which the two categories are based are somewhat different, the distinction should still be noted. In addition, the distinctions are convenient and useful. Engineers, for example, speak in terms of major and minor drainage. One must never forget, however, that these waters are part and parcel of the same hydrologic system.

Law of Watercourses

Watercourse law is based on the rights and duties established between riparian property owners, that is, owners of land along the banks of a river or lake. The fundamental principle of the riparian system is that each riparian has an equal right to make a reasonable use of the water of a stream subject to the equal rights of the other riparians to do likewise. A riparian right is reciprocal in character as to other riparian rights. Therefore, a riparian owner must exercise his rights in a reasonable manner and extent so as not to interfere unnecessarily with the corresponding rights of others. Applying these principals to flooding situations, a riparian owner does not have the right to protect his property from the ordinary flood if this causes damage to others in time of flooding. This would prohibit, for example, a riparian from building a dike which would divert ordinary floodwater onto his neighbor's property.

Law of Surface Waters

There are two basic doctrines which courts have adopted regarding surface waters. These are the "common enemy rule" and the "civil law rule". A third has evolved in recent years called the "reasonable use rule".

As originally conceived, under the common enemy rule a landowner may do anything he pleased with surface waters to protect his property from the "common enemy" regardless of the harm it might do to others. The upper landowner could divert or drain surface waters onto the lower land, or the lower landowner could block surface waters flowing onto his property, even if it flooded the upper property. Since the water must go somewhere, this would appear to inevitably result in contests in engineering where might makes right. Therefore, most courts have modified the rule, giving landowners the right to obstruct or divert surface waters, but only where it is incidental to the ordinary use, improvement or protection of their land, and is done without malice or negligence.

Under the civil law rule, the upper landowner has an easement for the natural drainage from his property over the lower property and the lower landowner must take such water. However, the key word here is "natural" meaning those waters which flowed from the land before alteration or development. If he does send down a greater volume, or at greater velocity, or in a different location, he is liable if it does more harm than would have occurred under the former conditions.

The reasonable use rule is based on tort rather than on property law. In tort law, liability is based on negligence. A person can be held negligent if he has not acted like the "reasonably prudent man" in a given situation, and such actions are the proximate cause of the injury. In surface water cases, the test for liability would be the same.

OKLAHOMA WATERCOURSE AND SURFACE WATER LAW

Oklahoma has adopted the usual riparian principles of watercourse law whereby landowners have reciprocal rights and duties towards each other. It has adopted the "common enemy" rule for surface waters, but modified by "the rule of reason". This rule results in liability for landowners who alter natural runoff if such alterations cause injury to others. There is a wealth of cases decided by the Oklahoma Supreme Court over the last 75 years, and they are remarkably consistent.

In the first two cases before the Oklahoma Supreme Court, in 1904 and 1908, the court analyzed the competing doctrines for both surface and watercourse waters and chose and articulated compatible principles which have controlled its decisions ever since. The Oklahoma courts have never had the dilemma of the surface waters/watercourse dichotomy because the results are virtually the same for both categories.

In the 1904 case, Davis v. Frey, 14 Okl. 340, 78 P. 180 (1904), surface waters flowed into a natural depression forming a 15-acre pond from which they evaporated or percolated into the ground. Defendant (upper landowner) cut a channel into the bank of this natural ponding area to drain it. Stormwaters, instead of being detained, flowed immediately onto the lower landowner's farm, damaging his crops. In finding the upper landowner liable, the court adopted the rule from an Iowa case:

> "If the ditch in question increased the quantity of water upon the plaintiff's land to his injury, or without increasing the quantity, threw it upon the plaintiff's land in a different manner from what the same would naturally have flowed upon it, to his injury, the defendant was liable for the damage thus occasioned, even though the ditch was constructed by the defendant in the course of the ordinary use and improvement of his farm. We recognize the fact ... that surface water ... is a common enemy, which each landowner may reasonably get rid of in the best manner possible, but in relieving himself he must respect the rights of his neighbors, and cannot be justified by an act having the direct tendency and effect to make that enemy less dangerous to himself and more dangerous to his neighbor." (14 Okl. 341, 78 P. 181.)

Then in 1908 the first of many railroad cases came before the court. Chicago, R. I. & P. Ry. Co. v. Groves, 20 Okl. 101, 93 P. 755 (1908). The railroad company had built an embankment across a ravine on the plaintiff's land with culverts which were inadequate to carry water which collected in the ravine after heavy rains. The railroad argued that the ravine was not a watercourse and, therefore, it was not violating a statute requiring railroads to restore streams and watercourses so as not to materially impair their usefulness. The court, however, held that the railroad had the duty to provide:

> "sufficient drainage and an outlet to carry off such waters as might be reasonably expected to flow along such channel ... so as to force the water off ... in like manner and in the same channel or place as it flowed prior to the construction of said embankment." (20 Okl. 101, 93 P. 755).

While a landowner has the right to improve his property, this right is qualified by the

> "golden maxim of the common law that one must so use his own property as not to injure the rights of another." (20 Okl. 101, 93 P. 755).

Interestingly, the cases cited and quoted are those which would generally be considered surface water cases, that is, they compare the civil law rule with the common enemy rule. It cites the Davis case as holding that an owner of land cannot collect water into an artificial channel and pour it upon the land of another to his injury, and goes on to state that such an owner cannot interfere with the flow of water in a natural channel either. In finding the railroad liable, the court does not appear to base its decision on statutory liability, but on common law principles; therefore it appears to be saying that whether or not these are surface waters or watercourse waters, such obstructions result in liability.

In one case we have surface waters injuring a lower landowner. In the other case we have watercourse waters injuring an upper landowner. The principle upon which liability is based is essentially the same -- one cannot change natural flow conditions to the detriment of another's property. These two cases set the stage for integrating the principles of surface water and watercourse water from the outset.

If there was any doubt regarding liability in such cases this was quickly dispelled in rapid succession by three more railroad cases and one against a city. Cole v. Missouri, K. & O. R. Co., 20 Okl. 227, 94 P. 540 (1908), held that where an upper riparian (the railroad) changes the channel and obstructs the flow of a watercourse so that at times of ordinary high waters it flows over the lower riparian's land in greater volume, with more violence, or in a different course or manner than would be permitted to flow to him in its natural state, he is liable. The railroad company still argued surface waters and the common enemy rule, but the court stated that water which overflows its banks in times of flooding does not thereby become surface water.

In Town of Jefferson v. Hicks, 23 Okl. 684, 102 P. 79 (1909), the facts were as follows: the plaintiff's farm on one side of the river was somewhat higher than the town site on the other side. Floodwaters would flow through the town, so the town put up a levee, forcing floodwaters onto the plaintiff's land. The court held that the owner of land situated on a watercourse may construct an embankment to protect his lands from flooding; but in so doing he must so place the embankment that the natural and probable consequences of the embankment in times of ordinary floods will not be to cause the overflow to erode, destroy or injure other proprietors on the watercourse. Since recurring floods would continue to cause injury, damages was not an adequate remedy. The plaintiff was granted an injunction; that is, the town had to remove its levee.

In Chicago, R. I. & P. Ry. Co. v. Johnson, 25 Okl. 760, 107 P. 662 (1910), the railroad had built a ditch which accummulated waters from upland farms and carried them through its roadbed, through which it flowed onto plaintiff's farm. In finding the railroad liable for the resulting damage, the court held that one cannot collect waters into an artificial channel or volume and pour it onto the land of another to his injury.

If there had still been any question regarding surface waters being treated any differently than watercourse waters, it was settled in Chicago, R. I. & P. Ry. Co. v. Davis, 26 Okl. 434, 109 P. 214 (1910). The court held that a railroad company has no more right to obstruct surface waters, or by collecting and conducting them, force them to be discharged upon lands of another, than it has in the same way to dispose of waters from a water course. It is liable for the resulting injury in the one case as in the other:

> "The wrong intended to be guarded against is the diversion of water, causing it to flow upon the lands of another without his will, which did not naturally flow there; and it is not deemed material whether the water is diverted from a running stream, or is surface water caused to flow where it did not flow before." (26 Okl. 438, 109 P. 218).

See also Culbertson v. Green, 206 Okl. 210, 243 P. 2d 648 (1952).

The basic theme which runs throughout the cases is that one may not alter the natural flow conditions if such changes cause injury to others. This fundamental theme has been amplified and fleshed out in many cases over decades, and the following legal principles have evolved:

1. Interference with a watercourse: A riparian landowner may take measures to protect himself generally from the harmful effects of flood waters, but it is fundamental that no one may change, divert, obstruct, or otherwise interfere with the natural flow of a watercourse without being chargeable in damages to persons and properties injured thereby. Liability was found in the following areas:

Atchison T. & S. F. Ry. Co. v. Hadley, 168 Okl. 588, 35 P.2d 463 (1934) (railroad embankment and jetties created a narrow "bottle neck", greatly increasing the natural velocity of the current).

Chicago, R. I. & P. Ry. Co. v. Groves, 20 Okl. 101, 93 P. 755 (1908) (obstructed watercourse by embankment with inadequate capacity).

Chicago, R. I. & P. Ry. Co. v. Maynard, 31 Okl. 685, 122 P. 149 (1911) (railroad embankment obstructed a watercourse, floodwaters damaged crops).

Castle v. Reeburgh, 75 Okl. 22, 181 P. 297 (1919) (dammed up a watercourse).

Lowden v. Bosler, 196 Okl. 205, 163 P.2d 957 (1945) (built jetties which restricted the flow, raised the water level; roiling waters deflected onto plaintiff's property).

Garrett v. Haworth, 183 Okl. 569, 83 P.2d 822 (1938) (obstructed a watercourse).

Chicago, R. I. & P. R. Co. v. Schirf, 267 P.2d 574 (Okl. 1954) (railroad trestle filled in, causing waters to back up onto plaintiff's land).

Godlin v. Hockett, 272 P.2d 389 (Okl. 1954) (to protect his subdivision, defendant dredged and deepened creek and built a dike up to 8 feet high, diverting floodwaters onto other riparian lands in increased volume and with greater depth).

Regier v. Hutchins, 298 P.2d 777 (Okl. 1956) (defendant put embankment across ox-bow of river, inundating plaintiff's land to a greater extent than formerly and preventing the water from receding as quickly).

Town of Jefferson v. Hicks, 23 Okl. 684, 102 P. 79 (1909) described above).

George v. Greer, 207 Okl. 494, 250 P.2d 858 (1952) (defendant built dike which caused water, which would otherwise have gone over his own land, to go upon plaintiff's land.)

2. Altering Surface Water Runoff: Where an upper landowner collects surface water, sends it down in a different manner or concentrated form, or in unnatural quantities or velocities, or discharges it in a different location, he is liable for any damage caused thereby. Conversely, a lower landowner may not cast surface waters back onto upper land to the detriment of the upper landowner. The basic principle is that one cannot prevent injury to one's own property by transferring that injury to one's neighbor's property. Oklahoma courts call this "the common enemy rule modified by the rule of reason." Chicago, R.I. & P. Ry. Co. v. Johnson, 25 Okl. 760, 107 P. 662 (1910); Gulf, C. & S. F. Ry. Co. v. Richardson, 42 Okl. 457, 141 P. 1107 (1914); Chicago, R.I. & P. Ry. Co. v. Taylor, 173 Okl. 454, 49 P.2d 721 (1935); Chicago, R. I. & P. Ry. Co. v. Davis, 26 Okl. 434, 109 P. 214 (1910); Kansas City Southern Ry. Co. v. Hurley, 61 Okl. 241, 160 P. 910

(1916); St. Louis & S. F. R. Co. v. Dale, 36 Okl. 114, 128 P. 137 (1912); Wichita Falls & N. W. Ry. Co. v. Stacey, 46 Okl. 8, 147 P. 1194 (1915).

3. Right to restore original bank of watercourse: A riparian owner on a watercourse may construct embankments or other structures necessary to maintain his bank of the stream, or to restore it to its original course when it has encroached upon his land, without becoming liable for injury that such action might cause to other riparian lands.

> Gulf C. & S. F. Ry. Co. v. Clark, 101 F. 678 (8th Cir. 1900) (defendant had built embankment and railroad on solid land, some distance from the bank of the river; river gradually washed away the bank until it swept away part of the embankment; so defendant built a dike which encroached on the new channel but not on the channel as originally located; defendant not liable).
>
> Sinclair Prairie Oil Co. v. Fleming, 203 Okl. 600, 225 P.2d 348 (1950) (defendant built a fence on the location of the original bank which had washed out in a flood, causing plaintiff's land to erode; defendant not liable).
>
> Pechacek v. Hightower, 269 P.2d 342 (Okl. 1954) (both the plaintiff and the defendant built levees; there was a question whether plaintiff did more than just restore, but the jury should have been instructed that she had a right to restore her bank).

4. Limited right to repel unnatural waters: Where one party has caused unnatural water to flow onto another's property, the second party has a right to repel such waters. This right is limited, however, to placing the parties in the same conditions as prior to any construction. Nor may a party, in repelling such waters, cause injury to innocent third parties.

In Dowlen v. Crowley, 170 Okl. 59, 37 P.2d 933 (1934), plaintiff built a dike which cast high waters onto defendant's land, whereupon defendant started to build his own dike. Plaintiff brought an action to stop him. The defendant showed that his dike would not cause more water to flow onto plaintiff's land than if there were no dikes at all. The court refused to halt defendant's dike, stating:

> "A riparian proprietor has no right to construct by dyke, dam, or otherwise, anything which in time of ordinary flood will throw the water in larger volume on the lands of another so as to overflow and injure them, and, when flood waters are diverted by one landowner to the land of another, that other has the right to repel it." (170 Okl. 59, 37 P.2d 933).

In a similar situation involving surface waters rather than a watercourse the court took the same position. Rainey v. Cleveland, 203 Okl. 283, 220 P.2d 261 (1950). Plaintiff (upper landowner) had built ditches and levees which in time of heavy local rains collected and discharged waters onto defendant's land in an excessive, unusual and unnatural volume. Defendant put up a levee for protection. Plaintiff's request for an injunction was denied. Since plaintiff had no right to discharge such waters, defendant had the right to protect himself. See also King v. Cade, 205 Okl. 666, 240 P.2d 88 (1951). The Lynn v. Rainey, 400 P.2d 805 (Okl. 1965), court went even further. Here the upper landowner (plaintiff) was discharging accumulated surface waters onto the lower property. Defendant bought the lower property with these conditions in place, and then built a protective barrier which flooded the upper property. In denying the plaintiff's request for an injunction, the court held that the plaintiff still has no legal right to discharge accumulated surface waters, either by easement, license or prescription. Therefore the defendant had the right to protect himself.

Where a dike built as protection to repel unnatural waters harms a third party, however, such dike may not be maintained. In Gregory v. Bogdanoff, 307 P.2d 841 (Okl. 1957) a drainage district had built a levee to protect a town. This levee turned a greater volume of water onto defendant's property, so he built a dike. This dike, however, caused damage to plaintiff's property (innocent third party), so the court ruled he had to remove it.

5. <u>Detention ponds:</u> Where a party interferes with natural detention, either by filling it in or by cutting through the banks, he is liable for injury to lower landowners caused by change in surface water runoff. Artificial ponds which catch surface water are recognized as beneficial for flood and erosion control, where they do not unreasonably interfere with water rights.

The very first surface water case decided by the Oklahoma Supreme Court in 1904, involved natural detention which created a 15-acre pond. As described in an earlier section, the upper landowner was liable for cutting through its banks resulting in injury to the lower farmer's lands. <u>Davis v. Fry,</u> already cited. In <u>Carter v. Gundy,</u> 259 P.2d 528 (Okl. 1953), defendant's land had formerly been in agriculture and had a low spot which constituted a natural lake in which water gathered and stood after rains. In preparation for residential development, he knocked down a bluff thereby filling in the natural lake. Water which formerly stood on his land now flowed onto plaintiff's land, carrying sand, silt, and debris. Defendant was liable.

In a water rights case a lower property owner objected to an upper proprietor's building of a dam to catch water which flowed across his land. The court held these waters to be surface waters, and not stream waters where riparian rights would attach. Regarding the benefits to be derived from such farm ponds in general, however, the court heard testimony by the Oklahoma Water Resources Board to the effect that there were almost 200,000 farm ponds along dry gullies, draws and intermittent stream channels and that such ponds aided in flood and erosion control. The court recognized that such ponds and lakes are beneficial and should be encouraged where they do not unreasonably interfere with the rights of others.

As such farmlands are converted into subdivisions the farm ponds may be destroyed. The lower property owners probably do not have a right to the maintenance of an artificial pond which causes <u>less</u> runoff than naturally, although the length of time the pond has been there and other factors may affect this decision. However, since urbanization of agricultural land creates more runoff than formerly, it may be prudent for a developer and a ity to retain the detention so that natural conditions are not exceeded by the development.

6. Ordinary and extraordinary floods: If injury to persons or property is due solely to an "extraordinary flood", there is no liability. If, however, someone's negligence, commingled with the "extraordinary flood", was a contributing proximate cause of the injury, such person is liable. Building structures which would injure others during ordinary floods is held to be negligence; therefore, such structures result in liability even during extraordinary floods.

Oklahoma, like most other jurisdictions, makes a distinction between the ordinary and the extraordinary flood, sometimes called an "act of God." If the injury is due solely to an extraordinary flood, then there is no liability. Chicago, R. I. & P. Ry. Co. v. Turner, 141 Okl. 267, 284 P. 855 (1930). It is the defendant's burden to prove that the event was an extraordinary one. Oklahoma City v. Tarkington, 178 Okl. 430, 63 P.2d 689 (1936). However, if the defendant was negligent, and his negligence commingled with the act of God caused the injury, then the defendant is liable. Chicago, R. I. & P. Ry. Co. v. Morton, 57 Okl. 711, 157 P. 917 (1916) (both bridge and culvert inadequate to pass ordinary floods). The plaintiff has the burden of proving defendant's negligence, and that, but for such negligence, the loss would not have occurred. Armstrong, Byrd & Co. v. Illinois Cent. R. Co., 26 Okl. 352, 109 P. 216 (1910). In Town of Jefferson v. Hicks, the distinction was made as follows, quoting 13 Ency. of Law (2d Ed.):

> "An ordinary flood is one, the repetition of which, though at uncertain intervals, might, by the exercise of ordinary diligence in investigating the character and habits of the stream, have been anticipated. An extraordinary flood is one of those unexpected visitations whose coming is not foreseen by the usual course of nature, and whose magnitude and destructiveness could not have been anticipated and prevented by the exercise of ordinary foresight." (23 Okl. 685, 102 P. 80).

Some cases have simply found that the subject floods were ordinary, and therefore the defendant is liable. Town of Jefferson v. Hicks, already cited. Regier v. Hutchins, 298 P.2d 777 (Okl. 1956). In most cases, however, the instructions to the jury (which inform the jury of the law controlling the case) are as follows:

> "You are .. instructed that an 'act of God' such as an unprecedented rainfall and resulting flood, which will excuse from liability, must not only be the proximate cause of the loss, but it must be sole cause. If, however, the injury is caused by an act of God, commingled with the negligence of the defendant as an efficient and contributing cause, and the injury would not have occurred except for such negligence, the defendant would be liable." Chicago, R. I. & P. Ry. Co. v. Morton, 57 Okl. 713, 157 P. 919 (1916).

When the jury finds the defendant liable based on this instruction, one cannot tell whether the jury decided the flood was ordinary, or whether it decided it was extraordinary but coupled with defendant's negligence. See the following cases where defendants were found liable: Missouri, K. & T. Ry. Co. v. Johnson, 34 Okl. 582, 126 P. 567 (1912); Chicago, R. I. & P. Ry. Co. v. McKone, 36 Okl. 41, 127 P. 488 (1912); Chicago, R. I. & P. Ry. Co. v. Bahr, 78 Okl. 78, 188 P. 1058 (1920); Walton v. Bryan, 188 Okl. 358, 109 P.2d 489 (1941); Steirs v. Mayhall, 207 Okl. 219, 248 P.2d 1047 (1952); Black v. Ellithorp, 382 P.2d 23 (Okl. 1963).

Four cases, all arising out of the same fact situation, help to clarify the interrelationship between the "act of God" and defendant's negligence. The floods of 1923 in the Oklahoma City area were held to be extraordinary floods. The June flood was higher than any previous floods, and the October flood was almost 5 times as great as the June flood. In prior years a railroad company had built a bridge and embankment which had sufficient capacity to pass ordinary floodwaters. Then Oklahoma City and the railroad closed these openings to create a settling basin for the city, raised the embankment, diverted the water and constructed a waterway through the embankment. In Oklahoma Ry. Co. v. W. H. Boyd, 140 Okl. 45, 282 P. 157 (1929), evidence showed that this new opening had only one-third the capacity of the former. A civil engineer testified that the new channel had a capacity of only 12,000 cfs, while in his judgment the amount of water to be reasonably anticipated required a capacity of 37,500. The defendant was found negligent. In two additional cases, arising from the same situation, only the measure of damages came before the appellate court, the defendants having been found liable. *Oklahoma Ry. Co. v. Woods*, 164 Okl. 215, 23 P.2d 217 (1933) and *Oklahoma Ry. Co. v. Mary Boyd,* 167 Okl. 151, 28 P.2d 537 (1934). Then in 1936, *Oklahoma City v. Rose,* 176 Okl. 607, 56 P.2d 775 (1936), came before the court involving the same city construction as before. Once again the jury at the trial court level had found the defendant liable. However, in this case the uncontradicted evidence in the record showed that the city's single opening in the embankment had more capacity than the prior three openings combined (about 30,000 cfs); that the greatest flood on record prior to construction was 13,640 cfs. In addition, the city had constructed these structures after consulting with nationally known authorities on the subject and the expenditure of a considerable sum of money in making such investigations. The recommendations of these authorities had been followed. With this evidence, the court reversed the jury's findings as a matter of law. It held that the defendant had not been negligent and that the injuries were due solely to an "act of God."

There have been very few cases in which the "extraordinary flood" has been a successful defense against liability. The first hurdle is proving that the flood was extraordinary. In only three cases has this really resulted in no liability. *Armstrong, Byrd & Co. v. Illinois Cent. R. Co.,* 26 Okl. 352, 109 P. 21[illegible] (1910); *Chicago, R. I. & P. Ry. Co. v. Turner,* 141 Okl. 267, 284 P. 855 (1930); *Oklahoma City v. Rose,* already cited. Note that two of these three cases involved the 1923 floods. In addition, when a flood of greater magnitude than the flood of record occurs, this becomes the new standard. Then one must respond in a timely fashion to the new flood conditions. In *Missouri, K. & T. Ry. Co. v. Johnson,* 34 Okl. 582, 126 P. 567 (1912), a company had built a roadbed, bridge and culvert across a narrow valley just below the plaintiff's property; these were adequate for conditions known at that time, that is, in 1903. Then came the May, 1908, flood which put eight feet of water onto plaintiff's land, more than ever before in the history of the river. Then in October of that same year an even larger flood occurred, flooding plaintiff's land twelve feet deep. In finding the railroad company liable the court made the following analysis:

> "(I)f nothing had occurred since the original construction of the road to demonstrate the insufficiency of the construction prior to the October flood, defendant would have been entitled to an instructed verdict. If, however, after the original construction of the road, and prior to the flood in question here, other floods of an unprecedented character came, demonstrating the faulty construction of the roadbed, or the inadequacy of the waterway left under the bridge, then ... a new standard of obligation was erected for the defendant, and it was its duty to meet the new conditions thus established." (34 Okl. 584, 126 P. 569).

Note that the "new standard of obligation" was created in May of 1908, that is, just five months prior to the flood injuries for which defendant was liable. See also Pahlka v. Chicago, R. I. & P. Ry. Co., 62 Okl. 223, 161 P. 544 (1916).

In addition, great strides have been made in meteorology and hydrology. Storms and floods are discussed in terms of their statistical probability. The federal insurance program, many state and local floodplain maps, and floodplain management programs are based on the one-percent flood (100-year flood). The U.S. Army Corps of Engineers uses the Standard Project Flood for design purposes (about a 500-year flood). It may, therefore, become increasingly difficult to convince a court or a jury that a given flood was one which could not be anticipated in the exercise of ordinary diligence, whose coming was unforeseen, and whose magnitude could not have been anticipated by the exercise of ordinary foresight (Oklahoma's definition of an extraordinary flood).

Then, of course, the second hurdle is that the defendant can still be held liable in the extraordinary flood situation if his negligent actions were a proximate and contributing cause of the injury. Here the cases hold that if *defendant's structures were inadequate for the ordinary, precedented, anticipatable flood*, then he is liable even in the extraordinary flood event. If one assumes that the one-percent flood could still be considered an extraordinary event, then one could still be held liable for injury resulting from the one-percent or greater flood if one has not accommodated the ordinary flood, which, at a minimum, is the flood of record. On the other hand, if one assumes that the one-percent flood is now considered to be an ordinary flood, then if one does not adequately provide for the one-percent flood, one can also be liable for the greater flood event.

7. Municipal liability: Municipalities are treated like private parties in watercourse and surface water cases.

In Gulf, C. & S. F. Ry. Co. v. Richardson, 42 Okl. 457, 141 P. 1107, (1914) the court had to rule specifically on the issue of whether or not municipalities were a breed apart. The city had gathered surface waters via its streets and discharged them onto the railroad right-of-way. The railroad, in turn, wished to place culverts through its roadbed which would discharge these waters onto plaintiff's land. The trial court held the railroad liable but discharged the city. In reversing and remanding the court stated:

> "The law makes no distinction in such cases between natural and artificial persons in the duty it imposes. The law holds the proprietor of the estate to the same obligation in the disposition of surface waters, whether he be a farmer, a municipality, or a railway corporation." (42 Okl. 457, 141 P. 1110).

Five years previously, of course, the court had already required the Town of Jefferson to remove its dike which was diverting floodwaters of a watercourse onto Hicks' property. Town of Jefferson v. Hicks, already cited. Other cases involving municipalities described in previous sections of this report are Oklahoma Ry. Co. v. W. H. Boyd, 140 Okl. 45, 282 P. 157 (1929); Oklahoma Ry. Co. v. Woods, 164 Okl. 215, 23 P.2d 217 (1933); Oklahoma R. Co. v. Mary Boyd, 167 Okl. 151, 28 P.2d 537 (1934); Oklahoma City v. Rose, 176 Okl. 607, 56 P.2d 775 (1936); Oklahoma City v. Tarkington, 178 Okl. 430, 63 P.2d 689 (1936). Additional cases are described below.

In Incorporated Town of Idabel v. Harrison, 42 Okl. 469, 141 P. 1110 (1914), the town had constructed drainage ditches along a number of streets. These ditches gathered surface waters which fell over a large area of land, conducting them to a street abutting plaintiff's residential lots. Heavy rains resulted in injury to plaintiff's property. The court held that it was settled law that the owner of the land has no right to gather and accumulate surface waters and conduct them in large volumes onto land of an adjoining proprietor to his injury.

In *Oklahoma City v. Bethel,* 175 Okl. 193, 51 P.2d 313 (1935) the city had built a municipal storm sewer system designed to drain a considerable area of the city. The outlet was to a ditch, which was inadequate to carry the collected storm waters from a 3.96-inch rain. The plaintiff's amusement park was flooded. The court held the following jury instructions to be proper:

> "(I)n the exercise of its corporate powers a municipal corporation has no power or authority to collect water by artificial means and to discharge it or permit it to discharge or overflow upon the premises of an adjacent owner in greater volumes or velocity than it would naturally flow there prior to the construction of such sewer." (175 Okl. 197, 51 P.2d 317).

In addition, it stated that the following was a general and almost universal rule (quoting 43. C. J. 1145):

> "A municipality cannot, without rendering itself liable for the resulting damage, exercise its right to construct drains or sewers and grade or otherwise improve streets so as to collect surface waters in artificial channels and discharge it in increased quantities, or in new and destructive currents, upon private property." (175 Okl. 197, 51 P.2d 317).

It should be noted that in neither of these two cases is there evidence that the city owns the lands which are being drained. The courts do not even discuss the matter. Apparently the rules of surface waters are not narrowly applied to actual owners of property; or, the ownership requirement, if any, is satisfied by the fact that the city owns the facilities. Taking this concept one step further, how would a court rule in the following situation: A subdivider takes agricultural land and builds thereon homes, carports, sidewalks, streets and storm sewers, all in accordance with city specifications as established in city ordinances. The city has annexed the property and approved the subdivision plat. The public facilities (streets, storm sewers, water lines, etc.) are dedicated to the city as part of the subdivision and annexation process. Because of the impermeability of the development, and because storm sewers and streets facilitate movement of runoff, the subdivision causes more surface water, with greater velocity, and in a different manner to be discharged onto lower proprietors. No compensating detention facilities were incorporated into the project in an attempt to maintain natural runoff conditions, nor were such detention facilities required by the city. The lower property owners sue both the developer and the city for the harm to their property caused by the changed runoff.

There are three additional cases which may be pertinent in the above hypothetical situation. These cases hold that the duty to prevent injury caused by altering surface water and watercourse conditions is a nondelegable duty. Oklahoma Ry. Co. v. W. H. Boyd, 140 Okl. 45, 282 P. 157 (1929), described in a previous section, involved raising the railroad embankment, closing culverts, and diverting water through a new culvert, in order to form a municipal settling basin. The defendant railroad raised the defense that the city, not the company, had actually done the construction, and was its only beneficiary. The court, however, was not convinced by this argument. It held that the railroad company, being:

> "under obligation imposed upon it by law to leave sufficient openings through its embankment for the flow of water to be reasonably anticipated, could not delegate the duty of rebuilding the embankment to another, so as to escape liability for the violation of a positive legal duty owing to third persons." (140 Okl. 50, 282 P. 162).

It held the city and the railroad to be joint tortfeasors. In Allied Hotels, Ltd. v. Barden, 389 P.2d 968 (Okl. 1964), a Ramada Inn was built which caused surface water to flow in greater volume onto plaintiff's residence. The motel owner argued that all of the construction had been performed by an independent contractor. Again, the court held that an owner owes a nondelegable duty to adjacent landowners to refrain from causing injury. One who owes such a duty to third persons cannot escape the obligation of performing his duty by engaging for its performance by a contractor. See also Garrett v. Haworth, 183 Okl. 569, 83 P.2d 822 (1938).

Large subdivisions annexing to cities or developing inside corporate boundaries are a fact of modern life. Many municipal facilities such as water lines sewers, streets, and storm drains in such subdivisions are no longer actually constructed by municipal crews but are constructed by the subdivider in conjunction with the homes themselves in accordance with city specifications and approval. In light of the fact that municipalities are treated like other parties in surface water cases, and in accordance with the duty imposed on municipalities in Oklahoma courts, would a court really refuse to "pierce the corporate veil" by discharging the city of responsibility in such situations? Or would it find that the city and the developer are joint tortfeasors; that since the city owns or will own the public facilities built by the developer, it cannot avoid liability by attempting to delegate a nondelegable duty to another party; and that it cannot, via a third party, collect water by artificial means and discharge it or permit to be discharged onto the premises of an adjacent owner in greater volumes or velocity than it would naturally flow there prior to such construction?

Municipalities have also been defendants in watercourse cases. In Herwig v. City of Guthrie, 182 Okl. 599, 78 P.2d 793 (1938), the city had built a dam across the channel creating a water supply reservoir. Plaintiff had property upstream and above the high water line of the reservoir and maintained that the lake retarded the ordinary rapid flow of water across her land to such an extent that sediment was deposited, forming a "secondary dam" and that this obstruction caused overflow and injury. The trial court had directed the verdict for the city, but the appellate court reversed. The question of whether the city had obstructed a natural watercourse, and whether this had resulted in injury to the upper riparian, were questions of fact for the jury.

A city has also been liable where it failed to remove a temporary dike, built to divert river water while it repaired a water line, which caused flooding to a farmer's land and crops. Elk City v. Rice, 286 P.2d 275 (Okl. 1955).

In Murduck v. City of Blackwell, 198 Okl. 171, 176 P.2d 1002 (1947), the city was liable for injury to plaintiff's land caused by interference with his drain tile. The city had built a water supply reservoir whose high-water line was higher than the outlet of the drain tile. When the river overflowed its banks, water which formerly could have been drained from plaintiff's land via the drain, backed up, causing injury to crops and buildings.

These cases, together with the cases cited at the beginning of this section, find cities liable for interfering with or obstructing watercourses. A municipality is liable when it constructs the obstruction itself, or when it contracts for such construction. Would it also be liable for granting a permit to a private party for constructing an obstruction if it knows or should have known such obstruction would cause injury to other properties? If the dike in Town of Jefferson v. Hicks (already cited) had been built not by the town to protect the town, but by a subdivider to protect his subdivision which was part of the town, and built with the town's approval, would Hicks have had a cause of action against the town? A city's permitting the placing of fill to elevate a subdivision to protect it from flooding would be a similar situation, if such fill diverts ordinary floodwaters onto property where it would not have flowed previously, or not to the same height or velocity. Another would be the channelizing of a watercourse by a developer as required by a city, which causes greater volumes and velocity of floodwater on downstream property. These are issues which will probably be raised in Oklahoma courts.

8. Governmental Immunity: Governmental immunity as a defense against liability has rarely been mentioned, and never successfully used, in Oklahoma watercourse and surface water cases.

As can be seen from the above cases, municipalities are treated like private parties in surface water and watercourse cases. Where is the traditional defense known as "governmental immunity"? The concept of governmental immunity was derived from the old English common law principle that "the King can do no wrong". While it has long since been abrogated in England, there are still vestiges of the doctrine in some states, including Oklahoma. A distinction which was made throughout the United States and in Oklahoma, however, was between a municipality's governmental and its proprietary functions, being immune to liability in the former, and liable for its tortious acts in the latter. See discussion in City of Oklahoma v. Hill, 6 Okl. 114, 50 P. 242 (1897). Which functions are governmental versus proprietary has given rise to many cases nationwide. Maintenance of public facilities such as water and sewers has generally been held to be a proprietary function. In City of Holdenville v. Moore, 293 P.2d 363 (Okl. 1956) where the injuries sustained were caused by the city's failure to properly maintain and repair its sewers, the city was liable. However, in City of Altus v. Martin, 268 P.2d 228 (Okl. 1954), the defendant city argued that the injury was caused by faulty design of its sewers, that design is a governmental function, and therefore that it was immune. The court decided that the injury was caused by improper maintenance and repair, however, so it did not have to decide whether design of sewers is a governmental function.

The distinction between governmental and proprietary may result in very arbitrary decisions. In Oklahoma City v. Taylor, 470 P.2d 325 (Okl. 1970), an automobile accident was caused by a city employee driving a city truck to a place where he was going to install some guardrail posts. The court decided that this activity was incidental to the maintenance and repair of the city streets, a proprietary function, and found the city liable. Justice McInerney, in his concurring opinion points out that the city's liability for tort is based on too tenuous a legal theory -- if the employee had been driving the truck to a place where he would repair a traffic signal, the city would apparently not have been liable! The dissenting opinions by Justice Hodges and Justice McInerney in Newman v. State ex rel. Board of Regents, etc., 490 P.2d 1079 (Okl. 1971) may foreshadow the abrogation of governmental immunity in Oklahoma altogether.

More importantly, however, in the area of watercourse and surface water law, there are only a few Oklahoma cases which even address governmental immunity. One of these is Oklahoma City v. Hoke, 75 Okl. 211, 182 P. 692 (1919), where the city rebuilt its water supply dam to a higher level after a flood, causing plaintiff's property to be flooded. Governmental immunity was raised but rejected on the traditional basis that in supplying water, a city is operating like a business corporation (proprietor). Whether this absence of governmental immunity as a defense is (1) because it is not raised, (2) because the activities which affect surface water and watercourses automatically fall into the proprietary category, (3) because surface water and watercourse law is based more on property than tort law, or (4) simply because the Oklahoma courts have established these rules and decided that municipalities are to be held to them also -- the fact is that in the final analysis municipalities have been found liable. It would, therefore, be foolhardy for any municipality to depend on governmental immunity as a defense against liability in watercourse and surface water situations.

9. Remedies: Wherever the law recognizes a right, it also provides a remedy. In stormwater law, several remedies are available.

If the illegal act has caused injury, such as destroying crops, damages are assessed. Castle v. Reeburgh, 75 Okl. 22, 181 P. 297 (1919). If the situation is such that injury could recur in future floods, the court may grant damages in the amount of the permanent depreciated value of the property. Chicago, R. I & P. Ry. Co. v. Davis, 26 Okl. 434, 109 P. 214 (1910).

A more appropriate remedy, however, may be to remove the offending structure, in which case the court will grant an injunction (after the fact). Town of Jefferson v. Hicks, already cited.

Where such a structure has not yet been built, but the court is convinced that it would cause injury in the future, it may grant an injunction to prevent its construction. McLeod v. Spencer, 60 Okl. 89, 159 P. 326 (1916).

Or the court may combine several remedies, Miller v. Marriott, 48 Okl. 179, 149 P. 1164 (1915) (damages and injunctions), or fashion a remedy appropriate for the situation. Where defendant's drainage ditch was causing erosion to plaintiff's land, and the land could be protected at small expense by structural improvements, the court denied the injunction but required the improvements. Kollman v. Pfennig, 196 Okl. 186, 163 P.2d, 534 (1945).

FINANCING THE PROJECT: THE DRAINAGE AND FLOOD CONTROL UTILITY AND FEE

Communities have long found it difficult to finance drainage projects. One community, Billings, Montana, developed an imaginative solution. It decided to view drainage projects as part of a drainage utility, just like water and sewer projects, and would charge customers for the services provided. Property owners whose runoff drained into city storm sewers would be considered customers of the storm sewer utility just like citizens whose homes used city water and sewer services. The fee charged would essentially be based on the difference between historic runoff and the amount of runoff from the property in its developed state. The reasoning was that under natural conditions a considerable amount of stormwater percolates into the ground. However, where land is covered with homes, carports, parking lots, etc., the surface is impermeable, producing much more runoff, at greater velocity, causing higher peak flows than naturally. Commercial establishments which usually have more impervious surface than residential property, would be charged a higher rate. The proposal was challenged in court in City of Billings v. Nore, 148 Mont. 96, 417 P.2d 458 (1966). The proposal was upheld as constitutional and equitable, and has since been implemented.

Other communities like Boulder, Colorado, have also adopted and implemented the drainage utility and fee concept. Additional refinements to the basic concept have been made, such as:

1. Giving credit for on-site detention; since the amount of runoff will be less, the drainage fee is reduced; giving credit is an incentive to on-site storage, which keeps runoff as close to natural as possible.

2. Providing that the revenue produced by the fee can be used not only for structural projects, but also for nonstructural measures such as purchase of land or easements to preserve a natural drainageway.

3. Providing for calculating actual runoff from a particular parcel, such as a shopping center, in order to more precisely determine the fee.

4. Adding a surcharge to the drainage fee for developed properties situated in a floodplain or flood hazard area because of the extraordinary public costs involved in protecting the properties and in providing emergency services in the event of a flood.

A drainage plan is of little value unless it is implemented. While some aspects can be implemented through zoning, subdivision regulations and building permits, corrective actions are usually costly, and financial resources are needed to implement such projects. This drainage fee concept, based on the difference between natural runoff and developed runoff, is particularly appropriate under Oklahoma's surface water law.

MANAGEMENT OF STORMWATER BY MUNICIPALITIES

Management of stormwater in a city is as important to the health, safety, and welfare of its citizens as providing water, sewer, transportation, streets, parks, and recreational facilities. It is part of the total urban system, and includes managing surface waters, watercourses, and their floodplains. As urbanization occurs, changes are made in natural flow conditions. Whether by default or inaction, or by positive action and policies, a city is affecting stormwater flows.

It is obvious from the many surface water decisions that if natural runoff conditions are changed -- in amount, velocity, location, etc. -- to do more harm than formerly, liability results. Where a city simply requires that a developer build streets, storm sewers, shopping centers and parking lots so as to move storm runoff as quickly as possible off the development, it is placing him in a very vulnerable position regarding liability to lower property owners. The city itself may be in a vulnerable position for authorizing or requiring such action. If, on the other hand, the city requires that the developer maintain natural runoff conditions, by whatever means are suitable, it is only complying with the basic principles of Oklahoma law.

Activities along the watercourse and its floodplain are considerably more complex. What makes implementing watercourse law in cities more complex than controlling surface waters is that (1) it may be the cumulative effect of many structures, rather than any single structure, which causes the harm, and (2) it may involve not only how the property is to be developed, but whether it can be developed at all. This immediately gets into the realm of constitutionality, as the prohibiting regulation is challenged as an unconstitutional "taking" of private property without compensation. It is important, however, to analyze such regulations in terms of Oklahoma watercourse law.

Oklahoma decisions state that it is unlawful to interfere with the flow of ordinary floodwaters to the detriment of other property owners. Ordinary floodwaters include those which can be anticipated by a reasonably diligent analysis of the stream, its characteristics, and its history. With today's technology, a diligent analysis would certainly include rainfall/runoff relationships and storm rainfall probability. The ordinary flood includes at least the flood of record and may include larger floods. If one affects the flow so that it would result in harm to others during an ordinary flood, one is also liable even when the flood damage occurs during an extraordinary flood:

Certainly the city's own activities should comply with watercourse law. Regarding private developments, the city may be the only entity which has the overview, and the overall authority, to implement the law. Most cities have zoned property for specific uses and require building permits and compliance with subdivision regulations. In a rural situation it may be fairly easy to point the finger at the transgressor who interferes with the flood flows. In the urban situation it may be an accumulation of filling, channelizing, diking and placing structures which results in the unlawful interference. As courts have said again and again, no one is permitted to sacrifice his neighbor's property for his own benefit. Floodplain regulation, then, should not be viewed as governmental interference with private property rights, but as protection of private property against unlawful use of other private property, which individually or cumulatively would cause flood injury which would not have occurred prior to development. On the other hand where, by its own policies and regulations, a city permits violations of Oklahoma watercourse law, the individual property owner who is harmed or sees a potential threat has to fend for himself by suing for damages or an injunction; it may be very difficult to prove cause and effect in an urban cumulative situation. In addition, the city itself may be vulnerable to liability where it authorized the developments.

The authority of municipalities to promulgate floodplain regulations is presently being tested in the Oklahoma courts. City of Tulsa v. Morland Development Company and Newcomb Cleveland, Oklahoma Supreme Court Case No. 49621. The court may rule that even though the authority to promulgate such regulations is not expressly granted, that such authority is necessarily implied in the granted powers. Morehead v. Dyer, 518 P.2d 1105 (Okl. 1974). The Attorney General of Oklahoma issued Opinion No. 70-234, approved October 8, 1970, which states:

> "Oklahoma cities and towns presently have authority under State statutes to participate in the National Flood Insurance Program of 1968, and to establish land use and control measures, and to adopt and enforce zoning ordinances, subdivision regulations, building codes and other regulations to provide safe standards of occupancy for and prudent use of flood prone areas

> pursuant to such participation." (See Appendix B, page 5).

Without going into the details of floodplain regulations, mention should be made of the minimum floodway criterion required by the federal insurance program. The federal one-foot rise criterion appears to be inappropriate in Oklahoma. Since it permits full development of the floodplain to the point where the one-percent floodwaters would be one foot higher than under natural conditions, it is almost by definition stating that the local government's regulations will result in cumulatively causing more harm than formerly by raising flood levels. Under watercourse law, if such changes actually cause injury, liability results.

APPENDIX III-A

CASES CITED

CASES CITED

Allied Hotels, Ltd. v. Barden, 389 P.2d 968 (Okl. 1964).

Armstrong Byrd & Co. v. Illinois Cent. R. Co., 26 Okl. 352, 109 P 216 (1910).

Atchison, T. & S. F. Ry. Co. v. Bishop, 201 Okl. 471, 207 P.2d 282 (1949).

Atchison, T. & S. F. Ry. Co. v. Eldridge, 41 Okl. 463, 139 P. 254 (1914).

Atchison, T. & S. F. Ry. Co. v. Hadley, 168 Okl. 588, 35 P.2d 463 (1934).

Baker v. Ellis, 292 P.2d 1037 (Okl. 1956).

Black v. Ellithorp, 382 P.2d 23 (Okl. 1963).

Branch v. City of Altus, 195 Okl. 625, 159 P.2d 1021 (1945).

Burkett v. Bayes, 78 Okl. 8, 187 P. 214 (1920).

Carter v. Gundy, 259 P.2d 528 (Okl. 1953).

Castle v. Reeburgh, 75 Okl. 22, 181 P. 297 (1919).

Caughlin v. Sheets, 206 Okl. 283, 242 P.2d 724, (1952).

Chicago, R.I. & P. Ry. Co. v. Bahr, 78 Okl. 78, 188 P. 1058 (1920).

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Chicago, R.I. & P. Ry. Co. v. Groves, 20 Okl. 101, 93 P. 755 (1908).

Chicago, R.I. & P. Ry. Co. v. Hale, 208 Okl. 141, 254 P.2d 338 (1953).

Chicago, R.I. & P. Ry. Co. v. Johnson, 25 Okl. 760, 107 P. 662 (1910).

Chicago, R.I. & P. Ry. Co. v. Maynard, 31 Okl. 685, 122 P. 149 (1911).

Chicago, R.I. & P. Ry. Co. v. McKone, 36 Okl. 41, 127 P. 488 (1912).

Chicago, R.I. & P. Ry. Co. v. Morton, 57 Okl. 711, 157 P. 917 (1916).

Chicago, R.I. & P. Ry. Co. v. Schirf, 267 P.2d 574 (Okl. 1954).

Chicago, R.I. & P. Ry. Co. v. Taylor, 173 Okl. 454, 49 P.2d 721 (1935).

Chicago, R.I. & P. Ry. Co. v. Turner, 141 Okl. 267, 284 P. 855 (1930).

City of Altus v. Martin, 268 P.2d 228 (Okl. 1954).

City of Billings v. Nore, 148 Mont. 96, 417 P.2d 458 (1966).

City of Holdenville v. Moore, 293 P.2d 363 (Okl. 1956).

City of Oklahoma v. Hill, 6 Okl. 114, 50 P. 242 (1897).

City of Tulsa v. Morland Development Company and Newcomb Cleveland, Oklahoma Supreme Court Case No. 49621.

Cole v. Missouri, K. & O. R. Co., 20 Okl. 227, 94 P. 540 (1908).

Conkin v. Ruth, 581 P.2d 923 (Okl. App. 1978).

Culbertson v. Green, 206 Okl. 210, 243 P.2d 648 (1952).

Cunningham v. McConnell, 382 P.2d 429 (Okl. 1963).

Davis v. Fry, 14 Okl. 340, 78 P. 180 (1904).

Dobbs v. Missouri Pacific R. Co., 416 F.Supp. 5 (E.D. Okl. 1975).

Dowlen v. Crowley, 170 Okl. 59, 37 P.2d 933 (1934).

Elk City v. Rice, 286 P.2d 275 (Okl. 1955).

Franks v. Rouse, 192 Okl. 520, 137 P.2d 899 (1943).

Garrett v. Haworth, 183 Okl. 569, 83 P.2d 822 (1938).

George v. Greer, 207 Okl. 494, 250 P.2d 858 (1952).

Godlin v. Hockett, 272 P.2d 389 (Okl. 1954).

Gregory v. Bogdanoff, 307 P.2d 841 (Okl. 1957).

Gulf, C. & S. F. Ry. Co. v. Clark, 101 F. 678 (8th Cir. 1900).

Gulf, C. & S. F. Ry. Co. v. Richardson, 42 Okl. 457, 141 P. 1107 (1914).

Haskins v. Felder, 270 P.2d 960 (Okl. 1954).

Hatmaker v. Gripe, 184 Okl. 26, 84 P.2d 418 (1938).

Herwig v. City of Guthrie, 182 Okl. 599, 78 P.2d 793 (1938).

Horschler v. Graham, 306 P.2d 701 (Okl. 1957).

Incorporated Town of Idabel v. Harrison, 42 Okl. 469, 141 P. 1110 (1914).

Iven v. Roder, 431 P.2d 321 (Okl. 1967).

Johnston v. Woodard, 376 P.2d 602 (Okl. 1962).

Kansas City Southern Ry. Co. v. Hurley, 61 Okl. 241, 160 P. 910 (1916).

King v. Cade, 205 Okl. 666, 240 P.2d 88 (Okl. 1951).

Kollman v. Pfenning, 196 Okl. 186, 163 P.2d 534 (1945).

Lowden v. Bosler, 196 Okl. 205, 163 P.2d 957 (1945).

Lynn v. Rainey, 400 P.2d 805 (Okl. 1965).

McLeod v. Spencer, 60 Okl. 89, 159 P. 326 (1916).

Miller v. Marriott, 48 Okl. 179, 149 P. 1164 (1915).

Missouri, K. & T. Ry. Co. v. Johnson, 34 Okl. 582, 126 P. 567 (1912).

Morehead v. Dyer, 518 P.2d 1105 (Okl. 1974).

Murduck v. City of Blackwell, 198 Okl. 171, 176 P.2d 1002 (1947).

Newman v. State ex. rel. Board of Regents, etc., 490 P.2d 1079 (Okl. 1971).

Nunn v. Osborne, 417 P.2d 571 (Okl. 1966).

Oklahoma City v. Bethel, 175 Okl. 193, 51 P.2d 313 (1935).

Oklahoma City v. Hoke, 75 Okl. 211, 182 P. 692 (1919).

Oklahoma City v. Rose, 176 Okl. 607, 56 P.2d 775 (1936).

Oklahoma City v. Tarkington, 178 Okl. 430, 63 P.2d 689 (1936).

Oklahoma City v. Taylor, 470 P.2d 325 (Okl. 1970).

Oklahoma Ry. Co. v. Mary Boyd, 167 Okl. 151, 28 P.2d 537 (1934).

Oklahoma Ry. Co. v. W. H. Boyd, 140 Okl. 45, 282 P. 157 (1929).

Oklahoma Ry. Co. v. Woods, 164 Okl. 215, 23 P.2d 217 (1933).

Pahlka v. Chicago, R. I. & P. Ry. Co., 62 Okl. 223, 161 P. 544 (1916).

Pechacek v. Hightower, 269 P.2d 342 (Okl. 1954).

Rainey v. Cleveland, 203 Okl. 283, 220 P.2d 261 (1950).

Regier v. Hutchins, 298 P.2d 777 (Okl. 1956).

Roberts v. Sterr, 312 P.2d 449 (Okl. 1957).

Sebranek v. Krivohlavek, 196 Okl.132, 163 P.2d 530 (1945).

Sinclair Prairie Oil Co. v. Fleming, 203 Okl. 600, 225 P.2d 348 (1950).

Sorge v. Graham, 312 P.2d 929 (Okl. 1957).

Stiers v. Mayhall, 207 Okl. 219, 248 P.2d 1047 (1952).

St. Louis & S. F. R. Co. v. Dale, 36 Okl. 114, 128 P. 137 (1912).

St. Louis - San Francisco Railway Co. v. Pinkston, 420 P.2d 537 (Okl. 1966).

Taylor v. Shriver, 82 Okl. 11, 198 P. 329 (1921).

Town of Jefferson v. Hicks, 23 Okl. 684, 102 P. 79 (1909).

Walton v. Bryan, 188 Okl. 358, 109 P.2d 489 (1941).

Wichita Falls & N. W. Ry. Co. v. Stacey, 46 Okl. 8, 147 P. 1194 (1915).

Zalaback v. City of Kingfisher, 59 Okl. 222, 158 P. 926 (1916).

APPENDIX III-B

OPINION OF ATTORNEY GENERAL

ON

NATIONAL FLOOD INSURANCE ACT

SEPTEMBER 17, 1970

1907

THE ATTORNEY GENERAL
OF OKLAHOMA
Oklahoma City, Okla. 73105

G. T. BLANKENSHIP
ATTORNEY GENERAL

September 17, 1970

Honorable Rex Privett
Speaker, House of Representatives
Honorable Finis W. Smith
President Pro Tempore of the Senate
Chairman and Vice Chairman of
Committee on Interstate Cooperation
State Capitol
Oklahoma City, Oklahoma 73105

Opinion No. 70-234

Gentlemen:

The Attorney General has had under consideration your recent letter relative to the National Flood Insurance Act, of 1968, as amended in 1969. You ask, in effect, the following questions:

> 1. Do cities, towns, and counties in Oklahoma have the authority to participate in this National Flood Insurance program?
>
> 2. Do they have the authority to establish land use and control measures, zoning ordinances, subdivision regulations, and other applications and extensions of the normal police power to provide safe standards of occupancy for, and prudent use of, flood prone areas?

Title 42 U.S.C., § 4011, provides in relevant part that:

> "(a) To carry out the purposes of this chapter, the Secretary of Housing and Urban Development is authorized to establish and carry out a national flood insurance program which will enable interested persons to purchase insurance against loss resulting from physical damage to or loss of real property or personal property related thereto arising from any flood occurring in the United States."

Title 42 U.S.C., § 4012, provides in part as follows:

> "(c) The Secretary shall make flood insurance available in only those states or areas (or sub-division thereof) which he has determined have --
>
> "(1) Evidenced a positive interest in securing flood insurance coverage under the Flood Insurance Program, and
>
> "(2) Given satisfactory assurance that by December 31, 1971, adequate land use and control measures will have been adopted for the State or area (or sub-division) which are consistent with the comprehensive criteria for land management and use developed under Section 4102 of this Title, and that the application and enforcement of such measures will commence as soon as technical information on floodways and on controlling flood elevations is available."

Title 42 U.S.C., § 4022, provides:

> "After December 31, 1971, no new flood insurance coverage shall be provided under this chapter in any area (or sub-division thereof) unless an appropriate public body shall have adopted adequate land use and control measures (with effective enforcement provisions) which the Secretary finds are consistent with the comprehensive criteria for land management and use under Section 4102 of this Title."

Title 42 U.S.C., § 4102(a), authorizes the Secretary to carry out studies and investigations, using available state, local and federal sources, with respect to the adequacy of state and local measures in flood prone areas, etc. It provides under (b) that such studies and investigations shall include, but not be limited to, laws, regulations, or ordinances relating to encroachments and obstructions on stream channels and floodways, the orderly development and use of flood plains of rivers or streams, floodway encroachment lines and flood plain zoning, building codes, building permits, and subdivision or other building restrictions. It further provides, under (c), that the Secretary shall from time to time develop comprehensive criteria designed to encourage where necessary the adoption of permanent state and local measures which, to the maximum extent feasible, will --

> "(1) Constrict the development of land which is exposed to flood damage where appropriate,
>
> "(2) Guide the development of proposed construction away from locations which are threatened by flood hazards,
>
> "(3) Assist in reducing damage caused by floods, and
>
> "(4) Otherwise improve the long range land management and use of flood prone areas."

Under 11 O.S.1961, §§ 401 through 412, as amended in 1968, 1969 and 1970, Oklahoma cities and towns are authorized to establish land use and control measures, and to adopt and enforce ordinances, subdivision regulations, building codes, and other regulations pertaining to the public health and welfare in respect to areas within the jurisdiction of their respective legislative bodies.

The 32nd Oklahoma Legislature, at its second regular session, enacted Senate Bill No. 320, effective April 28, 1970, which provided in its Title for "County Planning and Zoning." However, the body of the Act contains no reference to zoning or authority to establish regulations, other than with respect to "Planning."

Title 19 O.S. 1961, §§ 863.1 through 863.29, as amended, provided for city county planning and zoning by counties having cities with a certain population and more than 50% of their incorporated area within the county. However, in Elias v. City of Tulsa, Okl. 408 P.2d 517 (1965), the Supreme Court held:

> ". . . that Chapter 19Aa, S.L. 1955, 19 O.S. Supp.1955, §§ 863.1-863.43 is unconstitutional."

Title 19 O.S. 1961, §§ 866.1 through 866.36, as amended, provides for the creation, by one or more counties and certain municipalities located therein, of Metropolitan Area Planning Commissions. Specific powers are given to participating counties to establish zoning regulations, building codes, construction codes, and housing codes, for all the area located within three miles of the municipality, or within one-fourth mile of any State or Federal Highway located anywhere in the county, or within one-half mile of any water supply or reservoir owned by the municipality, excluding, however, any incorporated area. . . .

Title 19 O.S. Supp.1969, §§ 866.2 and 866.36 were respectively §§ 1 and 2 of O.S.L. 1965 Regular Session, Thirtieth Legislature, ch. 403, which was approved July 5, 1965, and contained the emergency clause and a provision for codification in Title 19 O.S. Supp.1965.

Section 866.2, as reenacted provides:

> ". . . .In every county of this state having an upstream terminal port and turnaround where navigation ends, or in any county containing all or any part of a reservoir or reservoirs constructed by the United States Army Corps of Engineers or by the Grand River Dam Authority, such county is hereby granted authority, at the discretion of the board of county commissioners, to establish zoning regulations, a building code and construction codes, and a housing code in accordance with the provisions of this act for all or any part of the unincorporated area within the county. . . ." (Emphasis added)

Section 866.2 was amended by the addition of the following paragraph:

> "In the counties in which a Lake Area Planning and Zoning Commission is authorized as provided above, said commission may be created by the Board of County Commissioners of said counties as provided in this act and said commission may exercise all the powers and authority hereinafter provided for City-County Planning and Zoning Commissions. The jurisdiction of any such Lake Area Planning and Zoning Commission is limited to a three mile perimeter from the normal elevation lake shoreline of any such lake." (Emphasis added)

Despite the lack of specific reference thereto in Section 866.2, it is apparent that the Legislature intended the first quoted portion thereof to be applicable to counties which were participants in the creation of a Metropolitan Area Planning Commission, and also had within their jurisdictions an upstream terminal and navigational turnaround or a reservoir built by the U.S.C.E. or G.R.D.A. Confirmation of the Legislative intent is shown by the language constituting a part of amended Section 866.36, hereinafter quoted.

Title 19 O.S. Supp.1969, § 866.36, provides for creation of a Lake Area Planning and Zoning Commission by any one or more counties having within their jurisdiction a lake constructed by the United States Corps of Engineers or by the Grand River Dam Authority.

Said section contains the following:

> ". . . .A Lake Area Planning and Zoning Commission may be formed to include all or any part of a county in which there is a lake constructed by the Corps of Engineers or by the Grand River Dam Authority regardless of the population of said county or the cities and towns therein. More than one county may cooperate in a joint Lake Area Planning and Zoning Commission. Funds for the operation of a Lake Area Planning and Zoning Commission may be appropriated by any county, city or town in the area affected by such Planning Commission. A Lake Area Planning and Zoning Commission when properly formed shall be authorized to exercise all the powers and duties set forth in this act." (Emphasis added)

It is therefore, the opinion of the Attorney General that your questions numbered 1 and 2 must be answered in the following manner: Oklahoma cities and towns presently have authority under State statutes to participate in the National Flood Insurance Program of 1968, and to establish land use and control measures, and to adopt and enforce zoning ordinances, subdivision regulations, building codes and other regulations to provide safe standards of occupancy for and prudent use of flood prone areas pursuant to such participation.

However, counties as such do not presently have such authority, or the power to establish such land use and control measures or to engage in such zoning and regulatory activities, acting in their individual capacities, but may, subject to the limitations and under the provisions of 19 O.S. Supp.1969, § 866.2, do so where they have created a Metropolitan Area Planning Commission, and under Section 866.36 where they can and have formed a Lake Area Planning and Zoning Commission.

Sincerely,

FOR THE ATTORNEY GENERAL

CARL G. ENGLING
Assistant Attorney General

ATTORNEY GENERAL OF OKLAHOMA
APPROVED
OCT - 5 1970
CONFERENCE
Official Copy

CGE:vw

APPROVED IN CONFERENCE:

G. T. BLANKENSHIP
ATTORNEY GENERAL

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CHAPTER IV

RECOMMENDED DESIGN TECHNIQUES & DRAINAGE CONSIDERATIONS

GENERAL

There are a variety of purposes for which an individual may use this Manual. Elected officials, administrators, and attorneys will have interest in the Goals and Objectives (Chapter II of Part I) and the Oklahoma Stormwater Law (Chapter III of Part I). Non-engineer professionals (such as planners and parks specialists) have interest in the technical prospects of this Manual because of the multipurpose aspects of drainage and because of the need to administer planning and zoning regulations.

Engineers have interest in Part I because the basic approach to drainage, the goals and objectives of the City of Stillwater and legal aspects in which the engineer can become entangled, are articulated in that part of the Manual. However, the technical aspects of the Manual will have varied interest to the engineer, depending on his role in the drainage process.

The private engineer's role may vary from the design of a small subdivision and/or small street extensions, to preparation of a master drainage plan for an entire drainage basin, and, ultimately, final design. The City staff engineers are involved in the entire range of these studies and designs and seek standardized methodology and criteria to facilitate review and approval.

The purpose of this Chapter is to give both the designer and the reviewer an overview of the design process, the techniques employed, and those items which must be considered in the design process. Depending on the engineer's role, not all items described in this Chapter or in the Manual are applicable.

The various steps to be utilized in developing and implementing a storm drainage plan are defined as follows:

o Get the facts. This is the most important aspect and relates to historic, future, and existing land use, historic and existing drainage paths, basic hydrology, (including rainfall, runoff, vegetation and infiltration), capacities of the existing facilities, presence of floodplains, impacts on adjacent properties, evaluation of the existing situation, and the presence (or lack of) a master drainage plan for the area and/or basin.

- Conceptual Design. Based on the fact situation, develop and analyze all reasonable alternatives. Depending on the size of area being considered, this phase may simply mean that facilities required to meet City standards are determined and shown on the proper sized drawings drawn to a specified scale. Or it may mean an extensive investigation in which the hydrologic, hydraulic, sociological, urban infrastructure, and cost interrelationships are investigated to develop a master plan.
- Master Planning. Based on the results obtained from the conceptual design process and upon the concurrence of the City and reviewing agency, a Master Plan is developed. This plan describes in detail the recommended alternative, shows sizes, types, and location for required drainage facilities, and is sufficient in detail for designing new roads, bridges, and other urban utilities. The Master Plan may only be a floodplain information report when structural solutions are not recommended or a more detailed delineation of facilities required to meet City standards for small subdivisions.
- Final Design. Detailed drawings and specifications are prepared. These are suitable for review approval, and construction of all, or segments, of the Master Drainage Plan.
- Construction. Physical placement of drainage facilities according to the final design drawings and specifications. This phase requires onsite supervision by the designer and/or City.
- Maintenance. Maintaining natural or artificial drainage facilities by the City or by others according to a procedure approved by the City. This includes snagging, mowing, silt and debris removal, erosion control, and periodic cleaning of inlets, pipes, ditches, and culverts.

Effect of Master Drainage Plans

An articulated policy of the City of Stillwater (See Chapter II of Part I) is to prepare master drainage plans on a basin by basin basis as funds are available. When these are completed, nearly all locations where major drainage requirements must be met will be known and appropriate recommendations made. Policies will be further articulated as to requirements for detention storage.

When completed, these studies will substantially reduce the amount of design work necessary for small tracts and large tracts alike; however, until a master drainage plan has been accepted relative to any specific tract, it will be necessary for the engineer on even small tracts to determine whether or not there are major drainage impacts on the parcel and to what degree ponding is necessary to meet legal constraints.

Drainage Law Considerations

Until the basin master plans are completed, it is quite important that drainage facilities for even small tracts be reviewed in light of the legal aspects presented in Chapter III of Part I. For basin master planning, it is necessary to include a legal professional as part of the team. For small and large parcels of a basin it may be wise to include an attorney; however, adherence to the principles of "Oklahoma Stormwater Law" will often be sufficient.

Floodplain Management

This concept has been adopted by the City through this Manual and through its participation in the Flood Insurance Program. It is to be considered an integral part of planning, especially for small tracts, where it is difficult to modify the floodplain without affecting adjacent properties. Modifications using the floodway criteria adopted by the City (See Chapter II of Part I) are acceptable; however, structurally oriented measures will be used only where:

- Existing conditions warrant their economical use, and
- The use of structural measures can be demonstrated to have no adverse affects downstream or upstream.

Major Versus Minor Drainage

Major and Minor Drainage are conceptually defined in Chapter I, Part I, but are discussed throughout the Manual. To the designer, the transition from one to the other is not always clear; therefore, criteria have been established which allow the designer to measure when the system analyses should be undertaken as major drainage versus minor drainage.

When a parcel is at the top of a basin, under 50 acres, and with no visible water course, it is reasonable to expect that a storm drainage system would be a minor drainage system. When a river, a continuous running brook, or a deep cut gulch are involved, it is apparent that a major drainage system is involved. It is the conditions in between which are difficult to determine where to start major drainage facilities. In fact, land use, development characteristics, street patterns, floodplain preservation, and types of minor and major drainage systems employed can be varied to affect when either a minor or major drainage system is needed.

With the items discussed under BASIC DATA of this Chapter obtained, the designer determines approximately where the storm sewer system begins. If roadside ditches are used, this first step can be eliminated. The analysis proceeds downstream. When the total tributary area exceeds 80 acres to any system, the designer should begin rough calculations to see if the 100-year event criteria are being exceeded. When the criteria begins to be exceeded for either the minor storm criteria or the major storm criteria, the size of the minor storm system (ditches or pipes) should be increased. When the size of the minor storm system facilities exceeds the 10-year runoff event, then the system is to be analyzed as a major drainage system.

The criteria used to determine when and where a minor system storm sewer begins are based on allowable encroachment of travel lanes (See Chapter II of Part II, "Streets, Curbs, and Gutters"). The criteria used to check the 100-year event are based on permissible inundation of the roadway surface or that structures adjacent to the streets are not flooded, whichever is most severe. In newly developed areas, the former criteria should normally be used and the street capacity curves in Chapter II of Part II are applicable. In established areas, it is frequent that the latter limit applies, in which case, the designer must employ the methodologies of Chapter II of Part II to develop his own street capacity curves.

BASIC DATA

The first step in drainage planning is to obtain the facts. These facts should be gathered at an early time in the planning process. This action in the planning process is necessary to acquire data to be used in conceptualization of alternatives. The required data are described as follows:

Location Maps

These maps should describe the general physical location of the basin of concern in relation to other major watercourses, various political boundaries, and other major topographic features.

Topographic Maps

Maps depicting the relief of the land and the stream network will be required. Aerial photos are very useful. Mapping scale will be 1" = 100' with a 2' contour, interval. If broad floodplain areas are involved, the City Engineer may require 1' contour initials.

Vegetation

Aerial photographs and various inventory maps that describe the kinds and quantities of vegetation in the basin are required to reliably depict the runoff process.

Soil Survey

A soil survey is required to understand the infiltration and runoff process in a basin. Information needed is that related to the physical condition of the soil (including infiltration tests) compaction, physical characteristics of the soil, and depth to ground water.

Geology

Data or maps are required to describe surface and bedrock geology. Hydrogeologic information is also needed to reliably describe the runoff response of a basin.

Stream Survey

When channels are involved, cross sections from the topographic maps and verified by field instrument surveys are needed for hydrologic stream routing and hydraulic calculations. For the same reasons, data relating to channel conditions such as roughness, vegetation, and meandering are required.

Historic Runoff Routes

The predevelopment runoff routes have frequently been obliterated, particularly for smaller drainage basins. Almost always, drainage problems have resulted. When looking for the outlet locations and routes, this historic route is often desirable physically for both urbanized and urbanizing areas; however, it is nearly mandatory to use this route to meet legal constraints (See Chapter III of Part I, "Oklahoma Stormwater Law").

Existing Urban Development

Existing development data are required to describe land use, transportation, water features, open spaces, and water oriented uses. This information will be used in other areas of the master planning process, such as runoff hydrology and conceptualization of alternatives.

Future Development

There are key items which need to be considered:

Overall Goals and Objectives. Apart from the drainage related goals and objectives, there are basin goals and objectives related to land use and quality of life. These can be elicited from the local and regional interests involved by an interviewing and interaction process. It is imperative that the designer ascertain these additional goals and objectives from the City of Stillwater Department of Community Development and from affected citizens in the impact area.

Land Use Forecasts. This information must be inventoried to the greatest detail available. Maps that depict areas of existing and proposed development are available from the City's Department of Community Development.

Redevelopment and Land Use Changes. Programmed and proposed land use changes shall be identified.

Drainage Management Strategy

A significant purpose of the preparation of basin-wide drainage master plans is to articulate the management strategy for any basin. The approach will not be clear until these plans are completed, and even the smallest of parcels will be affected.

The primary variable relates to the use of onsite detention and the size of parcel to which this concept may be applied. Until basin-wide management strategies are completed, each parcel and system is to be analyzed according to Chapter II of Part I, "Oklahoma Stormwater Law." Primarily, these are the activities which affect runoff from one parcel on other parcels. If onsite detention is necessary to mitigate the effects of runoff on other lands, then it is to be used. The designer is encouraged to work with other property owners to develop onsite detention facilities which can serve other properties.

The other significant management concept which has already been adopted by the City of Stillwater is the use of floodplain management.

Hydrometeorology

The Hydrology Chapter of this Manual provides much of the pertinent information for use in most planning efforts. However, it is important to recognize that little data regarding probable maximum precipitation are provided herein. The engineer should contact the U.S. Weather Bureau when probable maximum precipitation data are needed.

Procedures are outlined in Chapter I of Part II which are to be used by the designer. For summary purposes, Table IV-1 (Table I-1 of Chapter I, Part II) is included in this Chapter.

The designer is required to establish the discharge for the design frequency for existing and future conditions. For situations which, in the opinion of the City Engineer for the City of Stillwater, have difficult outlet conditions, the historic rate of runoff will be determined for the design runoff and the 100-year runoff event (if the design frequency is less than the 100-year event).

Problem Inventory

Based on a hydrologic/hydraulic analysis, through interviews with City officials and local citizens and by review of records of past events (newspapers, etc.), it is normally readily possible to do a problem inventory. For parcels under fifty acres this step will not be necessary unless the City Engineer identifies constraints at or below the outlet. For all other parcels and for basin-wide studies, this step is to be completed.

The problem inventory is needed to assess the affects of new urbanization and drainage of facilities and to identify those areas which specifically may need structurally oriented facilities. It assists the designer in basin-wide studies to break down the stream into reaches with similar properties.

MAJOR DRAINAGE - CONCEPTUAL DESIGN, MASTER PLANNING, AND FINAL DESIGN

This portion of the planning process will require an repetative approach in which each step reveals new information and thus points out the possible need for further analysis. The major steps included herein are:

- Runoff,
- Floodplain delineation,

TABLE IV-1

HYDROLOGY GUIDE

CATEGORY	TYPES OF CALCULATIONS PERFORMED	REFERENCE SECTIONS
I - Development drainage planning and design for small areas, generally less than 100 acres and not involving floodplains.	Peak discharges for local drainage system and estimates of runoff volumes for purposes of onsite detention storage.	Rainfall Intensity Duration Curves Rational Method
II - Development Drainage Planning and design for larger areas, generally than 40 acres and involving floodplains.	Local drainage will be handled in a fashion as described for Category I. As the tributary area increases, reliance will shift to design storms, runoff hydrographs, stream and reservoir routing, and ultimately use of sophisticated computer models.	Rainfall Intensity Duration Curves Design Storm Development Rainfall Excess and Infiltration Synthetic Unit Hydrograph Procedure Computer Modeling Approaches
III - Master planning of drainage basins, usually involving many parties and with heavy emphasis on economics.	Simplified problems can be handled by runoff hydrographs and basic stream routing. Problems involving complicated drainage basins and intricate alternative points toward use of computer tools.	Rainfall Intensity Duration Curves Rational Method Design Storm Development Rainfall Excess and Infiltration Synthetic Unit Hydrograph Procedure Computer Modeling Approaches

- Problem inventory,
- Alternative components,
- Conceptualization of alternatives,
- Multipurpose planning alternatives,
- Hydrologic analysis of possible alternatives,
- Cost benefit analysis and evaluation,
- Stormwater law considerations,
- Development of drainage management strategy,
- Preliminary design,
- Articulation of basin goals and objectives,
- Preparation of design drawings, and
- Preparation of construction specifications.

Until basin-wide plans are adopted, even small land parcels will be required to go through many of the steps of conceptual design, although floodplain management will normally be applied. After basin-wide plans are completed, small and even large parcels may only be required to illustrate floodplain information and to conceptually display those facilities required to meet adopted City plans.

Basic Design Information

The information required under BASIC DATA of this Chapter are required for analysis of major or minor drainage. The following discussion relates more specifically to information required for major drainage analysis.

Runoff. This step establishes the important baseline conditions by quantitatively describing the runoff character of the drainage basin. The Hydrology Chapter of Part II of this Manual describes several methods, one or more of which are to be used.

A major step in runoff hydrology is to identify subbasins which take into consideration the following major points:

- Major tributaries that have significantly different characteristics, or for which discharge flow information is desired,
- Areas of existing development,

- Areas of proposed development,
- Possible floodwater storage sites, and
- Reaches of stream to which significant alterations may be made in the future.

There are many other reasons for subdividing a basin, but it is important to realize the practical need for not being overly detailed. This is a significant concern in the hydrologic analysis of a drainage basin.

An important role of the hydrologist is to identify the key runoff phenomena that are occurring, and to represent these with a reasonable mathematical simplification. Accordingly, he must select an appropriate runoff model and input appropriate data.

Floodplain Delineation. The flood hydrographs that are produced in the runoff step are then used as input to a hydraulic analysis procedure that gives an approximate water surface profile for the given flow. This information is used to establish probable trouble areas and to evaluate proposed solutions. It is recommended that HEC-2 (or a similar model) be used for water surface profile determination. As well as the runoff flows, there are three basic types of data necessary for floodplain delineation. These are:

- Representative cross sections of the stream or channel,
- Information regarding the hydraulic character of the sections and their relationships, such as roughness, stream slope, and meandering, and
- Hydraulic information regarding bridges, culverts, and other constructions which can create a different water surface than that which could be caused by the cross section alone.

The analysis of this information as explained in the Major Drainage Chapter will result in a profile (or longitudinal section of the stream) and a plan view plot which together depict the extent of the floodplains for a given flow. This information, when combined with other analysis information regarding depths of flow and velocity, can characterize the extent of probable damages from a particular rainfall pattern. Until basin-wide plans are adopted (which will contain floodplain information), all developing parcels will be required to develop future development floodplains when in the opinion of the City Engineer a parcel may be affected by a floodplain(s).

Problem Inventory. One of the most important steps in the planning process is to identify and describe quantitatively and qualitatively the existing and potential drainage problems in a basin. There are several actions that are usually necessary as part of the problem inventory:

- Concerned special governmental entities, individuals, special technical groups, and the City should be interviewed to understand their views regarding drainage problems. It is necessary to identify the symptoms that are discussed in relation to the actual problems. This is important because often the so-called problems are described in such limited terms that certain possible alternatives are needlessly eliminated or included.
- The floodplains delineated for existing and possible future alternatives should be carefully analyzed to assess the scope of potential damages, to identify existing and future problems, and to also identify certain advantageous situations which can be a key to preventing or relieving problems.

Alternative Components. There are many possible alternative components that could be used to correct or prevent drainage problems. These components are commonly used in combinations, but in some cases, are used on an individual basis. The components can be categorized in many ways, but for the purposes of this Manual, they are identified as preventive and corrective components. Preventive components, which seek to mitigate the effects of a flood, are an approach that recognizes the floodplain as nature's prescribed easement. Corrective components seek to affect the flood event by changing its distribution in time and space. Frequently, this involves correcting past mistakes.

- Preventive Components. Preventive components generally keep development from occurring in the floodplains. They inform occupants of existing development in the floodplain of the flood hazard and suggest mitigating actions that would limit the damages incurred during a flood period.

  Preventive components include:
    Delineation of floodplain,
    Control of floodplain land uses,
    Acquisition of selected floodplains,
    Subdivision regulations,
    Floodplain information and education,
    Flood forecasts and emergency measures,
    Floodproofing, and
    Flood insurance.

- Corrective Components. Most corrective measures are structural in nature. Acquisition of flood prone structures is a non-structural flood control measure. Various constructed works can be used to store or convey floodwaters to reduce damages. It is the intention of this Manual to provide the designer with most of the design aids required to perform hydraulic analysis and design of structural urban drainage facilities; however, there are certain special design requirements which cannot be designed from information contained in this Manual. Structural analysis of hydraulic structures is not contained in this Manual. Listed below are four general types of structural components:

  Channels. These include numerous different possible solutions such as concrete-lined channels, grass-lined channels, use of natural channels with some clearing and erosion control works, and hybrid solutions such as European channels. These are channels with grass-lined bottoms and retaining walls on the sides.

  Pipes and Conduits. These are usually used in the upper reaches of streams and may be precast or cast in-situ structures.

Constriction Removal. This would be the replacement of various bridges and culverts to allow design flows to pass without undue backwater effects.

Man-made storage. This storage is used to supplement the natural storage. There are three possible types of man-made storage: detention storage, retention storage, and conveyance storage. The first two are dealt with in detail in Chapter VI of Part II,"Man-Made Storage." Conveyance storage is discussed in Chapter VII of Part II.

Conceptualization of Alternatives

Conceptualization of alternative drainage plans is another major point in drainage planning. It is a challenging process because of the large number of component combinations. There is a strong need to come up with a feasible number of alternatives for analysis and the final designation of one alternative. Sound judgment is always required in the process of delineating the alternatives. The following paragraphs describe a simplified procedural outline for conceptualization of alternatives.

As part of any drainage plan, there are common items and points which shall be addressed.

- The Floodplain Areas. These will be delineated. The Floodplain Regulation of the City will be utilized to guide and limit development in these areas. The flood-prone area should be based upon the expected future development and present stream conditions. If, subsequently, the basin develops differently, and/or different drainage works are undertaken, the flood-prone area should be modified to suit the new conditions.
- Land-Use Planning Unit. Because land-use has a significant effect on the drainage response of a basin, it is important to incorporate existing and proposed land uses. Land-use planners must be made aware of the drainage impacts of development and must seek to modify development so as to prevent the creation of drainage problems.

- o <u>Interaction</u>. Drainage planning is a dynamic process. Previously unrecognized problems, flexibilities and constraints (as anticipated from the City, other agencies, and persons directly concerned) must be integrated into the planning process. It is most important that this information is made available to appropriate interest groups during the planning process.

<u>Multipurpose Planning Opportunities</u>. Drainage planning should incorporate compatible multipurpose planning concepts as delineated in Chapter V of Part I of this Manual.

<u>Floodplain Management</u>. Floodplain management is to be used on all drainage studies in which the land use is affected by floodplains of major drainage features. Because it is a unique approach which does not lend itself to traditional analysis as articulated in the Chapters of Part II, a more detailed discussion is presented here.

Floodplain management includes all measures for planning and action which are needed to determine, implement, revise, and update comprehensive plans for the wise use of floodplain lands and their related water resources. This includes both preventive and corrective action, which is listed earlier in the Alternate Components under MAJOR DRAINAGE, Conceptual Design. This information is intended for use in basinwide planning and design but will be useful to others to understand the impact of this policy on floodplain properties.

Land-use management is the keystone of preventive floodplain management. In its broadest terms it involves both land-use and runoff controls. Land-use management is used in this Manual to describe policies of land management that lead to prudent and productive use of hazardous areas. It involves a set of actions at the local government, semigovernment, and state and federal governmental levels which can be relied upon to guide the wise use of public and private flood-prone land. These actions include acquisitions, legislative controls, taxation, fiscal policies, land valuations, and dissemination of information.

Floodplain management is an alternative which must be considered when analyzing major drainage alternatives on a basin-wide basis; however, it applies universally to all floodplain affected properties in Stillwater.

Preventive floodplain management measures are to be applied to existing development, future development, and heavily encroached floodplains. Each condition met in the field requires a specially developed combination of methods. Existing development and heavily encroached floodplains require a combination of corrective and preventive actions. When a preventive approach is programmed, it may be supplemented with corrective measures to achieve the desired degree of protection. The preventive measures can, however, move independently of the supplementary corrective measures. Implementation of the preventive measures help achieve a reduction in flood damage potential without waiting for the balance of the program.

The following items are discussions of corrective actions:

- Land-Use Adjustments. Corrective measures include land-use adjustments such as relocation of structures, programmed removal of noncompatible structures, and purchase of floodplain properties. Properties purchased may be leased back for temporary use with scheduled razing in the future.
- Nonconforming Uses. When land is rezoned, the existing use of the land or the buildings thereon may no longer conform to the zoning now in effect. A change in building regulations has the same effect on existing buildings. Such use or building is then held to be "nonconforming." The general policy is that such uses and buildings are permitted to continue, but that nonconformity should cease eventually.

  Frequently, floodplain regulations require that when a building is destroyed or damaged to the extent of more than 50 percent, and lies within the floodway, it shall not be restored, reconstructed, or repaired. In addition, if a building requires repairs for any reasons whatsoever, which at any one time are in excess of 50 percent of its value, it shall be removed if it lies within the floodway zone of the floodplain.
- Structural Control Measures. These measures (channels, detention ponds, etc.) seemingly remove properties from floodplains. Larger

floods can and do occur and it is necessary to view structural control measures from this viewpoint. It is not prudent to rely on structural measures without conceptualizing whether the resulting conditons from a larger flood would be worse than would have been if not built. This is particularly true in regard to flow control reservoirs. Further, the integrity of the structural control facilities must be maintained. It is still necessary to treat water courses with structural treatment (channels) floodplains; however, once a structural channel is in place, the floodway criteria in regard to filling cannot be applied.

A preventive floodplain management plan must call upon the use of many components to be effective and practical. It is more demanding upon the engineers and planners engaged in flood control, and it requires a high degree of performance and professionalism. On the other hand, many nonstructural measures can be implemented without heavy capital expenditures.

The components of a preventive program fall under the broad categories of land use management, early warning, land runoff control, flood-proofing, insurance, and relief and rehabilitation. The components are described below along with the type of basic data needed and the important flood-prone area maps.

- Control and Floodplain Land Uses. Floodplain land uses may be controlled by either floodplain controls or land acquisition. By controlling the amount and type of economic and social growth in the floodplain, flood losses are reduced and net benefits from suitable floodplain use increases. Methods used include land use controls, land acquisitions, subdivision regulations, and control of water, sewer, and other utility extensions.

- Floodplain Information and Education. The development of floodplain information should be accompanied by an information program. Floodplain maps accompanied by information on measures which a property owner can undertake to mitigate his potential losses can be provided to the various landowners when available.

Various methods can be used to mark and make known the floodplain in the field. These include showing the 1 percent flood level on public buildings, at bridges, and by using other easily recognized techniques for providing warning signs. Public and private financing institutions can be drawn into the floodplain management process.

- <u>Flood Forecasts and Emergency Measures</u>. Prediction of floods with appropriate forecasts or warnings delivered with credibility can trigger a series of emergency measures which will reduce flood damages. This is true even where only a short lead time is available. A simple warning can significantly reduce the adverse effects of a flood.

  With more sophistication, forecast and warning of a flood can be very effective. This would be the case when communications are used to quickly assess information on upstream flows and basin rainfall patterns. When such data are available, a hydrologist will be able to predict the level of anticipated flooding at downstream locations so that warnings could be given with considerable lead time for the larger rivers and creeks.

  A flood forecasting system will reduce flood losses if it stimulates appropriate emergency actions before the floodwaters reach the vulnerable areas. People, equipment, and materials located on floodplain sites should be moved from the floodplain to sites above anticipated flood heights. Equipment that cannot be moved should be treated to mitigate water damage. Where flood proofing measures have been incorporated into structures, they should be activated, e.g., sand-bagging or bulkheads secured in place in anticipation of the flood. Adjustments should be made to utilities to assure continuation of vital services. This may include the interruption of services (such as gas and electricity) to the floodplain areas. Equipment and personnel should be dispatched to critical channel constrictions (bridge openings) or control works at storage basins to remove debris to assure proper functioning. In essence, a disaster preparedness plan must be developed by appropriate government agencies to assure that the proper emergency measures are implemented in an effective and timely manner.

- Flood Proofing. The use of flood proofing is an important nonstructural floodplain management. Flood proofing consists of those adjustments, to structures and building contents, which are used for commercial and industrial purposes. This concept allows private property managers to take actions to reduce their flood risks.

  Many adjustments can be made to structures and contents which will mitigate the effects of flooding. Flood proofing should be carried out under the direction of a professional engineer or architect.

  General measures which may be mutually exclusive are tabulated below:
  - Anchorage to resist flotation and lateral movement. Floating residential homes are a hazard to both public and private property and contribute significantly to the downstream debris problem and utility and communication disruption,
  - Watertight doors, bulkheads, shutters, and sandbags,
  - Reinforcement of walls to resist collapse from hydraulic pressure,
  - Water-proofing walls to control seepage,
  - Addition of mass to resist flotation,
  - Installation of pumps to control seepage,
  - Check valves on sewerage and stormwater drains,
  - Reduction or management of water table to relieve hydraulic pressures,
  - Raising of electrical control panels, above the anticipated waterline,
  - Protection of sewer manholes from entrance of floodwaters,
  - Distribution of contents stored within the structure.

Building codes should be modified as required to include flood proofing measures. Flood proofed construction can be accomplished with a cost increase as small as one percent.

- Flood Insurance. Stillwater is a participant in the Federal Flood Insurance program which balances public subsidies with requirements for appropriate land use controls. Existing flood-prone properties can obtain subsidized insurance. New buildings would be subject to actuarial rates. The program has shown that actuarial rates virtually rule out any further development of floodplains. Experience has shown that the benefits derived from a floodplain site are not able to offset the costs associated with the flood risk if the owner must absorb the losses.

  Existing buildings in the floodway that have suffered substantial (more than 50 percent) damages from flooding, fire, or other causes, are no longer eligible for subsidized insurance. Through mortgage institutions, flood insurance is required on all buildings for which new mortgages are to be made following the publishing of Flood Insurance Rate Maps.

Hydrologic Analysis of Possible Alternatives. The procedures described in the Hydrology Chapter will be followed again for each of the possible alternatives. This would include runoff routing and production of runoff hydrographs which will show volumes and peak discharge values in comparison to the existing conditions and future conditions with no modifications to the stream network. Determination of water surface profiles by HEC-2 computer runs shall be made to determine residual floodplains remaining, when alternatives being considered do not completely contain the 100-year event.

It is important, at this point, to analyze these results to identify both the negative and positive modifications made to the runoff response characteristics of the basin with various alternatives.

Cost and Benefit Analysis. The analysis of the cost and benefits of each of the alternatives is a demanding process requiring consistency, equity, and reliability.

All evaluations should be made for the total system required for a given reach in the drainage network. The reaches should be chosen so that the evaluations are equitable to all alternatives.

The depth of this analysis should be scaled appropriately to the magnitudes and complexities of the basin and study level involved. This depth has to be chosen with careful judgment and should be agreed upon by the designer and the City. In fact, for parcels where only floodplain management is to be used, no cost-benefit study will be required.

In general, cost-benefit studies will be required for all basinwide planning studies and for all parcels involved with major drainages where structural measures are considered which may require expenditure of public monies either for capital improvements or for maintenance.

There are two major areas that need to be addressed to assess the cost of alternatives. They are capital costs for construction of works, and the operation and maintenance costs. Some of the components of each of these are presented below:

- Capital
    - Pipes
    - Channels
    - Erosion protection
    - Embankments
    - Structures
    - Clearing and removal of obstruction
    - Land acquisition
- Operation and Maintenance Costs
    - Pipes
    - Channel repairs
    - Erosion protection
    - Mowing grass and other vegetation maintenance

Structure (including embankments) repairs
Clearing of sediment, debris and obstructions
Inspection of waterways or facility conditions
Inspection of detention, retention, and flood-prone areas

A difficult area of analysis of potential alternatives concerns the accrued benefits. This is because many benefits related to environmental and social aspects cannot be expressed readily in dollars. Attempts at expressing the benefits in dollars can be a guide, but certainly are easily questioned. The depth to which the benefits ought to be inventoried should be scaled to the magnitude, complexity, and expense of possible alternatives. There are several factors which would normally be covered in any planning report. These are:

- Damages and Damages Relieved. Damage assessments should usually be cursory for most cases, as measurement of damages due to an expected water surface are speculative. The U.S. Army Corps of Engineers, U.S. Flood Insurance Administration, and the U.S. Soil Conservation Service, along with other groups, have procedures that can provide potential damage information for relative water levels of certain structures.

- Other Economic Factors. These would include approximate assessments for inconvenience due to such things as transportation and utility disruption.
- Multiple Use Benefits. This includes an evaluation of the multi-purpose planning opportunities. When possible, economic benefits and costs will be assigned.
- Social Factors. These include consideration of peace of mind to communities when potential drainage problems are reduced. Assessments of impacts on the quality of life will also be made.

Drainage Law Considerations. As part of the planning process, it is important to identify the legal and administrative constraints which have an impact on drainage. A summary is presented in the Legal Aspects Chapter of

Part I. It is usually wise to include a legal professional as part of the planning team.

- Evaluation of Concepts. The list of alternative components previously presented should be thoroughly evaluated for assistance in the decision making process prior to proceeding into Preliminary Design. The following general procedure should be used:
  - Recognize and identify combinations of components,
  - Relate to overall goals and objectives,
  - Assess and eliminate unreasonable combinations,
  - Identify the most likely combinations; the least likely combinations should be held aside in case it becomes apparent that the more likely combinations have serious problems, or if there should be other reasons for including such alternatives at a later time,
  - Identify the major management areas within the basin which are of critical importance to the drainage system.

With this process, the basic alternatives are identified along with the major areas of activities. Again, this planning process is an iterative process in which the earlier steps may be readdressed in more detail as areas of concern are exposed in the later steps. For example, in the conceptualization step it may become apparent that it is necessary to acquire more data and identify possible problem areas more thoroughly.

Preliminary Design

From the conceptual plan analysis, a management plan is agreed upon by the City which will be studied in greater detail. The preliminary design phase culminates in a master planning document which is suitable for rights-of-way acquisition, sizing of bridges, and constructing segments of the master plan with knowledge that they will be compatible with facilities to be constructed later. The master plan document can vary greatly depending on the size of parcel involved and relation to major drainage.

Small parcels need not have extensive analyses; however, for investigation, basic data as previously discussed must be developed and the required information shown on the drawings. A supporting report must be prepared documenting the investigation which cover the topics listed in this Section.

A master plan report documents the process and information of the conceptual design process, describes in detail the agreed upon plan, costs, and phasing

The requirements are nearly the same as those listed for preliminary design requirements for storm sewer design and are not repeated here. The only additional requirement is that all topographic mapping in the major drainageway will be at a scale of 1" = 100' with 2-foot contours.

Final Design

This level of design must be done in great detail. For small parcels involving only floodplain mangement is involved mapping at a scale of 1" = 100' will normally be suitable; however, for all cases involving structural channel modification, the mapping scale will be at a scale of 1" = 50'. A contour interval of 1 foot will be used when, in the opinion of the City Engineer, it is necessary to clearly define the proposed facilities. Contours will be required to illustrate proposed earthwork.

Profiles will be provided for all facilities and the hydraulic grade lines for both the design runoff event and the 100-year event (if they are different). These will be provided on prints for review by the City.

A complete list of information required for design submittals is provided in the Section of the Chapter on Minor Drainage Design. The requirements are similar and not repeated here.

Final Hydraulic Design. The general procedure for final hydraulic design is the same for natural as well as artificial channels. The final hydraulic design for natural channels may be more limited to small reaches around bridges, culverts, and the larger erosion control structures.

The first step is to determine the design water surface elevation in 1) the drainage system to which the new facility is tributary, or 2) the downstream water surface (hydraulic grade line) in the facility (channel or conduit) into which the improvements drain. This step for Item 2 is for localized improvements or natural channels or when the proposed improvements are entirely contained within a segment of the major drainage system.

Prior to commencing final hydraulic design, the second determination that must be made is the discharge in the drainage facilities being designed when the receiving stream is carrying the 100-year flood event. Either condition may govern design in the lower reaches of a major drainage facility being designed.

A third possible factor is the hydraulic grade line resulting from an energy dissipator at the downstream end of the proposed improvements, particularly if the receiving stream has no influence on the hydraulics of the drainage facility being designed or merely influences the hydraulics of the dissipator.

Regardless of whether or not the last downstream segment is subcritical or supercritical, design should proceed upstream until the influence of the receiving stream is no longer a factor. If the last segment is supercritical, the designer should look for hydraulic jumps which are caused by a higher hydraulic grade line in the receiving stream than in the tributary being designed. The designer should also be aware that the hydrology of storm drainage is dynamic and that a hydraulic jump can move up and down the channel depending on the flood stage in the receiving stream and the flood stage in the segment being designed. For this reason, supercritical channels should be avoided in the bottom reach of drainage facilities. If absolutely necessary, special design techniques and/or scale modeling will be necessary to avoid an undesired hydraulic phenomenon.

The design should proceed upstream in subcritical channel reaches and in conduits under pressure, and downstream in supercritical channel reaches.

When a subcritical reach is below a supercritical reach, the design proceeds to a common point where a hydraulic structure will normally be required. Layout should be done using both plan and profile to properly adhere to other constraints. This is an repetative process where adjustments are made according to physical, sociological, and cost constraints. The designer is referenced to Chapter V of Part II, Major Drainage, for assistance in structure sizing.

The preliminary elements of structural design should commence early in the procedure to provide input as to opportunities and constraints to the ongoing design process. Actual final structural design must often wait until all physical constraints concerning the conveyance, traffic,and other aspects are known. The importance of adequate structural design cannot be understated. Good hydraulic design is ineffective if structural elements cause failure of the system. In addition to the normal earth, hydrostatic (uplift) and traffic forces, the structural engineer must consider:

- The dynamic forces of water
- Erosion due to high velocity
- Impact from debris lodging in bends or on piers and abutments
- Debris plugging the inlet to conduits and causing the conduit to flow partially full
- Vibration
- Cavitation (mostly in outlet structures and in bends of high velocity conduits)

Man-made storage facilities are frequently designed by hand and should be compatible with the final design of other facilities. Computer models such as MITCAT or SWMM (see Chapter I of Part I, "Hydrology") may be used to route the design flood through the man-made detention facilities. The design of man-made storage is also an iterative process and continues until the desired hydrologic characteristics are obtained. If the precise characteristics are unattainable, downstream facility design may have to be modified. The designer should utilize Chapter VI of Part II of this Manual for assistance in the design of man-made storage facilities.

MINOR DRAINAGE - PLANNING, PRELIMINARY DESIGN AND FINAL DESIGN

Analyses of the major drainage system is the first step to planning for the minor drainage system. The previous part of this Chapter has provided a systematic process description for the first step.

The minor drainage system includes street gutters, roadside drainage ditches, culverts, storm sewers, small open channels, and any other feature designed to handle runoff from the minor storm.

A well-planned and designed major drainage system will reduce, and sometimes eliminate, the need and cost of minor drainage system. The major system may be considered to be the skeleton upon which the minor system is added.

Planning for Minor Drainage

Planning and design for the minor storm drainage system must be considered from the viewpoint of both the regularly expected storm (the minor storm) and the major storm occurrence. Depending on land use, street classifications, and inundation criteria (see Chapter II, Part II) the minor design storm will have a frequency ranging from once in two years to once in five years. There are criteria similar to that for the minor storm which also must be met for the major storm or 100-year event. The minor storm drainage system must be capable of handling both types of event within the criteria established.

Major questions facing the designer relate to:

- When does a storm sewer system become necessary to serve the needs of a minor system?

- When does the minor system (including storm sewer system at its designated design frequency) become incapable of meeting the needs of the major drainage criteria?

By utilizing a generally standard design/planning procedure, the designer can test the adequacy of the system performance against the specific and general criteria contained in this Manual. In newly developing areas, this process is fairly straight-forward. When working in already urbanized areas, however, economic, political, and sociological considerations frequently enter the decision-making process.

The specific methods used in the storm water management analysis are described in the following parts of this Chapter or elsewhere in this Manual. The generalized process is described as follows:

1. Using methods described in the Hydrology Chapter of Part II of this Manual and as subsequently described, the designer computes the runoff rates for the design storm starting at the uppermost reaches of the basin.
2. The storm sewer system begins when the design storm runoff exceeds the gutter (or roadside ditch) capacity. The design proceeds downstream until the system outfalls into the major drainage facilties.
3. Again, from the upper-most reaches of the basin,the designer computes the runoff from the 100-year storm. When street capacity criteria are exceeded for this major storm, the designer should increase the size of the storm sewer that was sized for the minor storm. This increase in sewer size should increase the flow in the pipe network and reduce the street flow to within the established criteria. The combined total of the allowable street carrying capacity for the major storm and the storm sewer capacity should equal the major design runoff.

The designer may find that the perceived point at which major drainage facilities begin is actually too low in the basin. This could happen in urbanized areas where the land use and basin size may require that the storm sewer system convey runoff from a 10- to 25-year runoff rather than the 2- to 10-year runoff. In this case, the lower areas should be designed in accordance with procedures outlined for Design of Major Drainage in this Chapter.

(Note: In most instances when analyzing the major storm in the minor storm system, the time of concentration will be based on street flow rather that the flow time in the storm water system.)

4. The previous three steps constitute preliminary design. Up to this point, junction losses in storm sewers are ignored and roughness coefficients are increased by 25 percent. The final design of a storm sewer system must include junction loss computation. This procedure is explained in Chapter IV of Part II.

A special point needs clarification before making a more detailed explanation of the design process. The designer may encounter the condition ~~where~~ a minor storm drainage system which serves an area requiring a 2-year design frequency is bisected by a commercial corridor which requires a 5-year design frequency. Once the storm sewer leaves the area requiring a greater degree of protection, it does <u>not</u> assume the higher design frequency downstream of the corridor. Unless the downstream area also requires a greater degree of protection, it is only necessary to convey the increment of additional runoff required to be removed from the surface through the corridor requiring greater protection.

Basic Data

The basic data needed is essentially the same as that described earlier in this Chapter. Additionally, the following information is necessary:

- A map of the drainage basin containing the area being studied.

- A layout of the area to be storm sewered showing existing or proposed streets, intersections, and development type. The drainage engineer should advise the planner in street layout and major drainageways in order to reduce the drainage problems.
- Typical street cross sections. If the streets are not yet constructed, the drainage engineer should develop the street design with the City Engineers.
- Street and intersection elevations or street profiles of the subject areas.
- Soil and water table data.
- Location and elevation of the outfall point of the storm sewer system.
- Information on existing and proposed utilities.

Mapping. For preliminary design, mapping at a scale of 1" = 100' is to be used except, when in the opinion of the City Engineer, the area is so large as to be better shown at a scale of 1" = 200'.

Determine Limits of Basin and Analyze. Classify probable future type of development within the basin as it affects both hydrology and hydraulic design. Classify streets as to storm water drainage carrying capacity. Determine design frequency for minor drainage design. Develop intensity duration frequency curves for both the minor design frequency and the major 100-year storm.

Develop Alternative Concepts. In many cases, numerous potential layouts are possible. Here the engineer should review the reasonable alternative concepts, selecting those that appear most practical from an intuitive standpoint.

If the surface runoff can be kept from concentrating in one street,the storm sewer system can begin at a greater distance from the top of the drainage basin. In storm sewer design, it should be remembered that a large part of the construction cost is represented by small diameter laterals.

Various layout concepts should be developed, reviewed, and critical analyses made to arrive at the best layouts. Stormwater detention is one concept which deserves specific consideration. As pointed out several times in this Manual, the City of Stillwater has not adopted a policy of stormwater detention; however, to meet legal constraints, some stormwater detention will certainly be required. The most significant value of onsite detention can be its effect on reducing the size of storm sewer facilities and even eliminating them in some instances. Storm-sewered residential areas will be most difficult because of the difficulty of maintaining small grassed areas. Grassed roadside channels may be quite effective as detention facilities. In commercial areas onsite detention may be easily achieved through roof top and parking lot ponding.

Planning of a storm sewer system should have as its objective the design of a balanced system in which all portions will be used to their full capacity without adversely affecting the drainage of any area. Although design flows are dependent upon assumptions that may not represent the actual conditions under which the sewer will usually operate, the designer must not be tempted by the inherent limitations of the basic flow data to become careless in hydraulic design.

Layout Preliminary Conduit Alignments for Design Purposes. Set grades to be used for preliminary design procedures. Several preliminary layouts should be considered.

Divide Basin into Subbasins for Design Points. When dividing into subbasins, it should be remembered that at various inlets on a continuous grade only a portion of street flow will be removed to the storm sewer system. At intersections of urban principal and minor arterials, it will be necessary to remove 100 percent of the minor runoff from the road surface to preclude cross street flow.

The subbasins should vary according to the actual storm sewer system layout being considered. Errors often occur by trying to apply the hydrology from one set of subbasins to other sewer layouts that may have different subbasins.

Location of Outlet. This point is covered more fully in Chapter IV of Part II, "Storm Sewers", however, certain points need special emphasis. First, the outlet should be located at the historic outfall point. In cases where this point has already been altered, the second point must be adhered to. The second point is that the resulting outflow should not do more harm than would have occurred if the improvement was not built. This second point applies even though the outlet is located at its historic point of outfall.

Utilities. All above-ground and below-ground utilities are to be located and shown in plan and profile.

Streets. Streets are to meet the criteria as set forth in Chapter II of Part II, "Streets."

Inlets. Inlets are to meet the criteria as set forth in Chapter III of Part II, "Inlets." In regard to location, it may be necessary to start the storm sewer earlier that might be required for street capacity when a street is crossed in which crosspans and cross flows are not permissible.

Layout Planning

The preliminary layout of the minor storm drainage system should be done with a sensitivity toward urban drainage objectives, urban hydrology, and hydraulics. The preliminary layout of the system has more effect on the success and cost of the system than the final hydraulic design, preparation of the specifications, and choice of materials.

The ideal time to undertake the early work on the layout of the storm sewers is prior to the finalizing of the street layout in a new development. Once the street layout is set, the options open to the drainage engineers are greatly reduced.

Preliminary Design

The general process for preliminary design is discussed herein. The reader is referred to the appendix to Chapter IV of Part II, "Storm Sewers" for an example of the preliminary design computational process.

For the purpose of the following discussion, it is assumed that major drainage facilities have been laid out, decided upon, and exist in one form or another.

The designer should note that non-conventional methods are available for minor storm drainage design, particularly with the extensive use of surface drainage channels and onsite detention; however, the design approach for the use of the advanced methods is nearly identical to major drainage planning and design. The following deals with the most common approach to storm sewer design in urban areas, the Rational Method.

The engineer must obtain the facts before commencing design work. This is particularly important in drainage work because there are no two fact situations which are identical. After having the facts, a logical and technically thorough procedure is necessary to design a hydraulically balanced storm sewer system.

Detention and Retention. As shown in Chapter VI of Part II, "Man-Made Storage", there are three types of detention and retention storage to be considered:

- Upstream
- Localized
- Downstream

All of these categories should be considered when considering conceptual alternatives for a minor storm drainage system. Of the three categories, the first two offer a large potential savings in on-site development costs through the reduction of the size and extent of storm sewer pipelines and/or channels. The later category is frequently used to meet legal constraints.

The land use, size of the parcel of land, and whether or not the parcel is developed have a significant impact on the type of storage considered.

- <u>Land Use.</u> Detention ponding within multi-family, commercial, and industrial areas is relatively easy to attain, and maintenance of grassed areas can be under the control of the landowner, and paved surfaces which require little maintenance are readily available. Regulation by the city can be accomplished in normal code enforcement activities. All three categories of storage can be readily adopted to these uses.

  Low density residential areas cannot be so easily regulated on an individual basis. Because of diverse ownership, perpetual maintenance is difficult, and in existing residential areas, sites for localized storage are difficult to obtain. Upstream and downstream storage is most frequently utilized. Except for large parcels and/or when provisions are made for perpetual maintenance, localized ponding will not be used in low density residential development.

- <u>Parcel Size.</u> As stated in the preceding text, parcel size is a significant consideration when considering localized detention in low density residential development. The previously stated provisions for this ponding will apply until the parcel size reaches 40 acres. Low-density, single family residential parcels greater than 40 acres should consider the use of localized detention. Perpetual maintenance must be insured; however, local park parcels which incorporate detention storage may be accepted by the City for future maintenance.

- <u>Existing Level of Development.</u> Fully developed parcels are the responsibility of the City to correct existing drainage problems, and all categories of ponding will be considered for all sizes and types of development. Totally developed parcels are the responsibility of the developer and the previously stated provisions apply. Drainage basins which contain both developed and undeveloped parcels should consider all categories of storage, particularly where existing drainage problems exist. The City will consider joint development of detention/retention storage with developers for a mixed level of development condition.

The preceding discussion applies to development of man-made storage for minor drainage systems. Costs to the City of construction and maintenance will be required when the City is requested to evaluate their participation in such projects.

System Sizing. The frequency of design runoff, or rainfall return period, to be used for the minor storm drainage system should range from once in two years to once in ten years. A summary of the design frequency to be used in Stillwater for storm sewer design is presented below in Table IV-2.

TABLE IV-2

STORM DESIGN FREQUENCY - MINOR STORM

Land Use	Return Period (Frequency)
1. Residential	2 years
2. General commercial area	5 years
3. Airports (does not include major drainages which traverse area)	5 years
4. Business/commercial areas	5 years
5. Special high value areas and transportation corridors	10 years

Once the overall design frequency has been set, the system should be reviewed for points where deviation is justified or necessary. For example, it is necessary to plan a storm sewer to receive more than the minor runoff from a sump area which has no other method of drainage.

An area must be reviewed on the basis of both the major and minor storm occurrence. When an analysis implies that increasing the storm sewer capacity is necessary to convey the major storm, the basic system layout of the major drainage system should be analyzed and changed as necessary.

Design of Minor Drainage System for Minor Storm. The method of determining design runoff values should be as developed in the Hydrology Chapter of this Manual. The maximum limit for the Summation Rational Method is 100-acres; however, in Stillwater, major drainage design will usually become a factor before the 100-acre limit is reached, and more advanced methods should be used.

Preliminary street grades and cross sections must be available to the storm sewer designer. The allowable carrying capacity for these streets can then be calculated (See the street rating curves in Chapter II of Part II). Beginning at the upper end of the basin in question, the designer should calculate the quantity of flow in the street until the point is reached at which the allowable carrying capacity of the minor storm in the street matches the design runoff computed by the Rational Method. Initiation of the storm sewer system would start at this point if there is no other method of removing runoff from the street surface. It is not necessary to remove 100 percent of the flow from the street surface at the beginning of the storm sewer system, nor at any given location along the system unless the intersection of streets requires termination of cross street flow. It is necessary for the allowable street capacity plus the storm sewer capacity to equal or exceed the design flow.

The portion of flow that is deemed necessary to remove from the street surface is used as the design flow in the storm sewer. For preliminary design purposes, a Manning's n value, or other roughness coefficient, about 25 percent above that contemplated for final design should be used in calculation. Maintain the crown of sewers continuous at manholes to compensate for head loss. Using the artifically high "n" value, the preliminary sewer grade established previously, and assuming the sewer flowing full, a required pipe size for the design discharge may be determined from any applicable chart or formula. This method for sizing pipes should be adequate for preliminary design purposes. Cost estimates can be derived using these sizes and assumed depths of excavation. The velocities of flow will be sufficiently accurate to use for flow time in Rational Method calculations.

Route the Major Storm Runoff Through the System. It is necessary to determine if the combined capacity of the street and storm sewer system is sufficient to maintain surface flows within acceptable limits for the major storm runoff. The combined total of the allowable street carrying capacity for the major storm and the storm sewer capacity should equal or exceed the major storm design runoff. At any given point along the storm sewer system, the capacity of the sewer should be assumed to be the same for major runoff as for the minor runoff for preliminary design purposes, unless special considerations indicate that it would be significantly otherwise. If it becomes evident that it is impractical to accomodate the major runoff design flow on the street and within the storm sewer without exceeding the allowable street carrying capacity, some revision in the major drainage planning should be considered. A major policy decision should be made at this point to determine if possible changes in the major drainage system could alleviate the problem. When routing major storm events through the system, the time of concentration should normally be based on flow time in the street.

It is important to remember that for storms having a recurrence interval of 25 years or greater, an adjustment factor C_f should be included in the Rational Method, as described in the Hydrology Chapter of Part II of this Manual.

Prepare Cost Estimates of Each Proposed System and List the Pros and Cons of Each System. Unbiased attention to all good and bad points of various systems is necessary to arrive at the decisions as to which system is actually most desirable.

Review Alternatives. Review alternative plans with all who are involved in the final decision. If any potential problems exist with reference to the system, such as effects on downstream systems, they should be thoroughly reviewed and resolved. A storm sewer cannot simply be designed through a particular developed area and be allowed to discharge onto the ground where it would adversely affect downstream systems.

<u>Checking of a Preliminary Design Submittal</u>. As an aid to the design engineer as well as to those reviewing drainage plan submittals, the following check lists are presented:

- Basic Data

  Map of total drainage basin
  Map of area to be storm-sewered
  Characteristics of streets
  Street grades and direction of slope
  Location and elevation of outfall points for minor and major drainage
  Rainfall curves
  Character of future development
  Degree of imperviousness
  Soil and water table data
  Utility information

- Hydrology

  Design criteria tabulation for minor and major storm runoff
  Peak discharge computations for pipe sizing
  Peak discharge computations for major storm runoff
  Assumptions as to upstream storage

- Layout

  Streets and street names
  Irrigation ditches
  Street drainage flow direction
  Drainage basin and subbasins
  Storm sewer layout with sizes
  Storm inlet locations
  Cross pan locations
  Open drainageways
  Layout of major drainage system showing flows and directions
  Scale
  North arrow
  Signature blocks for review approvals
  Location map and subdivision names
  Conflicting utilities

- Cost Estimate and List of System Pros and Cons

Prior to proceeding into final design, the City shall approve the preliminary design layout, sizing, and computations.

Final Design - Minor Drainage System

The final design process begins with a review of all previous work. Often, refinements are made to the actual program during the evaluation stage following preliminary design. These refinements generally are not a cause for major new study efforts and the actual impact can be determined in the final design process.

Hydrology. Depending on the impact of refinements made in the alternative selection process, the final design hydrology may range from a review of the preliminary design hydrology to additional hydrologic modeling. The same hydrologic techniques (and often the same hydrology) are used for final design as for preliminary design. The type of hydrologic method to be used is defined in Chapter I of Part II, "Hydrology."

Mapping. For many large minor storm drainage facilities, it will be necessary to utilize mapping at a scale of 1" = 20' to 1" = 50' with 2-foot contours along the route, unless the City Engineer determines that 1-foot contours are necessary. While a subjective choice, the scale of mapping is to be approved by the City Engineer, however, the larger scale mapping will generally be necessary where numerous utility conflicts exist.

Streets and Utilities. Prior to commencing final hydraulic design, it is necessary to obtain detailed information on street grades, utilities, and final grades adjacent to the improvements where the grade is likely to change due to development. This information should be displayed on plan and profile drawings and used as constraints in the final hydraulic design. The location of other utilities which serve a local function only should not be considered as a major constraint.

Hydraulically Designed Sewer System. The water level in the receiving major drainageway should be determined for the design storm frequency. If this elevation is above the crown of the storm sewer, it is less likely that special outlet control devices will be necessary to prevent erosion. If the major drainageway is flowing at less than the design depth, the outlet should be reviewed for possible erosion tendencies.

Erosion control measures must be taken when the possibility exists of affecting the outfall channel. These may vary from stilling basins to simple riprap.

The final hydraulic design of a system should be on the basis of procedures set forth in Chapter IV of Part II of the Manual. A realistic "n" value for final design should be used based on actual pipe roughness. The conduits should be treated as either open channels or conduits flowing full, as the case may be. For open channel flow, the energy grade line should be used as a base for calculation. For conduits flowing full, the hydraulic grade line should be calculated.

The design engineer must review the hydraulic grade line for runoff conditions exceeding the initial design storm. This is to insure that the hydraulic grade line does not rise above the ground surface and thus cause unplanned discharge to the street. Because of the greater opportunity for management of excess runoff, the closed conduit approach to design shall generally be used to prevent transporting a problem to another area with unknown and often damaging results.

The design generally proceeds upstream from the outfall utilizing the hydraulic procedures from determining pipe losses and junction losses as shown in Chapter IV of Part II of this Manual.

Design Inlets. Utilizing City standard inlets, the design of inlets should be carried on simultaneously with the design of the remainder of the storm sewer system. The allowable street carrying capacity should be continuously equated to the design runoff from the Rational Method to determine where inlets will be necessary.

Determine Structural Aspects. The structural aspects of pipe and appurtenances to be utilized in the storm sewer system should be designed by thorough methods to insure that they are both adequate and economical. Good hydraulic design is ineffective if structural aspects cause the failure to the system.

Certain of these decisions must be made prior to hydraulic design of the system since the geometry of junctions, the type of inlets to be utilized, and the pipe material will influence the design.

Draw Final Construction Plans. Final construction plans and specifications should be of sufficient accuracy and clarity to guarantee that the designer's ideas are carried to completion by field installation.

Checking of Final Design Submittal. The following check list is included as an aid to the design engineer to help insure completeness.

- General

    Title Block (lower right-hand corner preferred)
    Scale
    Date and revisions
    Name of professional engineer or firm
    Professional Engineer's seal
    Statement as to specifications
    Approval spaces with data spaces
    Drawing numbers
    Statement as to adherance to drainage policies and criteria in the Drainage Criteria Manual

- Drainage Area Plan

    North Arrow
    Contours (maximum 2-foot intervals)
    Location and elevation of USGS bench marks
    Property lines
    Boundary lines (counties, districts, tributary area, etc.)
    Streets and street names and approximate grades with width
    Subdivision (name and location by section)
    Existing irrigation ditches
    Existing drainageways and structures including flow directions
    Drainage subbasin boundaries
    Easements required
    Proposed curbs and gutters and gutter flow directions
    Proposed cross pans and flow directions
    Proposed inlet locations and inlet sizes

Proposed piping and open drainageways
Critical minimum finished floor elevations for protection from major storm runoff.

- o Construction Plans

  North Arrow
  Property lines and ownership or subdivision information
  Street names and easements with width dimensions
  Testhole locations and log
  Existing utility lines (buried), location and depth
    - Water
    - Gas
    - Telephone
    - Storm drain
    - Irrigation ditches
    - Sanitary Sewers

- o Vertical and horizontal grids with scales
  Ground surface existing and proposed
  Existing utility lines where crossed
  Pipes
    - Plan showing stationing
    - Profile
    - Size, lengths between manholes and type
    - Grades
    - Inlet and outlet details
    - Manhole details (station number and invert elevation)
    - Typical bedding detail

- o Open Channels
    - Plan showing stationing
    - Profile
    - Grades
    - Typical cross section
    - Lining details

- o Special structures (manholes, head walls, trash racks, etc)
    - Plan
    - Elevations
    - Details

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CHAPTER V

MULTIPURPOSE BENEFITS FROM URBAN DRAINAGE AND FLOOD CONTROL

INTRODUCTION

The urban drainage and flood control policy of Stillwater, Oklahoma, is systematically related to overall water resources management. For instance, drainage basin management to protect the surface of the land against erosion benefits both drainage and water development systems. Retention and/or detention of stormwater reduces downstream flooding, while at the same time, enchances water supply management opportunities. The beneficial effects of local urban drainage and flood control programs will often have a magnifying effect on extra-local or regional water resources management.

The watercourses of Stillwater must be managed in a manner that will accommodate floodwaters without causing under flood losses. Wise management of these water courses requires that urban drainage and flood control be considered in the broad context of floodplain management. This approach emphasizes the development of multipurpose programs supported by multiple-approach strategies.

Well-managed watercourses have the potential to improve the quality of life experienced in a community. The City of Stillwater has adopted goals recognizing the need for a unified program for drainage and flood control which then becomes an integral part of the overall comprehensive planning process.(4) More specifically, adopted flood control measures will emphasize the reduction of public and private costs by minimizing the interference afforded flood water conveyance in non-urbanized floodplains. This objective will thereby reduce the exposure of people and property to the flood hazard. This can best be accomplished by acquiring and maintaining a combination of recreation and open space systems utilizing, whenever feasible, floodplain lands. Watercourses can serve as green arteries through urban areas, where parks and trails can be incorporated for the benefit of the community. Concurrently, this largely preventive program will be supported by corrective approaches which will reduce the existing level of flood damages. This multi-purpose, multi-approach program will contribute to enhanced environmental quality, social well-being, and economic stability by emphasizing orderly community growth.

MULTIPURPOSE PLANNING OPPORTUNITIES

Drainage planning should incorporate compatible multipurpose planning concepts. These concepts represent opportunities for achieving a wide variety of community benefits. Some of these representaive benefits include:

- Reduced urban and social disruption which can often occur during minor and major flooding events.
- Reduced street construction and maintenance costs due to improved surface and subsurface drainage.
- Improved traffic movement, thereby allowing the unobstructed passage of emergency vehicles during periods of high stormwater runoff.
- Improved groundwater management, thereby avoiding subsurface foundation drainage problems due to high shrink-swell potential, poor percolation, or high groundwater levels.
- Better quality non-point discharges of urban runoff and resultant storm water quality.
- Improved erosion control can be realized through the use of subdivision, slow-flow drainage swales, onsite detention, terracing and energy dissipation devices. This will result in improved water quality due to the removal of sediment and debris.
- Public health benefits due to a decrease in standing water and related improvement in pest and insect control.
- Close-in solid waste disposal sites due to the availability of cover material from excavated storage ponds. Sculptural land forms and sound barrier embankments can be the result of well-planned solid waste disposal sites.
- Acquisition of community open space can be accomplished when integrated with undeveloped floodplain preservation. Such open space can at once serve as a conveyor or storm water runoff, while meeting passive open space needs.

- Active recreational requirements can be met when combined with floodplain acquisition. Trails for hiking, biking, and horseback riding can be tied to the development of community recreational facilities. Such facilities could include athletic fields, tennis courts, and playgrounds. Land acquisition costs are generally lower than for non-floodplain property.
- Reduced subdivision costs due to adequate drainage improvements associated with multi-use open space/recreational facilities. By utilizing comprehensive storm water management planning, developers can realize substantial cost savings due to lessened storm sewer improvement costs and emphasis on community open space.

RECREATION AND OPEN SPACE

Traditionally, drainageways have been considered undesirable elements within the community; however, they can serve a variety of purposes in addition to their stormwater conveyance and storage functions. They offer sites where recreational facilities would serve as buffers for developed areas; they can provide linkages to adjacent neighborhoods and public facilities, provide wildlife habitats, and protect areas of scenic importance to the community.

Public parks, open space, and recreation facilities are highly visible and are among the most popular services provided to citizens by a community. In a growing community, open space and park planning should anticipate future needs as well as determine what lands and facilities are needed to meet current demands. It is important in a developing community to identify open space opportunities which can be utilized for multipurpose community uses. Several important functions of open space are to:

- Protect drainageways and floodplains,
- Allow for the provision of park and recreation facilities,
- Protect a natural resource,
- Serve as a buffer between otherwise incompatible land uses,
- Define urban development through the use of open spaces; most trees are found along creeks and streams;
- Serve as an open space corridor or link between urban activities such as schools, parks, and shopping centers;

- Stabilize land values on adjacent properties and encourage the maintenance of city areas utilizing multiple-use strategies,
- Enhance the image of a community and encourage tourism,
- Provide visual relief from the developed urban landscape,
- Preserve unique geologic or scenic areas.

The City of Stillwater has recently adopted a policy of combining floodplain lands and park and recreation facilities, where possible.(2) Specifically, future development will be prohibited in the floodway and necessary easements for channel clearing equipment will be obtained. This policy will allow waterways and adjacent floodplain lands to be incorporated into a linear open space system for pedestrian, bicycle, and general recreational uses.

In addition, park lands and schools will be combined with public recreation facilities, providing lower cost public facilities to the taxpayer. When utilizing floodplain open space lands for combined recreation/school purposes, an added benefit accrues. School children will be able to walk safely to school and recreation areas which will be separated from streets and vehicular traffic.

In May, 1971, the Stillwater Greenbelt Committee and City officials initiated the Greenbelt Planning Program.(1) Impetus for the program came from local citizen concern over the U.S. Army Corps of Engineers' plan for controlling flooding along Boomer Creek. Recognizing, at once, the hazard that mature wooded areas would be destroyed due to the channel improvement program and that an opportunity to begin building a greenbelt system was at hand, concerned citizens began planning for the future. Work also began on preserving the attractive stands of mature trees along Stillwater Creek and its largely undeveloped floodplain. The Plan envisions a greenbelt along the two streams, with areas of concentrated development "nodal areas" connected by linear open space elements. The Greenbelt Planning Program will ultimately consist of all principal streams and connecting lakes and ponds in the urban area together with appropriate adjoining floodplain lands.

More importantly, it represents a multipurpose planning concept with multiple community benefits. The plan will preserve and enhance scenic natural areas, while providing uninterrupted movement to cyclists, pedestrians, equestrians, and in the case of parkways, to motorists. In doing so, the plan will maximize the benefits and minimize possible adverse effects of flood control measures.

Transportation

The majority of drainage basins and their stream networks are significantly changed by transportation facilities such as freeways, local roads, railways and airports. Combined efforts of transportation and drainage planning can often result in improved facilities at a lesser cost. When proper planning is done, necessary fill for road embankments can come from drainage work construction; when unplanned, the carrying capacity of natural waterways can be severely limited by filling in the floodplain. Often new roads for a subdivision can be planned to provide embankments for detention and retention storage sites.

When drainage and transportation planning are integrated, the inconvenience to traffic movement can be minimized during severe flooding events. On a larger scale, major transportation facilities such as freeways can be planned so that when a large flood storage basin (retarding basin) is designed, it can be combined with the construction of a major roadway crossing of a stream or river.

Water Supply

Water for irrigation, industry, and other purposes can be obtained in conjunction with drainage planning, particularly with regard to man-made storage. In particular, the case of stormwater for irrigation of farm, open space and park land is a use which should be developed to provide water quality enhancement, and to reduce the demand upon central water systems as a conservation measure.

Opportunities also exist for improving groundwater availability. The recharge of groundwater is an important environmental consideration in an urban area. Impervious surfaces, closed stormwater systems, and water wells often serve to deplete the natural water table in developed areas, although a significant amount of lawn watering would tend to counteract this. Natural drainage courses and stormwater detention facilities allow for percolation of stormwater into the ground to recharge the groundwater.

Wastewater Treatment

Quality of runoff water can be degraded by discharge of heated process water from some industries and by contaminants during rainfall events. There are many ways to reduce water quality degradation in a drainage program. Some possibilities are sedimentation and debris basins, aerated ponds, and land treatment (grass filtration) methods, combined with open space and grassed channel solutions.

In August, 1975, a wastewater facility plan was completed for Stillwater.(2) Plans for the treatment plant included modification and refurbishing of existing equipment. Of the several treatment alternatives examined one involved trickling filters plus land application of wastewater. Other innovative alternatives evaluated were treatment and reuse of resultant non-potable water and the utilization of appliances which use less water in an attempt to reduce the amount of wastewater flow.

The American Public Works Association, in 1972, conducted an on-site field survey of approximately 100 facilities in all climatic zones, where community or industrial wastewaters are being applied to the land, as contrasted to the conventional method of treating such wastes and discharging them into receiving waters,(3)

Land application of sewage effluent may be employed for a variety of reasons:

1. To provide supplemental irrigation water;
2. To give economical alternative solutions for treating wastes and discharging them into receiving waters, without causing degradation of the receiving waters, and
3. To overcome the lack of suitable receiving waters and eliminate excessive costs of long outfall lines to reach suitable points of disposal.

Economics of construction costs, operating costs, energy requirements, and efficiencies of performance of land application systems must be balanced with the ability to acquire the right to apply wastewater upon the required land areas. The cost of advanced waste treatment by conventional means must be weighted in light of the costs and complexities of land application systems.

The APWA study verified the relative success of present land application systems for supplementing groundwater sources, providing economical means of effluent disposal, improving effluent quality by soil uptake of constituents which would adversely affect receiving waters, enhancing crop growths and silviculture, and augmenting indigenous water supplies for recreation and aesthetic purposes. Relatively little need was found for providing special environmental protection measures in land application areas. Rather, such facilities were often found to enhance the environment.

Solid Waste Disposal and Extractive Industry

As a catchment develops, there is increasing demand to fill in the floodplain. There is a need to acquire land for development and space for related solid waste. When solid waste disposal sites are not provided at convenient locations, this material often ends up in the streams and waterways of the basin. Solid waste disposal sites can be planned in conjunction with drainage planning. A few examples of multipurpose planning for solid waste disposal are:

- New land forms outside the floodplain can be created with solid waste mounds which are covered with excavated material from drainage improvements. These land forms are especially constructed in clay capsules to prevent groundwater contamination. They can provide a sculptured landscape which has beneficial uses.
- Sound barrier embankments outside the floodplain can be created using solid wastes that are covered with material excavated from drainage projects.
- While often used for solid waste disposal sites, extractive activities which remove sand, clay, and rock for building purposes can be planned and controlled so that the exhausted sites can be developed as storage

facilities for floodwaters. Eventually, these storage sites can be converted to solid waste disposal sites, if properly lined to prevent groundwater contamination. In that event, adequate compensatory storage for floodwaters previously stored at that site would have to be provided. Such multiple sequential land-use emphasizing multipurpose planning would provide considerable cost savings to a community.

INTANGIBLE BENEFITS

Natural resource planning organizations have begun to recognize the importance of the intangible components of resource utilization. Almost by definition intangible benefits were, until recently, considered to be immeasurable and were not included as a part of project evaluation.

Intangibles include those components of environmental appreciation which are not directly quantifiable in terms of dollar value or dollars spent for their use. Normally, intangibles accrue from the aesthetic, scientific, educational, historical, and recreational aspects of natural and man-made environments. One additional intangible benefit, peculiar to residents of flood hazard areas, is the peace of mind which can be enjoyed by those safeguarded from future flood damages.

Although the appropriateness of placing dollar values on intangibles is certainly open to question, several techniques have been developed by researchers to estimate the value of intangibles in terms of dollars.

One of the least subjective and most practical techniques is to evaluate intangible benefits in terms of someone's willingness to pay for that benefit. In the case of privately owned recreational and scenic areas, the admission charge is generally representative of the value of an experience. In the case of publicly owned facilities, monetary values are indirectly expressed in the expenditures incurred in a recreational trip or the cost of admission at a similar private area.

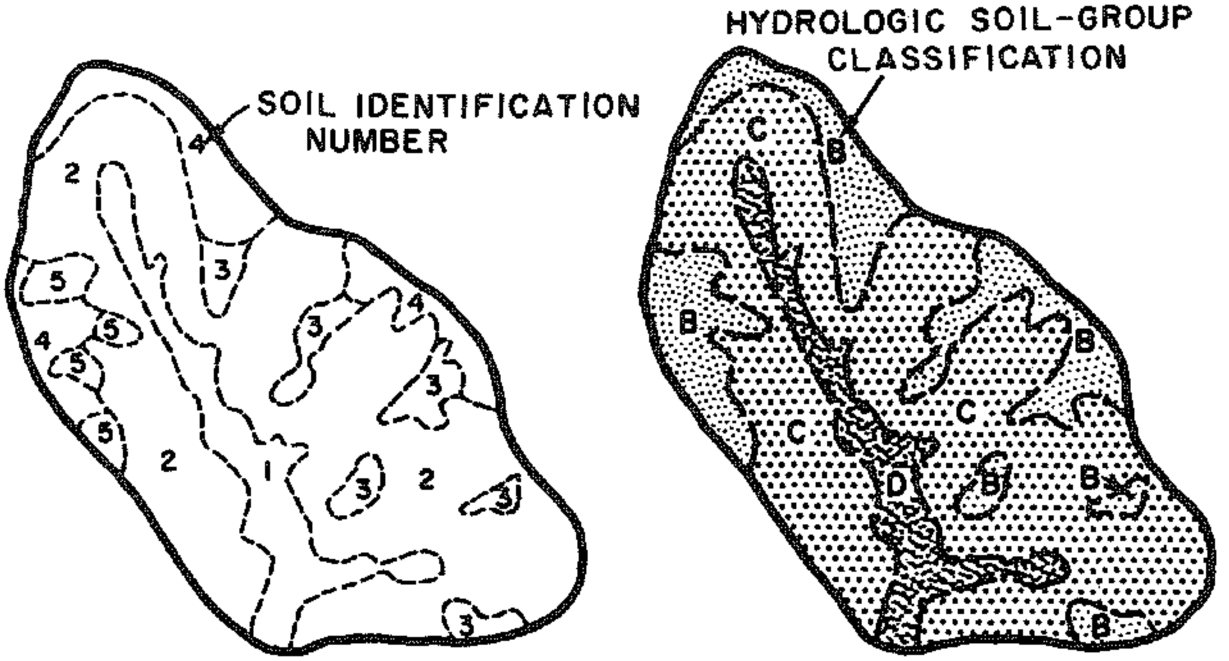
REFERENCES

1. Greenbelt Plan (Preliminary), Technical Document prepared for the City of Stillwater, Oklahoma, Erling Helland Associates/Robert M. Black, Tulsa, Oklahoma, September 1972.

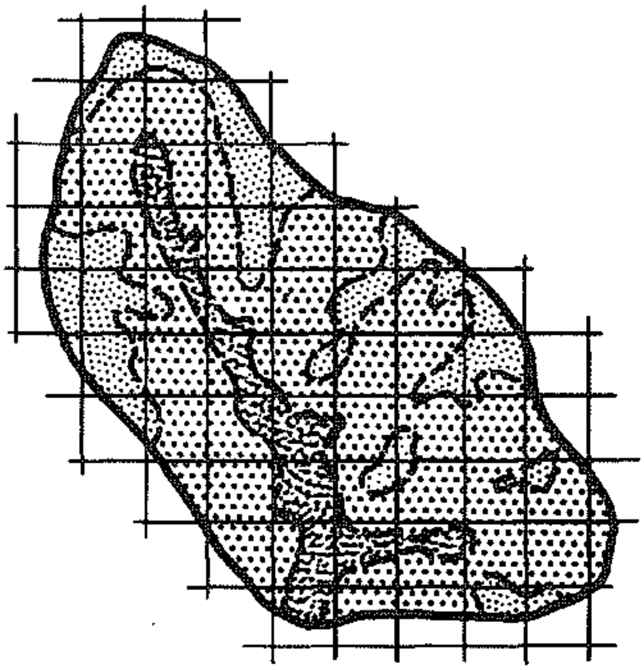
2. Stillwater Comprehensive Plan - 2000, (Technical Document), City of Stillwater, Oklahoma.

3. Survey of Facilities Using Land Application of Wastewater, American Public Works Association, and U.S. Environmental Protection Agency, July 1973.

4. Urban Drainage and Flood Control - Goals, Objectives, Policy, and Principles, Technical Document Preparation for the City of Stillwater, Oklahoma, Wright-McLaughlin Engineers, Smith-Biffle and Associates and R.D. Flanagan and Associates, March, 1979.



(a) DETAILED SOILS MAP (b) HYDROLOGIC SOIL GROUP MAP



(c) GRID ON SOIL GROUP MAP

SOIL GROUP	NUMBER GRID INTERSECTIONS	PERCENT
B	12	23*
C	32	63
D	7	14
TOTAL	51	100

*PERCENT FOR B:

$$(100)\frac{12}{51} = \underline{\underline{23}}$$

(d) COMPUTATIONS

FIGURE 1-7
STEPS TO DETERMINE PERCENTAGES OF SOIL GROUPS
(REF. 22)

(d) The tabulation and a typical computation of a group percentage is shown. In practice, simplified versions of this procedure are generally used.

Often one or two soil groups are predominant in a watershed, with others covering only a small part. Whether the small groups should be combined with those that are predominant depends on their classifications. For example, a hydrologic unit with 90 percent of its soils in the A group and 10 percent in D will have most of its storm runoff coming from the D soils. Putting all soils into the A group will cause a serious underestimation of runoff. If the groups are more nearly alike (A and B, B and C, or C and D), the under- or over-estimation may not be as serious, but a test may be necessary to show this. Rather than test each case, follow the rule that two groups are combined only if one of them covers less than about 3 percent of the hydrologic unit. Impervious surfaces should always be handled separately because they produce runoff even if there are no D soils (Ref. 10).

Determination of the Runoff Curve Number (CN). Once the soil group is known, the runoff curve number is determined by consideration of the surface conditions, vegetation cover, and other cover factors. References 10, 14 and 18 present detailed information for selection of the runoff curve number, as well as guidance toward determining a curve number for areas having mixtures of different soil and cover conditions.

Table I-9 lists CN's for agricultural, suburban, and urban land use classification. The suburban and urban CN's are based on typical land use relationships that exist in some areas. They should only be used when it has been determined that the area under study meets the criteria for which these CN's were developed (noted in table).

There will be areas to which the values in Table I-9 do not apply. The percentage of impervious area for the various types of residential areas or the land use condition for the pervious portions may vary from the conditions assumed in Table I-9. A curve for each pervious CN can be developed to determine the composite CN for any density of impervious area. Figure I-8 has been developed assuming a CN of 98 for the impervious area. The curves in Figure I-8 can help in estimating the increase in runoff as more and more land within a given area is covered with impervious material.

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HYDROLOGY

The determination of flood flows, flow patterns, and volumes is an important part of the process of drainage planning and design. Various rainfall and runoff hydrologic methods are used to determine this information. There are usually several methods that could be used, thus a key step is determining which are appropriate for a given problem. Guideline criteria and several suitable methods are presented here.

This Chapter has two sections as outlined in the Table of Contents: Section A, Rainfall; and Section B, Runoff.

The user of this Chapter will be preparing hydrologic calculations which can usually be categorized into three types. Table I-1 presents these categories, an explanation of the types of calculations required, and guidance as to which subsections to refer to for criteria and calculation procedures.

TABLE I-1

HYDROLOGY GUIDE

CATEGORY	TYPES OF CALCULATIONS PERFORMED	REFERENCE SECTIONS
I - Development drainage planning and design for small areas, generally less than 100 acres and not involving floodplains. Complicated storage facilities are referred to Category II	Peak discharges for local drainage system and estimates of runoff volumes for purposes of onsite detention storage.	-Rainfall Intensity Duration Curves -Rational Method
II - Development Drainage Planning and design for larger areas, generally greater than 40 acres and involving floodplains.	Local drainage will be handled in a fashion as described for Category I	-Rainfall Intensity Duration Curves -Rational Method
	As the tributary area increases, reliance will shift to design storms, runoff hydrographs, stream and reservoir routing, and ultimately use of sophisticated computer models.	-Design Storm Development -Rainfall Excess and Infiltration -Synthetic Unit Hydrograph Procedure -Computer Modeling Approaches
III - Master planning of drainage basins and design of improvements.	Simpler problems can be handled with runoff hydrographs and basic stream routing. Problems involving complicated drainage basins and intricate alternatives point toward use of computer tools.	-Design Storm Development -Rainfall Excess and Infiltration Synthetic Unit Hydrograph Procedure -Computer Modeling Approaches

CHAPTER I

SECTION A - RAINFALL

BACKGROUND DATA

The rainfall data presented here is derived from two sources:

1. Rainfall Frequency Atlas of the United States, for Durations from 30 Minutes to 24 Hours and Return Period from 1 to 100 years, Technical Paper No. 40 (Ref. 1).
   Five to 60-Minute Precipitation Frequency for the Eastern and Central United States, Technical Memorandum NWS HYDRO-35 (Ref. 2).

The development of specific rainfall curves for the City of Stillwater from the above references involved the selection of precipitation values for each frequency or return period from the maps and equation provided. This data is presented in Table I-2. These values were then used to construct rainfall depth-duration-frequency graphs as illustrated in Figure No. I-1. Because of the small variance in rainfall potential within the area only one set of curves was derived.

TABLE I-2

DEPTH-DURATION-FREQUENCY RAINFALL DATA POINTS
CUMULATIVE RAINFALL DEPTH (INCHES)

	Frequency					
	2-year	5-year	10-year	25-year	50-year	100-year
5 minute	0.48	0.57	0.63	0.72	0.80	0.87
10 minute	0.79	0.94	1.04	1.20	1.33	1.45
15 minute	1.01	1.20	1.34	1.54	1.70	1.86
30 minute	1.41	1.73	1.96	2.29	2.61	2.81
1 hour	1.83	2.29	2.61	3.08	3.44	3.80
2 hour	2.14	2.75	3.28	3.84	4.37	4.88
3 hour	2.23	3.09	3.60	4.19	4.75	5.36
6 hour	2.68	3.63	4.25	5.00	5.55	6.18
12 hour	3.25	4.29	5.06	5.89	6.55	7.36
24 hour	3.77	4.96	5.74	6.75	7.61	8.56

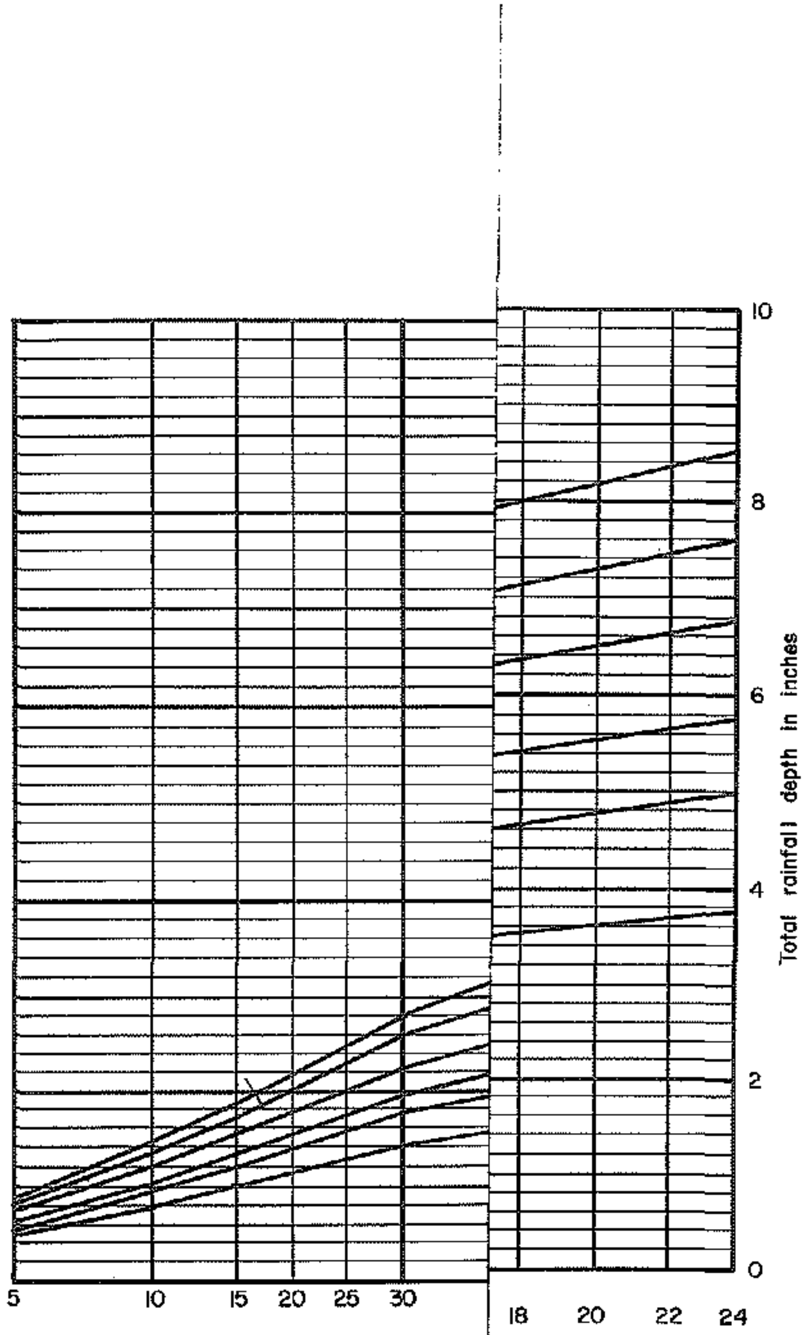


FIGURE I-I
RAINFALL DEPTH - DURATION -
FREQUENCY GRAPH
STILLWATER, OKLAHOMA

It is important to recognize that rainfall events of a greater magnitude than those illustrated above can and do occur. For greater risk situations, such as dams, probable maximum precipitation data is also used for analysis and design purposes. Table I-3 illustrates this data for the Stillwater area which was taken from Reference 3 as prepared by the Weather Bureau, Corps of Engineers, and the Bureau of Reclamation.

TABLE I-3

STILLWATER PROBABLE MAXIMUM PRECIPITATION DATA
(Ref. 3)

Duration (Hours)	Depth (Inches)
6	30.6
12	33.8
24	36.1
48	38.6

RAINFALL INTENSITY DURATION CURVES (RATIONAL METHOD)

The Rational Method for estimating runoff (explained in the following section) uses rainfall data from Figure I-1 and converts it into units of intensity (inches of rainfall per hour). Figure I-2 presents intensity-duration curves for various frequency storms. It is based on the data presented in Table I-2.

DESIGN STORM DEVELOPMENT (UNIT HYDROGRAPH AND MODELING METHODS)

The data presented in Table I-2 is developed into a design storm appropriate to the drainage basin and other considerations discussed herein. The design storm(s) is used as input to hydrologic methods which vary from synthetic unit hydrograph procedures to computer modeling.

One of the implicit assumptions of this Manual, and one that is common in drainage practice, is that a certain frequency rainfall event results in essentially the same frequency runoff event. It would be academically more correct to use a complete rainfall history (such as a 25-year weather bureau daily rainfall tape) to generate a synthetic runoff record. The statistical analysis of such a synthetic record should result in better runoff

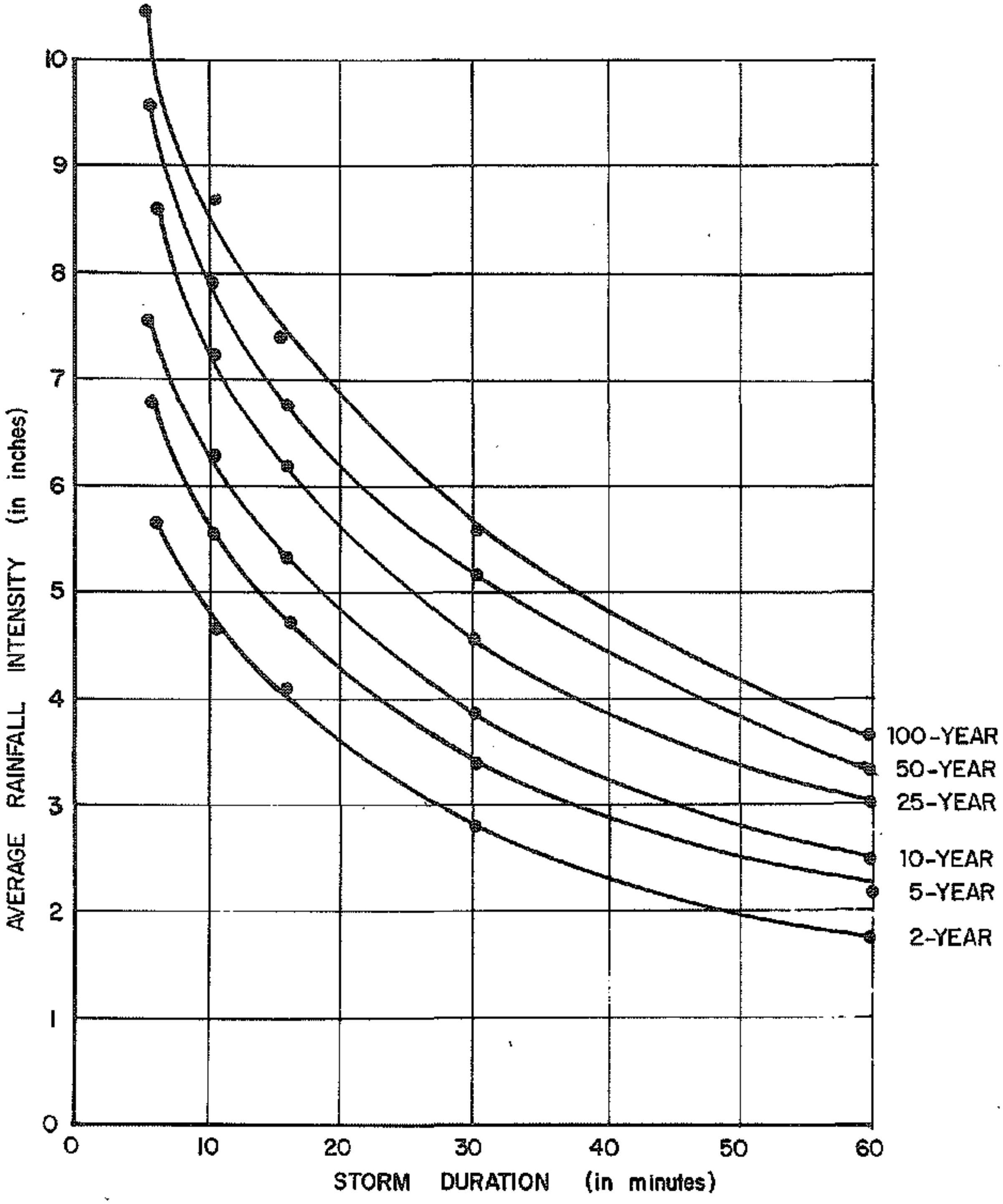


FIGURE I-2
RAINFALL INTENSITY-DURATION CURVES
for STILLWATER, OKLAHOMA

frequency information; however, the analysis costs are usually prohibitive and result in little appreciable improvement. Thus the assumption that a given frequency event results in the same frequency runoff event is deemed valid.

Spatial Corrections

The data presented in Table I-2 is based on rainfall at point locations. The average rainfall over a watershed will be smaller than at any point location. The correction for this effect is presented in Figure I-3 (Ref. 1). Normally, the correction factor for areas less than 10-square miles is small and can be neglected. For larger areas, the factor indicated should be used to reduce each of the values in Table I-2 and curves in Figure I-1. Generally speaking, the factor for the smallest subarea of interest should be used for multiple-subarea study.

Duration of Interest

The minimum storm duration to be used should not be less than two times the time of concentration (see Runoff Section) of the total study area. For studies involving storage alternatives or proposals that could significantly affect flows and discharge volumes downstream of the immediate study areas, then design rainfall events with durations appropriate to each should be used.

Time Increment

A design storm is input to the given runoff algorithm in terms of incremental rainfall amounts for a given time period. For the synthetic unit hydrograph procedure (see Runoff Section) it is often defined as:

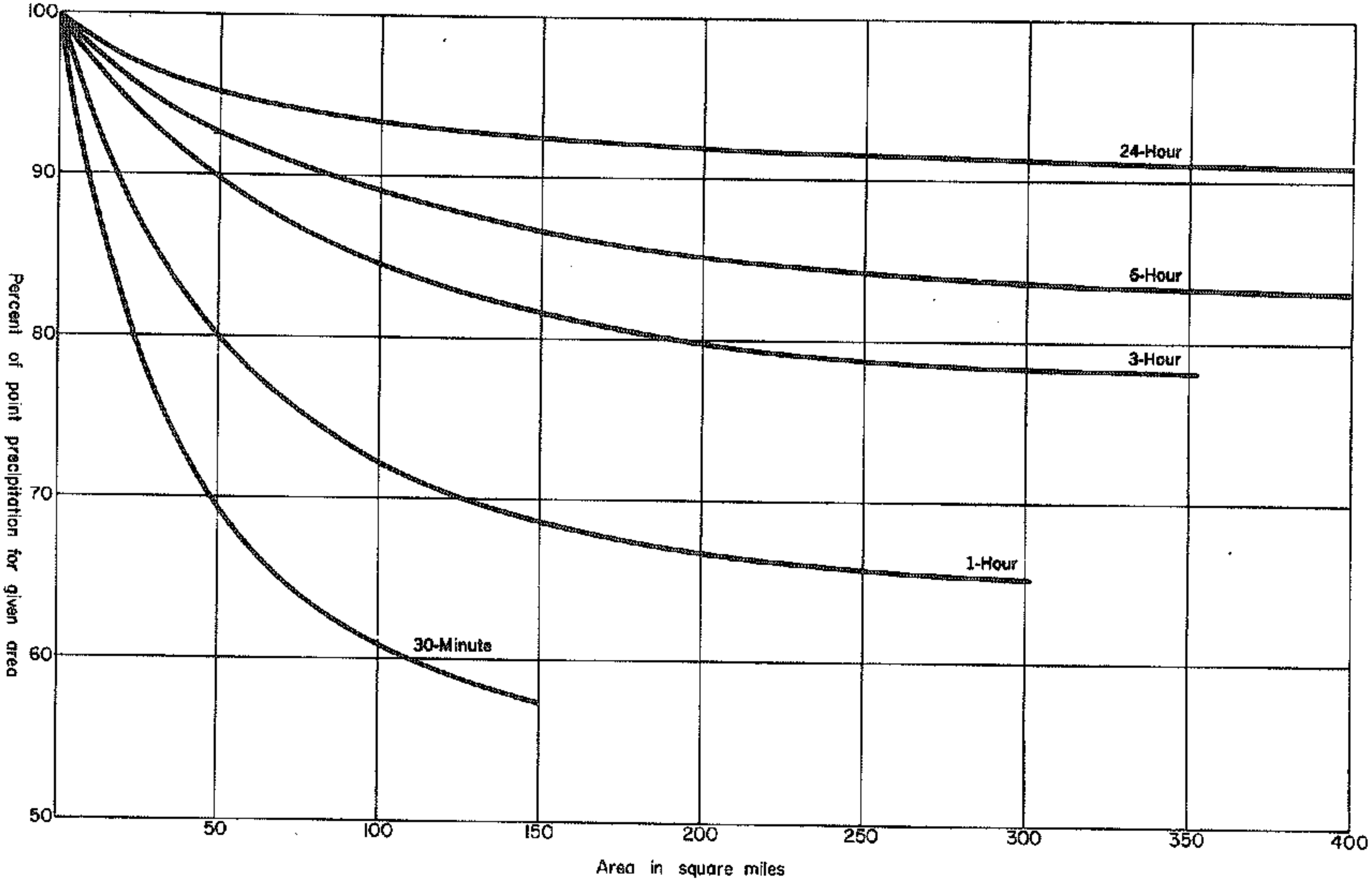
$$t_u = \frac{t_c}{7.5} \qquad \text{Eq. I-1}$$

where

t_u = incremental rainfall time increment

t_c = time of concentration (Ref. 4)

FIGURE I-3
AREAL ANALYSIS GRAPH



This value should be computed, but rounded appropriately. Rainfall is rarely measured in intervals of less than 5 minutes, thus the use of smaller increments is generally for numerical analysis purposes. Well-shaped hydrographs may not be achieved for small basins if the increment is too large.

Other runoff modeling approaches usually have guidelines as to computational time increments which should be used as the basis for rainfall time increment selection.

Temporal Patterns

There has been limited documentation (Ref. 5) that illustrates that a given duration rainfall event will tend toward a typical rainfall pattern that will be different from the pattern of events of other durations. The common practice is to arrange the rainfall in a critical pattern. The following procedure is essentially that used by the Bureau of Reclamation, the City of Tulsa, and other cities and is similar to procedures used by the SCS:

1. The heaviest incremental rainfall will be placed in the time increment immediately following the first half of the design storm duration.
2. The succeeding lower valves will be placed symmetrically around that peak increment.

Example Design Storm Calculation

The following information illustrates an example preparation of a 100-year design storm for a basin with a time of concentration of approximately 75 minutes.

Step 1. Given t_c = 75 minutes then,

$$t_u = \frac{t_c}{7.5} = 10 \text{ minutes}$$

and the recommended duration would be at least

75 minutes x 2 = 150 minutes

then use 3-hour duration.

Step 2. Design rainfall data from Table I-2 is listed below:

Time (Minutes)	Rainfall (Inches)
5	0.87
10	1.45
15	1.86
30	2.81
60	3.80
120	4.88
180	5.36

Additional values can be interpolated from Figure I-1 for various times.

Time	Rainfall
20	2.20
40	3.20
50	3.50
80	4.20
100	4.55
150	5.15

Step 3. Note that for later times the incremental rainfall amounts will be essentially equal for several consecutive time increments. After the incremental rainfall values are tabulated, it can be rearranged into the design event.

Time (minutes)	Cumulative Rainfall from above	Incremental Rainfall	100-year 3-hour Duration Design Storm (Rearranged Incremental Rainfall)
10	1.45	1.45	0.07
20	2.20	0.75	.07
30	2.81	.61	.09
40	3.20	.39	.16
50	3.50	.30	.17
60	3.80	.30	.20
70	-	.20	.30
80	4.20	.20	.39
90	-	.18	.75
100	4.55	.17	1.45
110	-	.17	.61
120	4.88	.16	.30
130	-	.09	.20
140	-	.09	.18
150	5.15	.09	.17
160	-	.07	.09
170	-	.07	.09
180	5.36	.07	.07
	Subtotal	5.36	5.36

RAINFALL REFERENCES

1. Rainfall Freqency Atlas of the United States for durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 years, Weather Bureau Technical Paper 40, U.S. Department of Commerce, Washington, D.C., May, 1961.

2. Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States, NOAA Technical Memoradum NWS HYDRO-35, U.S. Department of Commerce, Silver Spring, MD, June 1977.

3. Design of Small Dams, Water Resources Technical Publication, Bureau of Reclamation, U.S. Department of Interior, Washington, D.C., 1973.

4. SCS National Engineering Handbook, Section 4, Hydrology, U.S. Department of Agriculture, Washington, D.C., January 1971.

5. Point Rainfall Intensity-Frequency-Duration Data, Capital Cities, C. L. Pierrehumbert, Department of Science, Bureau of Meteorology, Bulletin No. 49, Australian Government Publishing Services, Canberra, August 1974.

CHAPTER I

SECTION B - RUNOFF

INTRODUCTION

The storm runoff peak flow rate, volume, and timing provide the basis for all planning, design, and construction of drainage facilities. The intent of this Chapter is to describe methods of approximating the characteristics of rainfall runoff and define acceptable ranges of application.

Analytical Methods

A review of current practice shows that four basic approaches can be used for determining the character of storm runoff. They are the Rational Method, the Synthetic Unit Hydrograph Procedure (SUHP) and similar adaptations, computer simulation modeling, and statistical analyses.

As the entire Stillwater area is or will potentially be urbanized, or modified, analysis of historical data cannot reveal future condition discharges, thus, statistical analysis methods are not described in this Manual.

Applicability of Methods

The multiple-characteristics of most drainage basins, development patterns, and complex alternative proposals often will necessitate the use of the SUHP or computer simulation modeling techniques. Drainage basin management is a space allocation problem which emphasizes the need to analyze for the time distribution of stormwater runoff and its volume.

For small drainage basins that are uncomplicated, the Rational Method is appropriate for planning and design purposes. It is in use throughout the world and has been found to be satisfactory for small and simple drainage systems.

RATIONAL METHOD

For basins that are not complex and have generally 100 acres or less, it is recommended that the design storm runoff be analyzed by the Rational Method.

Even though this method has frequently come under academic criticism for its oversimplifications, no other practical drainage design method has evolved to a level of general acceptance by the practicing engineer. The Rational Method properly understood and applied can produce satisfactory results for urban storm sewer design (Refs. 1, 2, 3, and 4)

Rational Formula

The Rational Method is based on the Rational Formula:

$$Q = CIA \qquad \text{Eq. I-2}$$

Q is defined as the maximum rate of runoff in cubic feet per second. (Actually Q has units of inches per hour per acre; however, since the conversion factor to cfs is less than a 1 percent adjustment, the more common cfs is used.) C is a runoff coefficient which is the ratio between the maximum rate of runoff from the area and the average rate of rainfall intensity, in inches per hour, for the period of maximum rainfall of a given frequency of occurrence having a duration equal to the time of concentration. I is the average intensity of rainfall in inches per hour for a duration equal to the time of concentration. The time of concentration usually is the time required for water to flow from the most remote point of the area to the point being investigated. A is the area in acres.

Assumptions

The basic assumptions made when the Rational Method is applied are:

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate during a period of time equal to the time of concentration.

   The maximum rate of rainfall occurs during the time of concentration and the design rainfall depth during the time of concentration is converted to the average rainfall intensity for the time of concentration.

3. The maximum runoff rate occurs when the entire area is contributing flow, which is defined as the time of concentration.

Limitations

The Rational Method is a wholly adequate method of approximating the peak rate of runoff from a rainstorm in a given basin. When the basins become complex and where subbasins come together, the Rational Method will tend to overestimate the actual flow, which results in oversizing drainage facilities. One reason the Rational Method is limited to small areas is that good design practice requires the routing of hydrographs for larger basins for economical design.

Another disadvantage of the Rational Method is that with typical storm sewer design procedures one normally assumes that all of the design flow is collected at the design point and that there is no carryover water running overland to the next design point. There must be some modification to the Rational Method, or another type of analysis used, when analyzing an existing system that is underdesigned or when analyzing the effects of a major storm on a system designed for the minor storm.

Time of Concentration

In the application of the method, the time of concentration must be estimated so that the average rainfall intensity of a corresponding duration can be determined from the rainfall intensity-duration curves (Figure I-2).

For urban storm sewers the time of concentration consists of an inlet time, or time required for runoff to flow over the surface to the nearest inlet, and time of flow in the sewer to the point under consideration. The latter time can be closely estimated from the hydraulic properties of the sewer. Inlet time, on the other hand, will vary with surface slope, depression storage, surface cover, antecedent rainfall, and infiltration capacity of the soil, as well as distance of surface flow. In general, the higher the rainfall intensity, the shorter the inlet time. Common urban practice varies the inlet time from 10 to 30 minutes. When dealing with pipe systems, the time of concentration can be readily calculated from the inlet time plus time of flow in each successive pipe run. The latter value is calculated from the velocity of flow as given by the Manning Formula for hydraulic conditions prevailing in the pipes.

The inlet time can be estimated by calculating the various overland distances and flow velocities from the most remote point. A common mistake is to assume velocities that are too high for the areas near the storm collecting drains. Often the remote areas have flow that is very shallow and in this case the velocities cannot be calculated by channel equations such as Manning's, but special overland flow analyses must be considered (Ref. 5). Figure I-4 can be used to help estimate time of overland flow. (Ref. 43)

Another common error is to only analyze the flow from the entire basin. When a smaller portion of the basin has a quicker response and a higher proportion of rainfall that becomes runoff it can thus have higher peak flow rates. This situation is often encountered in a long basin, or a basin where the upper portion contains rural areas or grassy park land and the lower portion is developed urban land. Thus, flow rates from homogeneous subbasins having high runoff potential should also be analyzed.

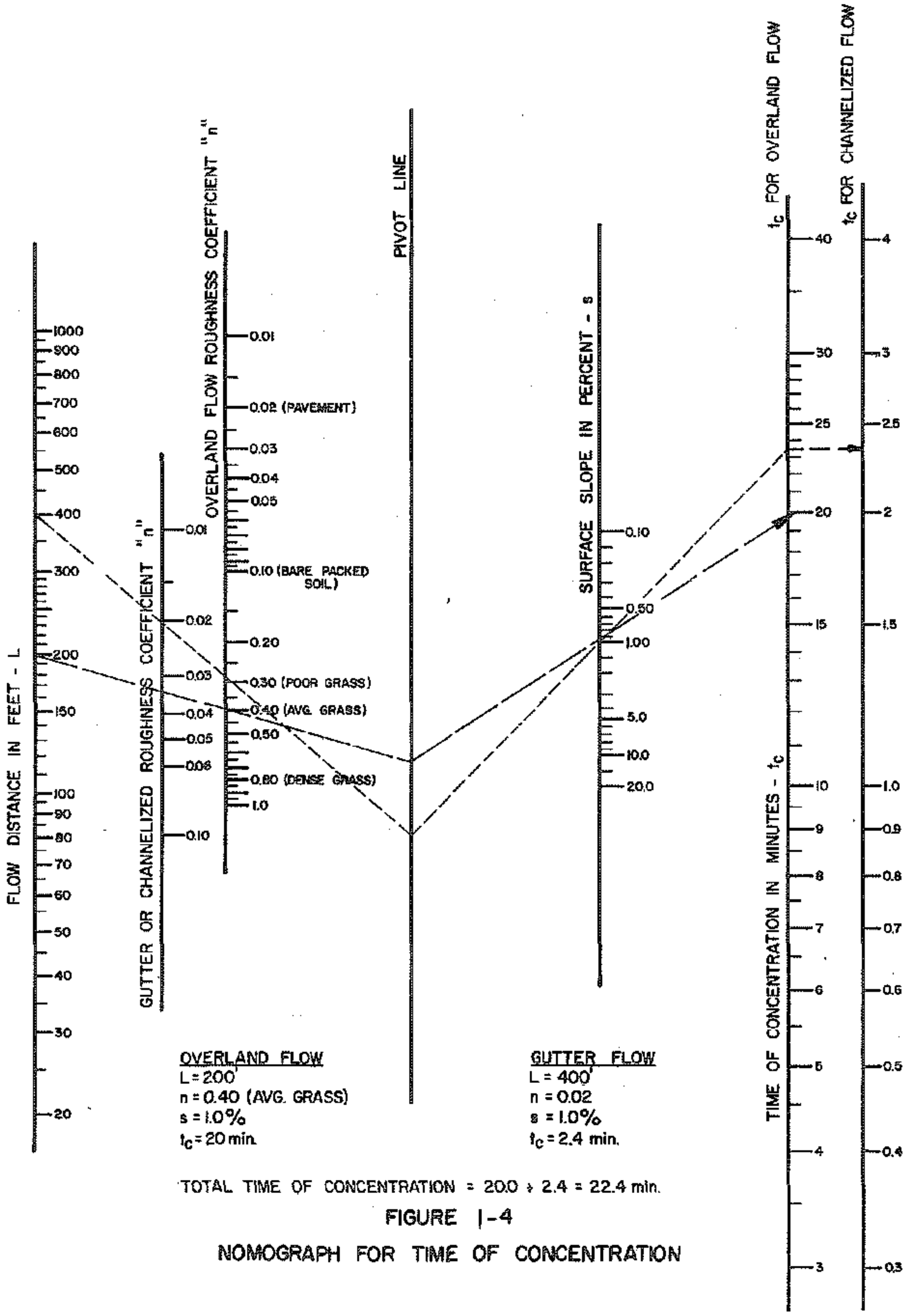
Figure I-4 is also a guide to be used for estimating the flow times in street channels and pipes. The drainage network characteristics should be carefully checked to determine if they are within the range of and appropriately represented by this graph.

Areas that have higher density development should include a time factor for runoff from roofs and paved areas. Values of 5 to 10 minutes have commonly been used.

When studying proposed subdivision land, do not take the overland flow path perpendicular to the contours since the land will be graded and swales will often intercept the natural contour and conduct the water to the streets thus cutting down on the time of concentration. The typical flow pattern after finished grading should be investigated.

Intensity

The intensity, I, is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration.



OVERLAND FLOW
L = 200'
n = 0.40 (AVG. GRASS)
s = 1.0%
t_c = 20 min.

GUTTER FLOW
L = 400'
n = 0.02
s = 1.0%
t_c = 2.4 min.

TOTAL TIME OF CONCENTRATION = 20.0 + 2.4 = 22.4 min.

FIGURE 1-4

NOMOGRAPH FOR TIME OF CONCENTRATION

After the design storm frequency has been selected, the appropriate intensity value for the time of concentration should be selected from Figure I-2.

Rational Runoff Coefficient

The runoff coefficient, C, is the variable of the Rational Method that is least susceptible to precise determination and requires judgment and understanding on the part of the engineer. Its use in the formula implies a fixed ratio for any given drainage area. In reality, this is not the case. The coefficient represents the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception which all affect the time distribution and peak rate of runoff.

Table I-4 presents C values given by the American Society of Civil Engineers (Ref. 2) and recommended values for Stillwater. The recommended values are based upon general soils types shown in Figure I-6 for Stillwater, consideration of average slopes and composite analysis of C factors presented in Table I-5.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. This procedure is often applied to typical sample subareas as a guide to selection of reasonable values of the coefficient for an entire area. Suggested coefficients with respect to surface type are given in Table I-5. The values for streets, drives, walks and roofs are ASCE values in Reference 2. The values for soils are from Reference 3. The soils classes given are SCS, and explained later. The soils in Group A are sandy with high infiltration capacity while D soils are heavy with low infiltration. These values are essentially the same as given by ASCE. Table I-7 presents typical estimates of pervious and impervious areas for different land uses.

The Oklahoma Department of Transportation has developed curves for different land uses and slopes. These are readily available in their technical manual, (Ref. 44).

TABLE I-4

RATIONAL METHOD RUNOFF COEFFICIENTS

Description of Area	ASCE Runoff Coefficients	Recommended Stillwater Coefficients
Business:		
Central Business areas	0.70 to 0.95	0.90
District and local areas	0.50 to 0.70	0.65
Residential:		
Single-family areas	0.35 to 0.45	0.45
Multi-units, detached	0.40 to 0.60	0.55
Multi-units, attached	0.60 to 0.75	0.65
Residential (1/2-acre lots or larger)	0.25 to 0.40	0.40
Industrial:		
Light areas	0.50 to 0.80	0.80
Heavy areas	0.60 to 0.90	0.90
Parks, cemeteries	0.10 to 0.25	0.24
Playgrounds	0.20 to 0.35	0.30
Railroad yard areas	0.20 to 0.40	0.35
Unimproved areas	0.10 to 0.30	0.28

TABLE I-5

RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS

For Impervious Surfaces (Ref.2)

Character of Surface	Runoff Coefficients
Streets:	
Asphaltic	0.70 to 0.95
Concrete	0.80 to 0.95
Drives and Walks	0.75 to 0.85
Roofs	0.75 to 0.95

For Pervious Sufaces (Ref. 3)

Slope		Runoff Coefficient			
		A Soils	B Soils	C Soils	D Soils
Flat	0 - 2%	0.04	0.07	0.11	0.15
Average	2 - 6%	0.09	0.12	0.16	0.20
Steep	Over 6%	0.13	0.18	0.23	0.28

Adjustment for Major Storms

The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f, which is used to account for antecedent precipitation conditions. The Rational Formula now becomes:

$$Q = CIAC_f \qquad \text{Eq. I-3}$$

The following table of C_f values can be used. The product of C times C_f should not exceed 1.0.

TABLE I-6

FREQUENCY FACTORS FOR RATIONAL FORMULA

Recurrence Interval (years)	C_f
2 to 10	1.0
25	1.1
50	1.2
100	1.25

When analyzing the major runoff occurring on an area that has a storm sewer system sized for a minor (2-10 year) storm, care must be used when applying the Rational Method. Normal application of the Rational Method assumes that all runoff is collected by the storm sewer. In the design of the minor system, the time of concentration is dependent upon the flow time in the sewer; however, during the major runoff the sewers should be fully taxed and cannot accept all the water flowing to the inlets. This additional water then flows past the inlets and continues overland, generally at a lower velocity than the water in the storm sewers. This requires an analysis of the split of total flow between underground flow and overland flow. Times of concentration and resulting peak discharge estimates should be evaluated for major and minor events considering this effect.

Hydrograph Approximations Using the Rational Method

The estimation of runoff hydrographs may be necessary for evaluation of very small and simple development drainage proposals, particularly when they are involved with onsite storage systems. However, such methods should not be used for larger areas or evaluation of costly structures, or where serious ramifications are possible downstream.

There are many versions but the usual concept is that a triangular hydrograph describes the response of the basin from a unit storm of duration equal to the time of concentration. The peak of the triangle (hydrograph) is equivalent to the peak discharge calculated by the Rational Formula. The time-to-peak of the hydrograph is equal to the time of concentration and both are measured from the beginning of the rainfall. The receding limb of the hydrograph is also equal to the time-to-peak, thus the hydrograph is an isosceles triangle.

Application of a design storm appropriate for a watershed can be made by revising the design storm into increments which have a time equal to the time of concentration. The rainfall for each revised time increment is converted to rainfall intensity and an incremental triangular hydrograph response calculated for each. The series of triangular responses can be summed to arrive at a design hydrograph.

The user is reminded that this method is a rough approximation and only appropriate for small simple areas. A safety factor should be used appropriate to the possible variances. One of the key variances is with regard to rainfall losses assumed by the Rational Coefficient C. For major storms, and particularly in urbanized areas, the usual total percentage of rainfall that becomes runoff will be different than that indicated by the factor C. One can refer to the following subsection for insight on typical values.

In practice, one can use the rational peak discharge as indicated, but the hydrograph approximated from this method should be adjusted to more accurately reflect the probable runoff volume and other special effects.

Methods are available for evaluation of storage systems such as the FAA storage method (Ref. 6) which is presented in the storage chapter and other references (Ref. 42). The key point is to remember that these are crude approximations appropriate to only small, simple facilities with no hazard potential.

RAINFALL EXCESS AND INFILTRATION (UNIT HYDROGRAPH AND MODELING METHODS)

Rainfall excess* is that portion of the precipitation which appears in surface channels and man-made subsurface channels during and after a rainstorm. Those portions of precipitation which do not reach the channels are called abstractions, and include: interception by vegetation, evaporation, infiltration, storage in all surface depressions, and long-time surface detention. The total design rainfall can be obtained from the Rainfall Section of this chapter. This subsection illustrates methods for determining the amount of rainfall that actually becomes runoff.

The methods described in this portion of the Runoff Section are for use with the Synthetic Unit Hydrograph Procedure (SUHP) and some computer modeling techniques and do not normally apply to the Rational Method because in the Rational Method the abstractions are accounted for in the C factor. However, the information presented here can aid the engineer in selecting a reasonable runoff coefficient for the Rational Method.

Representations of Infiltration and Other Rainfall Losses

There are many representations of rainfall losses to arrive at rainfall excess, though none are completely satisfactory. There are many texts which describe different methods (Refs. 7, 8, 9, 10, 11, 12, and 13). For the purposes of this Manual, any of these can be used when substantiated by appropriate documentation. The following methods are summarized for use in Stillwater:

1. Use of guideline values.
2. Soil Conservation Service (SCS) Methods (Refs. 10, 14, and 40)

Normally, the use of guidelines is appropriate for simpler hydrologic investigations. Where more precise hydrology is required then the SCS or

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*Rainfall excess is also commonly referred to as effective rainfall or effective precipitation.

other more sophisticated and realistic methods should be used, such as with Master Planning efforts.

Guideline Values

Approximations for detention, depression and infiltration losses are presented following for various surfaces:

Pervious-Impervious Areas. All parts of a basin can be considered either pervious or impervious. The pervious part of a drainage basin is that area where water can readily infiltrate into the ground. The impervious part is the area that does not readily allow water to infiltrate into the ground, such as areas that are paved or covered with buildings. As urbanization occurs, the percent of impervious area increases and the rainfall runoff changes significantly. With the total amount of runoff normally increasing and the time of concentration decreasing, and the peak runoff rates increase substantially (Refs. 15 and 16).

When analyzing an area for design purposes, the probable future percent of impervious area must be estimated. Table I-7 is presented as a guide.

TABLE I-7

LAND USE VERSUS PERCENT OF PERVIOUSNESS/IMPERVIOUSNESS

Land Use	Percent Pervious	Percent Impervious
Central Business zone area, shopping centers, etc.	0 to 5	95 to 100
Residential:		
Dense (apartment houses)	40 to 55	45 to 60
Normal (detached houses)	55 to 65	35 to 45
Large lots	60 to 80	20 to 40
Parks, greenbelts, passive recreation	90 to 100	0 to 10

Depression and Detention Losses. Rainwater that is collected and held in small depressions and does not become part of the general runoff is called depression storage. Most of this water eventually infiltrates or evaporates. Detention losses include water intercepted by trees and bushes, and water that is retained and detained on the surface.

The following table can be used as a guide for estimating the amount of depression and detention storage. It does not include planned ponding areas.

TABLE I-8

TYPICAL DEPRESSION AND DETENTION FOR VARIOUS LAND COVERS

Land Cover	Depression and Detention Values in Inches	
	Range	Recommended
Impervious:		
Large paved areas	0.05 - 0.15	0.1
Roofs, flat	0.1 - 0.3	0.1
Roofs, sloped	0.05 - 0.1	0.05
Pervious:		
Lawn grass	0.1 - 0.5	0.3
Wooded areas and open fields	0.2 - 0.6	Assess each Situation

When an area is analyzed for depression and detention storage, the various pervious impervious storage values must be considered according to the percent of areal coverage.

There are other losses in the impervious area that are not readily quantified such as water lost to evaporation off warm surfaces and water lost due to the natural amount of water that attaches to the surface and cannot run off. This is referred to here as a general pervious area loss. An amount of 0.1 inches will typically be used.

Infiltration. The penetration of water through the soil surface is called infiltration. In urban hydrology, much of the infiltration occurs on areas covered with lawns and gardens. Urbanization normally decreases the total amount of infiltration (Ref. 17).

Soil type is an important factor in determining the infiltration rate. When the soil has a large percent of well-graded fines, the infiltration rate is low. In some cases of extremely tight soil there may be, from a practical standpoint, essentially no infiltration. If the soil has several layers or horizons, the least permeable layer will sometimes control the steady infiltration rate. The infiltration rate is the rate at which water enters the soil at the surface and which is controlled by surface conditions, and the transmission rate is the rate at which the water moves in the soil and which is controlled by the horizons. The soil cover also plays an important role in determining the infiltration rate.

Normally, infiltration rates are higher at the beginning of the storm event. Then as the rainfall continues, the infiltration rate decreases. When rainfall occurs on an area that has little antecedent moisture (the ground is dry), the infiltration rate is much higher than it is with a high antecedent moisture such as from a previous storm or from irrigation. The designer can use a higher infiltration rate at the beginning of the storm and a lower rate as the storm progresses.

Antecedent precipitation can satisfy, wholly or partially, the higher initial infiltration. A high intensity storm can, in some cases, affect the soil surface sufficiently to cause a change in the infiltration.

Other factors affecting infiltration rates include: slope of land, temperature, quality of water, age of lawn, and soil compaction (Ref. 10).

Although it is desirable to have each basin being analyzed for storm runoff field tested for its specific pattern of infiltration, guideline infiltration values for preliminary storm runoff analysis and sewer design can be made. This value is 0.5 inches per hour, expressed as a constant value. It must be emphasized, however, that infiltration rates should normally be further documented for master planning and critical projects which would be sensitive to variations in infiltration rates.

Soil Conservation Service Method

The following discussion of the SCS Method of determining rainfall excess presents many of the components which are more often used for hydrology in developing areas. For more detailed explanations one should refer to SCS and/or the key references (10, 14, and 40).

This technique uses three variables to estimate the rainfall excess during a given event. These variables are rainfall, the antecedent moisture condition, and the hydrologic soil cover complex. The general equation is:

$$I_E = \frac{(P - I_a)^2}{(P - I_a + S)} \qquad \text{Eq. I-4}$$

where I_E = accumulated direct runoff
P = accumulated rainfall
I_a = initial abstraction including surface storage interception, and infiltration prior to runoff
S = potential maximum retention

Although this equation is written for cumulative rainfall and rainfall excess to any given time, a time varying record of the rainfall excess can be easily derived.

Since the above variable S includes I_a, an empirical relationship has been developed from data on watersheds in various parts of the United States. This generally can be expressed as

$$I_a = 0.2\ S \qquad \text{Eq. I-5}$$

The Soil Conservation Service has made extensive experiments and analyses of watershed data to determine the best way to relate the variable S to the soil water storage and the infiltration rates of a watershed. The method adopted is the curve-number (CN) technique. This is simply a method of combining the properties of the soil groups in the watershed with both the land use and treatment classes, and the antecedent moisture conditions.

The variable S is related to the CN by the following relationship:

$$S = \frac{1000 - 10\ CN}{CN} \qquad \text{Eq. I-6}$$

The SCS technique is a useful and reliable method of representing the infiltration characteristics of a watershed. Once the CN is obtained, Figure I-5 can be used to determine the rainfall excess in inches. The first step in arriving at the curve number is to determine the SCS soils group.

The reader is encouraged to review related references for details on the SCS method (Refs. 10, 14, and 18).

Determination of the Soil Group. The major soil groups are defined for the estimated watershed soil conditions. The groups, as defined by the SCS, are:

Group A. (Low runoff potential.) Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well- to excessively drained sands or gravels. These soils have a high rate of water transmission. Soils of this nature are very infrequent in the Stillwater area.

Group B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well-drained to well-drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

Group C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

Group D. (High runoff potential.) Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

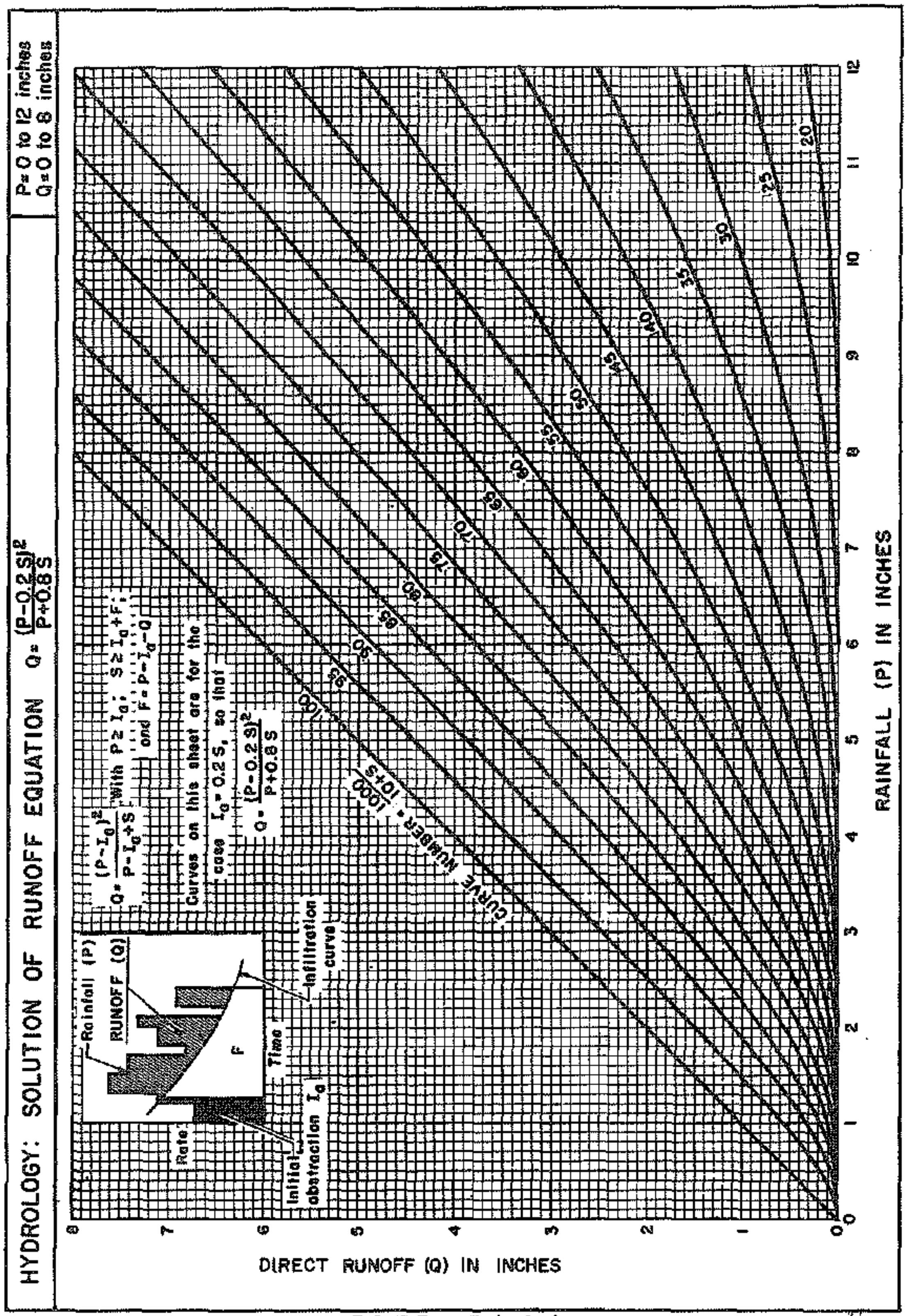


FIGURE I-5 (1of 2)

HYDROLOGY: SOLUTION OF THE SCS RUNOFF EQUATION

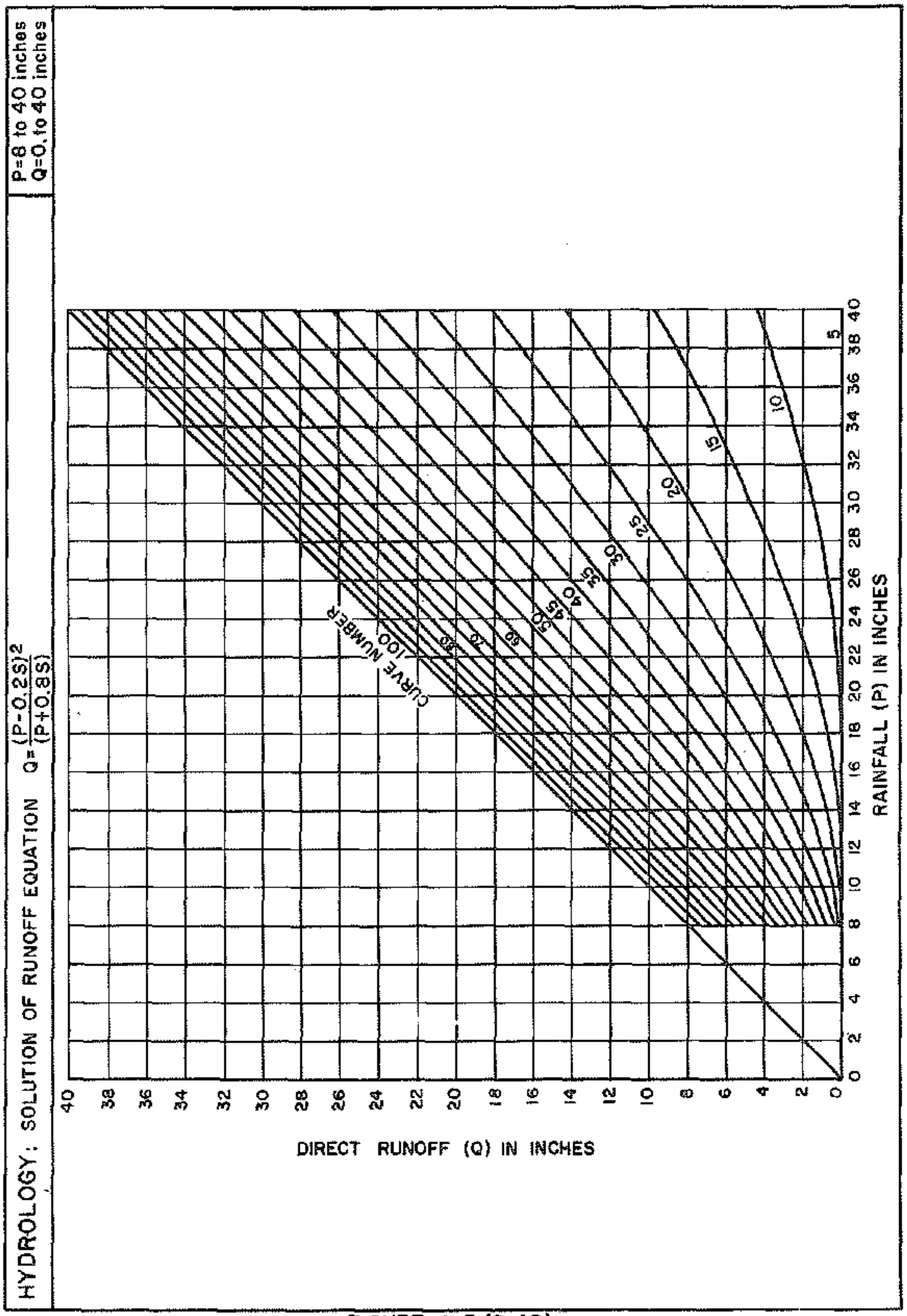


FIGURE 1-5 (2of2)

HYDROLOGY: SOLUTION OF THE SCS RUNOFF EQUATION

The SCS has published the Hydrologic Soil Group for the majority of the soil types found within the United States (Ref. 10).

Figure I-6 is a map that has been derived from local SCS soils maps (Ref. 19) and depicts the generalized location of the SCS type soil groups. This figure should be used with other available data from soils testing and onsite inspections. The soil groups shown should usually prove adequate for hydrologic analysis for conceptual planning; however, more detailed evaluation would be appropriate for hydrologic analysis of final design projects and master planning. Figure I-6 does not depict small or isolated areas of different groups of soils. Therefore, when undertaking studies of small basins, it is recommended that the engineer review the specific soil types in the field and as contained in References 10 and 19.

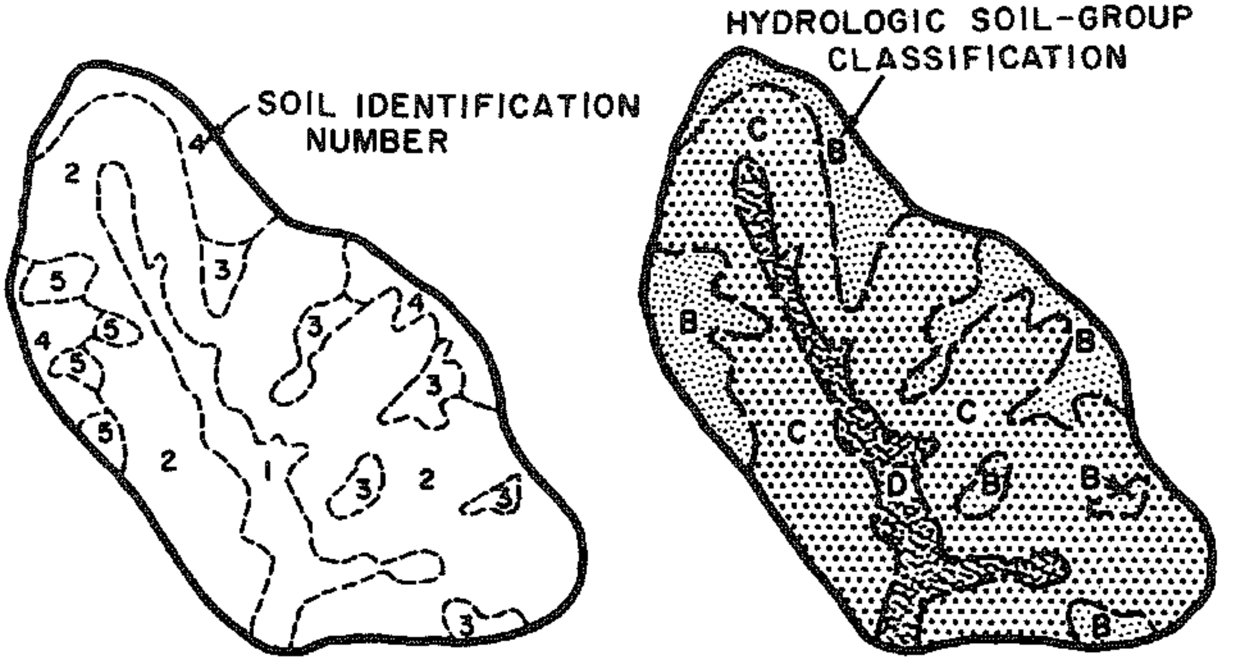
When determining urban CN's, consideration should be given to whether heavy equipment compacted the soil significantly more than natural conditions, whether much of the pervious area is barren with little sod established, and whether grading has mixed the surface and subsurface soils causing a completely different hydrologic condition. Any one of the above could cause a soil normally in hydrologic group A or B to be classified in group B or C. In many areas, lawns are heavily irrigated which may significantly increase the moisture content in the soil over that under natural rainfall conditions (Ref. 14).

SCS Guidelines for Determining Extent and Number of Groups. Precise measurement of soil-group areas, such as by planimetering soil areas on maps or weighing map cuttings, is seldom necessary for hydrologic purposes. The maximum detail need not go beyond that illustrated in Figure I-7:

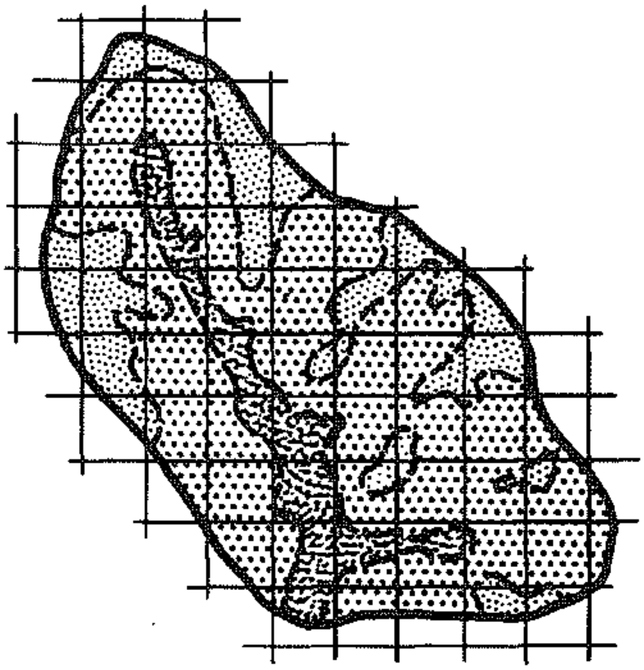
(a) The individual soils in a hydrologic unit are shown on a sketch map;

(b) The soils are classified into groups;

(c) A grid, or "dot counter" is placed over the map and the number of grid intersections falling on each group is counted and tabulated; and



(a) DETAILED SOILS MAP (b) HYDROLOGIC SOIL GROUP MAP



(c) GRID ON SOIL GROUP MAP

SOIL GROUP	NUMBER GRID INTERSECTIONS	PERCENT
B	12	23*
C	32	63
D	7	14
TOTAL	51	100

* PERCENT FOR B:

$$(100)\frac{12}{51} = \underline{\underline{23}}$$

(d) COMPUTATIONS

FIGURE I-7
STEPS TO DETERMINE PERCENTAGES OF SOIL GROUPS
(REF. 22)

(d) The tabulation and a typical computation of a group percentage is shown. In practice, simplified versions of this procedure are generally used.

Often one or two soil groups are predominant in a watershed, with others covering only a small part. Whether the small groups should be combined with those that are predominant depends on their classifications. For example, a hydrologic unit with 90 percent of its soils in the A group and 10 percent in D will have most of its storm runoff coming from the D soils. Putting all soils into the A group will cause a serious underestimation of runoff. If the groups are more nearly alike (A and B, B and C, or C and D), the under- or over-estimation may not be as serious, but a test may be necessary to show this. Rather than test each case, follow the rule that two groups are combined only if one of them covers less than about 3 percent of the hydrologic unit. Impervious surfaces should always be handled separately because they produce runoff even if there are no D soils (Ref. 10).

Determination of the Runoff Curve Number (CN). Once the soil group is known, the runoff curve number is determined by consideration of the surface conditions, vegetation cover, and other cover factors. References 10, 14 and 18 present detailed information for selection of the runoff curve number, as well as guidance toward determining a curve number for areas having mixtures of different soil and cover conditions.

Table I-9 lists CN's for agricultural, suburban, and urban land use classification. The suburban and urban CN's are based on typical land use relationships that exist in some areas. They should only be used when it has been determined that the area under study meets the criteria for which these CN's were developed (noted in table).

There will be areas to which the values in Table I-9 do not apply. The percentage of impervious area for the various types of residential areas or the land use condition for the pervious portions may vary from the conditions assumed in Table I-9. A curve for each pervious CN can be developed to determine the composite CN for any density of impervious area. Figure I-8 has been developed assuming a CN of 98 for the impervious area. The curves in Figure I-8 can help in estimating the increase in runoff as more and more land within a given area is covered with impervious material.

TABLE I-9
RUNOFF CURVE NUMBERS FOR SELECTED AGRICULTURAL, SUBURBAN, AND URBAN LAND USES
Antecedent moisture condition II, and $I_a = 0.2S$, (Ref. 14)

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

[1] For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.

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

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

[4] The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers

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

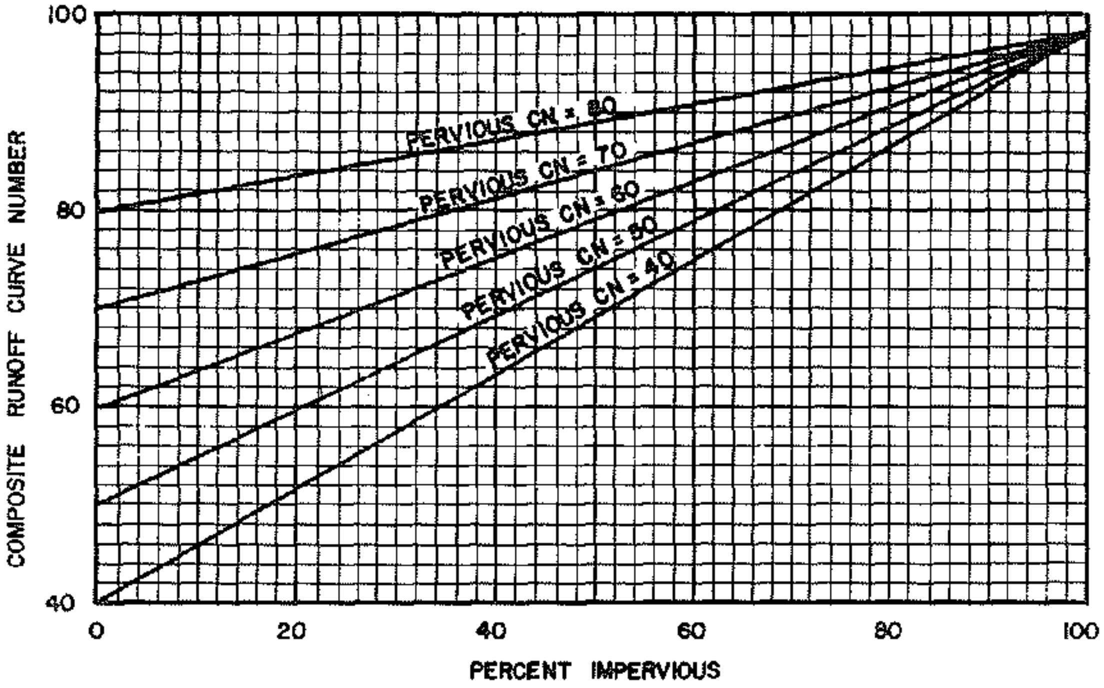


FIGURE I - 8

PERCENTAGE OF IMPERVIOUS AREAS
vs.
COMPOSITE CN's FOR GIVEN PERVIOUS AREA CN's
(REF. 30)

Curve Number Modification for Antecedent Moisture and Special Conditions. Retention parameters can be modified for various situations and to allow for antecedent conditions. A key example of the need for modificaion is when major storage facilities are planned that would be adversely affected by wet antecedent conditions.

The SCS runoff equation states that 20 percent of the potential maximum retention S is the initial abstraction I_a, which is the interception, infiltration, and surface storage occurring before runoff begins. The remaining 80 percent is mainly the infiltration occurring after runoff begins. This later infiltration is controlled by the rate of infiltration at the soil surface, or by the rate of transmission in the soil profile, or by the water storage capacity of the profile, whichever is the limiting factor. A succession of storms, such as one a day for a week, reduces the magnitude of S each day because the limiting factor does not have the opportunity to completely recover its rate or capacity through weathering, evapotranspiration, or drainage; but there is enough recovery, depending on the soil-cover complex, to limit the reduction. During such a storm period, the magnitude of S remains virtually the same after the second or third day even if the rains are large so that there is, from a practical viewpoint, a lower limit to S for a given soil-cover complex. Similarly, there is a practical upper limit to S, again depending on the soil-cover complex, beyond which the recovery cannot take S unless the complex is altered.

In the SCS method, the change in S (actually in CN) is based on an antecedent moisture condition (AMC) determined by the total rainfall in the 5-day period preceding a storm. Three levels of AMC are used: AMC-1 is the lower limit of moisture or the upper limit of S, AMC-II is the average for which the CN of Table I-10 apply, and AMC-III is the upper limit of moisture or the lower limit of S. The CN for high and low moisture levels were empirically related to the CN of Table I-10, the results of S and I_a for the CN in Column 1. Comparisons of computed and actual runoffs show that for most problems the extreme AMC can be ignored and the average of CN of Table I-10 applied (Ref. 10).

TABLE I-10

CURVE NUMBERS (CN) AND CONSTANTS FOR THE CASE $I_a = 0.2$ S (Ref. 10)

1	2	3	4	5
CN for Condition II	CN for Conditions		S Values*	Curve Starts Where P =
	I	III	(inches)	(inches)
100	100	100	0	0
99	97	100	0.101	0.02
98	94	99	.204	.04
97	91	99	.309	.06
96	89	99	.417	.08
95	87	98	.526	.11
94	85	98	.638	.13
93	83	98	.753	.15
92	81	97	.870	.17
91	80	97	.989	.20
90	78	96	1.11	.22
89	76	96	1.24	.25
88	75	95	1.36	.27
87	73	95	1.49	.30
86	72	94	1.63	.33
85	70	94	1.76	.35
84	68	93	1.90	.38
83	67	93	2.05	.41
82	66	92	2.20	.44
81	64	92	2.34	.47
80	63	91	2.50	.50
79	62	91	2.66	.53
78	60	90	2.82	.56
77	59	89	2.99	.60
76	58	89	3.16	.63
75	57	88	3.33	.67
74	55	88	3.51	.70
73	54	87	3.70	.74
72	53	86	3.89	.78
71	52	86	4.08	.82
70	51	85	4.28	.86
69	50	84	4.49	.90
68	48	84	4.70	.94
67	47	83	4.92	.98
66	46	82	5.15	1.03
65	45	82	5.38	1.08
64	44	81	5.62	1.12
63	43	80	5.87	1.17
62	42	79	6.13	1.23
61	41	78	6.39	1.28
60	40	78	6.67	1.33
59	39	77	6.95	1.39
58	38	76	7.24	1.45
57	37	75	7.54	1.51
56	36	75	7.86	1.57
55	35	74	8.18	1.64
54	34	73	8.52	1.70
53	33	72	8.87	1.77
52	32	71	9.23	1.85
51	31	70	9.61	1.92
50	31	70	10.0	2.00
49	30	69	10.4	2.08
48	29	68	10.8	2.16
47	28	67	11.3	2.26
46	27	66	11.7	2.34
45	26	65	12.2	2.44
44	25	64	12.7	2.54
43	25	63	13.2	2.64
42	24	62	13.8	2.76
41	23	61	14.4	2.88
40	22	60	15.0	3.00
39	21	59	15.6	3.12
38	21	58	16.3	3.26
37	20	57	17.0	3.40
36	19	56	17.8	3.56
35	18	55	18.6	3.72
34	18	54	19.4	3.88
33	17	53	20.3	4.06
32	16	52	21.2	4.24
31	16	51	22.2	4.44
30	15	50	23.3	4.66
25	12	43	30.0	6.00
20	9	37	40.0	8.00
15	6	30	56.7	11.34
10	4	22	90.0	18.00
5	2	13	190.0	38.00
0	0	0	infinity	infinity

*For CN in Column 1.

Rainfall Excess Examples

Examples of summary calculations using guideline values and by the SCS method are presented following. Both examples are areas with single-family (1/4-acre lot) residential development with a nominal 40 percent impervious percentage and slow infiltration rates (SCS Group C). The example design rainfall storm in the Rainfall Section is to be used.

Example Using Guideline values. Table I-11 presents the rainfall excess calculation procedure with example data.

Column 1 For the design location select a rainfall time interval according to the guidelines in the Rainfall Section or one appropriate for the runoff model being used.

Column 2 Tabulate the design storm in incremental values according to the Rainfall Section.

Column 3 Tabulate increments of infiltration for each time period for the pervious area. If the assumed rate is 1/2-inch per hour, then use 0.083-inch for each 10-minute interval in the example.

Column 4 The total pervious detention and depression storage is determined from Table I-8 and shown as a total at the bottom of Column 4. For each time period, the detention and depression storage in Column 4 is found by subtracting infiltration, Column 3, from precipitation, Column 2. If the result is negative, there is no excess for this period, and Column 4 and Column 5 are zero. If the result is positive, the amount is entered in Column 4 as the detention and depression storage for the time period. When the cumulative amount in Column 4 equals the total shown at the bottom of the column, the detention storage is fully used and all remaining values are zero. Note that the last value of detention storage will usually be less than Column 2 minus Column 3 and will be determined by the difference between the total amount of Column 4 and the amount accumulated through the previous time period.

Column 5 Rainfall excess (effective precipitation) for the pervious area is Column 5 minus Columns 3 and 4, positive values only.

TABLE I-11

DETERMINATION OF RAINFALL EXCESS USING GUIDELINE VALUES (EXAMPLE)

(All values in inches)

		Pervious Area 60%				Impervious Area 40%				
Time (Min.) (1)	Design Rainfall (2)	Maximum Infiltration (3)	Detention & Depression Storage (4)	Rainfall Excess (5)	60% of Pervious Area Rainfall Excess. (6)	Detention and Depression (7)	General Loss (8)	Rainfall Excess (9)	40% of Impervious Rainfall Excess (10)	Total Rainfall Excess (11)
0	0	0	0	0	0				0	0
10	0.07	0.083	0	0	0	0.07			0	0
20	.07	.083	0	0	0	.03	0.04		0	0
30	.09	.083	0.007	0	0		.06	0.03	0.012	0.012
40	.16	.083	.077	0	0			.16	.064	.064
50	.17	.083	.087	0	0			.17	.068	.068
60	.20	.083	.117	0	0			.20	.080	.080
70	.30	.083	.012	0.205	0.123			.30	.120	.243
80	.39	.083	0	.307	.184			.39	.156	.340
90	.75	.083	0	.667	.400			.75	.300	.70
100	1.45	.083	0	1.367	.820			1.45	.580	1.40
110	.61	.083	0	.527	.316			.61	.244	.560
120	.30	.083	0	.217	.130			.30	.120	.250
130	.20	.083	0	.117	.070			.20	.080	.150
140	.18	.083	0	.097	.058			.18	.072	.130
150	.17	.083	0	.087	.052			.17	.068	.120
160	.09	.083	0	.007	.004			.09	.036	.04
170	.09	.083	0	.007	.004			.09	.036	.04
180	.07	.083	0	0	0			.07	.028	.028
	5.36	1.494	0.30		2.16	0.10	0.10		2.064	4.225

Column 6 Column 5 times the (decimal) percent of the pervious area gives the area-weighted depth of water that will runoff in each time increment for the pervious area.

Column 7 Enter the total assumed impervious detention and depression storage determined from Table I-8 at the bottom of Column 7. The impervious detention and depression storage in Column 7 is then either the amount of precipitation in Column 2 or the amount avialable as determined by deducting the total accumulated amount from the total assumed value shown at the bottom of Column 7. When the total assumed amount is fully used, all remaining values are zero.

Column 8 After all of the impervious storage has been filled, the general impervious area loss is assigned in a similar fashion to Column 8.

Column 9 Rainfall excess for the impervious area is Column 2 less Columns 7 and 8.

Column 10 Column 9 times the (decimal) percent of impervious area gives the area-weighted depth of water that will runoff in each time increment for the pervious area.

Column 11 Add Column 10 and Column 6 to obtain the total rainfall excess. This will be applied to the unit hydrograph method described later in this chapter or other runoff methods to obtain the design storm runoff hydrograph.

Example Using the SCS Method. Table I-12 reveals the final summary computations for the SCS method. Though this final summary calculation procedure appears simpler, the derivation of the selected curve number in actual practice will require more effort.

Column 1 According to the guidelines in the Rainfall Section or for the appropriate runoff model, a time increment is selected.

Column 2 The incremental design storm as computed according to the Rainfall Section is entered here.

Column 3 Cumulative rainfall values are computed by summing all incremental values in Column 2 up to a given time. The last value should check with the same in Column 2.

Column 4 Based on soils Group C and the residential description given, Table I-9 can be used to select a curve number of 83. However, it is noted that the subdivision grading will be extensive and irrigated lawns are planned. Thus, an adjustment to Group D is made and curve number 87 is finally selected. Values can be computed or taken from Figure I-4.

Column 5 Incremental effective precipitation values suitable for input to runoff methods are calculated by subtracting the previous cumulative value from the cumulative value for the time of interest.

TABLE I-12

DETERMINATION OF RAINFALL EXCESS USING THE SCS METHOD (EXAMPLE)

(All values in inches)

Time (minutes) (1)	Incremental Design Rainfall (2)	Cumulative Design Rainfall (3)	CN 87 Cumulative Rainfall Excess (4)	Incremental Rainfall Excess (5)
0	0	0	0	0
10	0.07	0.07	0	0
20	.07	.14	0	0
30	.09	.23	0	0
40	.16	.39	0.01	0.01
50	.17	.56	.03	.02
60	.20	.76	.11	.08
70	.30	1.06	.25	.14
80	.39	1.45	.51	.26
90	.75	2.20	1.07	.56
100	1.45	3.65	2.33	1.26
110	.61	4.26	2.88	.55
120	.30	4.56	3.16	.28
130	.20	4.76	3.35	.19
140	.18	4.94	3.52	.17
150	.17	5.11	3.67	.15
160	.09	5.20	3.76	.09
170	.09	5.29	3.84	.08
180	.07	5.36	3.91	.07
	5.36			3.91

SYNTHETIC UNIT HYDROGRAPH PROCEDURE (SUHP)

For basins that are larger than about 100 acres, and for some complex basins that are less, it is recommended that the design storm runoff be analyzed by deriving synthetic unit hydrographs or similar methods, such as those described in SCS references (Refs. 10, 14, and 40). The unit hydrograph procedure provides a realistic volume distribution with respect to time. The unit hydrograph principle was originally developed by Sherman in 1932 (Ref. 20). The synthetic unit hydrograph, which is used for analysis when there is no rainfall runoff data for the basin under study, was developed by Snyder in 1938 (Ref. 21). Since that time, there have been many developments and modifications to these methods which basically derive varying empirical relations but have not significantly modified the basic concepts (Refs. 9, 10, 12, 14, 22, 23, 24, 25, 26, 27, 28, 40, and 41).

Definitions

A unit hydrograph is defined as the response of one unit (usually an inch) of direct runoff from a drainage area. A unit storm is a rainfall of such duration that the period of surface runoff is not appreciably less for any rain of shorter duration. This unit hydrograph represents the integrated effects of factors such as tributary area, shape, street pattern, channel capacities, and street and land slopes (Refs. 12, 23, 24, and 25).

The runoff response (design hydrograph) to a design storm is determined by multiplying the ordinates of the unit hydrograph by incremental rainfall excess depths of the design storm and summing each of the resulting incremental runoff hydrographs sequentially with respect to time.

A unit hydrograph for a watershed may be derived by analyzing actual rainfall and runoff data. Often, this type of analysis cannot be completed because of a lack of data, and it has limited value because it allows no direct prediction of the effects of urbanization. However, empirical relationships have been derived by analysis of hydrographs from many different kinds of watersheds. These relationships lead to synthetic unit hydrographs for basins without historical data and for various development conditions.

Basic Assumptions

The derivation and application of the unit hydrograph are based on the following assumptions:

1. The rainfall intensity is constant during the storm that produces the unit hydrograph.
2. The rainfall is uniformly distributed throughout the entire area of the drainage basin.
3. The base or time duration of the design runoff due to rainfall excess of unit duration is constant.
4. The ordinates of the design runoff with a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph.
5. The effects of all physical characteristics of a given drainage basin, including shape, slope, detention, infiltration, drainage pattern, channel storage, etc., are reflected in the shape of the unit hydrograph for that basin.

General Equations and Relationships

Snyder proposed two basic equations used in defining the limits of the synthetic unit hydrograph (Ref. 21). The first equation defines the lag time of the basin in terms of time to peak, t_p, which, for the SUHP Method, is defined as the time from the center of the unit storm duration to the peak of the unit hydrograph.

$$t_p = C_t \ (LL_{ca})^{0.3} \qquad \text{Eq. I-7}$$

where t_p = time-to-peak of the hydrograph from midpoint of unit rainfall in hours

L = length along stream from study point to upstream limits of the basin in miles

L_{ca} = distance from study point along stream to the centroid of the basin in miles

C_t = a coefficient reflecting time to peak.

The second equation defines the unit peak of the unit hydrograph.

$$q_p = \frac{640C_p}{t_p} \qquad \text{Eq. I-8}$$

where q_p = peak rate of runoff in cfs per square mile,

C_p = a coefficient related to peak rate of runoff.

Victor Mockus (Ref. 10) derived a dimensionless unit hydrograph which is presented in Table I-13. It can provide general shaping information and is useful in deriving other basic relationships helpful to the Synthetic Unit Hydrograph Procedure; however, it should not be used directly for representing urban areas.

TABLE I-13

RATIOS FOR DIMENSIONLESS UNIT HYDROGRAPH (REF. 10)

Time Ratios (t/T_p)	Discharge Ratios (q/q_p)	Time Ratios (t/T_p)	Discharge Ratios (q/q_p)
0	0.000	1.6	0.560
0.1	.030	1.7	.460
.2	.100	1.8	.390
.3	.190	1.9	.330
.4	.310	2.0	.280
.5	.470	2.2	.207
.6	.660	2.4	.147
.7	.820	2.6	.107
.8	.930	2.8	.077
.9	.990	3.0	.055
1.0	1.000	3.2	.040
1.1	.990	3.4	.029
1.2	.930	3.6	.021
1.3	.860	3.8	.015
1.4	.780	4.0	.011
1.5	.680	4.5	.005

The dimensionless curvilinear unit hydrograph in Table I-13 has 37.5 percent of the total volume in the rising side, which is represented by one unit of time and one unit of discharge. This dimensionless unit hydrograph also can be represented by an equivalent triangular hydrograph having the same units of time and discharge, thus having the same percent of volume in the rising side of the triangle (Figure I-9).

The base of the approximate triangle in Figure I-9 can be derived with the following results:

$$T_b = \frac{1.00}{0.375} = 2.67 \text{ units of time or } 2.67\ T \qquad \text{Eq. I-9}$$

$$T_r = T_b - T_p = 1.67 \text{ units of time or } 1.67\ T_p \qquad \text{Eq. I-10}$$

These relationships lead to the following equation:

$$q_p = \frac{484}{T_p} \qquad \text{Eq. I-11}$$

where $$T_p = \frac{t_u}{2} + t_p \qquad \text{Eq. I-12}$$

However, SCS notes that the numerator of 484 is known to vary from 600 to 300, similar to variances noted by Snyder (Refs. 10 and 21). Further, a relationship is derived to define the recommended unit of rainfall duration for the unit hydrograph as:

$$t_u = 0.133\ t_c \text{ or } \frac{t_c}{7.5} \qquad \text{Eq. I-13}$$

where t_c = time of concentration as illustrated in Figure I-9

and that the average relationship between the lag time t_p and the time of concentration is:

$$t_p = 0.6\ t_c \qquad \text{Eq. I-14}$$

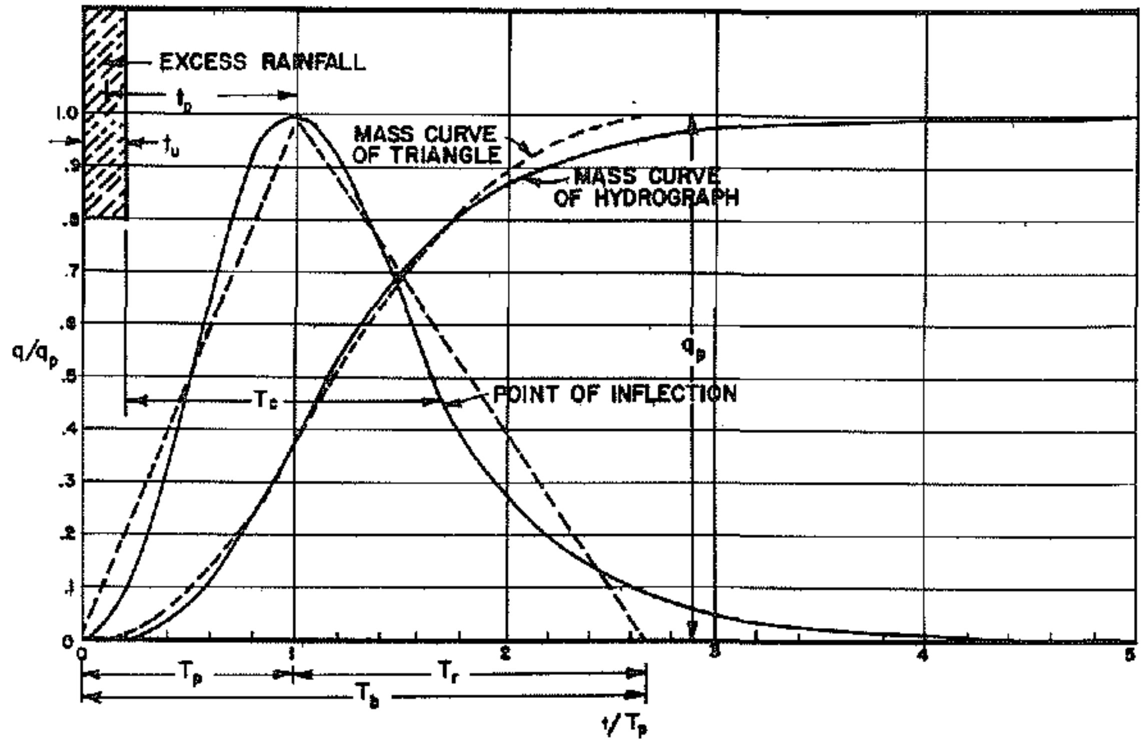


FIGURE I-9

SCS DIMENSIONLESS CURVILINEAR UNIT HYDROGRAPH
AND
EQUIVALENT TRIANGULAR HYDROGRAPH
(REF. 22)

General Guidelines for Determining Parameters

The previous discussion presents basic relationships of synthetic unit hydrograph theory. Final determination of a synthetic unit hydrograph for a given watershed required additional information. For example, values of C_T in Equation I-7 are needed to determine the lag time. The basic approach is to determine these types of coefficients or equations by analysis of rainfall runoff data from other similar watershed(s). The HEC-1 computer program (Ref. 29) provides an efficient means of analyzing such data.

Fortunately, many other hydrologists have already performed such analyses of local, regional and national data and have arrived at coefficients and variations of and additions to the basic unit hydrograph equations. Any of these relationships, which are presented below have potential application to the basin under study and thus provide a savings in study effort. But the hydrologist must determine which equation(s) are applicable based upon the similiarity between the study basin and the watersheds for which the equations were derived.

The relationships available can generally be put into three categories:

1. Estimation of lag time, t_p.
2. Estimation of a synthetic unit hydrograph discharge peak, q_p.
3. Unit hydrograph shape factors.

Estimation of Lag Time, t_p

The various unit hydrograph methods are sensitive to lag time or other peak response time factors. The unit hydrograph peak discharge and shaping factors are usually a function of the lag time and other parameters; and, most methods are usually derived with algorithms that are a direct or indirect function of the lag time. Thus the determinatin of the lag time is critical to the reliability of the results.

Rural Areas. The Tulsa District Corps of Engineers (Ref. 30) has derived a relationship for t_p based upon data for natural watersheds in the central and northeastern Oklahoma area which is:

$$t_p = \left(\frac{L\ L_{ca}}{\sqrt{S}}\right)^{0.39} \quad \text{Natural watershed relationship Eq. I-15}$$

where S = watershed slope in feet/mile

L = stream length in miles

L_{ca} = length along stream to centroid of basin in miles

This equation, illustrated in Figure I-10, is recommended for natural watersheds. It can be checked for unusual cases by estimating the time of concentration and multiplying by a factor of 0.60 as indicated earlier.

A report by Espey, Morgan, and Masch (Ref. 41) for watersheds in Texas, New Mexico and Oklahoma resulted in the following recommended equation.

$$T_R = 2.65\ L_f^{\ 0.12}\ S_f^{\ -0.52} \qquad \text{Eq. I-16}$$

where

T_R = time of rise in minutes which can be assumed to be equal to

$$T_p = \frac{t_u}{2} + t_p \qquad \text{Eq. I-17}$$

and

L_f = stream length in feet

S_f = slope in feet/feet

This equation is based on data from small watersheds ranging as follows: L(3,250 to 25,300 ft.), S(0.008 to 0.015 ft./ft.) and T_R(30 to 150 minutes).

Urbanized Areas. Several approaches will be presented here which should be used with judgment and in comparison to arrive at a recommended t_p.

The Tulsa Corps has derived parallel relationships for 50 and 100 percent urbanized basins as follows.

$$t_p = 0.92 \left(\frac{L\ L_{ca}}{\sqrt{S}}\right)^{0.39}$$

for 50 percent urbanized
Eq. I-18

$$t_p - 0.59 \left(\frac{L\ L_{ca}}{\sqrt{S}}\right)^{0.39}$$

for 100 percent urbanized
Eq. I-19

The Corps estimates that the 100 percent urbanized basin would have approximately 50 percent impervious area. The Corps reflects that these relationships are questionable in extremely large basins and in small basins less than 1/3 to 1/2 square miles, and that these are based on a limited data base. Thus, the information should be used carefully.

Interestingly, Eagleson presents synthetic unit hydrograph data of fully storm sewered basins (Ref. 23). When corrected for slope defined as feet/mile, this data can be plotted on Figure I-10 and a parallel relationship drawn to the Corps'. The plotted line has the relationship:

$$t_p = 0.32 \left(\frac{L\ L_{ca}}{\sqrt{S}}\right)^{0.39}$$

for fully sewered urbanized basins
Eq. I-20

However, in this equation the slope is the weighted slope of the storm sewers. Since the Tulsa Corps data was based on a mixture of basins that carried mainstream flows in open channels, either artificial or natural waterways, all the various relationships are deemed reasonable and usable for Stillwater. Also, use for small watersheds is reasonable.

However, another precaution is noted by the Corps. That is that the routing effects of the mainstreams and the associated valley storage should be evaluated. It may be necessary to derive hydrographs for individual subbasins and route the discharges through a main channel. Such streamflow routing is discussed later.

Espey (Ref. 41) proposed the following relationship in urban areas for the time from the beginning of effective rainfall to the peak.

$$T_R = 20.8\ U\ L_f^{\ 0.29}\ S_f^{\ -0.11}\ I^{-0.61} \qquad \text{Eq. I-21}$$

where U = 1.0 for natural conditions
0.8 for watersheds with some storm sewers and channelization
0.6 for watersheds with extensive urban development

and

L_f = stream length in feet
S_f = slope in feet/feet
I = percent impervious

This equation is based on data from watersheds ranging as follows: L(200 to 54,800 ft.), S(0.0064 to 0.0104 ft./ft.), I(2.7 to 100 percent) and T_R(30 to 720 minutes).

The USGS compared data from Wichita, Kansas against a relationship by Putnam and found satisfactory results (Ref. 31). Putnam notes that "the estimates are most reliable for smaller size floods at sites where the drainage area ranges between 0.3 and 150 square miles, where the $L/\sqrt{S}$ ratio ranges between 0.1 and 9.0 and where impervious cover of less than 30 percent is uniform and distributed over the basin." (Ref. 45)

$$t_p = 0.49\left(\frac{L}{\sqrt{S}}\right)^{0.5} I_w^{\ -0.57} \qquad \text{Eq. I-22}$$

where

L = length of the main watercourse in miles
S = slope in feet/mile, and
I_w = Impervious area/total area

Another method follows from the original Snyder Equations, except that

$$C_t = \frac{7.81}{(I_a)\ 0.78}\ r_2 = 0.95 \text{ (coefficient of determination)} \qquad \text{Eq. I-23}$$

where I_a = percent of watershed which is impervious
(for 90 percent impervious I_a = 90)

This equation was developed in 1975 as a revision to earlier information in the Denver Urban Drainage Criteria Manual (Ref. 26). Figure I-11 presents the equation. This relationship was developed by Colorado State University after studying new gaged rainfall-runoff relationships in the Denver area. Also, many other points were developed based on data from other areas as indicated on Figure I-11. However, it is noted that there is significant scatter for areas less than 10 percent impervious, thus, it is not advisable to use this curve for such areas.

Also, as another check the user is reminded that a reasonableness check can be made by calculating 60 percent of the time of concentration as calculated by evaluation of flow time through the basin.

Estimation of Synthetic Unit Hydrograph Peak, q_p

The Snyder Equation for q_p is a function of t_p and a coefficient C_p; however, empirical correlations to various parameters are quite poor.

The most reasonable and recommended approach found was to use a basic relationship provided by the Tulsa District Corps as shown in Figure I-12. The data discussed earlier by Eagleson (Ref. 23) was also plotted against this relationship and is found in agreement.

All of the data presented is found by determining the best fit of derived unit hydrographs (which are found by fitting effective rainfall and runoff flow data to the synthetic unit hydrograph equations). Thus, the good relationship shown is not suprising, and one can draw the conclusion that the accurate determination of the lag time discussed earlier is critical.

Reference 41 poses equations for smaller basins of:

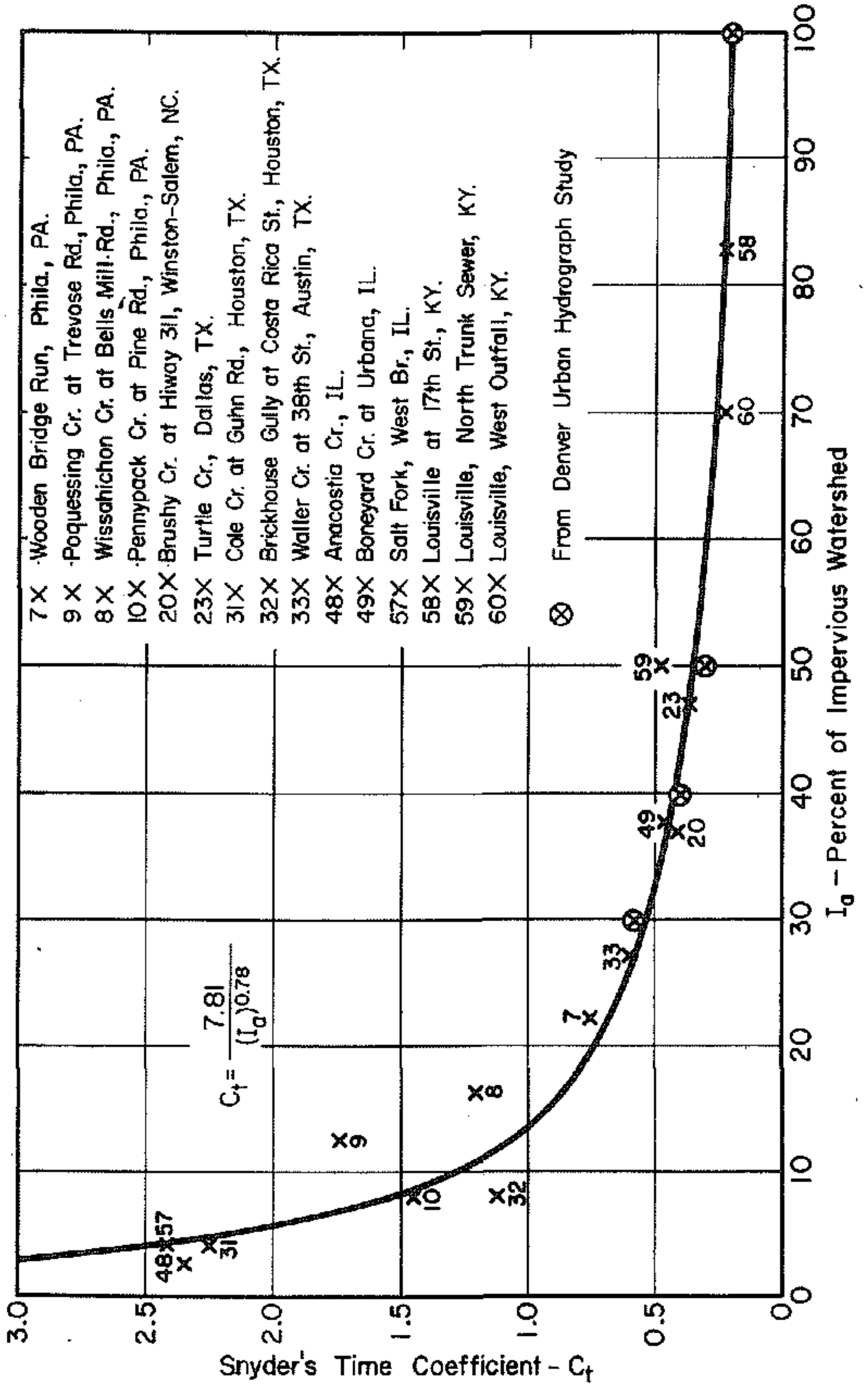


FIGURE 1-11
RELATIONSHIP BETWEEN C_t AND IMPERVIOUSNESS

$$q_p = 1.70 \times 10^3 A^{-0.12} T_R^{-0.30} \text{ Rural} \qquad \text{Eq. I-24}$$

and

$$q_p = 1.93 \times 10^4 A^{-0.09} T_R^{-0.94} \text{ Urban} \qquad \text{Eq. I-25}$$

where

T_R is the time from the beginning of effective rainfall to the peak runoff

A = are in square miles

The rural equation is based on watersheds with the following range: A (0.134 to 7.01 square miles) and T_R (30 to 150 minutes). The urban equation based upon A (0.0128 to 92 square miles) and T_R (30 to 720 minutes). The equations have acceptable correlations, but not as good as the previously discussed equations for T_R.

Synthetic Unit Hydrograph Shape

The shape of the unit hydrograph is a function of the physical characteristics of the drainage basin. The shape is developed from empirical relationships such as those discussed previously and as follows. The following equations may be used for urbanized areas to estimate the width of the unit hydrograph at 50 percent and 75 percent of the peak discharge (Ref. 26). These values are similar to those proposed by Eagleson (Ref. 23).

$$W @ 50\% Q_p = \frac{500}{q_p} \text{ (hours)} \qquad \text{Eq. I-26}$$

$$W @ 75\% Q_p = \frac{260}{q_p} \text{ (hours)} \qquad \text{Eq. I-27}$$

Corps' numerator values for natural watersheds are 470 and 280 respectively and indicate a more rounded peak (Ref. 22).

Reference 41 provides guidance equations for small watersheds previously discussed.

$$\begin{array}{lcc}
 & \underline{\text{Rural}} & \underline{\text{Urban}} \\
W \text{ @ } 50\% \ Q_p = & \dfrac{1230}{q_p^{1.13} A^{0.02}} , & \dfrac{690}{q_p^{1.04} A^{0.01}} \\
W \text{ @ } 75\% \ Q_p = & \dfrac{740}{q_p^{1.13} A^{0.07}} , & \dfrac{223}{q_p^{0.94} A^{0.02}} \\
T_b \text{ (base) } = & \dfrac{123 A^{0.11}}{q_p^{0.53}} , & \dfrac{7400}{q_p^{1.19} A^{0.02}}
\end{array}$$

Eq. I-28

Eq. I-29

Eq. I-30

Once q_p is determined, Q_p (the maximum unit hydrograph peak for the basin) can be computed by:

$$Q_p = q_p A \qquad \text{Eq. I-31}$$

where A is the area of the basin in square miles.

The time from the beginning of rainfall to the peak of the unit hydrograph is determined by:

$$T_p = t_p + 0.5 \ t_u \quad \text{(check units)} \qquad \text{Eq. I-32}$$

Once Q_p is located, the unit hydrograph can be sketched with the aid of the approximate widths at $Q_{50\%}$ and $Q_{75\%}$. Sketching of the hydrograph can be assisted by comparison with the shape of the dimensionless unit hydrograph shown in Figure I-8. However, the area under the hydrograph should always be planimetered to determine the volume of runoff.

This volume should equal the volume of the unit runoff, i.e., 1-inch depth from the entire basin, area A. If the two volumes are within 5 percent, then the sketched unit hydrograph is acceptable. The final step is to define the unit hydrograph in tabular form showing time versus rate of flow. If Q_p does not fall on a chosen time interval so that the tabulation does not represent the graph, then the graph may be shifted so that the table will more truly represent the graph.

The SCS, In Reference 40, presents the methodology for using the SCS triangular synthetic hydrograph as somewhat of a simplification to that discussed above.

Design Storm Runoff. After the unit hydrograph has been calculated and the rainfall excess from the design storm determined, the design storm hydrograph can be calculated. The time units of the unit hydrograph abscissa should be the same as the time units of the rainfall excess which for convenience should all be equal to the unit storm duration.

By multiplying the incremental rainfall amounts and the unit hydrograph values, a response can be obtained for each rainfall increment. Approximate lagging and addition of the responses from each rainfall increment result in the design storm hydrograph.

SUHP Example

Determine the 100-year hydrograph for a developed basin of generally single family homes (1/4-acre lots) in an area with soils Group C before development:

Area = 2.0 square miles
L = 4.0 miles
L_{ca} = 2.5 miles
Pervious area = 60 percent
Impervious area = 40 percent
Slope = 1 percent = 53 feet/mile

Step 1. Determine t_c

a. Using the Corps' Equation I-19 for 100 percent urbanization

$$t_p = 0.59 \left(\frac{L\ L_{ca}}{\sqrt{S}}\right)^{0.39} = 0.59\left(\frac{4 \times 2.5}{\sqrt{53}}\right)^{0.39} = 0.67 \text{ hr.}$$

b. Using Espey's Equation I-21 for urban areas

$$T_R = 20.8\ U\ L_f^{0.29}\ S_f^{-0.11}\ I^{-0.61}$$

$$= \frac{20.8(0.8)(4 \times 5280)^{0.29}}{(.01)^{0.11}\ (40)^{0.61}} = 52.2 \text{ minutes} = 0.87 \text{ hrs.}$$

but from Equation I-17

$$t_p = T_R - \frac{t_u}{2} = 0.87 = 0.75 \quad \text{(Note: see } t_u \text{ in step 4)}$$

c. Using the Equation I-22 by Putnam

$$t_p = 0.49 \left(\frac{L}{\sqrt{S}}\right)^{0.5} I_w^{-0.57} = \left(0.49 \frac{4}{\sqrt{53}}\right)^{0.5} 0.4^{-0.57} = 0.61 \text{ hr}$$

d. Using the suggested Equation I-23 for C_T

$$C_T = \frac{7.81}{(I)^{0.78}} = \frac{7.81}{(40)^{0.78}} = 0.44$$

and then the Snyder Equation I-7

$$t_p = C_T \ (L \ L_{ca})^{0.3} = 0.44 \ (4 \times 2.5)^{0.3} = 0.80 \text{ hr}$$

e. Recommend a value for t_p

The 0.67 values given by the Corps Equation I-19 seems reaonsable as it fits within the scatter of data points on Figure I-10 and is based on Oklahoma watersheds.

The 0.75 value from the Epsey Equation I-21 is reasonable, but note that the 1 percent slope of the basin being analyzed fits on the upper range of the data from which the equation was divided.

The 0.61 value given by Putman's Equation I-22 is reasonable, but because of the 40 percent impervious area it is out of the data range from which the equation was derived.

The 0.80 value from Equation I-23 and Synder's Equation I-7 is reasonable, but based on a wide range of watersheds.

The 0.67 and 0.75 values are judged to be the most reasonable and 0.67 is recommended as being conservative.

Step 2. Determine Q_p

a. From Figure I-12, q_p is 600 cfs/sq.mile/inch

b. $Q_p = q_p A = 600 \times 2 = 1{,}200$ cfs/inch

c. Equation I-25 from Reference 41 results in $q_p = 480$ cfs/sq.mile/inch, thus the result above is reasonable.

Step 3. Determine the width of the unit hydrograph at 50 percent and 75 percent and the 75 percent of the Q_p

$$W \text{ @ } 50\% \; Q_p = \frac{500}{Q_p/A} = \frac{500}{600} = 0.83 \text{ hrs.} \qquad \text{Eq. I-26}$$

$$W \text{ @ } 75\% \; Q_p = \frac{260}{Q_p/A} = \frac{260}{600} = 0.43 \text{ hrs.} \qquad \text{Eq. I-27}$$

Interestingly, Reference 41 equations I-28, 29 and 30 give values 0.88 hours and 0.54 hours for W @ 50% Q_p and W @ 75% Q_p; and for T_b gives 3.6 hours.

Step 4. Determine the unit time increment t_u from equations I-13 and I-14.

$$t_u = \frac{t_c}{7.5} = \frac{t_p/0.6}{7.5} = \frac{0.67/0.6}{7.5} = 0.15 \text{ hr.} = 8.9 \text{ min.}$$

Use t_u = 10 minutes

Step 5. Determine the time to peak from the beginning of rainfall using Equation I-12.

$$T_p = t_p + \frac{t_u}{2} = 0.67 \text{ hr.} + \frac{0.167}{2} = 0.75 \text{ hr. (rounded)}$$

Step 6. Using the results of Steps 5, 6, and 7, sketch a unit hydrograph. See Figure I-13.

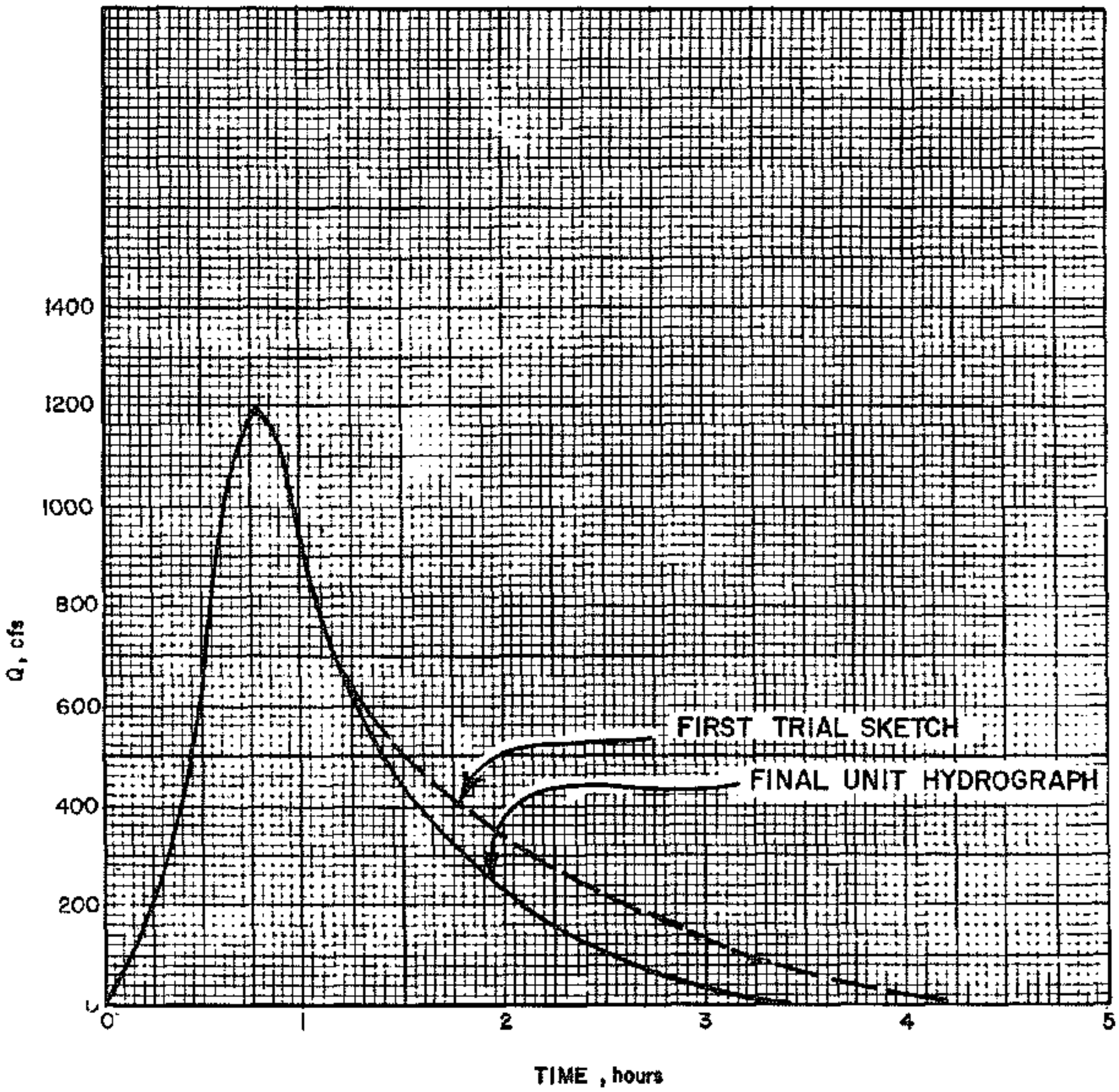


FIGURE I - 13

UNIT HYDROGRAPH EXAMPLE

Step 7. The volume of the unit hydrograph should be 107 acre-feet, i.e., 1-inch runoff from 2 square miles. A revision was made to lower the volume from the first estimate of 130 acre-feet to 109 acre-feet.

Step 8. Present the unit hydrograph in graphic form as shown on Figure I-13.

Step 9. Obtain the design excess precipitation values in unit duration increments. See previous rainfall and rainfall excess example per SCS method. Remember the duration storm should be greater than $2 \times t_c = 2 \times t_p/0.06 = 2.24$ (i.e. use 3-hour storm).

Step 10. Set up Table I-14.

Step 11. Multiply the precipitation value at the top of each column by each of the unit hydrograph ordinates and put the product in the corresponding time. Note that the first rainfall excess increment occurs during the period from 30 to 40 minutes, thus, the product hydrograph begins in time from 30 to 40 minutes. As the next increment of rainfall excess is 10 minutes later, the product hydrograph is also 10 minutes later.

Step 12. The final design storm hydrograph is checked and found to have a volume of 400 acre-feet which compares favorably with the volume of 3.91 inches x 2 square miles which equals 417 acre feet.

Routing

As indicated previously, streamflow routing of runoff can substantially modify discharge hydrographs.

A good procedure to use when performing hydrologic calculations is to subdivide a drainage basin of interest into a number of small subbasins of 1 to 3 square-miles that have similar character. Hydrographs should be calculated for each subbasin and the resulting hydrographs should then be routed down the main stream channels. There are numerous routing methods which are best referred to directly (Refs. 9, 10, 12, and 29).

COMPUTER MODELING APPROACHES

During the last decade, a number of hydrologic and/or water quality computer models have been developed and are in practical use in engineering. The

TABLE I-14 DETERMINATION OF STORM HYDROGRAPH

Time (min.)	Unit Hydrograph (cfs)	Rainfall Excess in Inches .01	.02	.07	.14	.26	.56	1.26	.55	.28	.19	.17	.15	.09	.08	.07	Design Storm Hydrograph (cfs)
0	0																0
10	150																0
20	320																0
30	700	0															0
40	1130	2	0														2
50	1160	3	3	0													6
60	900	7	6	12	0												25
70	680	11	14	26	22												73
80	540	12	22	56	44	39	0										173
90	420	9	23	90	98	83	84	0									387
100	330	7	18	93	158	182	179	189	0								826
110	280	5	13	72	162	294	392	403	28	0							1369
120	220	4	11	54	126	302	633	882	176	42	0						2230
130	170	3	8	43	94	234	650	1424	385	90	29	0					2960
140	140	3	7	34	76	177	504	1460	622	196	61	26	0				3166
150	100	2	6	26	58	140	381	1134	638	316	133	54	23	0			2911
160	80	2	4	22	46	109	302	857	495	325	215	119	48	14	0		2558
170	60	1	3	18	40	86	235	680	374	252	220	192	105	29	12	0	2247
180	40	1	3	14	30	73	185	529	297	190	171	197	170	63	26	11	1960
190	20	1	2	11	24	57	157	416	231	151	129	153	174	102	56	22	1686
200	0	1	2	8	20	44	123	353	182	118	103	116	135	104	90	49	1448
210		0	1	6	14	36	95	277	154	92	80	92	102	81	93	79	1202
220		0	1	5	12	26	78	218	121	78	63	71	81	61	72	81	968
230			0	3	8	21	56	176	94	62	53	56	63	49	54	63	758
240			0	1	6	16	45	126	77	48	42	48	50	38	43	47	587
250				0	3	10	34	101	55	39	32	37	42	30	34	38	455
260					0	5	22	76	44	28	27	29	33	25	26	29	344
270						0	11	50	33	22	19	24	26	20	22	23	250
280							0	25	22	17	15	17	21	15	18	20	170
290								0	11	11	11	14	15	13	14	15	104
300									0	6	8	10	12	9	11	12	68
310										0	4	7	9	7	8	10	45
320											0	3	6	5	6	7	27
330												0	3	4	5	6	18
340													0	2	3	4	[illegible]
														0	2	3	
															0	1	
																0	0

advanced computer models allow the use of sophisticated algorithms to represent various phenomena and complicated flow routing. Without computer models, such algorithms could not be practically used. These computer models have made possible a better understanding and analysis of runoff from both urban developing and rural land. The end result of computer modeling in the case of drainage is an improved runoff system.

Various publications are available that describe and test these methods (Refs. 32, 33, 34, and 35). The publication Evaluation of Mathematical Models for the Simulation of Time-Varying Runoff and Water Quality in Storm Combined Sewerage Systems (Ref. 35) presents the results of a comprehensive review of most computer models available in North America and Europe. The authors of this paper also arrived at the following recommendations regarding routing applications.

"Various models stand out due to their completeness of hydrologic and hydraulic formulations, the ease of input data preparation, the efficiency of computational algorithms, and the adequacy of the program output. Other models, although deficient in some of these respects, merit consideration due to special features which are not included in the more comprehensive models, but may be required for specific applications.

"The following models are consequently recommended for routine applications:

1. Battelle Urban Wastewater Management Model for real-time control and/or design optimization considering hydraulic, water quality and cost constraints, provided the hydrologic and hydraulic model assumptions are adequate for particular applications (lumping of many small subcatchments into few large catchments, neglect of downstream flow control, backwater, flow reversal, surcharging, and pressure flow).
2. Corps of Engineers STORM Model for preliminary planning of required storage and treatment capacity for storm runoff from single major catchments, considering both the quantity and quality of the surface runoff and untreated overflows.
3. Dorsch Consult Hydrograph Volume Method for single-event flow analysis considering most important hydraulic phenomena (except flow reversal).

A Quantity-Quality Simulation Program for continuous wastewater flow and quality analysis is not available, but the model was completed too late for evaluation.

4. Environmental Protection Agency Stormwater Management Model (SWMM) for single-event wastewater flow and quality analysis provided the hydraulic limitations of the model are acceptable (neglect of downstream flow control and flow reversal, inadequate backwater, surcharging, and pressure flow formulation) A new version patterned after the Corps of Engineers STORM is now available for continuous simulation, but this version was completed too late for evaluation.
5. Hydrocomp Simulation Program for single-event and continuous wastewater flow and quality analysis provided the hydraulic limitations of the model are acceptable (approximate backwater and downstream flow control formulation, neglect of flow reversal, surcharging, and pressure flow).
6. Massachusetts Institute of Technology Urban Watershed Model (MITCAT) for single-event flow analysis provided the hydraulic limitations of the model (neglect of backwater, downstream flow control, backwater, flow reversal, surcharging, and pressure flow), or the use of a separate model for these phenomena is acceptable.
7. Seattle Computer Augmented Treatment and Disposal System as an example of an operating real-time control system to reduce untreated overflows. A more comprehensive computer model simulating both wastewater flow and quality and including mathematical optimization should be considered, however, for new systems.
8. SOGREAH Looped Sewer Model for single-event wastewater flow and quality analysis considering all important hydraulic phenomena.
9. Water Resources Engineers Stormwater Management Model for single-event wastewater flow and quality analysis considering all important hydraulic phenomena."

The hydrologist and other technical staff members should carefully evaluate which model would be appropriate for a particular engineering investigation. The authors of this Manual generally find SWMM and MITCAT the most usable and reliable for urban drainage hydrology. No one computer model can reasonably meet the needs of all hydrological investigation types. A mix of a few of the models would be required to meet such a comprehensive demand.

The following list tabulates references for some additional computer programs. Refer to available references for complete descriptions and tests of these models as brief statements tend to be misleading.

CURM	Cincinnati Urban Runoff Model	(Ref. 36)
HEC-1	Hydrologic Engineering Center, United States Corps of Engineers	(Ref. 29)
RRL	Road Research Laboratory Hydrograph Method	(Refs. 37 and 38)
SCS	SCS-TR-20 Computer Program for Project Formulation	(Ref. 39)

These models can give good results and test a number of possible alternatives to drainage problems when used in a professional manner.

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CHAPTER II

STREETS, CURBS, AND GUTTERS

Streets, curbs and gutters serve an important and necessary drainage service, even though their primary function is for the movement of traffic. Traffic and drainage uses are compatible up to a point, beyond which drainage is, and must be, subservient to traffic needs.

Gutter flow in streets is necessary to transport runoff water to storm inlets and to major drainage channels. Good planning of streets can substantially help in reducing the size of, and sometimes eliminate the need for, a storm sewer system in newly urbanized areas.

GENERAL

An overall approach to storm runoff management includes using the street system to transport runoff to inlets during the minor storm and to transport runoff from storms that are greater than the storm sewer capacity. According to the street classification and/or the surrounding land use, certain criteria (set forth herein) are used to determine at what point the minor and major drainage facilities begin, such criteria are being based on encroachment (maintenance of traffic lanes) for the minor storm and on inundation limitations for the major storm.

For the purpose of this work, the existing Stillwater street classification system is utilized as described following:

Principal Arterial System

This system of streets and highways should serve the major centers of activity of a metropolitan area, the highest traffic volume corridors, and the longest trip desires, and should carry a high proportion of the total urban area travel on a minimum of mileage. The system should be integrated, both internally and between major rural connections.

The principal arterial system should carry the major portion of trips entering and leaving the urban area, as well as the majority of through movements desiring to bypass the central city. In addition, sufficient intra-area travel, such as between central business districts and outlying residential areas, between major inner city communities, or between major suburban centers should be served by this class of facilities. Frequently, the principal arterial system will carry important intra-urban as well as inter-city bus routes. Finally, this system in urbanized areas should provide continuity for all rural arterials which intercept the urban boundary.

Minor Arterial Street System

The minor arterial street system should interconnect with and augment the urban principal arterial system and provide service for trips of moderate length at a somewhat lower level of travel mobility than major arterials. This system also distributes travel to geographic areas smaller than those identified with the higher system.

Collector Street System

The collector street system differs from the arterial systems in that facilities on the collector system may penetrate neighborhoods, distributing trips from the arterials through the area to the ultimate destination which may be on a local or collector street. Conversely, the collector street also collects traffic from local streets in the neighborhood and channels it into the arterial systems. In some cases, due to the design of the overall street system, a minor amount of through traffic may be carried on some collector streets.

Local Street System

The local street system comprises all facilities not on one of the higher systems. It serves primarily to provide direct access to abutting land and access to the higher order systems. It offers the lowest level of mobility and usually contains no bus routes. Service to through traffic movement usually is deliberately discouraged.

Freeway

A freeway permits rapid and unimpeded movement of traffic through and around a city. Access to the freeway is completely controlled by interchanges at major arterial streets. There may be up to eight lanes of traffic and parking is not permitted on the freeway right-of-way.

Effects of Runoff on Pavement

The storm runoff which influences the traffic-carrying capacity of a street can be classified as follows:

- Sheet flow across the pavement as falling rain flows to the edge of the pavement,
- Runoff flowing adjacent to the curb,
- Stormwater ponded at low points,
- Flow across the traffic lane from external sources, cross street flow, (as distinguished from water falling on the pavement surface), and
- Splashing of any of the above types of flow on pedestrians.

Each of these types of storm water runoff must be controlled within acceptable limits so that the streets' main function as a traffic carrier will not be unduly restricted.

The efficient removal of storm runoff from pavement surfaces has a positive effect on street maintenance, and street maintenance procedures can, in turn, affect the efficiency of a street as a runoff carrier. Research has indicated that pavement deterioration is accelerated by the presence of storm runoff.

Deterioration is promoted when stormwater enters pavement cracks and the joints between concrete and asphalt (such as at gutter-pans and cross-pans), and when high velocities peel pavement surfacing. When pavement overlays are undertaken, great care must be exercised not to fill the gutter section which, in turn, reduces the stormwater transport capabilities of the street.

Roadside channels are susceptible to erosion, but if properly designed for velocity and depth control, this problem can be minimized. Hydrologically, advantage may be gained due to decreased hydrologic peaks, runoff rates, and facilities' costs can be reduced compared to conventional storm sewer systems.

DESIGN

The typical Stillwater street cross section is shown in Figure II-1. In addition to the requirements shown in Figure II-1, the street should have a minimum grade of 0.4 percent. Inlets should be a minimium of 25 feet downstream of any curb cut. In locating curb cuts near inlets in already storm sewered areas, the same spacing should be utilized to locate the curb cut. Figure II-2 illustrates typical standard curb configurations to be used in Stillwater.

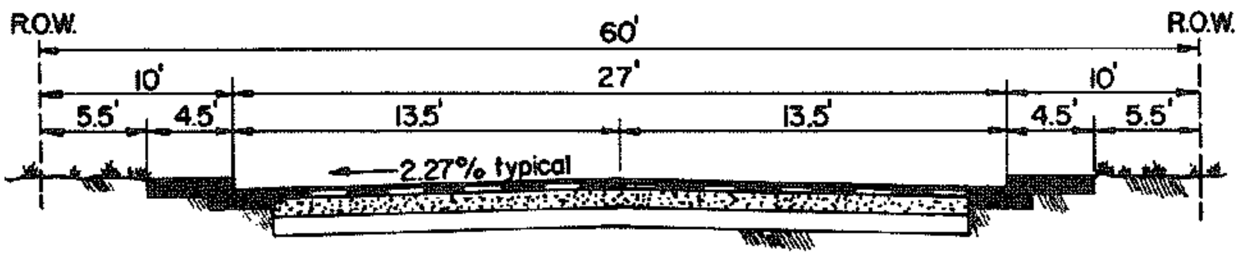
Figure II-3 illustrates the typical cross section to be used when cross fall occurs from one gutter to the other. This configuration is important to prevent sheet flow across the street, which reduces the street capacity during frequently occurring rainfall events or ice formation during the winter. Some sheet flow across the centerline of local streets is acceptable during the design minor frequency storm event, but should not occur for the rainfall events which occur more frequently than the one-year event nor during the design minor frequency storm event. Cross flow should not be allowed on streets whose designation is equal to or greater than the collector.

On local streets, where cross fall is necessary due to the existing topography, inlets may be placed in the lower curb, and the street crown removed to allow flow from the upper curb to reach the inlet in the lower curb at specified locations when approved by the City Engineer.

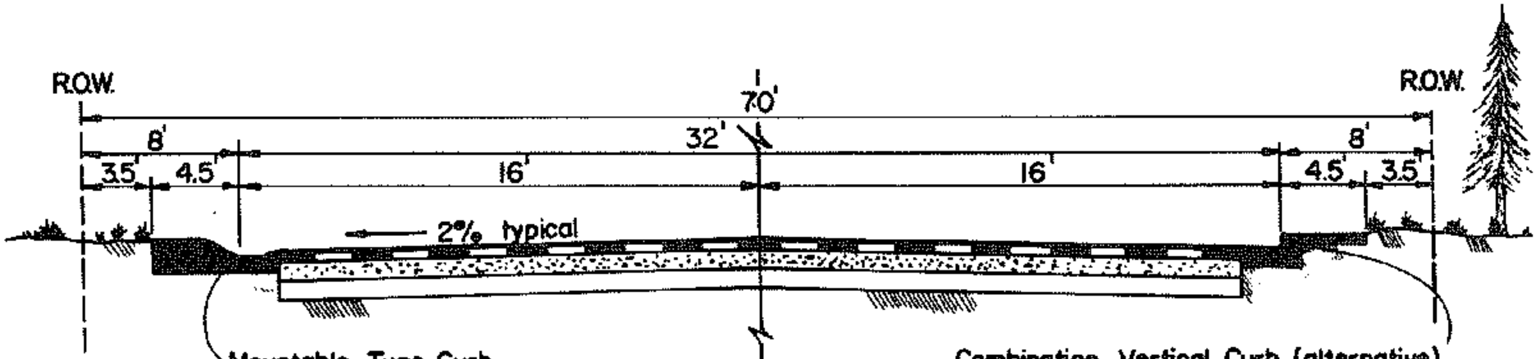
Driveway entrances should be recessed into the curb and not be made by building up in the gutter. The driveway should slope up to an elevation equal to the top of the curb so runoff within the street cannot flow onto adjacent property through the driveway entrance.

WME, June, 1979, II

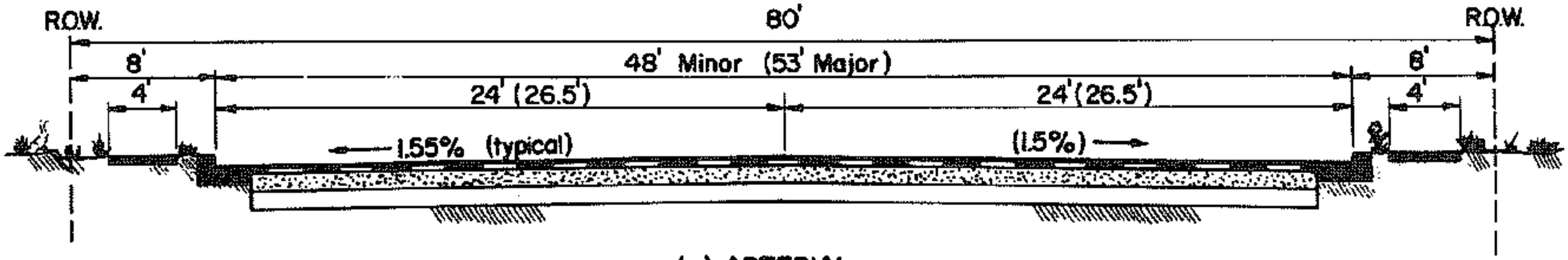
FIGURE II-1
TYPICAL STREET CROSS SECTION



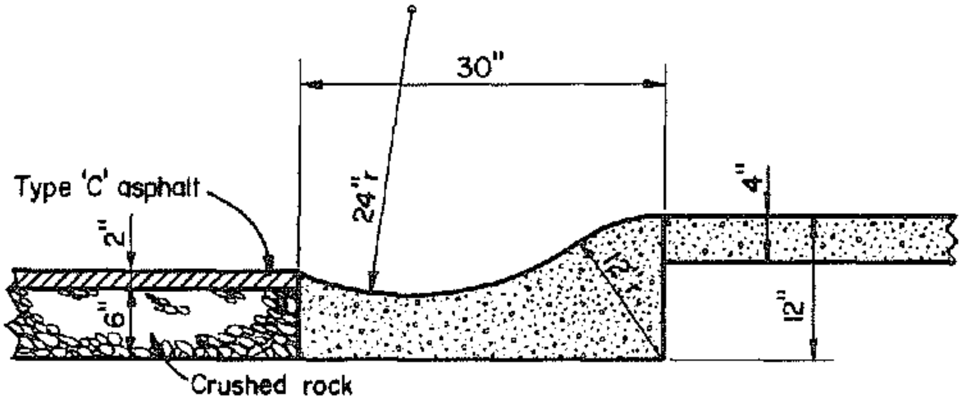
(a) LOCAL



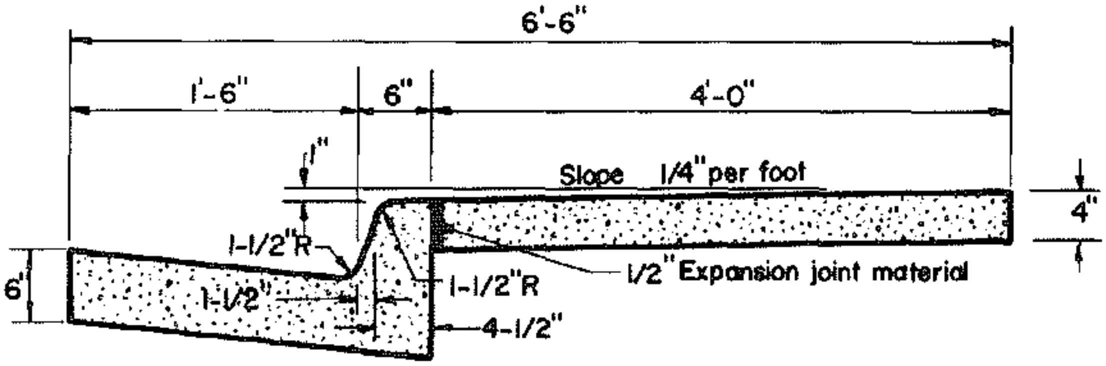
(b) COLLECTOR



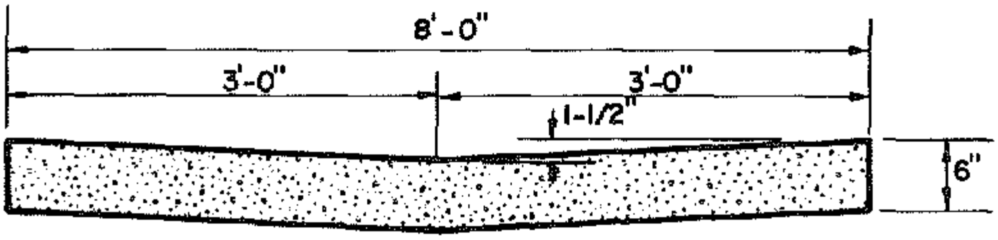
(c) ARTERIAL



30" MOUNTABLE CURB AND GUTTER



COMBINATION 6-INCH VERTICAL CURB, GUTTER, AND SIDEWALK



CROSS-PAN

FIGURE II-2
STANDARD CURB CONFIGURATIONS

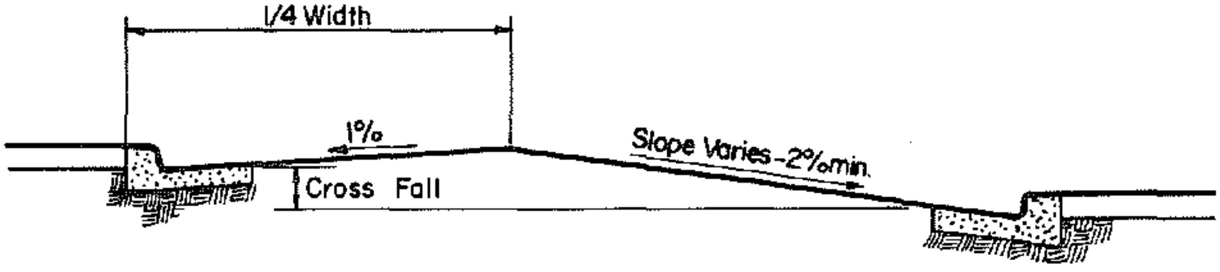


FIGURE II-3

TYPICAL STREET CROSS SECTION W/CROSS FALL

Inverted crown or dished streets should not be utilized for local, collector or arterial streets, or for freeways, as their primary aim is the carrying of extra storm runoff water. Since alley ways are used for limited volume and velocity of traffic, the use of the inverted crown does not adversely affect their function.

When local streets intersect, variations in grade shall be at the option of the designer. Figure II-4 illustrates the principles that will be used. When local streets intersect arterial or collector streets, the grades of the arterial or collector street should be continued uninterrupted. Figure II-5 illustrates the typical street cross sections necessary for such an intersection. For the figure, it is assumed that the arterial or collector street grade is 5 percent,the maximum allowable crown slope is 4 percent, the minimum allowable crown slope is 1 percent, and the crown must be maintained within the one-quarter points of the street.

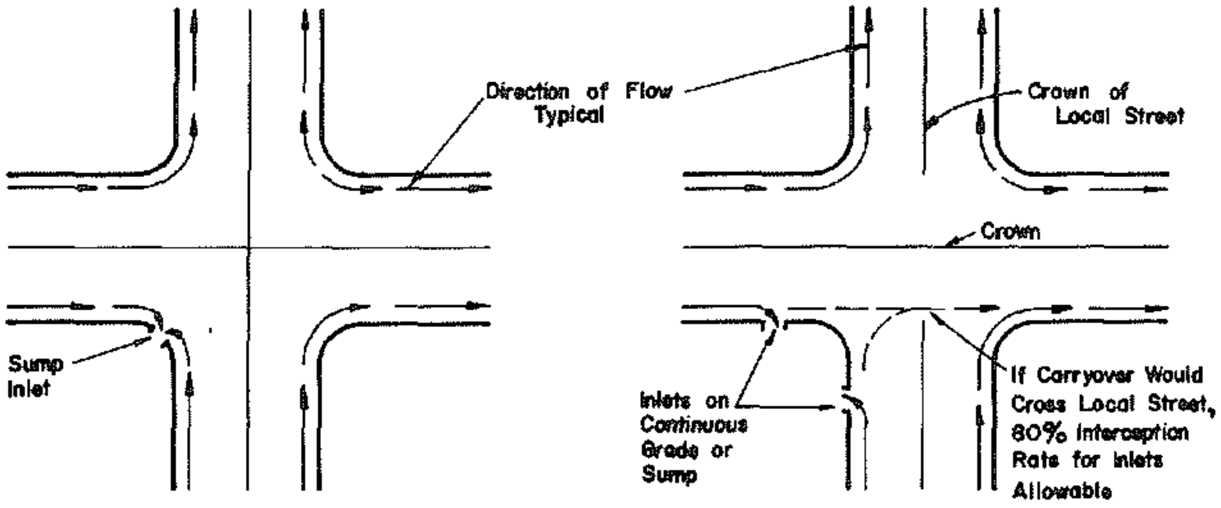
When collector and arterial streets intersect, the grade of the more major street should be maintained insofar as possible. No form of cross pan should be constructed across an arterial street for drainage purposes.

Conventional cross pans may be utilized to transport runoff across local streets when a storm sewer system is not required. The cross pan size and slope should be sufficient to transport the runoff across the intersection with encroachment equivalent to that allowed on the street. Infrequently, pans may be used on collector streets.

Street Capacity for Minor Storm

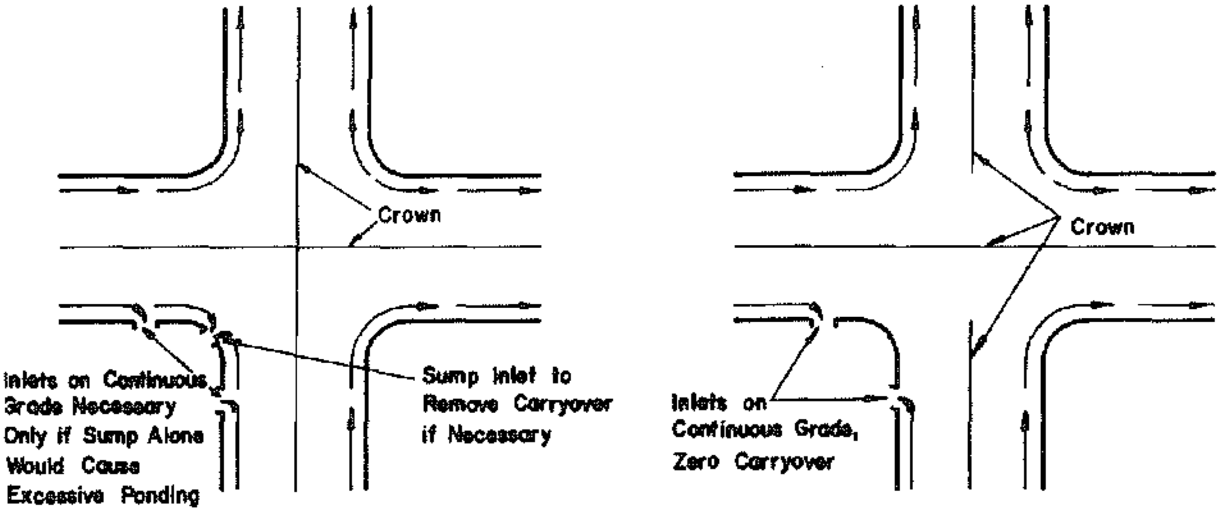
The following sections present specific design requirements for storm drainage on urban type streets. The methods employed to meet these requirements are at the designer's option, as long as they are in compliance with the criteria in other parts of this Manual.

Determination of street carrying capacity for the minor storm shall be based upon two considerations:



LOCAL ST. TO LOCAL ST.

LOCAL ST. TO ARTERIAL



ARTERIAL TO ARTERIAL
Where crowns are to be maintained

ARTERIAL TO ARTERIAL
One continuous crown

The examples show the minimum required inlets. Additional inlets may be necessary based upon allowable carrying capacity of gutters.

FIGURE II-4

TYPICAL STREET INTERSECTION
DRAINAGE TO STORM SEWER SYSTEM

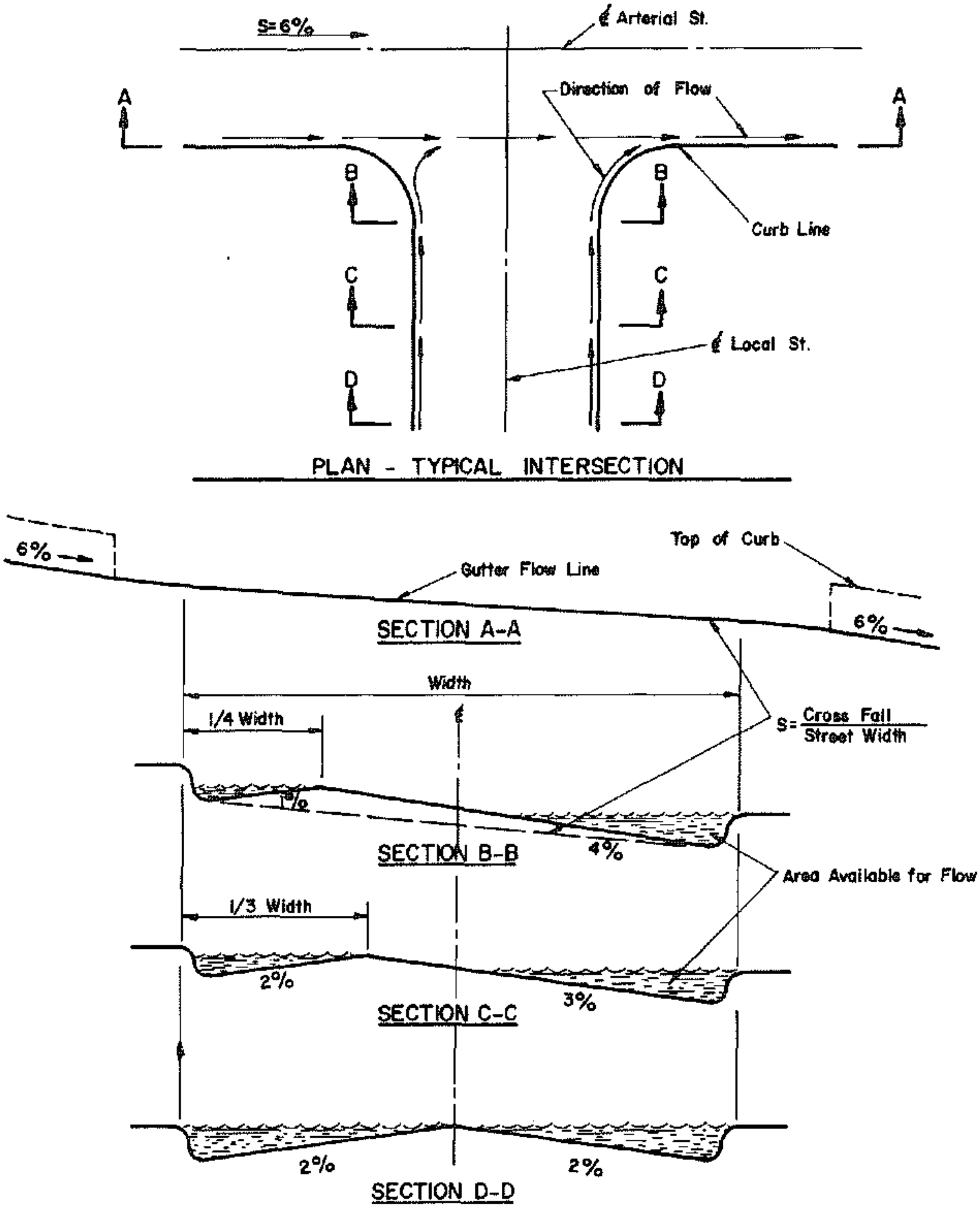


FIGURE 11-5

TYPICAL INTERSECTION CONSTRUCTION AT JUNCTION OF LOCAL AND ARTERIAL ST.

A. Pavement encroachment for computed theoretical flow conditions.

B. An empirical reduction of the theoretical allowable rate of flow to account for practical field conditions.

The pavement encroachment for the minor storm shall be limited as set forth in the following Table.

TABLE II-1

ALLOWABLE USE OF STREETS FOR MINOR STORM RUNOFF IN TERMS OF PAVEMENT ENCROACHMENT

Street Classification	Maximum Encroachment
Urban Local	No curb over-topping.* Flow may spread to crown of street.
Urban Collector	No curb over-topping.* Flow spread must leave at least one lane free of water.
Urban Minor Arterial	No curb over-topping.* Flow spread must leave at least one lane free of water.
Urban Principal Arterial	No curb over-topping.* Flow spread must leave at least one lane free of water in each direction.
Freeway	No encroachment is allowed on any traffic lane.

*Where no curbing exists, encroachment shall not extend over property lines.

The storm sewer system should commence at the point where the maximum encroachment is reached, and should be designed on the basis of the minor storm. Development of the major drainage system is encouraged so that the minor runoff is removed from the streets, thus reducing the extent of the storm sewer system.

When the allowable pavement encroachment has been determined, the theoretical gutter carrying capacity for a particular encroachment shall be computed using the modified Manning's formula

$$Q = 0.56 \frac{z}{n} s^{1/2} y^{8/3} \qquad \text{(II-1)}$$

for flow in shallow triangular channel, as shown on Figure II-6.

Figure II-6, Nomograph for Flow in Triangular Gutters, may be utilized for all gutter configurations. To simplify computations, graphs for particular street shapes may be plotted. An n value of 0.016 shall be utilized unless special considerations exist.

The actual flow rate allowable per gutter shall be calculated by multiplying the theoretical capacity by the corresponding factor obtained from Figure II-7. Discharge curves have been developed for standard streets. The designer will be able to develop discharge curves for non-standard streets and for streets with crossfall.

Street Capacity for Major Storm

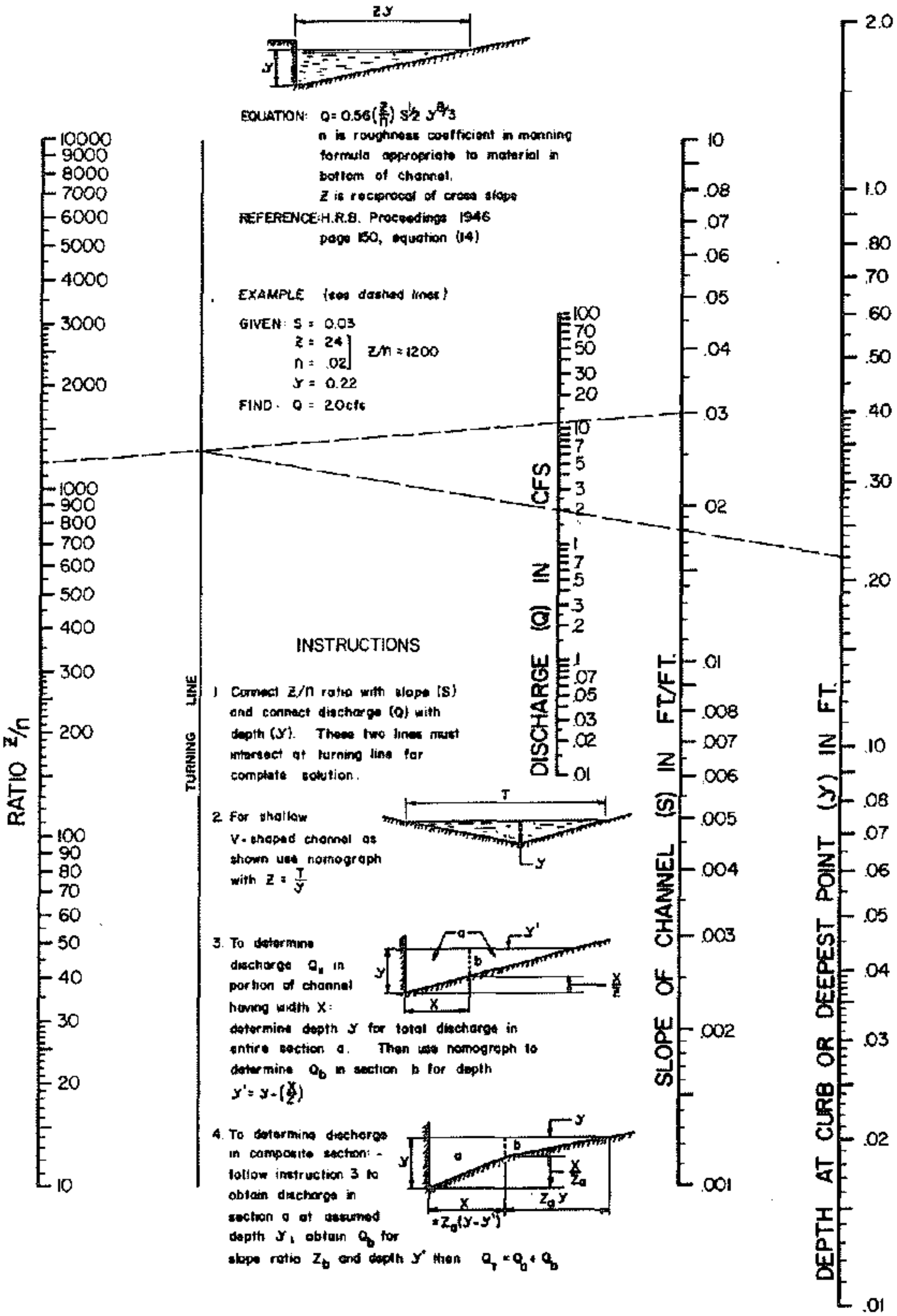
Determination of the allowable flow for the major storm shall be based upon two considerations:

A. Theoretical capacity, based upon allowable depth and inundated area.
B. Reduced allowable flow due to velocity considerations.

The allowable depth and inundated area for the major storm shall be limited as set forth in Table II-2.

Based upon the allowable depth and inundated area as determined from Table II-2, the theoretical street carrying capacity shall be calculated. Manning's formula shall be utilized with an n value applicable to the actual boundary conditions encountered which may include grassed areas and sections with differing geometry.

The actual flow allowable within the street right-of-way shall be calculated by multiplying the theoretical capacity by the corresponding factor obtained from Figure II-7.



EQUATION: $Q = 0.56(\frac{z}{n}) S^{1/2} y^{8/3}$

n is roughness coefficient in manning formula appropriate to material in bottom of channel.

Z is reciprocal of cross slope

REFERENCE: H.R.B. Proceedings 1946 page 150, equation (14)

EXAMPLE (see dashed lines)

GIVEN: $S = 0.03$

$z = 24$, $n = .02$] $Z/n = 1200$

$y = 0.22$

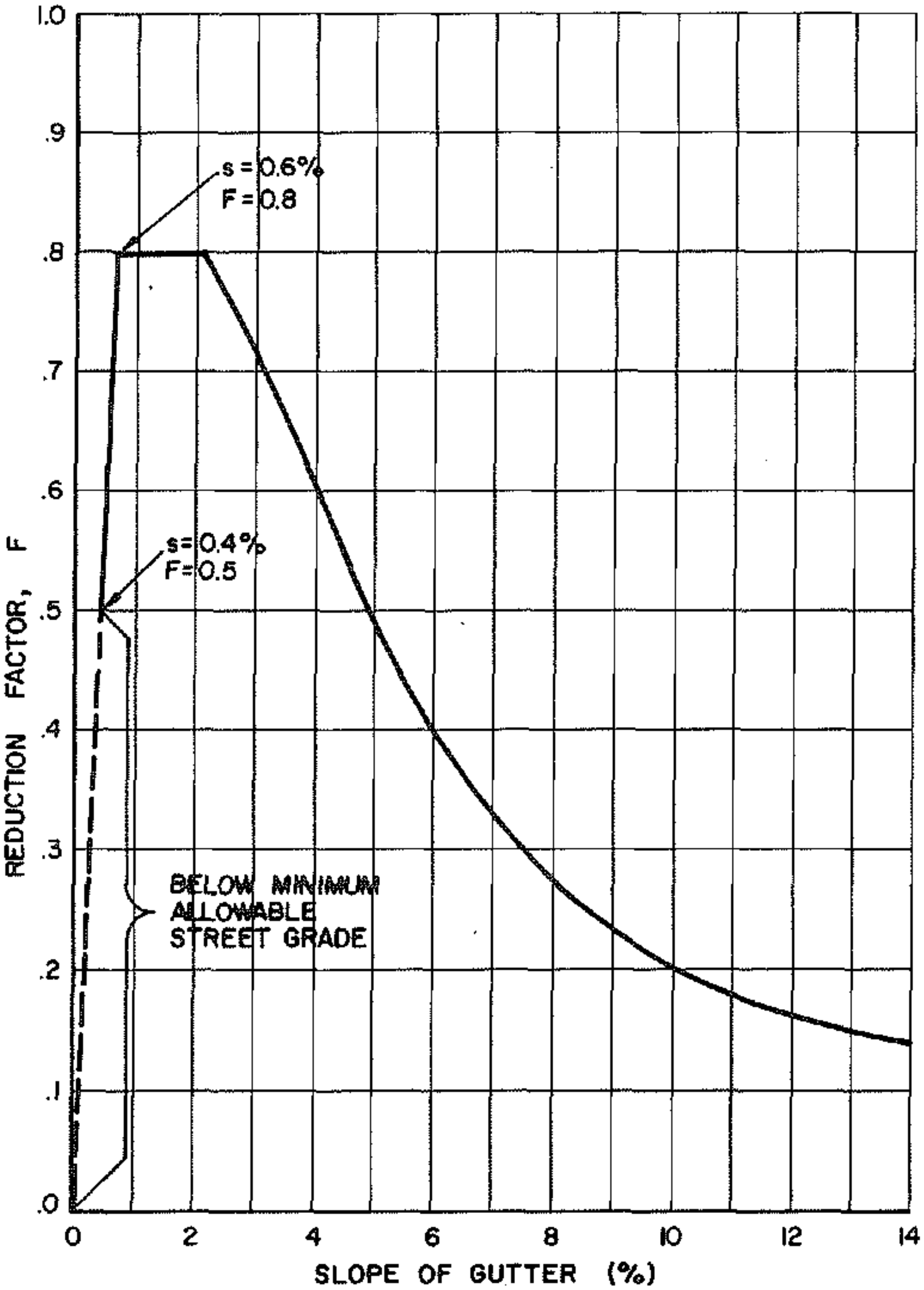
FIND: $Q = 2.0$cfs

INSTRUCTIONS

1. Connect Z/n ratio with slope (S) and connect discharge (Q) with depth (y). These two lines must intersect at turning line for complete solution.
2. For shallow V-shaped channel as shown use nomograph with $Z = \frac{T}{y}$
3. To determine discharge Q_x in portion of channel having width X: determine depth y for total discharge in entire section a. Then use nomograph to determine Q_b in section b for depth $y' = y - (\frac{X}{Z})$
4. To determine discharge in composite section: - follow instruction 3 to obtain discharge in section a at assumed depth y_1 obtain Q_b for slope ratio Z_b and depth y' then $Q_T = Q_a + Q_b$

FIGURE 11-6

NOMOGRAPH FOR TRIANGULAR GUTTERS



Apply reduction factor for applicable slope to the theoretical gutter capacity to obtain allowable gutter capacity.

FIGURE II-7

REDUCTION FACTOR FOR ALLOWABLE GUTTER CAPACITY

Limitations for depth and inundated area for Major Storms shall be those presented in Table II-2, Major Storm Runoff Allowable Street Inundation. These limitations shall determine the allowable depth at inlets, gutter turnouts, culvert headwaters, etc.

Where allowable ponding depth would cause cross street flow, the limitation shall be the minimum allowable of the two criteria set forth in Table II-2 or Table II-3.

When the direction of flow is toward a principal arterial street, the allowable carrying capacity shall be calculated by applying the reduction factor from Figure II-8 to the theoretical gutter capacity. The grade used to determine the reduction factor shall be the same effective grade used to calculate the theoretical capacity.

TABLE II-2

MAJOR STORM RUNOFF ALLOWABLE STREET INUNDATION

Street Classification	Allowable Depth and Inundated Areas
Local, Collector and Minor Arterial	Residential dwellings, public commercial, and industrial buildings shall not be inundated at the ground line unless buildings are flood proofed. The depth of water over the gutter flow line shall not exceed 12 inches.
Principal Arterial and Freeway	Residential dwellings, public commercial, and industrial buildings shall not be inundated at the ground line, unless buildings are flood proofed. Depth of water at the street crown shall not exceed 6 inches to allow operation of emergency vehicles. The depth of water over the gutter flow line shall not exceed 12 inches.

Cross Street Flow is in two general categories: The first type is runoff which has been flowing in a gutter and then flows across the street to the opposite gutter or to an inlet. The second type is flow from some external source, such as drainageway, which will flow across the crown of a street when the conduit capacity beneath the street is exceeded.

Cross Street Flow Depth shall be limited as set forth in Table II-3.

TABLE II-3
ALLOWABLE CROSS STREET FLOW

Street Classification	Minor Designer Runoff	Major Design Runoff
Local	6-inch depth at crown or in cross pan	12 inches of depth above gutter flow line
Collector and Minor Arterial	Where cross pans allowed, depth of 6 inches	12 inches of depth above gutter flow line
Principal Arterial	None	6 inches or less over crown
Freeway	None	6 inches or less over crown

STREET INTERSECTIONS

The following design criteria are applicable at intersections of urban streets. Such limitations as gutter carrying capacity covered previously shall apply along the street proper, while this section shall govern at the intersection.

Gutter Capacity, Minor Storm

Pavement Encroachment: Limitations at intersections for pavement encroachment shall be given in Table II-1, Allowable Use of Streets for Minor Storm Runoff.

Theoretical Capacity. The theoretical carrying capacity of each gutter approaching an intersection shall be calculated, based upon the most critical cross section, as covered in this section.

Continuous Grade Across Intersection. When the gutter slope will be continued across an intersection, as when cross pans are utilized, the slope used for calculating capacity shall be that of the gutter flow line crossing the street.

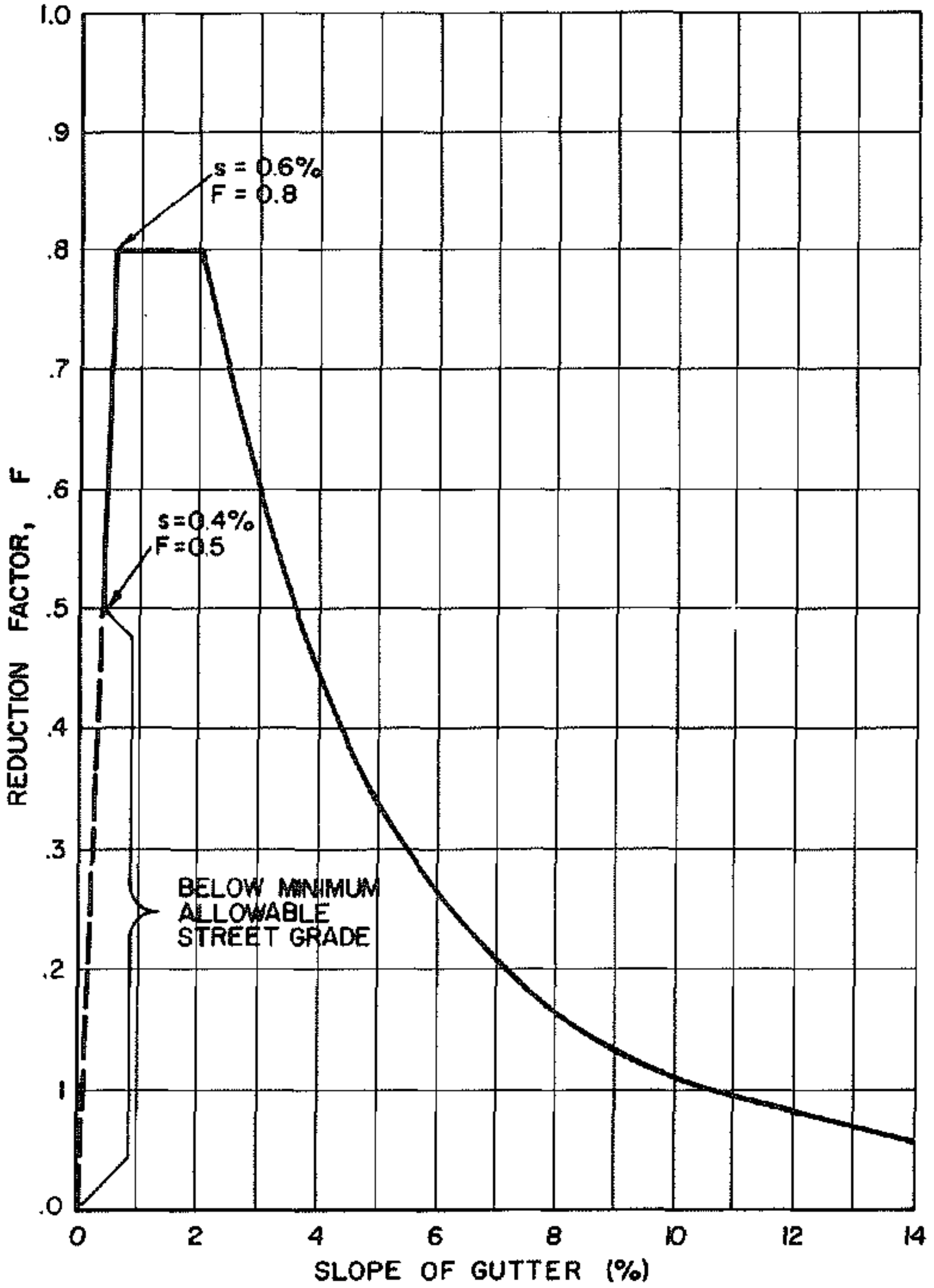
Flow Direction Change at Intersection. When the gutter flow must undergo a direction change at the intersection greater than 45°, the slope used for calculating capacity shall be the effective gutter slope, defined as the average of the gutter slopes at 0 feet, and 50 feet from the point of direction change.

Flow Interception by Inlet. When gutter flow will be intercepted by an inlet on continuous grade at the intersection, the effective gutter slope shall be utilized for calculations. Under this condition, the points for averaging shall be 0 feet, 25 feet, and 50 feet upstream from the inlet.

Allowable Capacity. The allowable capacity for gutters approaching an intersection shall be calculated by applying a reduction factor to the theoretical capacity as covered in the following sections.

Flow Approaching Principal Arterial Street. When the direction of flow is towards an arterial street, the allowable carrying capacity shall be calculated by applying the reduction factor from Figure II-8 to the theoretical gutter capacity. The grade used to determine the reduction factor shall be the same effective grade used to calculate the theoretical capacity.

Flow Approaching Streets Other Than Principal Arterial. When the direction of flow is towards a non-principal arterial street, the allowable carrying capacity shall be calculated by applying the reduction factor from Figure II-7 to the theoretical gutter capacity. The slope used to determine the reduction factor shall be the same effective slope used to calculate the theoretical capacity.



Apply reduction factor for applicable slope to the theoretical gutter capacity to obtain allowable gutter capacity approaching arterial street.

FIGURE 11-8

REDUCTION FACTOR FOR ALLOWABLE GUTTER CAPACITY WHEN APPROACHING A PRINCIPAL ARTERIAL STREET

Special Considerations for Business Areas and Heavily-Used Pedestrian Areas. In highly concentrated business areas, where large volumes of pedestrian traffic are likely, the use of walk-over curbs at intersections should be considered. If utilized, it would appear necessary that no flow be allowed to continue around the corner and, therefore, inlets would be required at nearly every corner. For the storm frequency being contemplated, the effect water may have on pedestrian walking area should be compatible with that on streets. Based upon vehicular traffic use, in a business area, all streets would probably classify as collector or arterial, which requires one water free travel lane for the minor design storm for the former and one water free travel lane each direction for the latter. The walk-over curbs should be available for limited pedestrian use.

Where concentration of pedestrians occurs, depth and area limitations may need modifications. As an example, streets adjacent to schools, are arterials from a pedestrian standpoint, and should be designed accordingly. The sociological aspects of designing for the pedestrian is at least as important as designing for vehicular traffic.

Where business buildings are constructed to property lines, the reduced clearance between buildings and heavy traffic must be considered. Splash from vehicles striking gutter flow may damage store fronts and make walking on sidewalks impossible. Ponding water and gutter flow exceeding 2 feet in width are difficult to negotiate by pedestrians.

Although not a necessity in many business areas, highly concentrated business areas shall be designed for use of reduced allowable pavement encroachment area, inundated areas, raised walk over curves at intersections, or additional inlets to intercept flow before it reaches intersections. Generally, these areas should be storm sewered even if other criteria do not so indicate.

Gutter Capacity, Major Storm

Allowable Depth and Inundated Area. The allowable depth and inundated area for the major storm shall be limited as set forth in Table II-2.

Theoretical Capacity. The theoretical carrying capacity of each gutter approaching an intersection shall be calculated, based upon the most critical cross section, as covered previously.

Allowable Capacity. The allowable capacity for gutters approaching an intersection shall be calculated by applying the reduction factor from Figure II-7 (or Figure II-8 for principal arterials) to the theoretical capacity. The gutter grade used to determine the reduction factor shall be the same effective grade used to calculate the theoretical capacity.

Ponding and Cross-Street Flow shall be limited as set forth in Table II-2 and Table II-3.

Based upon the limitations in Table II-3 and other applicable limitations (such as ponding depth), the theoretical quantity of cross street flow shall be calculated. Because the nature of the flow will vary, no general rule for computational method can be made.

Once the theoretical cross street capacity has been computed, the allowable quantity shall be calculated by multiplying the theoretical capacity by the corresponding factor from Figure II-7. The slope of the water surface crossing the street shall be used in lieu of the gutter slope.

Where cross street flow is caused by exceeding the capacity of a major drainage structure crossing beneath the street, measures shall be taken to insure that the street will not be unnecessarily damaged by the cross street flow. This may require protective headwalls, riprap on embankments or other measures.

No specific limitations are set for sheet flow. Designers should be aware of its existence and effects and take precautions to limit its occurrence.

As a result of the quest for large areas of flat, inexpensive land, industries are often located in areas particularly subject to flooding. As stated in Table II-2, industrial areas not flood proofed shall not be inundated by the Major Storm. Flood proofing may be an acceptable solution where large amounts of runoff are concentrated in industrial areas.

Drainage Design Criteria for Roadside Ditches

When roadside drainage ditches are used for drainage purposes, as opposed to curbs and gutters, the majority of requirements set forth for typical urban streets are applicable for rural streets. Certain special considerations necessary for proper design of rural streets are set forth in the following sections.

Street Capacity, Minor Storm

Determination of street carrying capacity for the minor storm shall be based upon the following considerations:

A. Pavement encroachment allowed.

B. Maximum allowable velocity to prevent scour.

The same limitations as expressed in Table II-1 shall govern rural streets. Once the pavement encroachment has been established, the maximum allowable velocity for the drainage channel shall be determined from Tables II-4 and II-5.

Design velocities for all linings should not fall below 2 fps for the minor runoff to minimize sediment depositional problems. The allowable capacity for the drainage ditch should be calculated using Manning's formula with an appropriate n value. If the natural channel slope would cause excessive velocity, drop structures, checks, riprap, or other suitable channel protection shall be employed. Design depths shall be limited to 1.5 feet, and preferably less than 1.0 feet.

Determination of street carrying capacity for the Major Storm shall be based upon the following considerations.

TABLE II-4
PERMISSIBLE VELOCITIES FOR ROADSIDE DRAINAGE CHANNELS
Channels with Erodible Linings

Soils Type of Lining (Earth, No Vegetation)	Permissible Velocity (fps)
Fine Sand (noncolloidal)	2.5
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Ordinary Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (noncolloidal)	5.5
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Shales and Hard Pans	6.0

TABLE II-5
ROADSIDE CHANNELS LINED WITH UNIFORM STAND OF VARIOUS GRASS COVERS AND WELL MAINTAINED

		Permissible Velocity (fps)*	
Cover	Slope Range (Percent)	Erosion Resistant Soils	Easily Eroded Soils
Berumuda Grass	0-5	8.0	5.0
Crested Wheat Grass	0-5	6.0	5.0
Buffalo Grass	0-5	6.0	5.0
Kentucky Bluegrass	5-10	5.0	4.0
Smooth Brome	over 10	4.0	3.0
Blue Grama			
Grass Mixture	0-5	4.0	3.0
	5-10	3.0	2.5
Lespedeza Sericea	0-5	3.0	2.0
Weeping Lovegrass			
Yellow Bluestem			
Kudzu			
Alfalfa			
Crabgrass			
Common Lespedeza			
Sundangrass			

*Higher velocities are permissible for major storm runoff events where grasses are irrigated and well maintained.

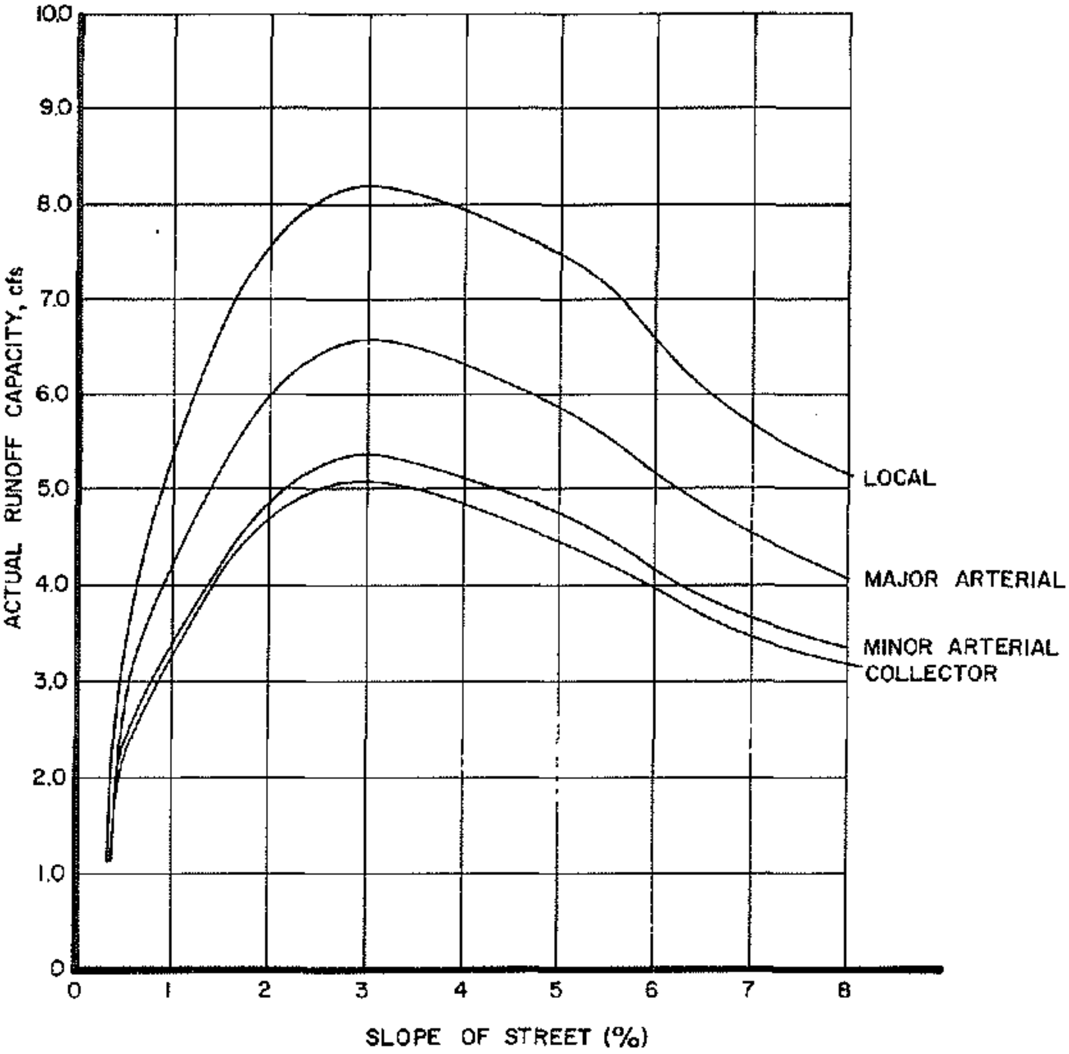
1. Allowable depth and inundated area.
2. Maximum allowable velocity for acceptable scour.

The same limitations as expressed in Table II-2 shall govern rural streets.

Based upon the allowable depth and inundated area, allowable capacity for the Major Storm shall be calculated as described for streets with curbs and gutters.

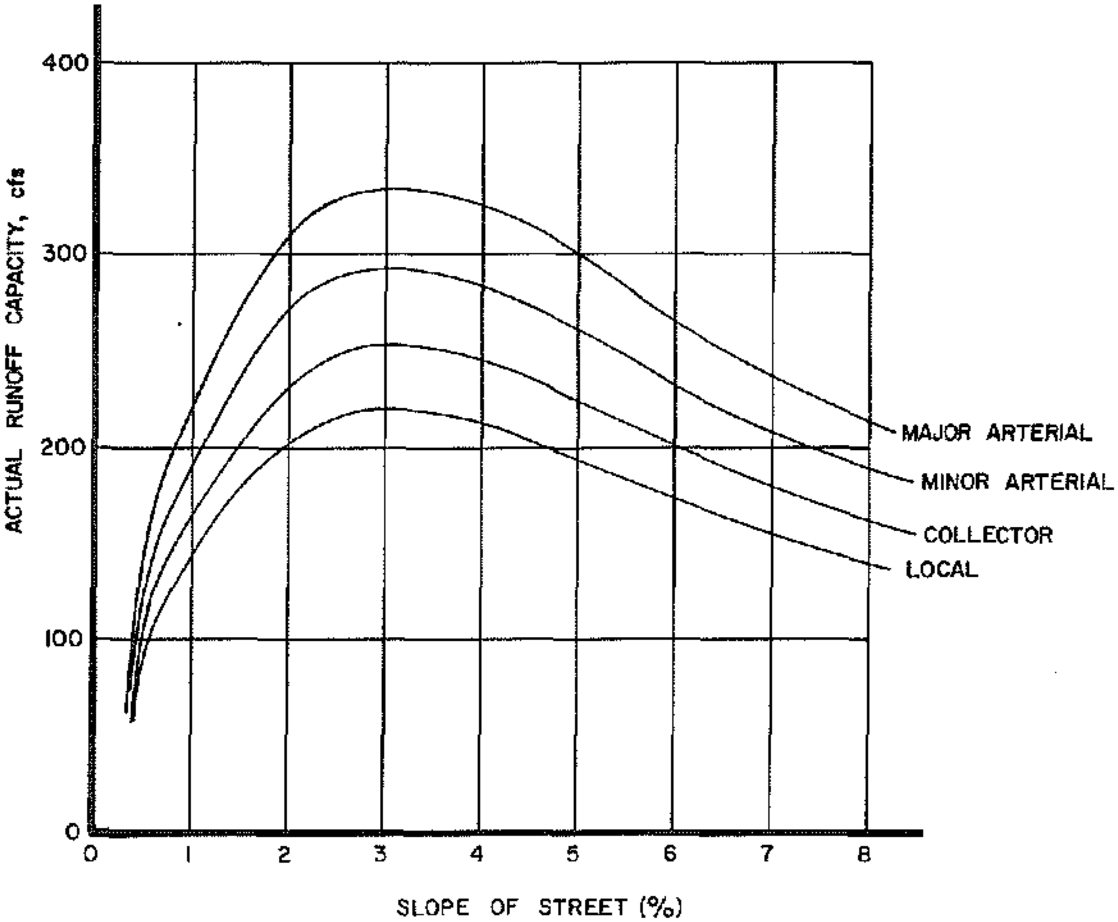
DESIGN CHARTS

Design Charts II-A and II-B represent rating curves for the various standard street sections in Stillwater. The values obtained from the graphs are for one gutter, and the reduction factor has already been applied.



RUNOFF CAPACITY PER GUTTER FOR TYPICAL STREET SECTIONS, STILLWATER, OKLAHOMA

CHART II-A
STREET CAPACITY FOR MINOR STORM



RUNOFF CAPACITY OF TOTAL SECTION FOR
TYPICAL STREET SECTIONS, STILLWATER OKLAHOMA

CHART II-B
STREET CAPACITY FOR MAJOR STORMS

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INLETS

Proper surface drainage of streets and highways is one of many requirements for the safe movement of traffic and is normally accomplished with stormwater inlets. A stormwater inlet is an opening into a storm sewer system for the entrance of surface storm runoff. Use of inlets should be delayed as long as possible because as soon as the runoff enters the pipe system, it is carried rapidly downstream. The placing of inlets is dictated by street encroachment criteria as listed elsewhere in this Manual.

INLET TYPES

There are four categories of inlets:

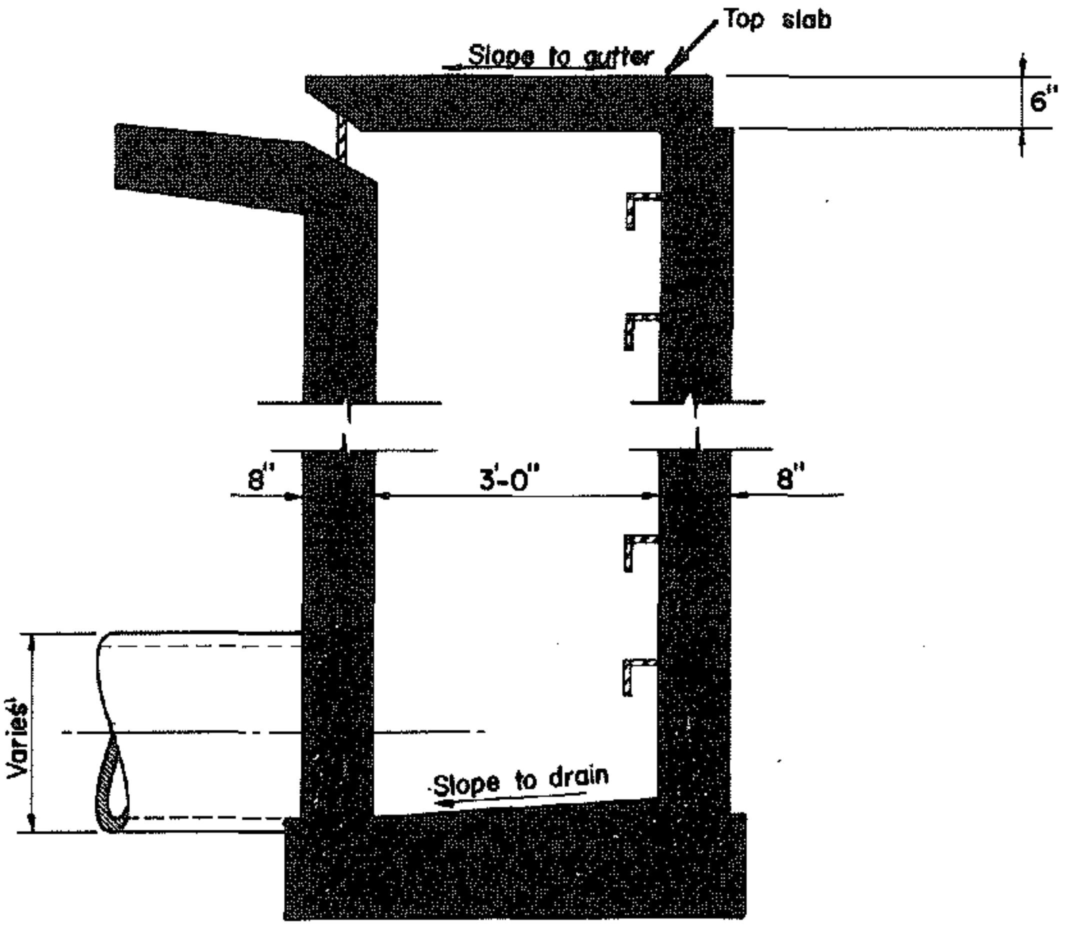
- Curb Opening Inlets (Figure III-1)
- Grated Inlets (Figure III-2)
- Combination Inlets (Figure III-3)
- Slotted Drains (Figure III-16)

In addition, inlets may be further classified as being on a continuous grade or in a sump.

The continous grade condition exists when the street grade is continuous past the inlet and the water flows past. The sump condition exists whenever water is restricted to the inlet area because the inlet is located at a low point. This may be due to a change in grade of the street from positive to negative or due to the crown slope of a cross street when the inlet is located at an intersection.

Curb opening inlets should be utilized in the design of storm sewer systems when a sump condition exists. Although a curb opening inlet will not guarantee against plugging, it is the most desirable type of inlet.

A curb opening inlet is a vertical opening in a curb through which the gutter flow passes. For safety reasons, the vertical opening should not be greater than 6 inches. The gutter may be undepressed or depressed in the



SECTION A-A

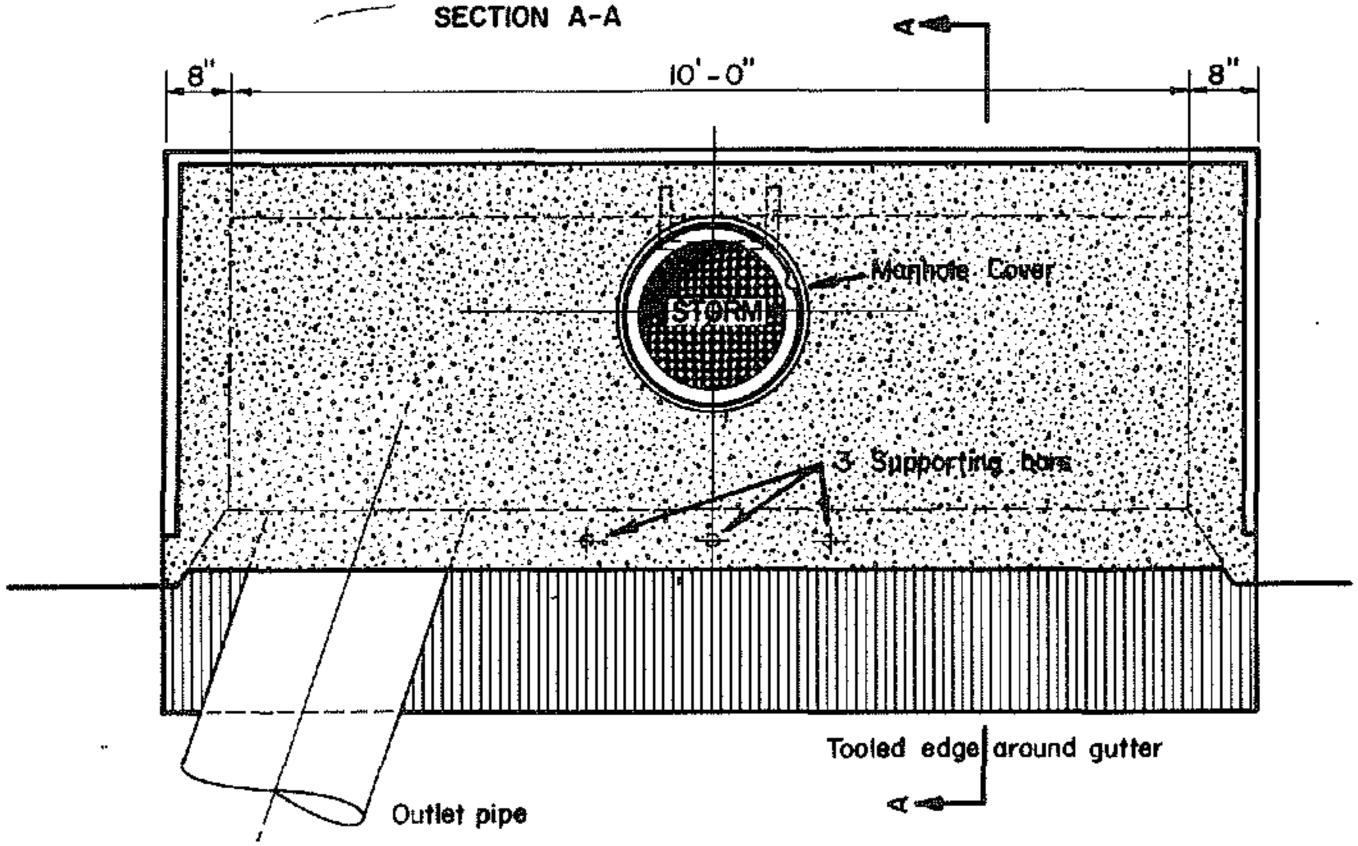


FIGURE III-1
TYPICAL 10FT. CURB OPENING INLET

area of the curb opening. The capacity of the curb opening is significantly increased by depressing the opening. Curb opening inlets will have a 2-inch depression of the face of the curb opening. A characteristic of the curb opening inlet is its relative inefficiency on streets of steep grade. The use of 5 foot long (or less) curb opening inlets is, therefore, not recommended on continuous grades; however, longer inlets can be quite efficient and should be considered for use on streets with slopes up to 1.0 percent.

The term, grated or gutter inlet, refers to an opening in the gutter covered by one or more grates through which the water falls. As with other inlets, grated inlets may be either depressed or undepressed and should be located on a continuous grade.

The engineer should use grated inlet designs which optimize hydraulic efficiency, bicycle and pedestrian safety, structural adequacy, economy, and freedom from clogging. (See combination inlet for recommendations.)

The term, slotted drain, refers to a slot opening in the pavement which will intercept sheet flow and convey the flow through a corrugated steel pipe. They are most effective when street slopes are shallow.

The term, longitudinal bar grate, refers to a grate in which the bars are oriented parallel to the direction of gutter flow. Transverse bars refer to bars located at some angle, usually perpendicular, to the direction of flow. Longitudinal bar inlets are far more efficient and less apt to be plugged by trash than are grates made wholly of transverse bars or incorporating transverse bars in the design of the longitudinal bar grate; however, for safety reasons, a grate with transverse bars shall be employed, and longitudinal bar grates without transverse bars will not be permitted. The designer should specify a grate having recessed transverse bars spaced about 9 inches apart (i.e., the spacing that will give a 1-inch vertical drop of a 20-inch bike wheel). In most instances, the standard grate for the City will be suitable for use and as specified later in this Chapter.

The major disadvantage of the grated inlet is its tendency to plug with trash. This significantly reduces efficiency from the theoretical value, and in some cases, renders the inlet inoperable. A grate's ability to handle debris without clogging is most dependent on the spacing of its longitudinal bars. Grates with wide longitudinal bar spacing consistently outperform grates with narrower longitudinal spacing, but this is in direct conflict to high safety performance characteristics. Therefore, a compromise has been made to optimize both hydraulic and safety criteria. in the Standard City grate.

Depressing the grated inlet will significantly increase its capacity, but the interference to traffic caused by the depression may make the depression undesirable. A 2-inch depression at the gutter flow line shall be included with all inlet details. At the intersection of the gutter and the street pavement, no depression will be permitted. Test have shown increases in inlet capacity of up to 100 percent in specialized cases and increases on the order of 30 percent to 50 percent in more generalized cases. The safety hazard associated with this marked increase in hydraulic capacity is considered minimal as the depression where bikes usually travel will be a fraction of one inch. See Figure II-2 for acceptable grated inlets in Stillwater.

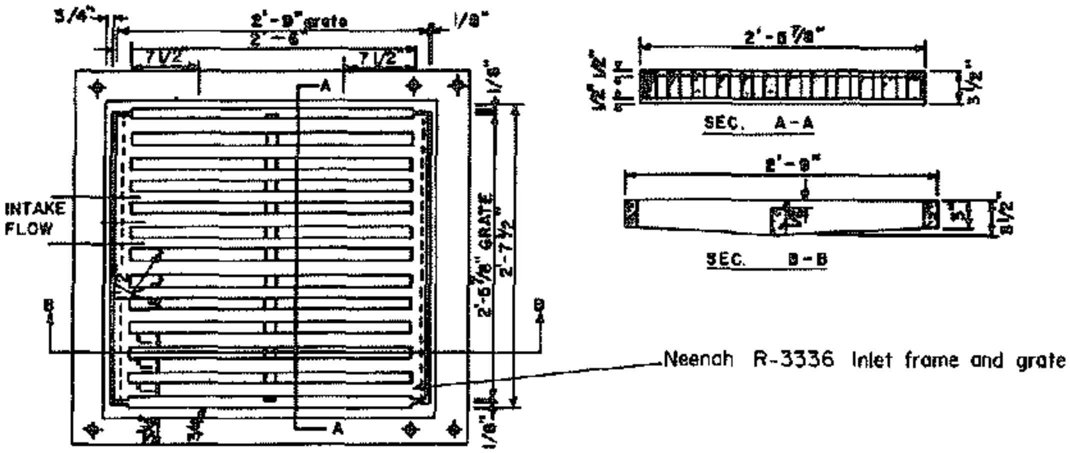
A <u>combination</u> inlet is composed of a curb opening and a grated gutter inlet acting as a unit. Usually the gutter opening is placed adjacent to the curb opening. As with other inlets, a combination inlet may be either depressed or undepressed and located in a sump or on a continuous grade. It is the most efficient type of stormwater inlet with regards to hydraulic interception capabilities and freedom from debris clogging. See Figure III-3 for the Stillwater combination inlet standard.

A <u>multiple</u> inlet is two or more closely spaced inlets acting as a hydraulic unit.

GRATE CONFIGURATION

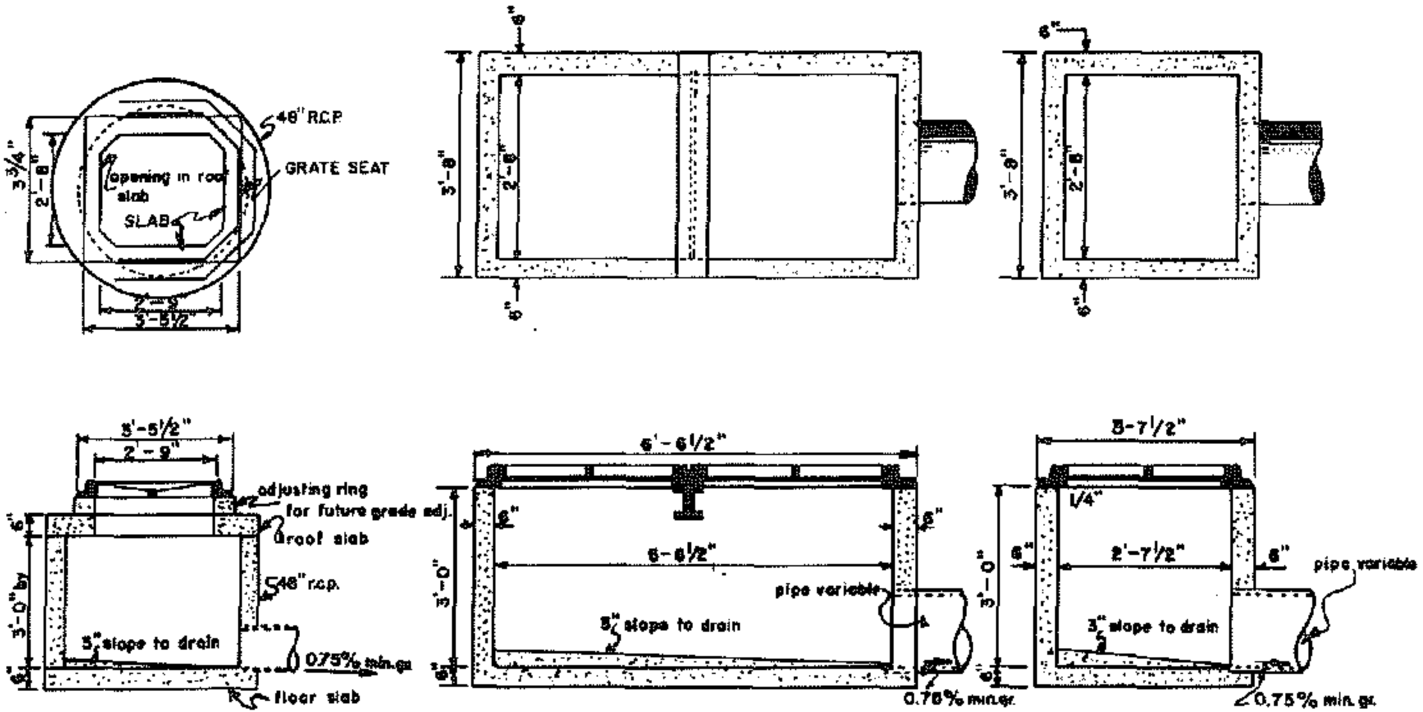
In recent years there has been a nationwide increase in interest in bicycling. The recent exposure of bicyclists to highways and streets has resulted in increased bicycle accidents with vehicular traffic as well as with various highway-related structures. Although curb opening inlets and combination inlets are used on flat street slopes, grate inlets are far more prevalent, particularly on steeper slopes.

Safety and hydraulic efficiency characteristics of a grate inlet are in conflict. Grate design must be a compromise that will optimize both characteristics. Numerous tests have found that high performance grate inlets for bicycle and pedestrian safety are generally in a low performance



DETAIL - DROP INLET COVER

NOTE: GRATE TO BE INSTALLED WITH BARS PARALLEL TO INTAKE FLOW



ROUND SINGLE DROP INLET

TYPE I

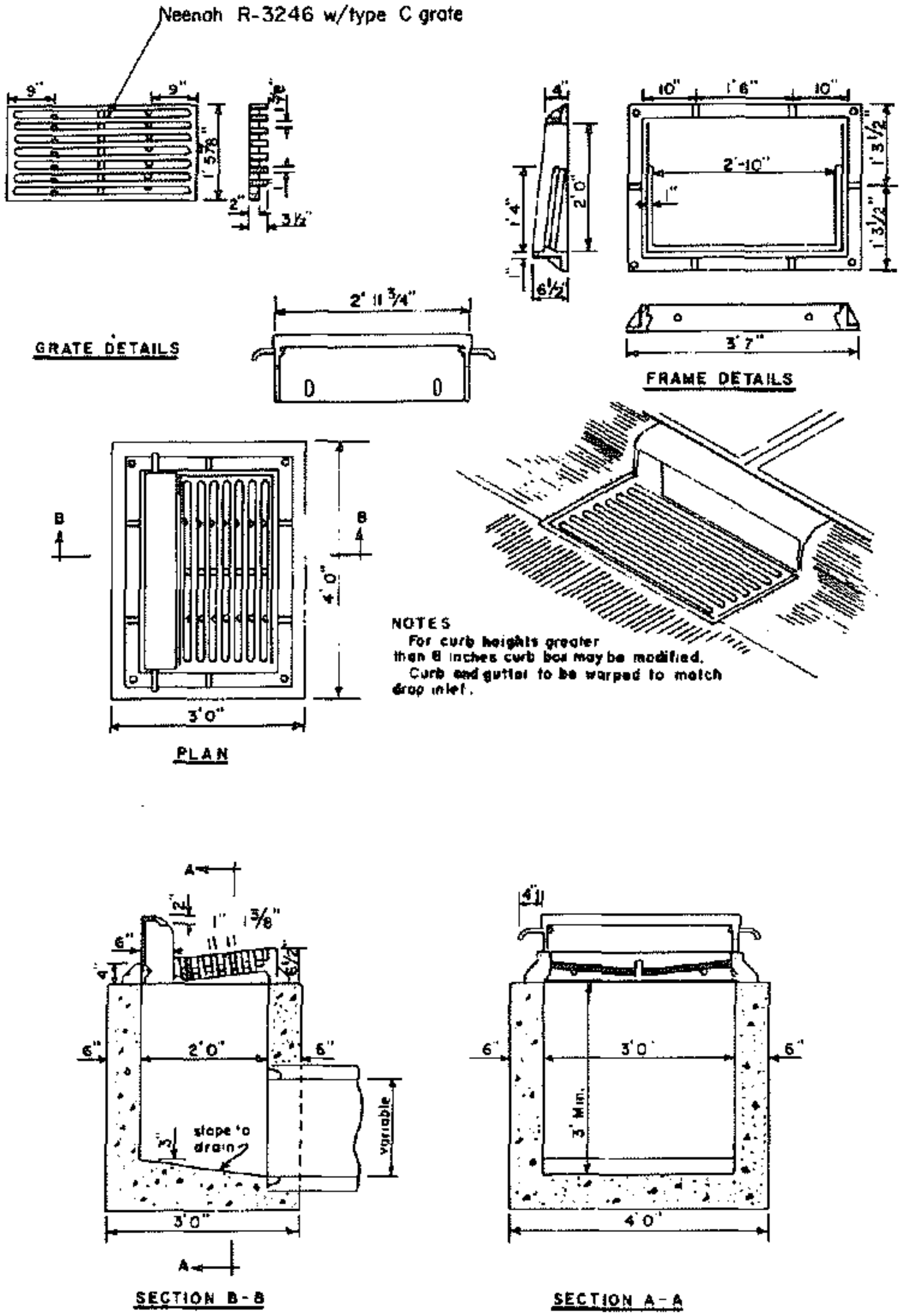
DOUBLE DROP INLET

TYPE II

SINGLE DROP INLET

TYPE III

FIGURE III-2
GRATED INLETS



GRATE DETAILS

FRAME DETAILS

PLAN

NOTES
For curb heights greater than 8 inches curb box may be modified.
Curb and gutter to be warped to match drop inlet.

SECTION B-B

SECTION A-A

FIGURE III-3
COMBINATION INLET

category for debris handling. Hydraulic efficiencies for high and low performance grates do not vary significantly under favorable flow conditions (see Figure III-4 for combination inlets), but under unfavorable flow conditions the good safety inlets will be 15 to 30 percent less efficient than inlets considered good debris handlers.

Grate Configuration Review

Longitudinal bar grates have long been recognized as a very efficient grate inlet. In recent years, however, it has become evident that the standard parallel bar grate with 1- to 2-inch clear openings between longitudinal bars is not safe for bicycle and pedestrian traffic. Transverse spacing of bars is a far more critical factor with regard to safety. Clear performance differences in terms of bicycle and pedestrian safety are noted between grates having transverse bar spacings over 9 inches (poor bicycle safety performance) and those having closer spaced transverse bars (better bicycle safety performance).

A grate's hydraulic efficiency is improved by a wider spacing of the longitudinal bars. Its ability to handle clogging is also most dependent on longitudinal bar spacing.

It is seen, therefore, that a compromise must be reached to optimize both hydraulic and safety criteria.

These recommendations are based on research conducted for the Federal Highway Administration and published in Bicycle Safe Grate Inlet Study, Volumes 1, 2, and 3. (8, 9, 10)

To ensure maximum bicycle and pedestrian safety, the grated inlet should not have longitudinal bars spaced further apart than 1-1/2-inches and transverse bars should not be spaced further apart than 9 inches. (This spacing allows a bicycle wheel to fall 1 inch across the bar grate and this is similar to normal road roughness). The Neenah Foundry Company details grates that are bicycle safe and review has determined that grates designated collectively as R-3246 (Type C grate only) are acceptable to both hydraulic and safety optimized designs.

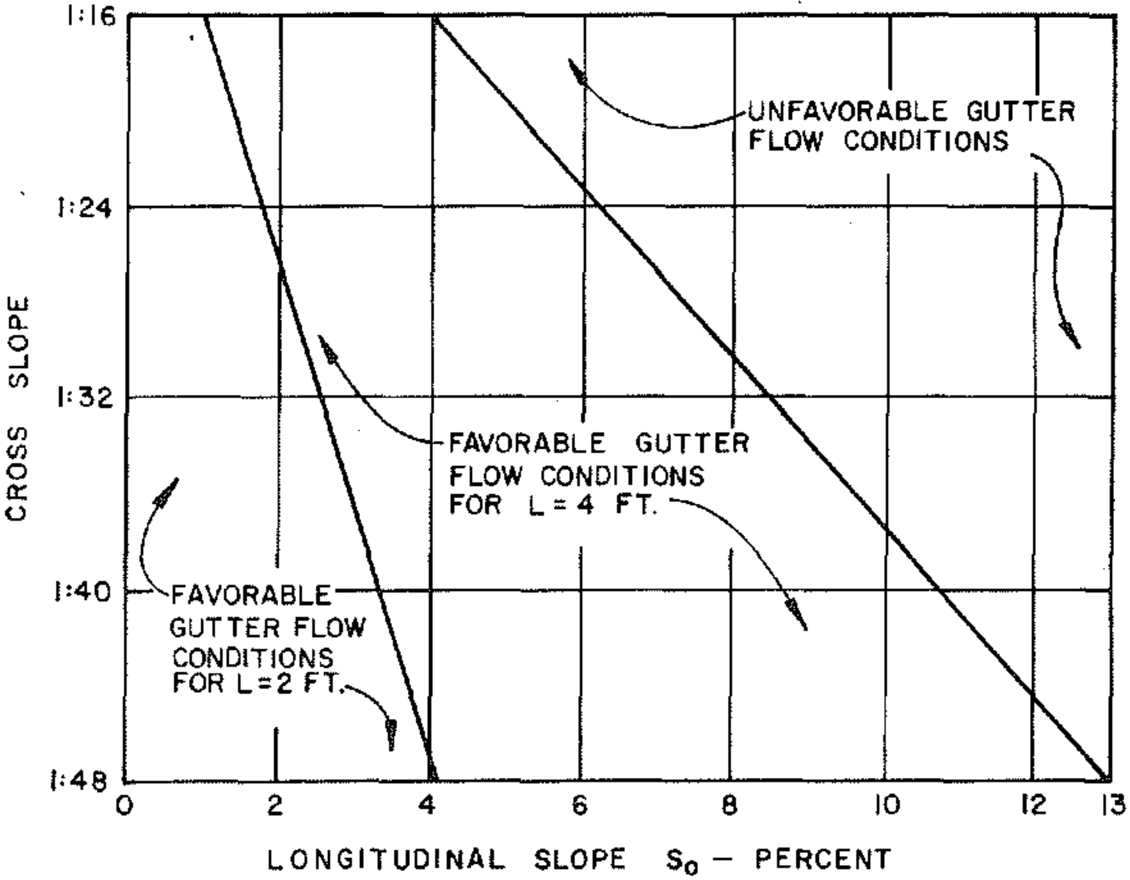


FIGURE III - 4

FAVORABLE AND UNFAVORABLE GUTTER FLOW CONDITIONS
FOR COMBINATION INLETS OF LENGTH, L

SOURCE: Bicycle-Safe Grate Inlet Study, Volume I - F.H.W.A.

USE OF INLETS

The following general recommendations are made for the utilization of different types of stormwater inlets.

Sump Conditions

- True Sump. The use of depressed curb opening inlets is recommended. Each true sump should be reviewed to determine if the area affected by ponding is within acceptable limits.

- Sumps Formed by Crown Slope of Cross Section at Intersection. The use of curb opening inlets is recommended, though combination inlets may be successfully utilized. A small amount of ponding may cause storm runoff to flow over the crown of the cross street and continue down the gutter.

Continuous Grade Conditions

Except as permitted by the City Engineer, combination inlets should be used on continuous grades.

Shallow Overland Flow Conditions

Except as permitted by the City Engineer, under certain conditions, slotted drains may be utilized.

Allowable Inlet Capacities

The following reduction factors should be applied to the theoretical calculated capacity of inlets based upon their type and function. The reduction factors compensate for effects which decrease the capacity of the inlet such as debris plugging, pavement overlaying, and in variations of design.

TABLE III-1
REDUCTION FACTORS TO APPLY TO INLETS

Condition	Inlet Type	% of Theoretical Capacity Allowed
Sump	Curb Opening	80
Sump	Grated	50
Sump	Combination	65
Continuous Grade	Curb Opening	80
Continuous Grade	Deflector	75
Continuous Grade	Longitudinal Bar Grate	75
	incorporating recessed transverse bars	60
Continuous Grade	Combination	110% of that listed for type of grate utilized.
Shallow Overland Flow	Slotted Drains	80

The allowable capacity of an inlet should be determinedby applying the applicable factor from Table III-1 to the theoretical capacity calculated in accordance with the appropriate design charts.

The percentage of theoretical capacity allowed may be even lower when the inlet is likely to intercept large amounts of sediment or debris. For instance, the first inlet to a pipe network draining a high debris-yielding area may actually accept only 20 percent of the theoretical capacity allowed because of clogging. Sediment traps will not be designed into the inlet box. A sediment trap formed by lowering the floor of the inlet box below the elevation of the outlet pipe is unnecessary and undesirable since there is too much turbulence for effective trapping, and cleaning is costly. Inlet boxes should be self-scouring, even under low-flow conditions.

DESIGN OF CURB OPENING INLETS

A curb opening inlet may operate under two different conditions of flows:

- Free flow, in which a free water surface is continuous into the inlet and the inlet acts as a weir, or
- Submerged conditions, in which the inlet acts as an orifice.

The following design procedures assume that the inlets will be designed to operate under the free flow conditions.

Figure III-5 is presented for use in designing curb opening inlets in the sump condition with a depression depth of 2 inches. This chart is an adaptation of a Federal Highway Administration chart and is applicable to both the free flow and the submerged cases.

For the average sump conditions, the presence of a vertical curve in the street will cause the grade to gradually become shallower as the inlet is approached. This shallow grade may cause the flow depth in the approach gutter to be greater than depth, y_o, read from Figure III-5. The street grade of s = 0.002 will generally occur within 30 feet of the sump and the uniform gutter flow water depth should be checked for this slope. If the value of y_o read from this Figure is less than y_o for $s = 0.002$,

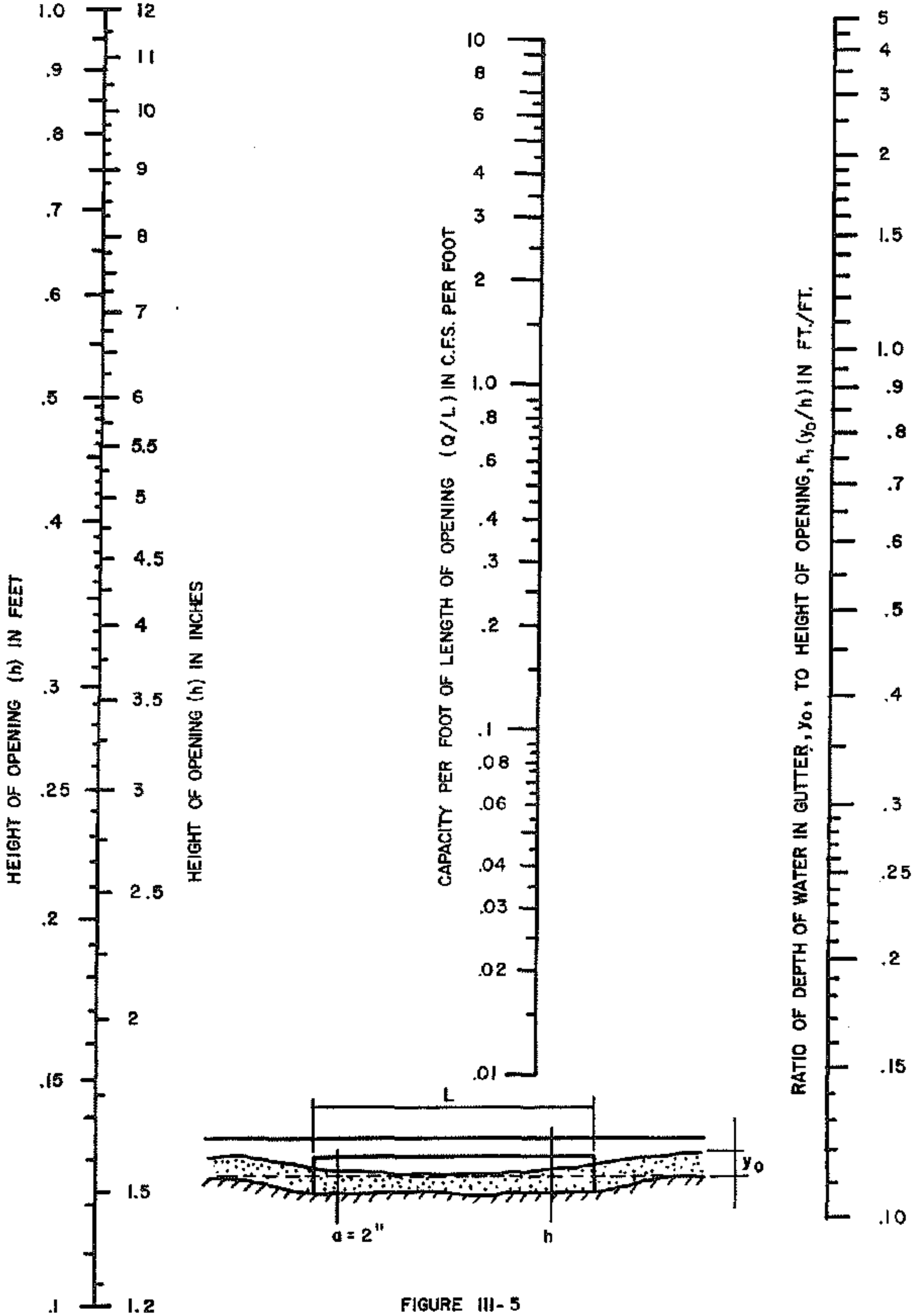


FIGURE III-5

NOMOGRAPH FOR CAPACITY OF CURB OPENING INLETS IN SUMPS, DEPRESSION DEPTH 2"
(FROM FHWA)

flow will tend to draw down as it approaches the inlet. On the other hand, if sump depth is greater, then the pool backs up water along the gutter. To facilitate this check, a curve of Q_o versus y_o for a slope of 0.002 should be developed for the street cross section being studied.

Even though an inlet is designed to operate under free flow conditions for the design frequency runoff, say 2 years, it may be desirable to compute the capacity of the inlet under the increased head which might result from the runoff generated by a 5-year storm when the inlet is located in a sump area where flooding would occur. For a relatively small increase in cost, the connector pipe might then be increased in size to accommodate the additional capacity of the inlet due to increased head. This approach may be particularly useful when a specific street has a design frequency higher than that of the surrounding area.

The following design charts for curb opening inlets on continuous grade, Figure III-6 (i), (ii), and (iii) are presented for the standard depression configuration as utilized by the City of Stillwater.

The curve applies only to free flow (not submerged) at the curb opening. Therefore, as a standard part of each design, the engineer should verify that the free flow condition will exist by generally making h greater than or equal to y_o.

The procedure for utilizing the curve is as follows:

1. Compute the gutter flow spread in feet for the particular Q_o and street geometry involved.
2. Enter the figure, with the gutter flow spread computed under Step 1.
3. Extend a vertical line from the width of flow spread to the line representing the street grade, extend a horizontal line from this point to the curve representing the crown slope of the street being studied, and extend a vertical line from this point to the inlet interception rate.
4. Multiplying the inlet interception rate times Q_o in the gutter results in the quantity of flow entering the curb opening inlet.

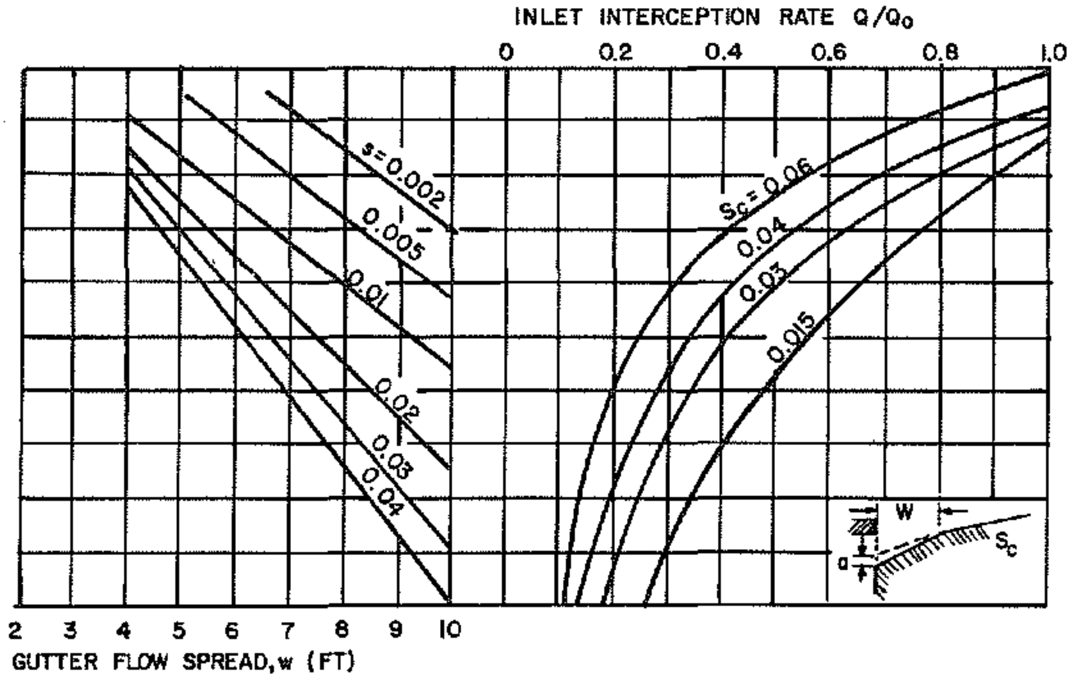


FIGURE III-6(l)

CAPACITY OF CURB OPENING INLET ON CONTINUOUS GRADE
$W = 1\ 1/2$ FT., $a \geq 2$ IN., $L = 5$ FT., $h \geq y_0$
(FROM FHWA)

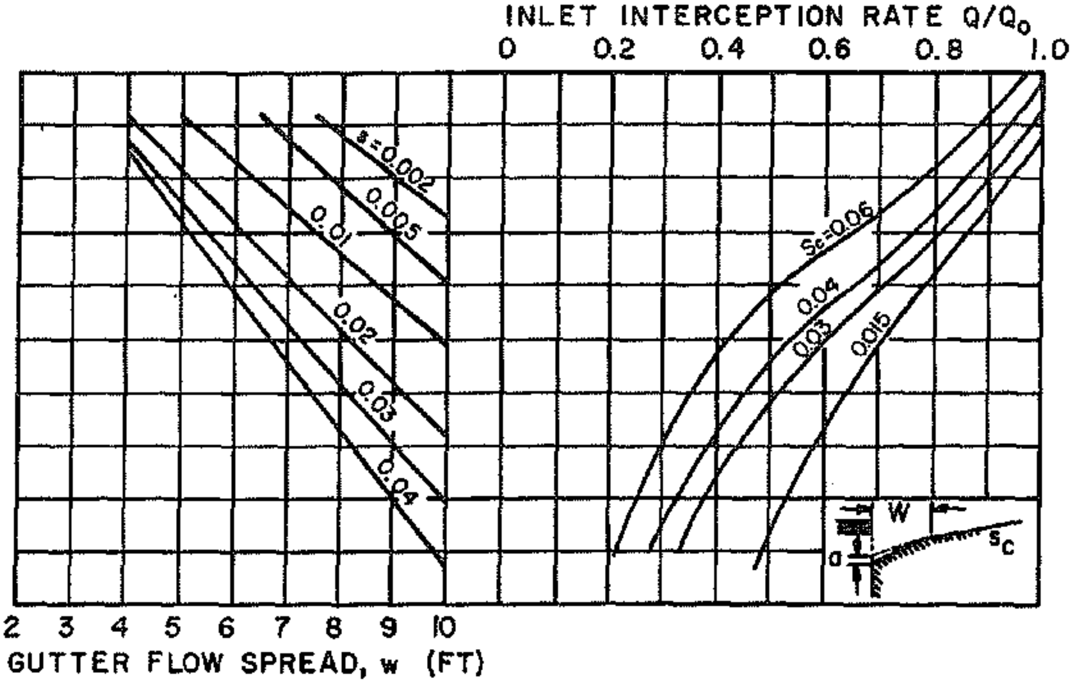


FIGURE III-6(ii) CAPACITY OF CURB OPENING INLET ON CONTINUOUS GRADE W=1½ FT., a≥2 IN., L=10 FT., h≥y_o
(FROM FHWA)

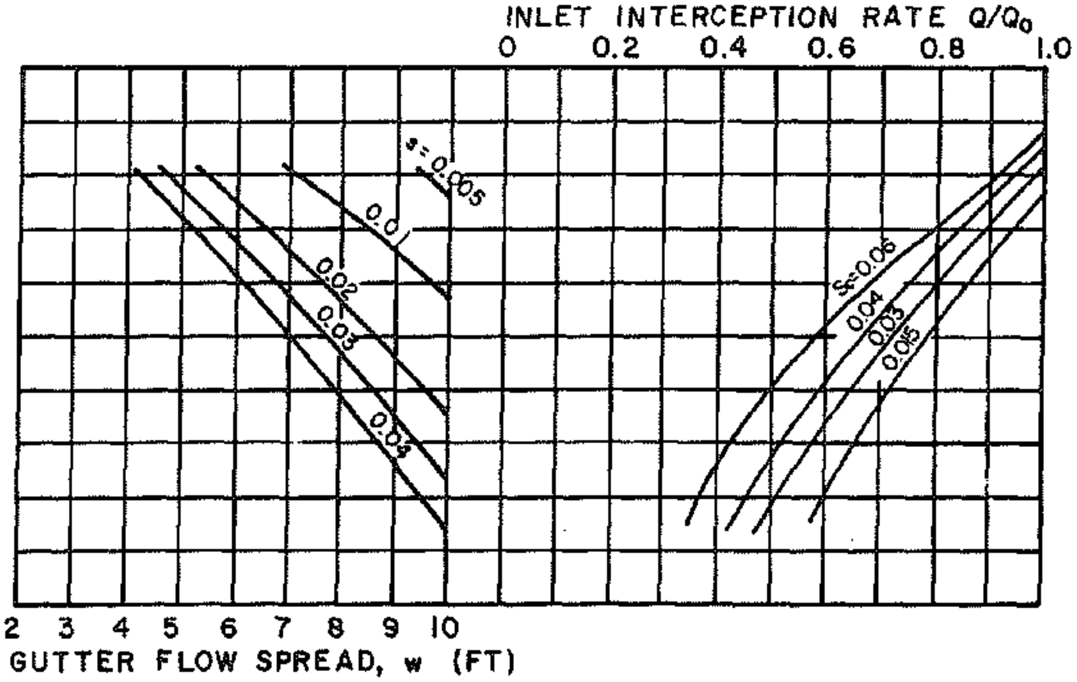


FIGURE III-6(iii) CAPACITY OF CURB OPENING INLET ON CONTINUOUS GRADE W=1½ FT., a≥2 IN., L=15 FT., h=≥y_o
(FROM FHWA)

5. The height of the curb opening inlet should be greater than the depth of the flow at the curb face to insure that the opening will act under free flow conditions.

Figure III-7 illustrates a curb opening on a continuous grade. The capacity of a curb opening inlet on a continuous grade is a function of how quickly the water can change its direction of flow from paralleling the curb to flowing toward the curb as it moves through the inlet. The change in direction of flow is primarily caused by the crown slope of the street. Therefore, increasing the effective crown slope of the street by utilizing a depression can significantly increase the capacity of the curb opening inlet on a continuous grade. The figures presented for calculation of capacity of curb opening inlets on continuous grades are for the general case. Local conditions may dictate that a depression shape other than that recommended will be necessary.

DESIGN OF GRATED AND COMBINATION INLETS

General

The design procedure presented in the following section is based upon the assumption that the grated inlet is clear from debris and is operating at its maximum efficiency. This is seldom the case for a grated inlet under operating conditions so the reduction factors of Table III-1 should be applied.

When located on a continuous grade, an inlet can only accept 100 percent of a very small flow. Above this value a portion of the flow by-passes the inlet in question and continues on. For this reason, the charts are in terms of the ratio of flow passing through the inlet to the flow in the street. This ratio is referred to as the interception rate of the inlet.

Sump Conditions

Under sump conditions a grated inlet acts essentially as a series of orifices once the depth of water is sufficient to submerge the grate as well as the curb opening. At lesser depths the grated inlet will act as a weir and the capacity may be determined by applying any of the accepted weir formulas. Most sump conditions will operate with design depths above the top of the curb opening. Tests show that the application of the orifice

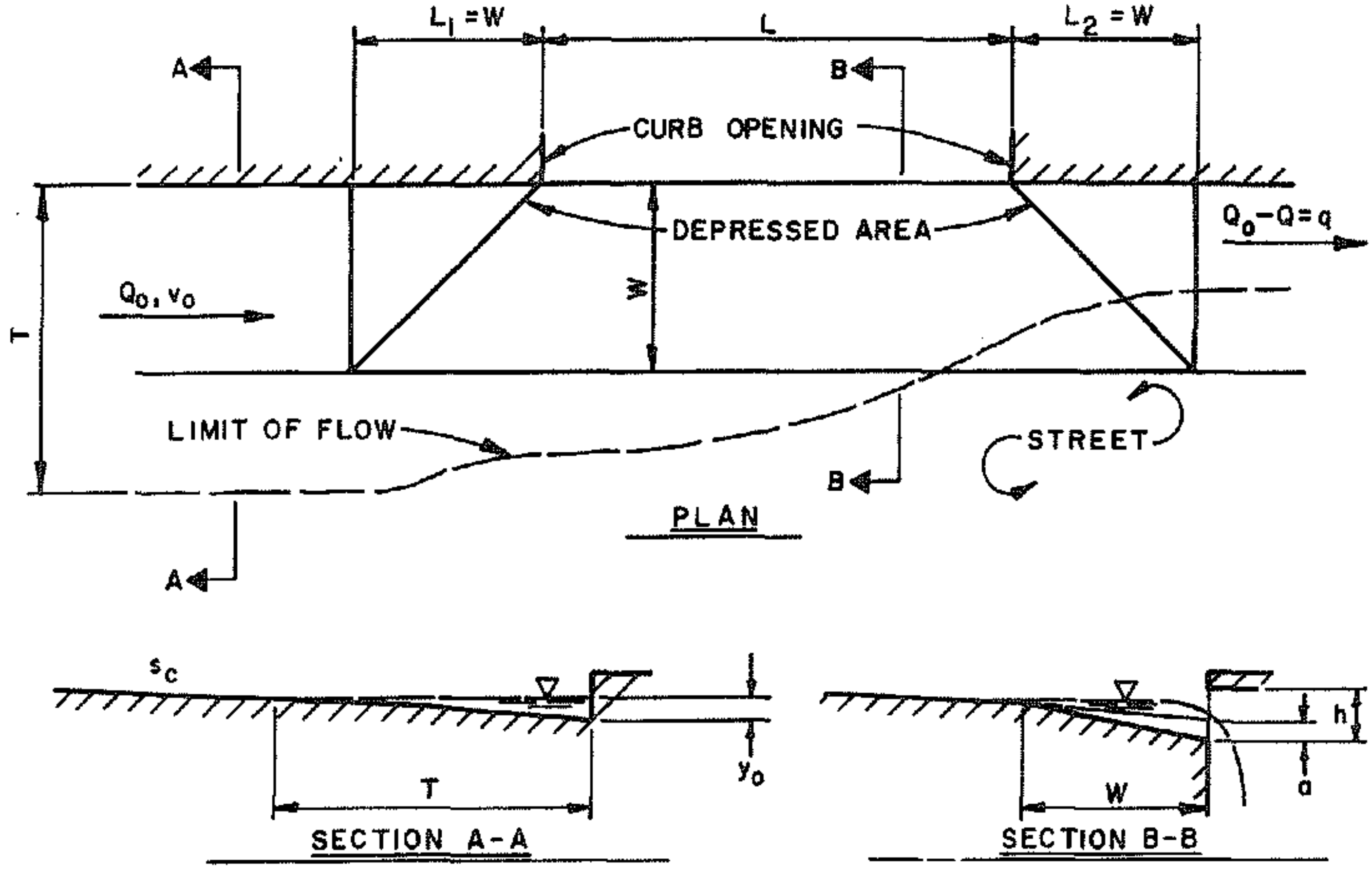


FIGURE III-7 CURB OPENING INLET FOR DESIGN CHARTS formula to the clear opening of the inlets under most operating conditions gives a good indication of the capacity of a clean inlet. Figure III-8 can be used to determine the capacity of a particular inlet. The head used should be determined by the allowable depth of ponding for the installation at the design storm frequency.

For combination inlets, the figures show the theoretical capacity to be the total of the grate capacity and the curb opening capacity at the design water depth. Hydraulic limitations are obvious in the assumption, but it offers the best design procedure available.

If it is impossible to utilize a curb opening inlet for a sump condition, such as in a driveway area to a filling station that exists in a sump, a grated gutter inlet would normally be utilized.

For the sump condition, the use of a longitudinal bar grate apparently does not significantly reduce the tendency of the grating to plug. The openings must be made as large as possible to allow trash to flow through the openings, but not so large that they pose a safety hazard to pedestrians and bicyclists.

Continuous Grade

The design method used to compile the capacity charts, Figures III-10 and III-11, is based on comprehensive research. The method is utilized here because it represents the best values obtainable at the present time and is based on the conditions illustrated in Figure III-9. The charts can be used to determine the capacity for the recommended inlet type (Neenah R 3246 and R 3246-1 with grate type C).

The design charts were derived for use in the most common situations encountered in Stillwater. The street, gutter, and inlet configurations assumed in the analysis are as described in this Manual in the relevant sections. A Manning's n value of 0.0016 has been used in the computations. The application of the figures is limited to those cases matching the design configurations. Above the dashed lines on the figures, carryover occurs across the surface of the grate and the figures are not applicable.

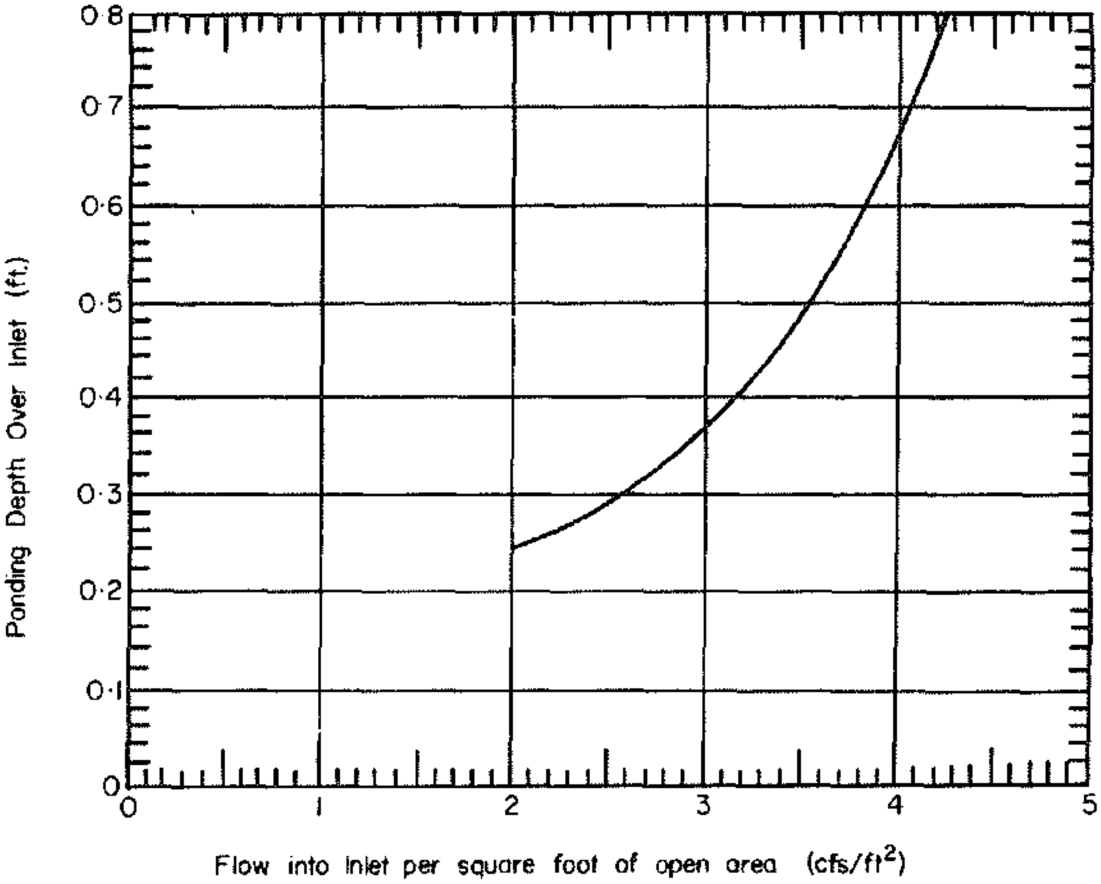
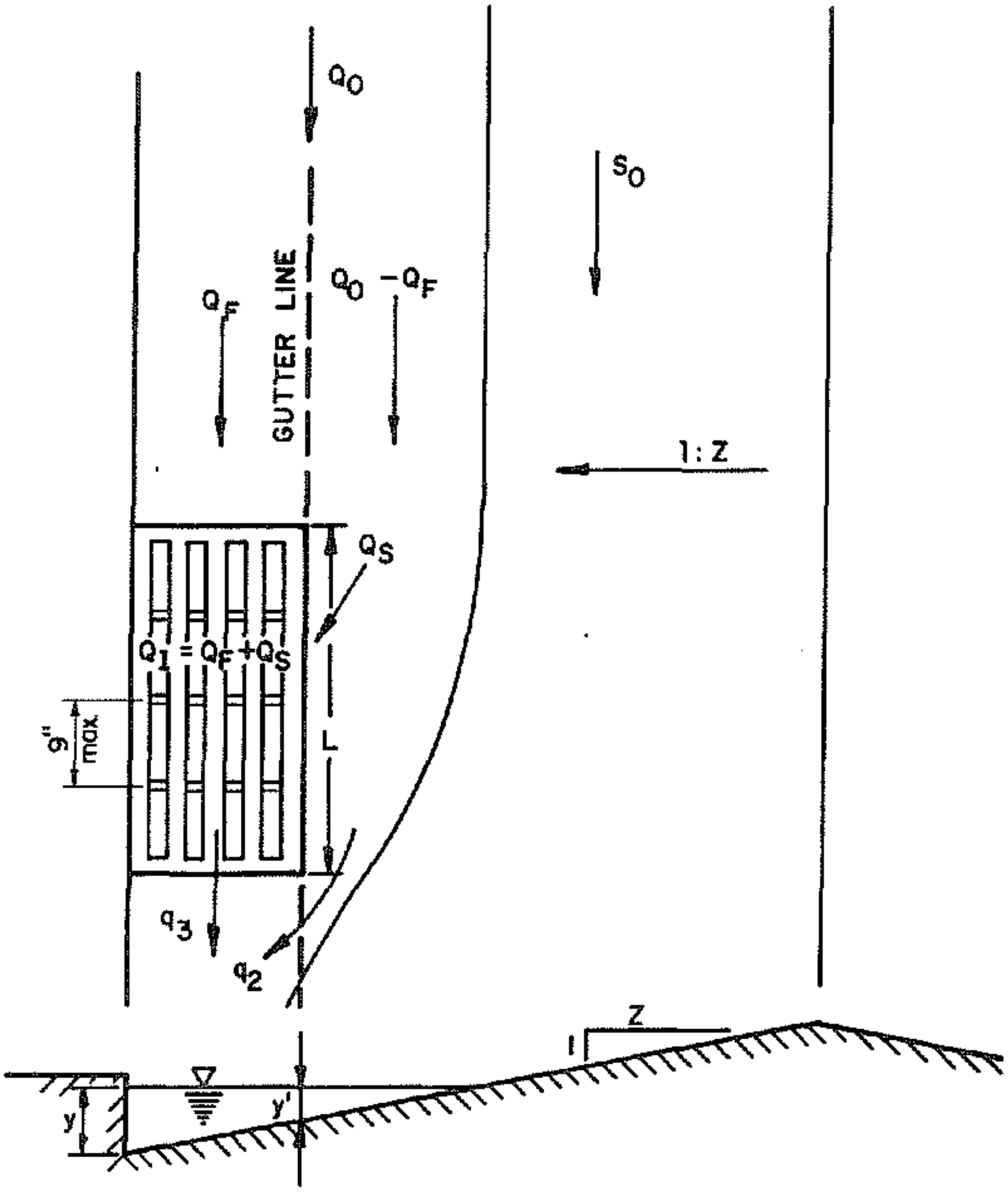


FIGURE III-8 CAPACITY OF GRATED INLET IN SUMP

NOTE: For ponding depths greater than 0.8 feet apply formula:

$$Q = 0.61\ A\sqrt{2gH}^{*}$$

*Source: Water Measurement Manual
Bureau of Reclamation, 1974.



DEFINITION SKETCH OF GUTTER FLOW NEAR AN INLET

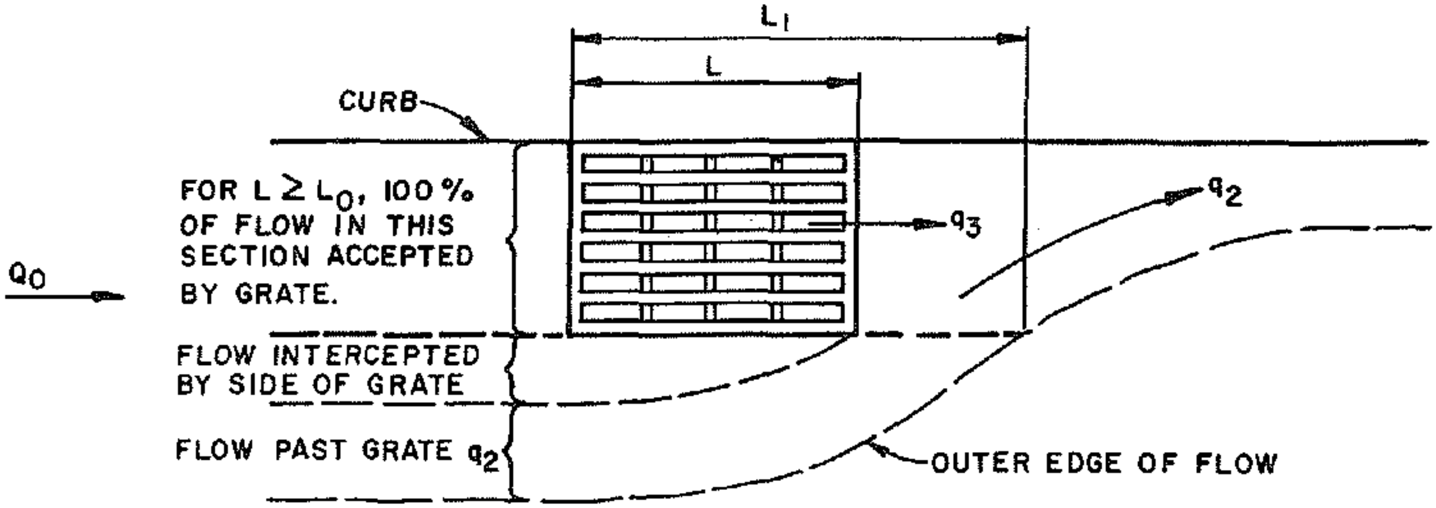
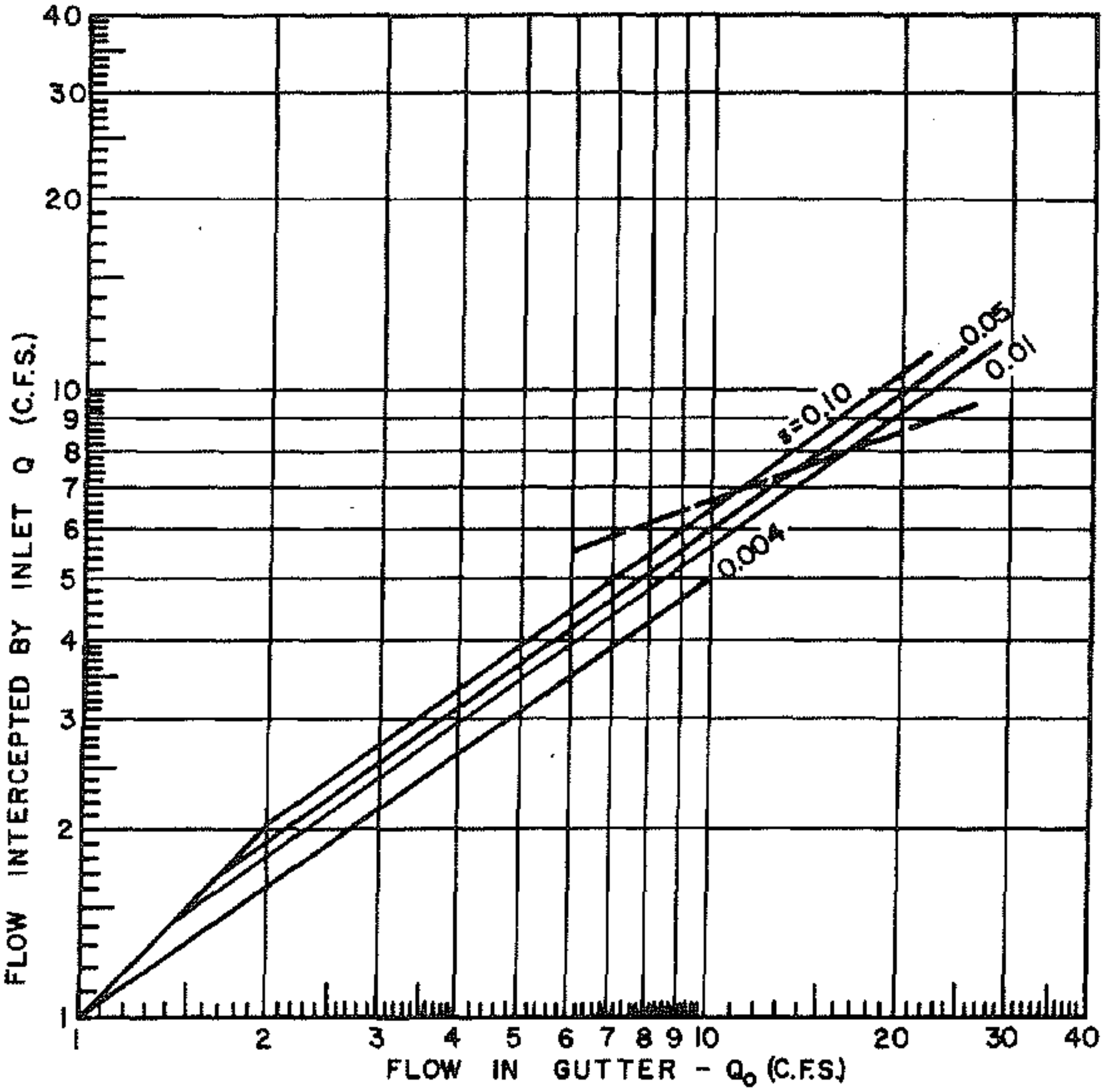


FIGURE III-9 PLAN OF GRATED INLET SHOWING FLOW LINES



NOTE: ABOVE DASHED LINE, CARRYOVER OCCURS ACROSS SURFACE OF GRATE

FIGURE III-10

CAPACITY CHART GRATED BAR COMBINATION INLET

L = 3ft., W = 1.5 ft., a = 2in., S_c = 0.02

(e.g. Neenah Foundry Company, Grated Inlet Type R-3246-A Grate Type C).

NOTE: ABOVE DASHED LINE, CARRYOVER OCCURS ACROSS SURFACE OF GRATE

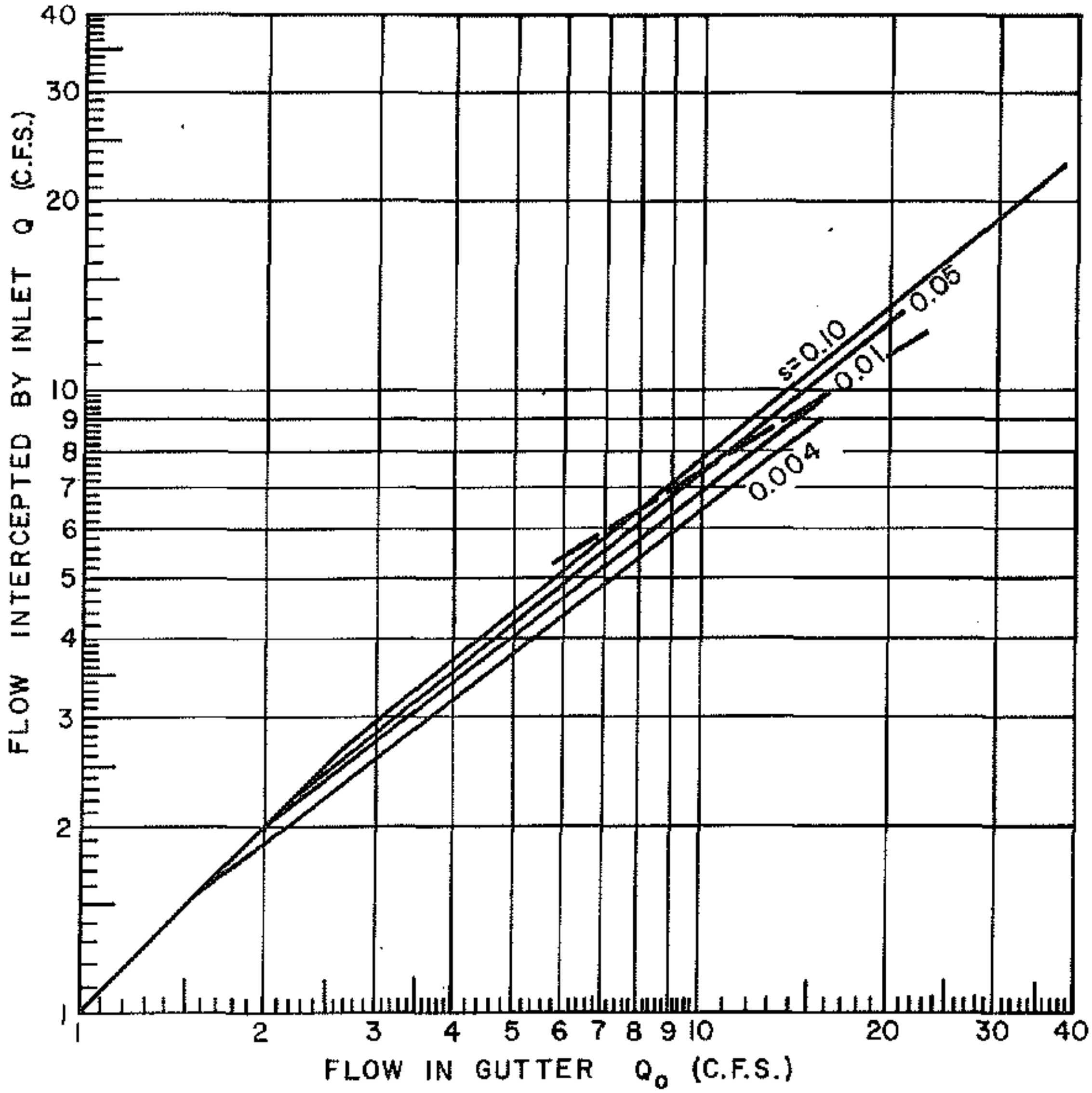


FIGURE III-11

CAPACITY CHART GRATED BAR COMBINATION INLET

L=3 ft., W=1.5 ft., a = 2 in., S_c =0.04

(e.g. Neenah Foundry Company, Grated Inlet Type R-3246-A Grate Type C).

The amount of direct carryover may be estimated by the equation

$$q_3 = Q_o \left[1 - \frac{L^2}{Lo^2}\right]^2 \qquad \text{Eq. III-1}$$

where q_3 = carryover directly over the grate (cfs)

Q_o = flow in gutter (cfs)

L = length of inlet (feet)

L_o = length of grate required to eliminate carryover directly across the inlet =

$$mv \sqrt{\frac{y}{g}} \qquad \text{Eq. III-2}$$

m = value given in Table III-2

v = velocity of flow at upstream end of grate (fps)

y = depth of flow at upstream end of grate (ft)

g = acceleration due to gravity

TABLE III-2

VALUES OF m FOR VARIOUS GRATING CONFIGURATIONS

Inlet Description	m
Combination inlets with bar width equal to or slightly less than the clearance between bars, and no large transverse bars flush with the surface.	3.3
Combination inlets with a bar width equal to or slightly less than the clearance between bars, and with several transverse bars.	6.6
Grated inlet without curb opening and with bar width equal to or slightly less than the clearance between bars, and no large transverse bars flush with the surface.	4.0
Grated inlet with a bar width equal to or slightly less than the clearance between bars, and with several transverse bars flush with the surface.	8.0

The carryover flow outside the grate (q_2) may be found:

$$q_2 = 0.25 \ (L^1 - L) \ (g)^{1/2} \ (Y \ )^{\ 3/2} \qquad \text{Eq. III-3}$$

where L^1 = length of inlet required to eliminate carryover flow outside the inlet (ft)

$$= 1.2 \ v \ \tan \theta \ \frac{y}{g}^{\ 1/2} \qquad \text{Eq. III-4}$$

$\tan \theta$ = tangent of the angle between the depression slope and vertical where the cross-slope changes beyond the inlet.

y_1 = depth of flow at outside edge of inlet (ft)

Therefore, the total flow bypassing the inlet is given by $q_2 + q_3$. Using equations (III-1) and (III-3) and Figures III-10 and III-11, the designer may now calculate the number of inlets required to intercept a certain percentage of street flow.

Example. Intercept 100 percent of storm runoff on a local street before it reaches an urban arterial street.

Street cross slope $s_c = 0.002$

Street longitudinal slope $s = 0.05$

Gutter Flow Q_o = 10 cfs

Inlet dimensions = 18" x 36"

Depression depth, a = 2"

Type of inlet = combination (m = 3.3)

How many inlets are required?

Solution. From Figure III-10, 6 cfs is theoretically intercepted. Apply the reduction factor from Table III-1 to get the actual interception for a combination inlet on a continuous grade = 0.66 x 6 = 4 cfs.

The design point is below the dashed line on Figure III-10 so there will be no carryover directly across the inlet.

Downstream from inlet, the gutter flow is now 6 cfs. From Figure III-10, 4 cfs is theoretically intercepted. Apply the reduction factor to get actual interception = 0.66 x 4 - 2.6 cfs.

$\therefore$ downstream from inlet 2, $Q_o = 3.4$ cfs

A total of four combination inlets, 18-inch by 36-inch, are required to intercept 100 percent of flow.

DESIGN OF SLOTTED DRAIN INLETS

An identified need to pick up sheet flow without the use of expensive and hazardous berms or dikes led to the innovation of slotted drains. The basic material is corrugated steel pipe with a slot opening incorporated in the crown. This type of pipe features continuous drainage at surface level and is, therefore, effective in minimizing the ice hazard caused by ponding, in reducing the chances of clogging by runoff-carried debris, providing an easily maintained minor drainage system and providing the designer with a practical, aesthetically pleasing solution to the disposal of surface water runoff in a variety of applications. Slotted drains can prove to be the most economical solution for certain drainage problems because of the low installation costs associated with shallow excavations and minimal backfill.

The principal application for slotted drains include:

- continuous inlets in medians, shoulders, and parking lots
- areas where sheet flow needs to be intercepted.

Where the safety of pedestrians, motorcycles, or bicycles is a concern, the designer should be careful to address the solution with careful detailing.

When slotted drain is installed in areas of heavy pedestrian traffic, expanded wire mesh is attached across the top of the drain opening. This mesh is welded directly to the grating and prevents shoe heels from being caught in the open slot. When two-wheel vehicular traffic safety should be ensured, bar spaces should be placed at the bottom and top of the drain opening. The top bar spacer should be placed flush with the surface. In ordinary circumstances, only one bar spacer at the bottom of the opening would be used.

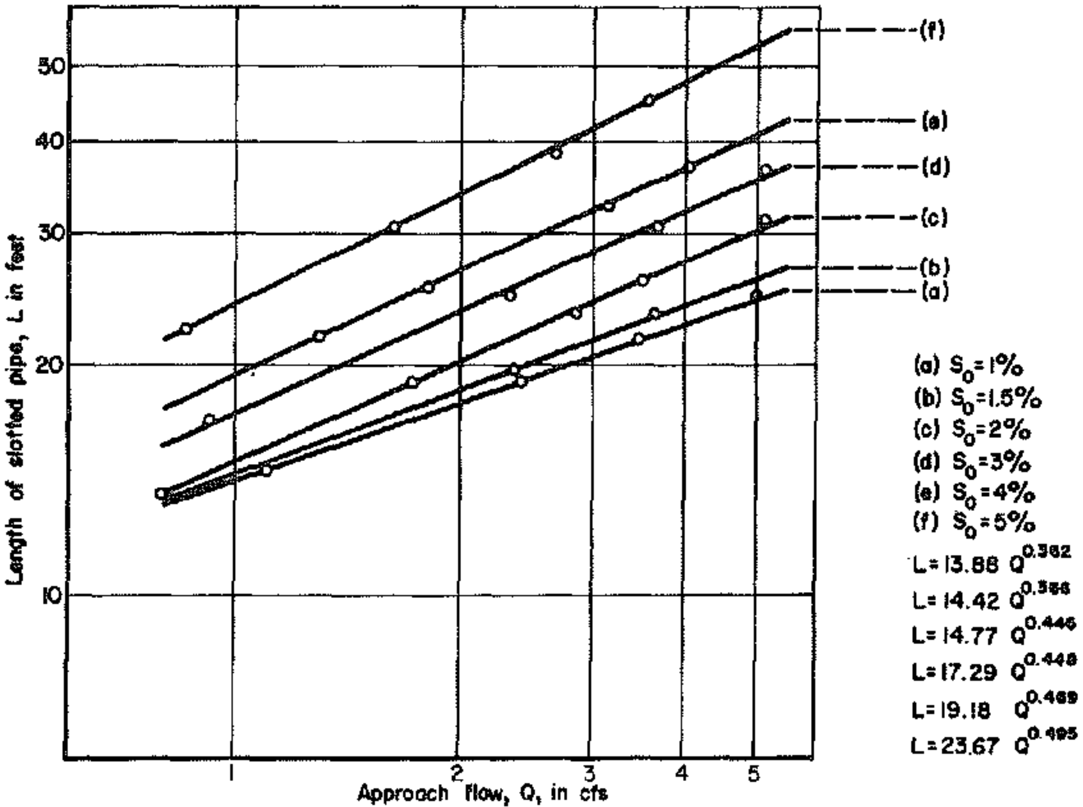
The most important feature of slotted drain is its ability to pick up runoff in an efficient manner. As a general rule, 40 feet of 18-inch slotted drain (1-3/4-inch drain opening) will collect as much water as two 36-inch drop inlets.

In the design of slotted drains, two types of flow have been identified, depending on the depth of water at the inlet. If the water flows at a depth of less than 0.2 feet, the drain acts as a weir and follows regular formulas for weir-type flow. If the water flows at a depth of more than 0.2 feet, however, the drain acts as an orifice and follows regular formulas for orifice-type flow.

The problem of interception capacity has been investigated by California Department of Transportation and the experimentral results compare favorably with theoretical predictions.

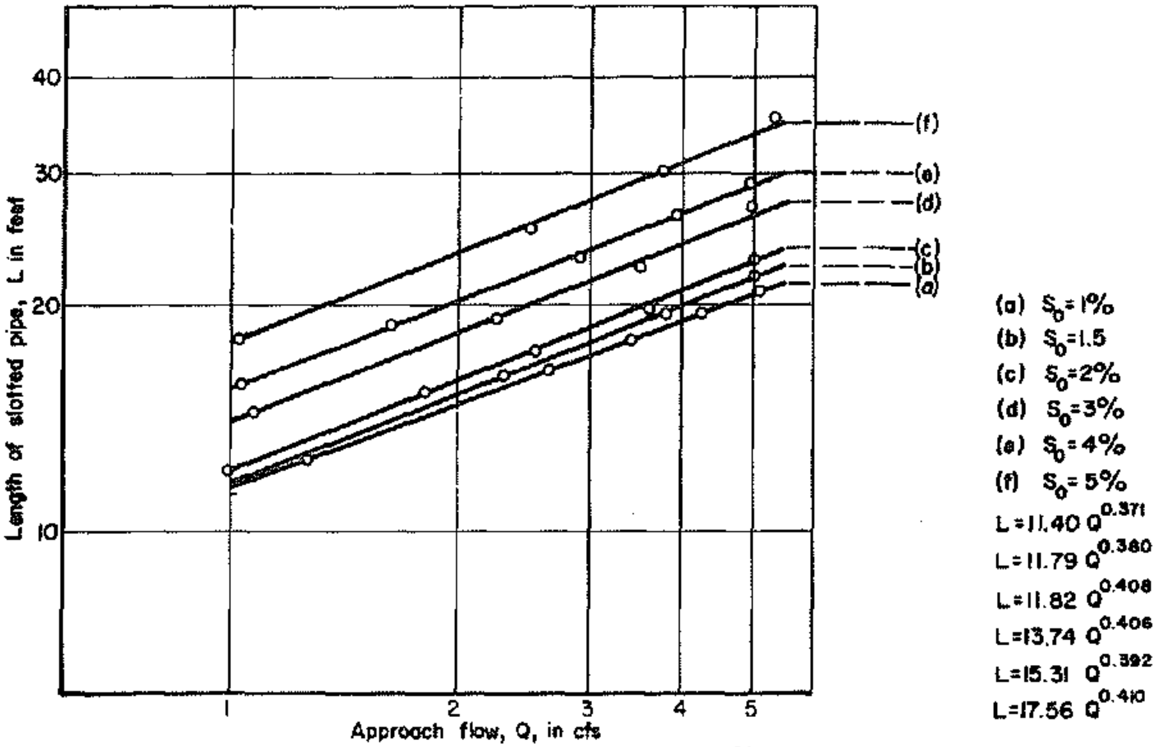
The charts shown in Figures III-12, III-13, III-14, and III-15 can be used to determine the capacity of a given length of slotted drain for various longitudinal slopes, S_o, and cross slopes, S_c. It is seem that as the cross slope increases, the length of slotted drain required to intercept a given discharge will decrease.

Care must be taken to carefully specify the correct construction practice to ensure the adequate performance of the system. Proper invert grades for slotted drain can be determined from Figure III-16. A common method of specifying installation practice is to shape the bedding on which the pipe is to be laid and backfill with a lean grout. Because of the small area, the use of lean grout is no more expensive than native material. Lean grout permits the pipe to build up side support and act in ring compression, making possible installations in areas subject to occasional wheel loadings.



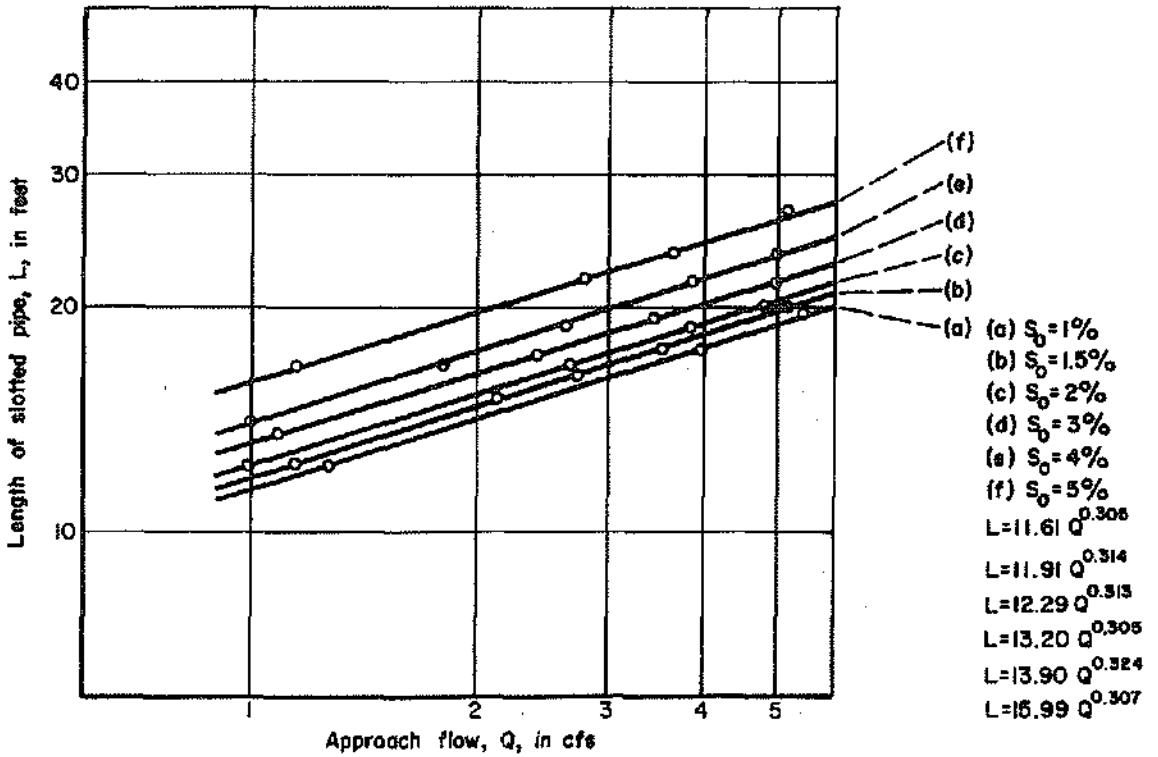
LENGTH OF PIPE VS. APPROACH FLOW, S_c=2%

FIGURE III-12



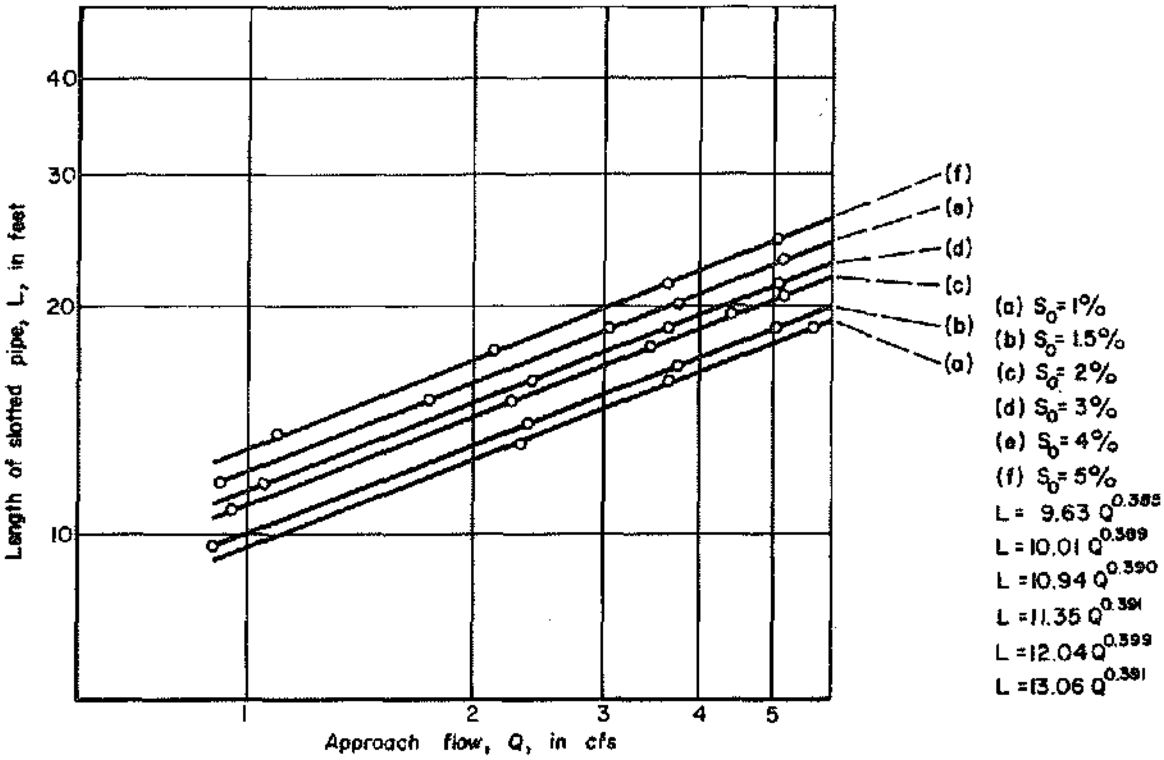
LENGTH OF PIPE VS. APPROACH FLOW, S_c=3%

FIGURE III-13



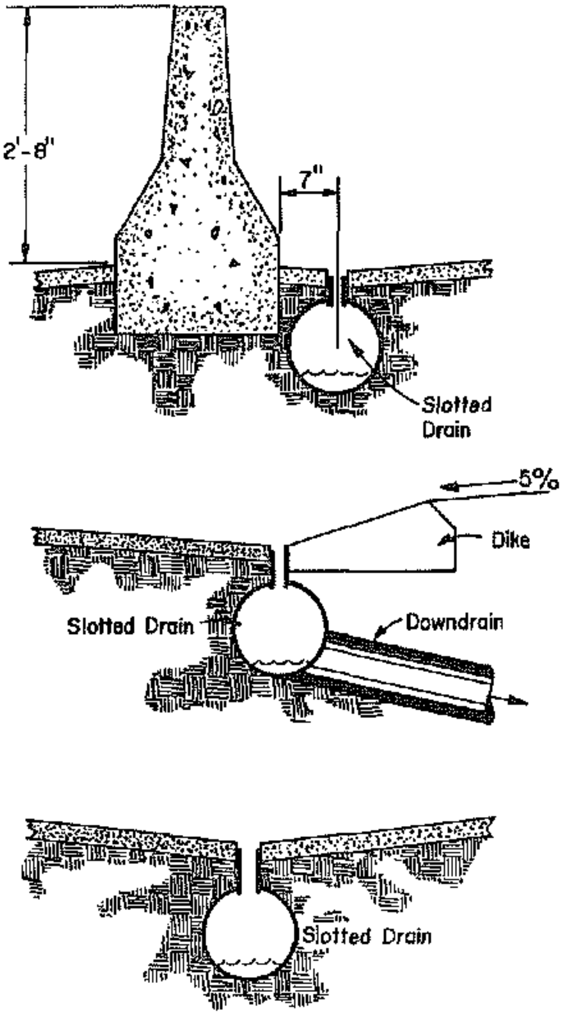
LENGTH OF PIPE VS. APPROACH FLOW, S_c = 4%

FIGURE III-14

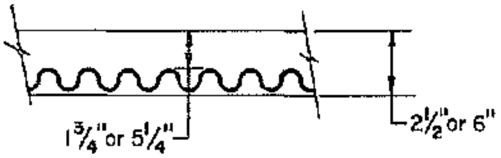


LENGTH OF PIPE VS. APPROACH FLOW, S_c = 5%

FIGURE III-15



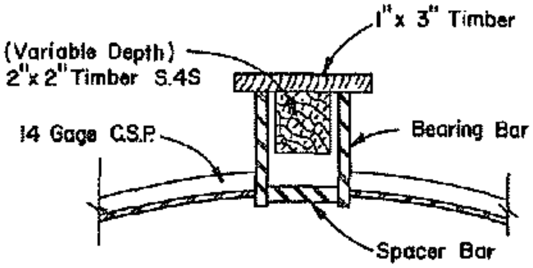
TYPICAL SLOTTED DRAIN INSTALLATIONS



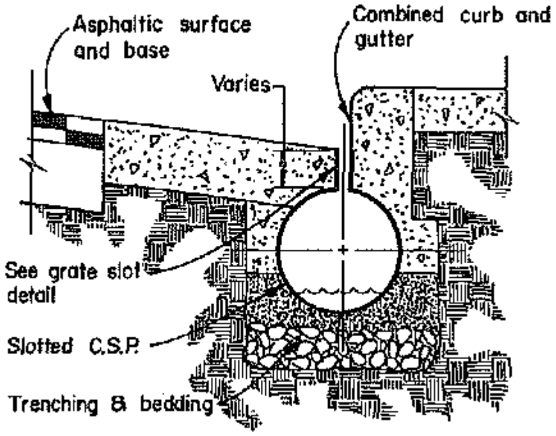
PROFILE SLOT GRATE

DIMENSION 'A' (in feet)

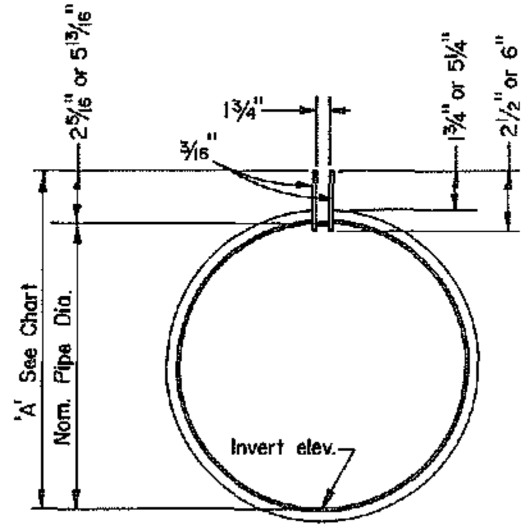
DEPTH OF GRATE	NOM. DIA. OF PIPE				
	12"	15"	18"	24"	30"
2½"	1.19'	1.44'	1.69'	2.19'	2.69'
6"	1.48'	1.73'	1.98'	2.48'	2.98'



GRATE SLOT DETAIL (typical)



TYPICAL SECTION



TYPICAL SECTION FOR DETAILING

FIGURE III-16

SLOTTED DRAINS

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FINAL HYDRAULIC DESIGN OF STORM SEWER PIPELINES

CHAPTER IV
FINAL HYDRAULIC DESIGN
OF
STORM SEWER PIPELINES

This Chapter is intended to cover the hydraulic design of storm sewer pipeline systems. The preliminary design phase preceeds the methods described in this Chapter and is generally described in Chapter IV of Part I, "Recommended Design Techniques and Drainage Considerations."

A specific method is described in Chapter I of Part II, "Hydrology." Computer programs are also available which are suitable for preliminary design and these programs, or the preliminary design techniques described in Chapter I, can be effectively used to choose the type and extent of the system and to determine pipeline sizes and costs for basic decision making. However, the steps outlined in this Section are critical to insure that runoff is managed so as to eliminate unplanned flooding problems and hazards.

It should be noted that while this Chapter is intended to cover the final hydraulic design of the storm sewer segment of the minor drainage system, the technical aids may also be used for hydraulic design of closed-conduit segments of the Major Drainage System. The technical information of this Chapter does have limitations and should not be used outside the allowable range of application for major drainage system components without judicious thought as to the physical conditions which will occur.

In addition to those chapters previously listed, this Chapter is also supported by Chapter III, "Storm Water Inlets" and Chapter II, "Streets, Curbs and Gutters." As shown in Chapter IV of Part I, "Design Procedure," the procedures and information contained in this Chapter are utilized once a basic system has been selected and the runoff rates have been determined. Many designers never proceed to this level and rely on rules of thumb approaches to prepare final design and drawings. For reasons which will be described later, this approach is not sufficiently detailed for use in Stillwater.

Often a closed conduit designed for open channel flow operates as a pressure conduit. This may result when storm runoff exceeds that used for design purposes or simply because junction losses were ignored in the design. In storm drain systems, it is found that junctions in closed conduits can cause major losses in the energy grade line across the junction. If these losses are not included in the hydraulic design, the capacity of the conduit may not be as large as that required for the design flow.

Even though a conduit may be designed to carry stormwater as open-channel flow, losses at bends and junctions will frequently cause pressure flow to occur for some distance upstream of the "loss" area. Situations may occur in steep terrain where the flow often interchanges between open channel and pressure flows. Because it is not economical to size conduits to avoid pressure flow under all storm runoff and flow conditions, it follows that it is reasonable and even necessary to design the conduits as flowing full. Planned management of stormwater runoff is also easier to achieve if the hydraulic grade line is kept higher than the crown of the conduit. The discharge through a circular pipe flowing full is constant for a given pipe diameter and hydraulic gradient. Once that discharge is reached in the pipe, no further runoff can be admitted to the pipe network if the hydraulic grade has risen to the elevation of the inlet. This procedure also allows for minimizing the capital expenditure required for a specified level of protection.

This phenomenon in the field would be evidenced by runoff passing directly over the inlet to flow down the street (or overland) until it enters the system elsewhere. Another indication is water standing in sumps (detention ponding) until there is sufficient capacity in the stormwater to admit the ponded water. If the designer deliberately sizes the pipes so that the hydraulic grade line is at or very near the inlet elevation, he has provided an "automatic valve" that will stop extra runoff entering the pipe network and cause unforeseen problems at other locations in the system.

Although not always feasible, the recommended procedure is to design storm sewers to flow under pressure. Whether or not the final design is made with the pipe flowing partially or completely full, the hydraulic grade line should be computed and displayed on a profile drawing of the conduit. Because this frequently has not been a design practice, stormwater has often been found shooting out of inlets or popping manhole covers. This situation arises when the hydraulic grade line rises above ground level and can lead to needless damage and inconvenience to pedestrian and vehicular traffic.

The general procedures for establishing quantities of flow and horizontal layout are the same for closed conduits flowing either as open channels or as pressure conduits. Because of the nature of hydraulic elements in circular conduits, it may be reasonably assumed that open channel flow will occur only when the flow depth is less than 80 percent of the conduit diameter. Once criteria have been set, computations may be made to size the conduits and the various appurtenances.

For minor drainage systems, the designer should size the pipes to carry runoff from the initial storm. This storm will have a design frequency of between 2 and 10 years depending upon the protection the City needs to afford the surrounding land uses. Once this runoff has been admitted to the pipe network, the excess runoff can be carried by surcharge in the street to a level of encroachment allowed by design criteria for the 100-year storm, or major storm. If this street encroachment criteria is exceeded for the 100-year storm, then the major drainage pipe sizes should be increased to carry the extra runoff and the runoff will flow as open-channel flow in the network during the initial storm.

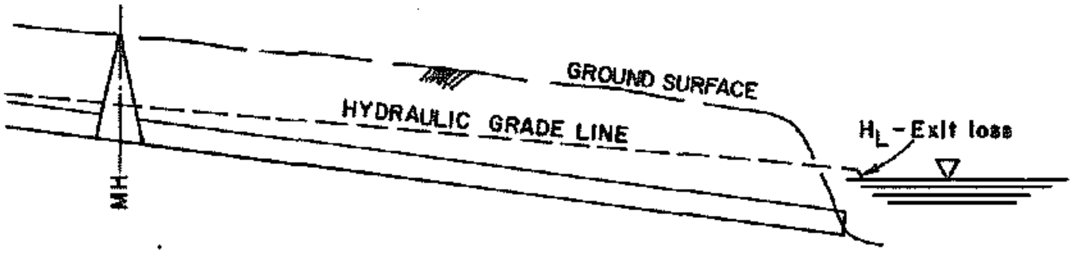
Final hydraulic design of storm sewers begins at the lowest point in the storm sewer system. The beginning hydraulic grade line (water surface in open channel flow or hydraulic grade line in pressurized conduits) in *the receiving facility must be determined coincident with the time of* peak flow from the storm sewer. If sophisticated hydrologic modelling techniques are utilized, the flow rate in the receiving facility (normally major drainage) may be known and the corresponding water surface elevation can be determined using techniques described in Chapter V, "Major Drainage," for open channel flow or the techniques described in this Chapter.

If the outlet is submerged or if the receiving water surface elevation is higher than normal depth in the storm sewer, the beginning hydraulic grade line is the hydraulic grade line in the receiving stream. With a submerged outlet, the design proceeds up the pipeline after inclusion of exit losses. For unsubmerged outlets, design can begin assuming normal depth at the first source of a point loss (lateral, manhole, or bend), unless this first loss is hydraulically close to the outlet. In this case, backwater techniques will be necessary (see Chapter V). For a conduit with an unsubmerged outlet and a greater hydraulic (and energy) grade line slope than pipe slope, the beginning water surface elevation is critical depth in the storm sewers.

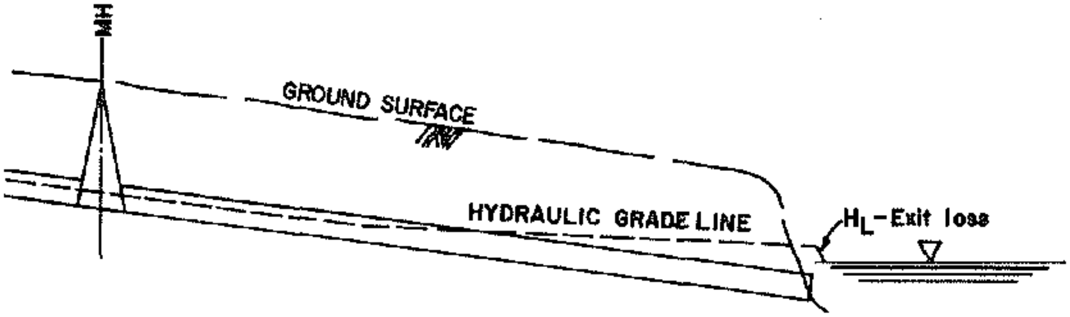
Figure IV-1 has been included to illustrate some of the exit conditions. Figure IV-2 illustrates the various hydraulic relationships for closed conduits and for open channel flow.

Calculations may then proceed upstream with checks being made at each manhole to verify whether or not the hydraulic grade line is above 80 percent of the pipe diameter. When this is the case, pressure conduit calculations should be used. If the water level should fall below the 80 percent level, open channel flow calculations should be used.

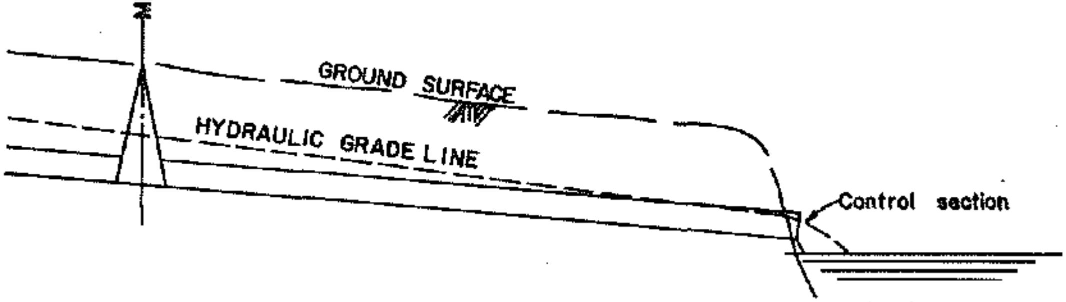
Figure IV-3 is used to determine the slope of the hydraulic grade line (also energy grade line) for pressurized conduits. Figure IV-4 is used to determine depths of flow in circular pipe. The latter figure was developed using the concept of roughness factors which vary with depth of flow. Since conduits generally are designed on the basis of their



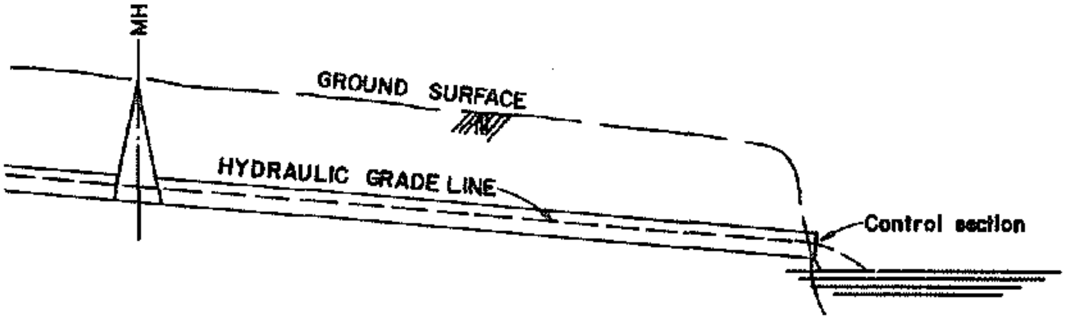
SUBMERGED DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



SUBMERGED DISCHARGED – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.

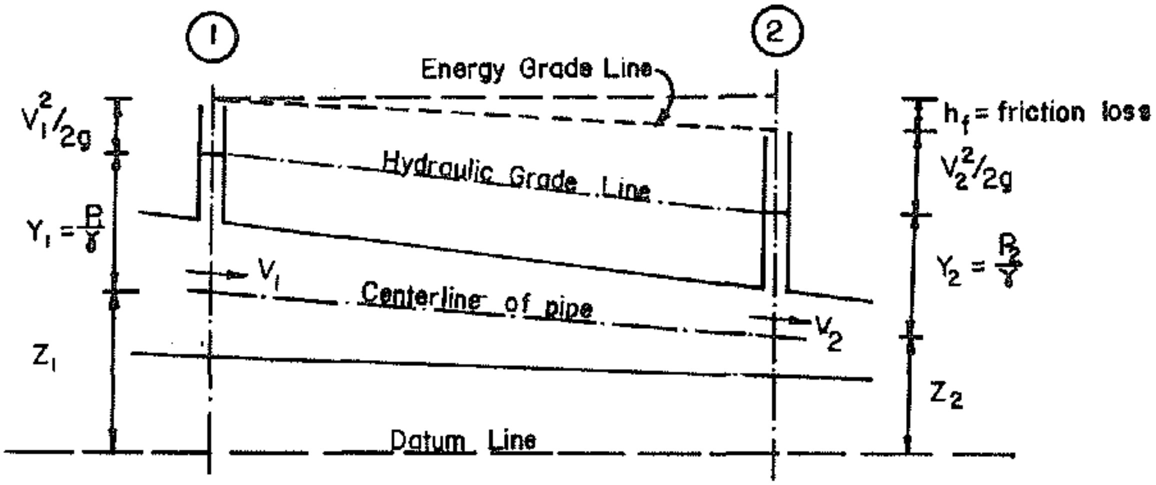


FREE DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.

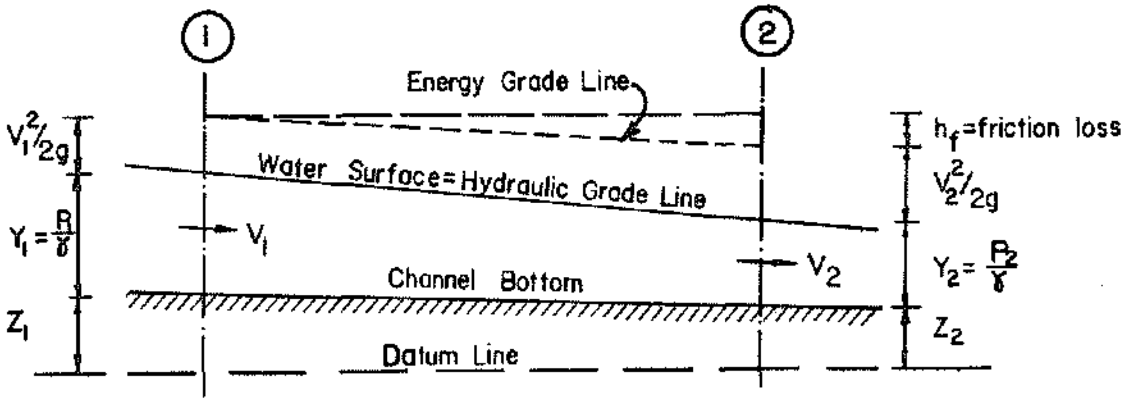


FREE DISCHARGE – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.

FIGURE IV-1 DETERMINING TYPE OF FLOW



(a) CLOSED-CONDUIT FLOW



(b) OPEN CHANNEL FLOW

FIGURE IV-2 COMPARISON BETWEEN CLOSED CONDUIT AND OPEN CHANNEL FLOW.

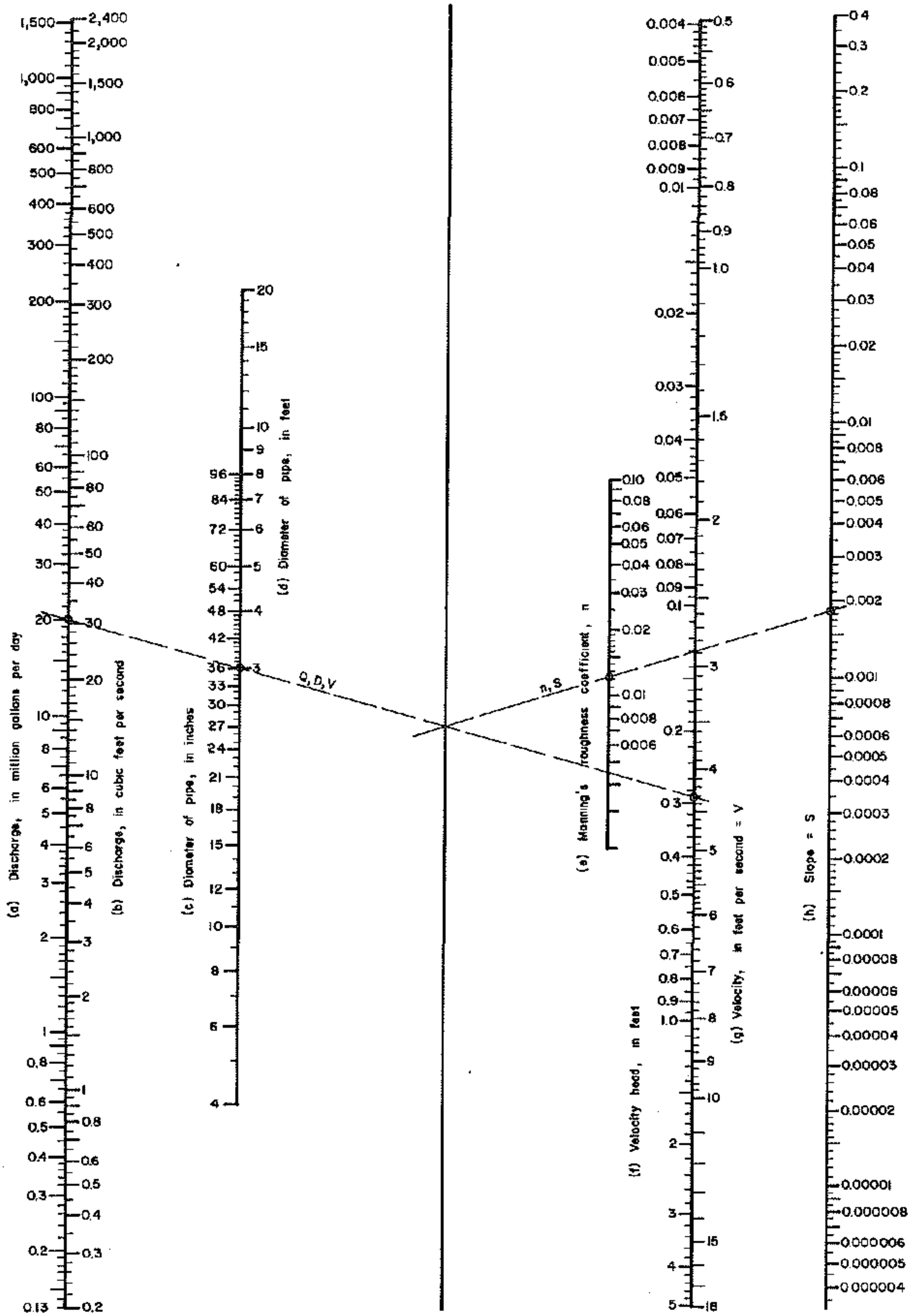
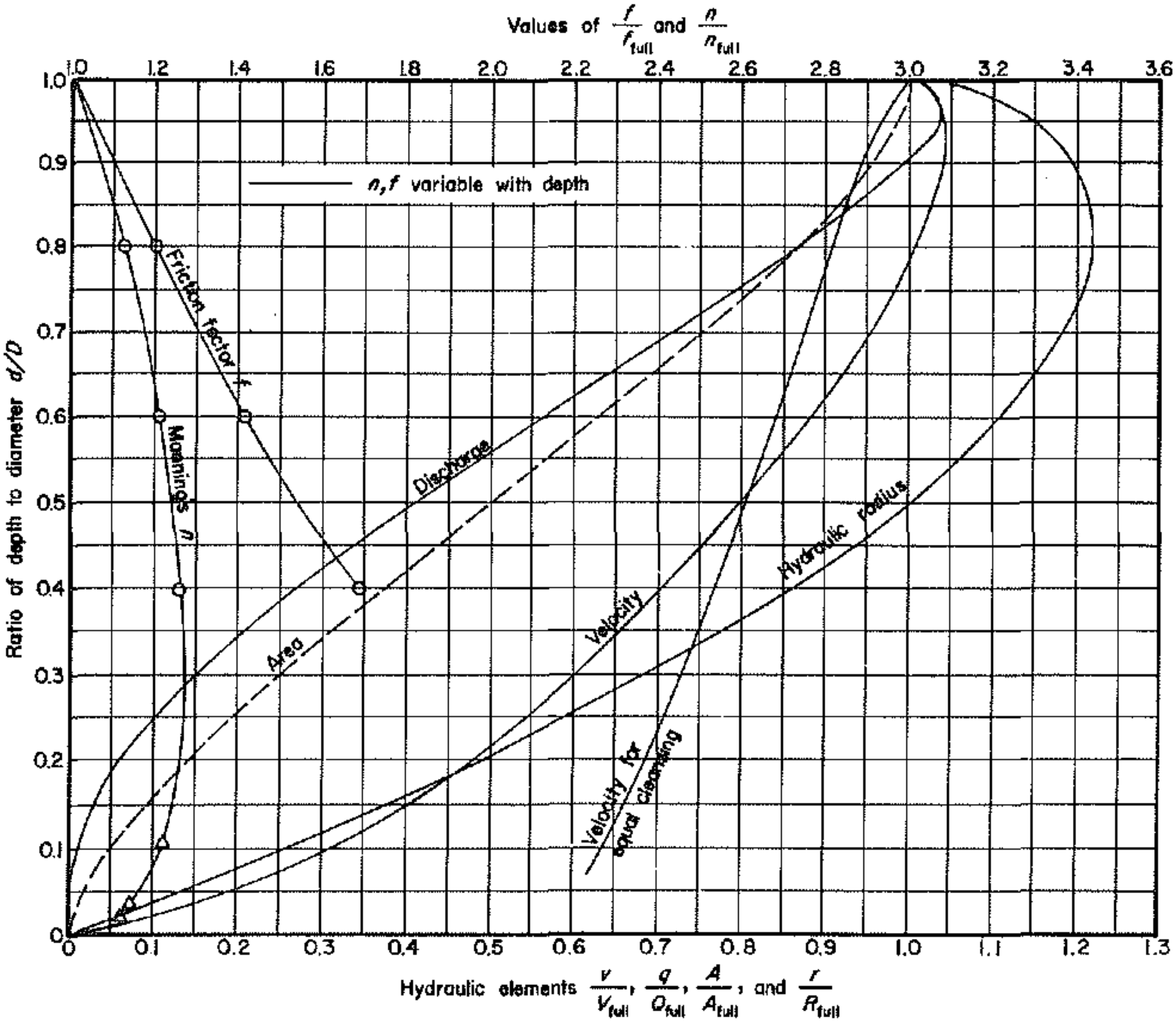


Figure IV-3 NOMOGRAPH for flow in round pipe - Manning's formula
(From 'Design and Construction of Concrete Sewers', Portland Cement Assoc.)



v = Actual velocity of flow (fps)

V_{full} = Velocity flowing full (fps)

q = Actual quanity of flow (cfs)

Q_{full} = Capacity flowing full (cfs)

A = Area occupied by flow (ft^2)

A_{full} = Area of pipe (ft^2)

r = Actual hydraulic radius (ft.)

R_{full} = Hydraulic radius of full pipe (ft.)

FIGURE IV-4 HYDRAULIC ELEMENTS OF CIRCULAR CONDUITS capacity when flowing full, or nearly full, the provision of a velocity adequate for self-cleansing under these conditions does not necessarily ensure prevention of deposits at all conditions of flow, especially in newly developing areas when the tributary quantities may be a small portion of the design capacity. Research shows that full flow in a pipe with a friction factor of 0.025, at 2fps, will barely move a coarse sand particle with a diameter of 1.8 mm. As the friction factor increases, the scouring velocity decreases. Since the friction factor increases with decreasing depth of flow in a pipe, equal self-cleansing will occur in partially full pipes at somewhat less than the critical velocity when flowing full. Based on these principles, a curve of the velocity required for equal cleansing ability at all depths of flow has been added to the graph of hydraulic elements.

Figures IV-5, IV-6, IV-7 and IV-8 illustrate the hydraulic properties for open channel flow in corrugated steel arches, concrete arches, concrete horizontal elliptical, and concrete vertical elliptical pipes, respectively.

Changes in the pressure across a junction in closed conduit flow are expressed in terms of the hydraulic grade line. Analysis shows that the pressure change from the outfall pipe to any upstream pipe at a junction can be expressed as the product of a coefficient and the mean velocity head in the outfall pipe. Many handbooks give coefficients for the loss of pressure for certain simple types of conduit transitions of pipe fittings. The lack of reasonable method of calculation of coefficients at more complex junctions has long hindered the design of storm sewers flowing full. This is in contrast to open-channel flow where pressure changes are related to the total head or energy grade line. See Figure IV-2.

Because of the complexities involved in junction design, a degree of simplification may be achieved by using design charts which were developed for pressure storm sewers. These charts give the pressure change coefficient for each pipe at a specific type of inlet or junction. When

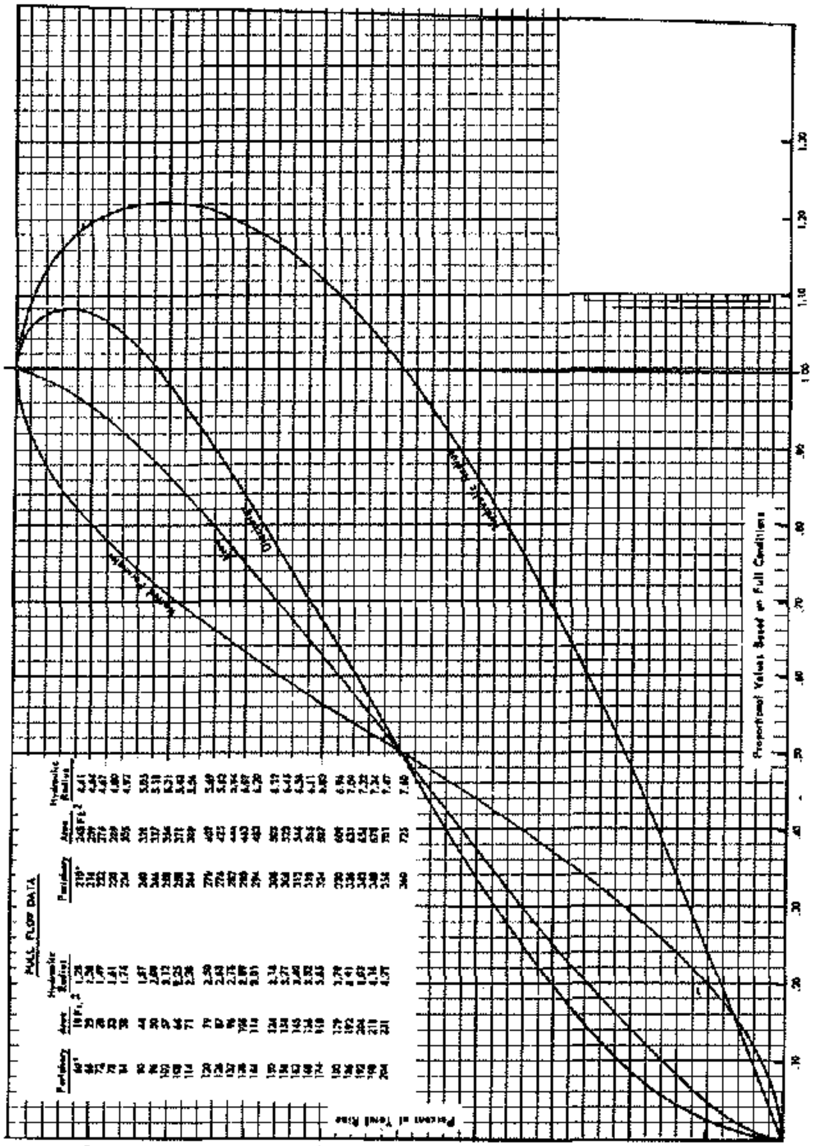


FIGURE IV-5
HYDRAULIC ELEMENTS OF CORRUGATED METAL
ARCH PIPE
(From Armco)

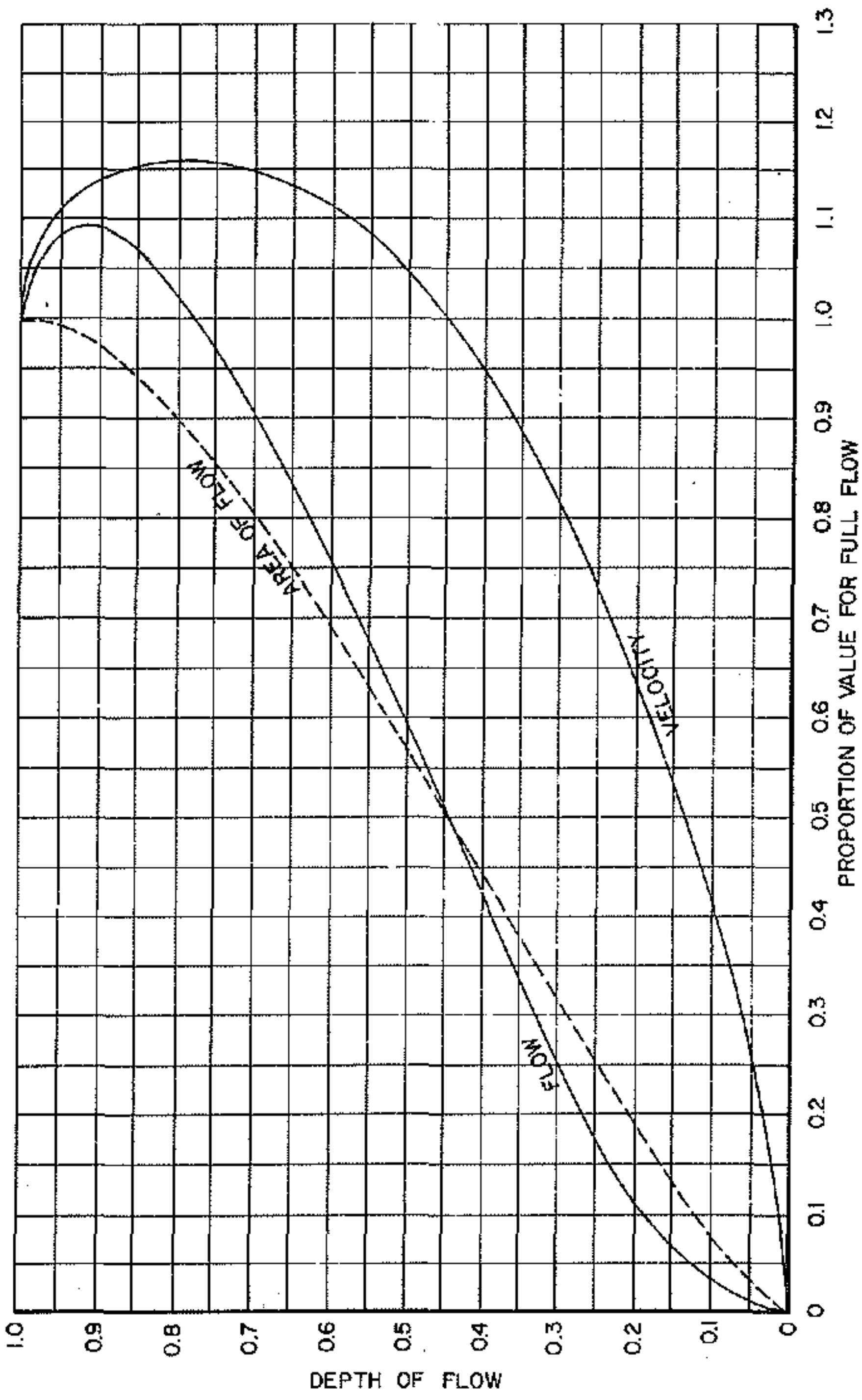


FIGURE IV-6
RELATIVE VELOCITY AND FLOW IN ARCH
PIPE FOR ANY DEPTH OF FLOW

WRIGHT-McLAUGHLIN ENGINEERS
2420 ALCOTT STREET DENVER COLORADO 80211

(From 'Design Manual, Concrete Pipe,' American Concrete Pipe Association)

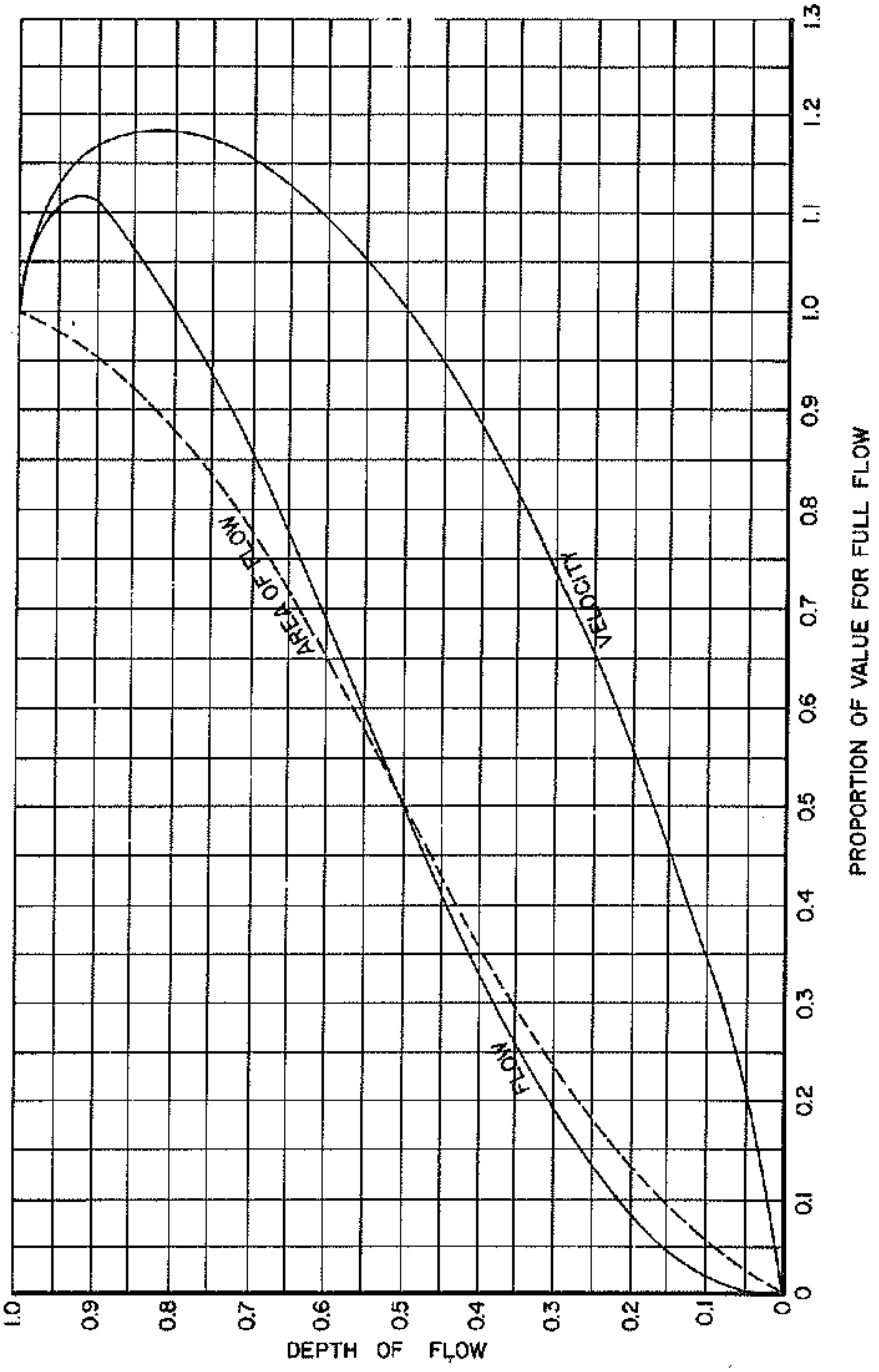


FIGURE IV-7
RELATIVE VELOCITY AND FLOW IN HORIZONTAL ELLIPTICAL PIPE FOR ANY DEPTH OF FLOW

WRIGHT-McLAUGHLIN ENGINEERS
2420 ALCOTT STREET DENVER COLORADO 80211

(From 'Design Manual, Concrete Pipe', American Concrete Pipe Association)

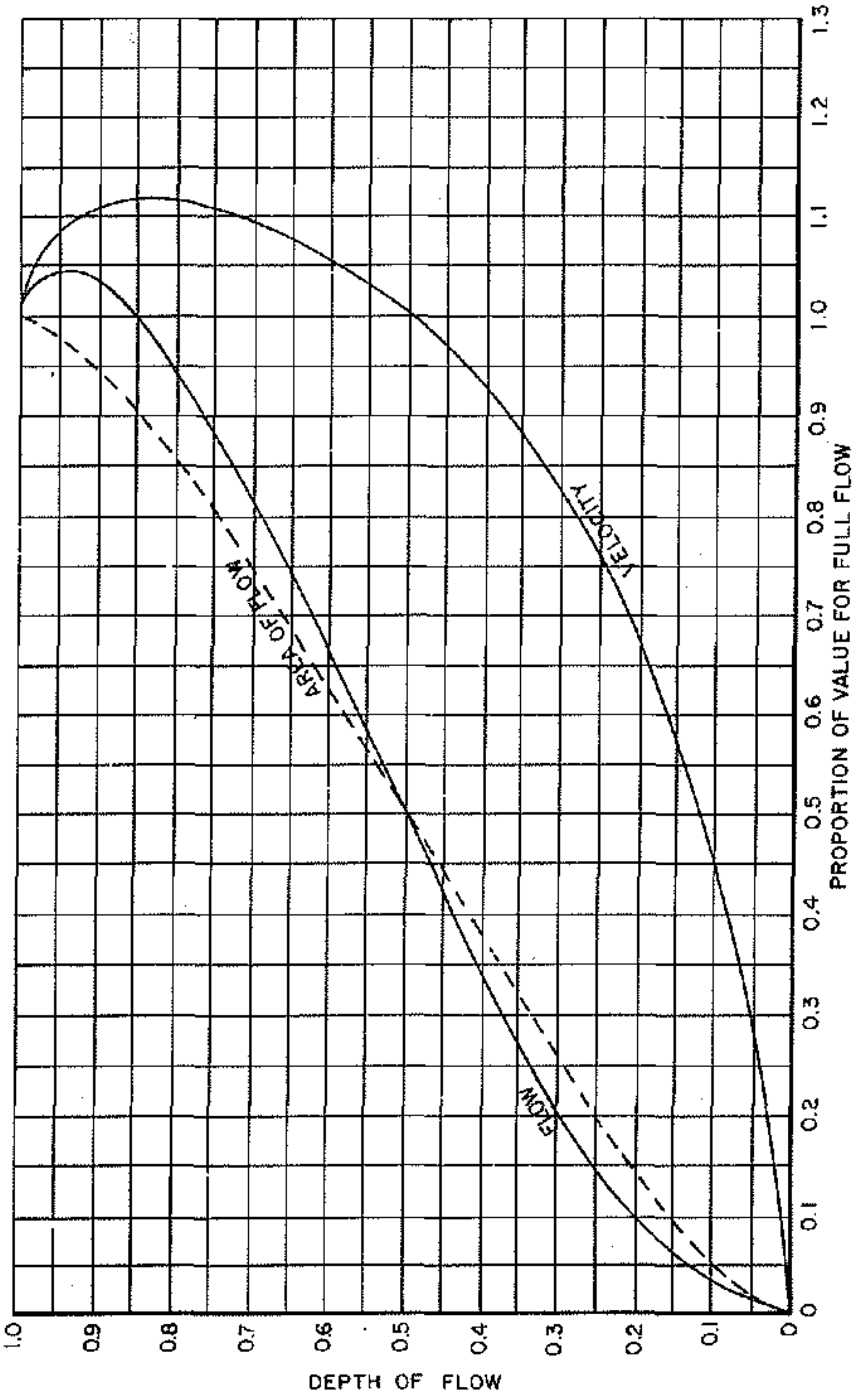


FIGURE IV- 8
RELATIVE VELOCITY AND FLOW IN VERTICAL
ELLIPTICAL PIPE FOR ANY DEPTH OF FLOW

WRIGHT-McLAUGHLIN ENGINEERS
2420 ALCOTT STREET DENVER, COLORADO 80211

(From 'Design Manual, Concrete Pipe,' American
Concrete Pipe Associate)

two or more upstream pipes are involved at a junction, the design charts provide a coefficient for each. The charts also provide pressure change coefficients for rectangular inlets fitted with an efficient bar grate capable of admitting relatively large rates of flow in to the storm drain system from a roadway gutter or sump.

Pressure losses are large at junctions involving any considerable proportion of joining flow and at right-angle turns of a pipeline at a manhole. The loss is often in the neighborhood of 1.5 downstream velocity heads which may be thought of as the friction loss in 100 to 250 feet of pipe. All losses of energy through resistance to flow in pipes or by changes of momentum and interference with flow patterns at junctions must be compensated by an accumulative series of rises of pressure along the system from its outlet to its initial upstream inlet. While the resistance loss in a pipeline is entirely or nearly compensated by the fall in the pipe grade in the downstream direction, losses at junctions are not. The purpose of the accurate determination of pressure changes at junctions is to include the values in the progressive calculation of pressure elevations proceeding upstream along a storm drain system. In this way it is possible to determine the water surface elevation which will exist at each appurtenance. With this information, the designer may adjust sizes and junction types to arrive at a final design of the storm drain system which may be expected to perform satisfactorily under the storm runoff conditions for which it was designed.

Because normal economical design procedures for open channel flow will result in nearly full storm sewer pipes, the junction loss curves for pressure conduits may be used to approximate junction losses for runoff in storm sewers flowing as open channels. Care must be exercised in unique situations where a segment of storm sewer may be designed for a depth less than 80 percent of the pipe diameter. In this case, the designer is encouraged to use other methods described in this Chapter to compute conduit losses when applied to open channel flow.

GENERAL ASPECTS OF STORM SEWER DESIGN

As previously described, calculations to check the pressure of water surface elevations in the storm drain system begin with a known surface elevation at some downstream point. The rise of the hydraulic grade line along the pipe to the first upstream junction is added to this known elevation to obtain the outfall hydraulic grade line elevation at the upstream junction. The hydraulic grade line rise through the pipe length is the resistance loss, or friction loss, in the pipe and is calculated by any of a variety of accepted methods.

If the hydraulic grade line is above the pipe crown at the upstream junction, full flow calculations may proceed. If the hydraulic grade line is below the pipe crown at the upstream junction, then open channel flow calculations must be used at the junction.

When the discharge is not submerged, a flow depth must be determined at some control section to allow calculations to proceed upstream. As illustrated in Figure IV-1, the hydraulic grade line is then projected to the upstream junction. Full flow calculations may be utilized at the junction if the hydraulic grade is above the pipe crown.

For open channel flow, the assumption of straight hydraulic grade lines as shown in Figure IV-1 is not entirely correct, since backwaters and drawdowns exist, but should be accurate enough for the size of pipes usually considered as storm sewers.

On steep storm sewer grades, the upstream storm sewer may enter the junction at an elevation somewhat higher than the crown of the downstream storm sewer pipe. In this case, it may be assumed that the upstream flow acts as though it were inlet flow since the jet is essentially broken up when it enters the junction. The designer may then use the relevant inlet design chart to calculate the pressure change likely across the junction.

When two laterals intersect a manhole, the alignment should be quite different. If lateral pipes are aligned opposite one another, so the jets may impinge upon each other, the magnitude of the losses are extremely high. A design chart for directly opposed laterals is included, although this arrangement can be avoided when the hydraulic losses from opposite/aligned laterals cannot be tolerated, as would be expected in Stillwater. However, the high losses of directly opposed laterals may be used to elevate the hydraulic grade line to control inflow in special cases.

If the installation of directly opposed laterals is necessary, the installation of a deflector as shown in Figure IV-9 will result in *significantly reduced losses. Research conducted on this type of* deflector is limited to the ratios of:

$$\left[D_O/D_L\right] = 1.25^* \qquad \text{Eq. IV-1}$$

The tests indicate coefficient pressure change at 1.6 for all flow ratios and pipe diameter ratios when no inlet flow is considered, and 1.8 when inlet flow is over 10 percent of Q_O.

When necessary to reduce head loss, it is suggested that lateral pipes should not be located directly opposite; rather, their centerlines should be separated laterally by at least the sum of the total lateral pipe diameters. Reference to the design figures shows that head losses are definitely reduced as compared to directly opposed laterals, even with deflectors. Insufficient data have been collected to determine the effect of offsetting laterals vertically.

The pressure change for each upstream pipe at the junction is then added to the common outfall pipe hydraulic grade elevation at the center of *the junctions. This accounts for all losses at the junction and gives a*

*See Figure IV-11 for nomenclature.

starting elevation for calculations to be made along each upstream pipe.

The water depth at each junction must be calculated to verify that the water level is above the crown of all pipes. Whenever the level is 80 percent of the pipe diameter, full flow methods are not applicable. When the pipe level exceeds the 80 percent level, pressurized pipe flow techniques are applicable.

Certain points concerning the application of the design method described herein may be noted. To expedite computations, the storm sewer hydraulic grade line elevation determined at a junction should first be compared to the elevation of the top of the downstream pipe and the gutter. Because of usual losses at the junction, it is known that upstream hydraulic grade elevations and the water elevation in the inlet are generally higher than the elevation of the downstream storm sewer hydraulic grade line. Comparison to limiting conditions will indicate whether the design may or may not be applicable at the junction.

Manhole Construction

As shown in Figure IV-9, construction details of manholes for storm sewer systems should deviate somewhat from standard manholes for sanitary sewers.

Alignment of Pipes in Manholes

For a straight through flow, research indicates that the pipes should be positioned vertically so that they are between the limits of inverts aligned or crowns aligned. An offset in the plan and/or profile is allowable provided the projected area of the smaller pipe falls within that of the larger. Aligning the inverts of the pipes is probably the most efficient as the manhole bottom then supports the bottom of the jet issuing from the upstream pipe.

Shaping Inside of Manhole

The fact that jets issue from the upstream and lateral pipes must be considered when attempting to shape the inside of manholes.

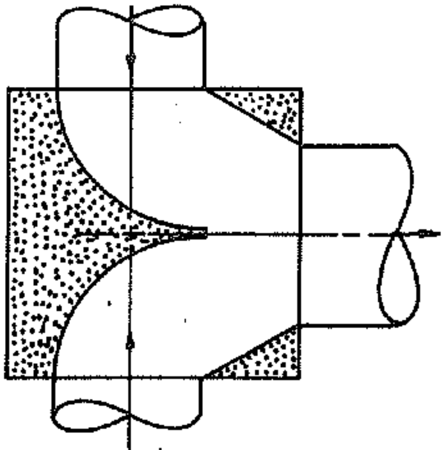
The tests for full flow revealed that very little, if anything, is gained by shaping the bottom of a manhole to conform to the pipe invert. Shaping of the invert may even be detrimental when lateral flows are involved, as the shaping tends to deflect the jet upwards, causing unnecessary head loss. Limited shaping of the invert to handle low flows is necessary from a practical point of view.

Figure IV-9 details several types of deflector devices that have been found efficient in reducing losses at junctions and bends. In all cases, the bottoms are flat, or only slightly rounded to handle low flows. Other devices which have been found inefficient are shown in Figure IV-10. The fact that several of these inefficient devices would appear to be improvements indicates that special shapings deviating from those in Figure IV-9 should be used with caution, possibly only after model tests.

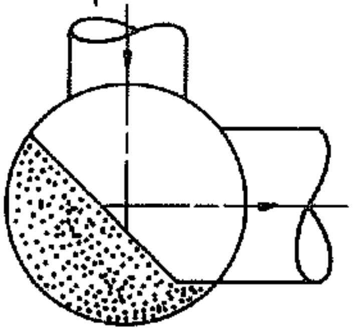
Entrances

Tests show that rounding entrances or the use of pipe socket entrances do not have the effect on reducing losses that might be expected. Once again, the effect of the jet from the upstream pipe must be considered. Specific reductions to the pressure change factors are indicated with each design chart. Special shaping of entrances is only recommended where pipeline size reductions can be gained from entrance shaping.

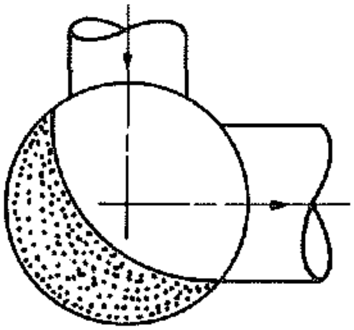
Catch Basins. Certain specific design procedures are necessary when designing catch basins for storm water inlets on systems flowing full.



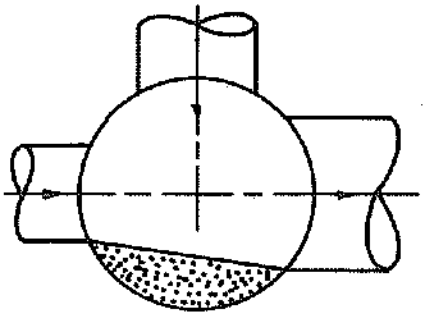
Directly opposed lateral with deflector
(head losses are still excessive with this method, but are significantly less than when no deflector exists.)



Bend with straight deflector



Bend with curved deflector



Inline upstream main & 90° lateral with deflector

FIGURE IV-9 EFFICIENT MANHOLES

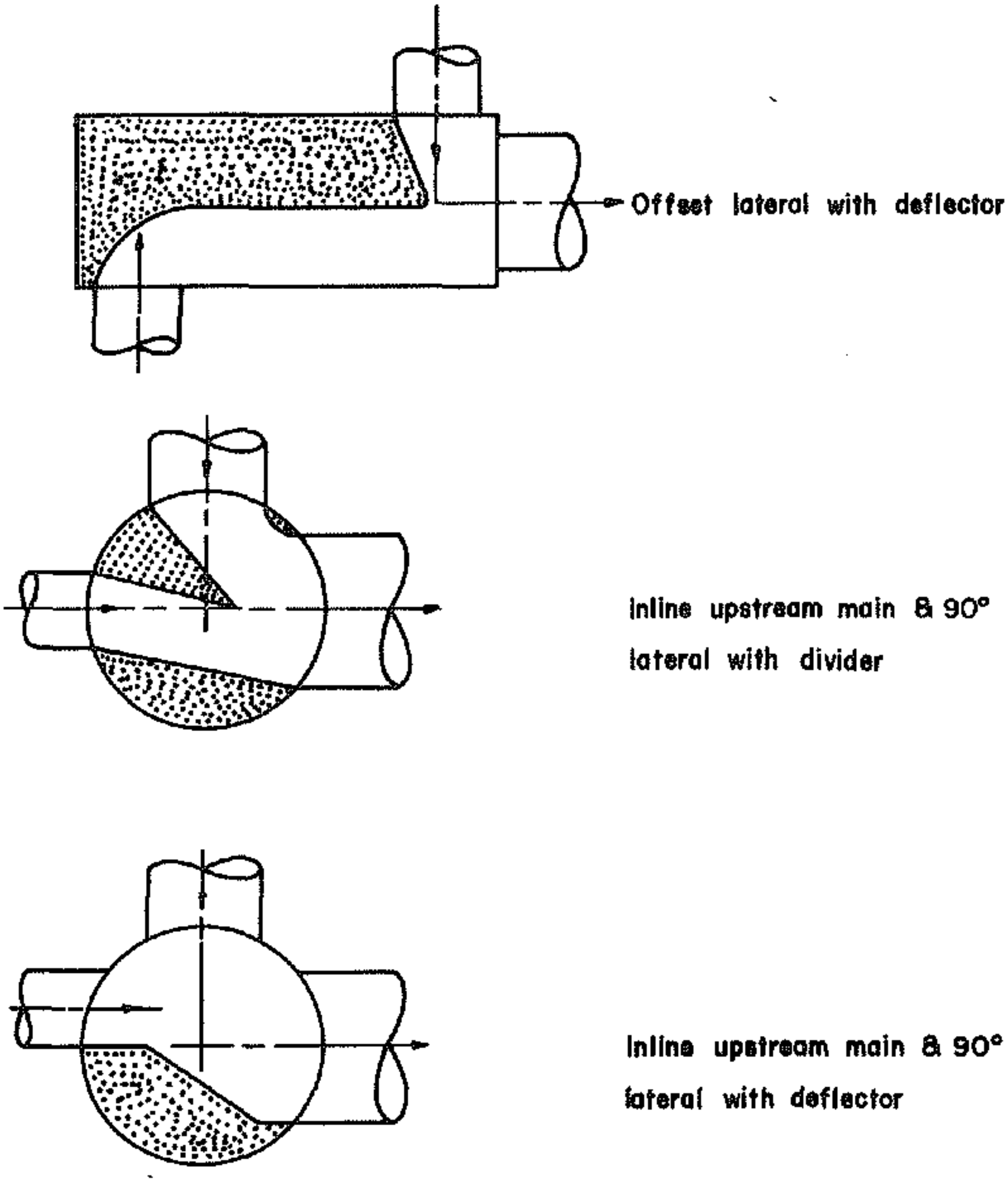


FIGURE IV-10 INEFFICIENT MANHOLE SHAPING

The above methods of shaping the interior of a manhole were found inefficient either due to increased head loss or tendency to plug with trash.

The design water surface should be at least 6 inches below the gutter grade at the inlet to allow the inlet to function properly. If there is any possibility of the hydraulic grade being above this level, the inlet should be considered not to accept any flow. In unusual cases, the hydraulic grade may exceed the inlet elevation, allowing flow to escape from the system. Methods of dealing with this water must then be included in the design.

DESIGN METHODOLOGY FOR PRESSURE CONDUITS

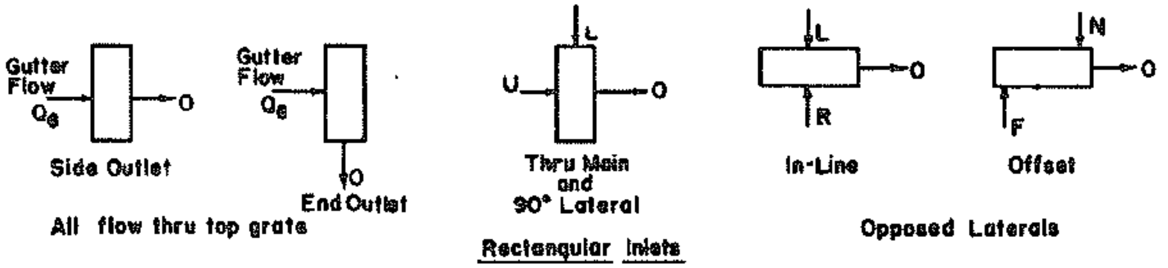
The methodology outlined in this Chapter allows computations for required storm sewer systems to be pursued with the degree of accuracy justified by the cost of subsequent construction. The charts in this Section offer one of the better design methodologies available.

These charts enable a designer to include manhole losses in a progressive calculation of pressure elevations proceeding upstream along a storm sewer system, determining the water surface elevation, hydraulic grade line, and total energy gradient. Emphasis must be placed on the fact that the charts are strictly applicable only when all pipes entering the manhole are flowing full.

The nomenclature used in the design charts and suggested for design purposes is explained in Figure IV-11, where sketches of the junction types also appear.

In this presentation of design methods, provision is made for reserving numbers to designate inlets and junctions by using a system of letter subscripts for identifying pipes. The letter subscript is applied to the pipe diameter D, its discharge Q, and the resulting velocity of flow V.

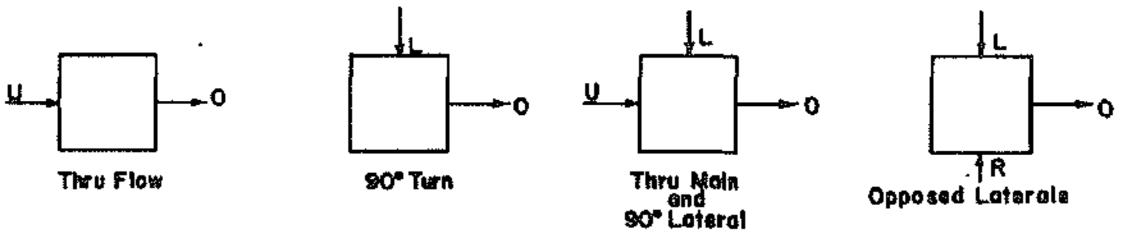
The letter subscript designates the function of that pipe at the particular junction under consideration. These letter subscripts are used consistently for each pipe of similar function at all junctions. Thus D_O designates the diameter of the outfall pipe at any junction,



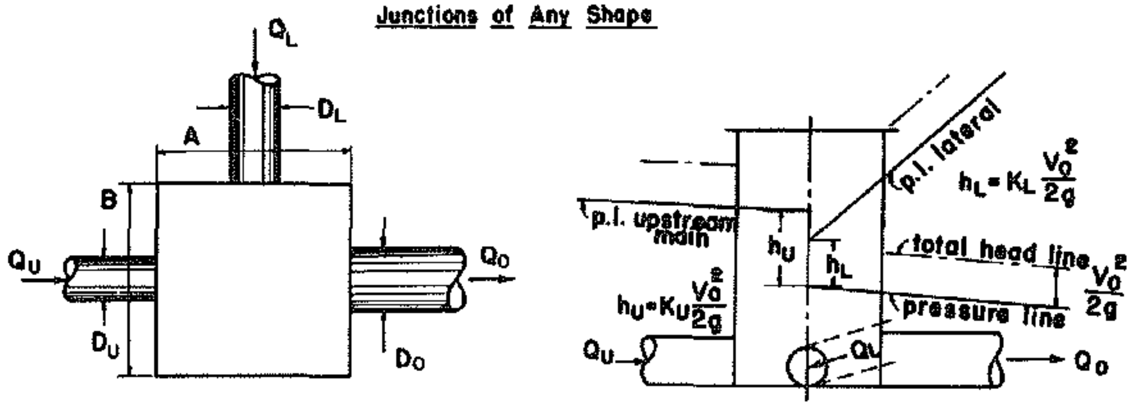
All flow thru top grate

Rectangular Inlets

Opposed Laterals



Junctions of Any Shape



Junction Dimensions

Diagram of Pressure Line Elevations at Junction of A Lateral with A Thru Main

Nomenclature

Q	rate of flow	
D	diameter of pipe	
A	dimension of junction in direction of outfall pipe	
B	dimension of junction at right angles to outfall pipe	
d	depth of water in inlet	
S	slope of pipe	
S_f	friction slope	
Q_g	flow into inlet thru top grate	
D_O Q_O	dia. and flow in outfall	
D_U Q_U	dia. and flow in upstream main	
D_L Q_L	dia. and flow in left lateral	
D_R Q_R	dia. and flow in right lateral	
D_N Q_N	dia. and flow in near lateral	of opposed laterals; designation with reference to position relative to outfall end of junction.
D_F Q_F	dia. and flow in far lateral	
$D_{h.v.}$ $Q_{h.v.}$	dia. and flow in lateral with higher-velocity flow	for in-line opposed laterals only
$D_{l.v.}$ $Q_{l.v.}$	dia. and flow in lateral with lower-velocity flow	

Pressure change coefficients for inlet water depth and on upstream pipe pressure relative to the outfall pipe pressure.

K_g	water depth with all flow thru grate	
K_U	upstream main pressure	
K_R or K_L	lateral pipe pressure	
K_N	near lateral pipe pressure	offset opposed laterals
K_F	far lateral pipe pressure	
$\overline{K_U}$, $\overline{K_L}$	pressure coefficient at $Q_L = Q_O$	
M_U, M_L	multipliers for $\overline{K_U}$ or $\overline{K_L}$ to obtain K_U or K_L	

FIGURE IV-11 MANHOLE JUNCTION TYPES & NOMENCLATURE

(University of Missouri)

that of the upstream pipe, D_L that of a lateral entering the left side of a junction, viewed looking downstream along the direction of the outfall pipe, and D_R a similar lateral at the right. Similarly, Q_O, Q_U, Q_L, and Q_R designate the rates of flow in the several pipes.

In the design applications, the outfall pipe is always used as the basic measurement. Pipe size ratios are stated as the ratio of upstream to outfall pipe diameter, e.g. D_U/D_O or D_L/D_O. Flow ratios are similarly stated, e.g. Q_U/Q_O. Pressure changes are stated in terms of outfall velocity head; that is, the pressure change coefficient K_U, equals the pressure change h_U in its ratio to $V_O^2/2g$.

The line diagrams of Figure IV-11 illustrate the pipe positions and the function of each as supply or outfall for each type of inlet and junction involved. A detail plan is included to show junction and pipe diameter dimensions used. These dimensions may be in inches, feet, or any other unit of linear measurement since they are used in the design charts only as ratios of one to another. The charts included in this Section are based on tests of round pipe, and apply to pipes of circular cross section. However, the charts will apply accurately enough to pipe of any cross section.

One of the diagrams on Figure IV-11 shows a through main at a junction of a 90° lateral, with pressure lines and total head lines superimposed. It will be noted that the relative elevations of the various total head lines are not dealt with in this sketch.

The same diagram shows the hydraulic grade lines projected to a point above the "branch point," this being the location in plan of the intersection of the outfall pipe and lateral pipe centerlines. A similar point of reference, the center of the junction box, is used for the upstream in-line pipe and its hydraulic grade line where no lateral is present. The change of pressure at a junction is measured by the difference in elevation between the outfall hydraulic grade line and an up-stream hydraulic grade line, along the vertical line through the branch point. The vertical dimensions h_U and h_L indicate the change of pressure for the upstream in-line and lateral pipes, respectively. The adjacent equations on Figure IV-11 state how each is calculated.

There will be situations where combinations of pipelines and/or flow conditions are not represented in the design charts. In the latter case, it is generally acceptable to extrapolate. Situations are rare where required.extrapolations are significant.

In those rare instances and where more than three pipelines enter the same manhole, it may be necessary to make simplifying assumptions in regard to the flow conditions and utilize the appropriate chart representing these assumptions. This approach is not considered a serious constraint as there are many similarities between the graphics for varying flow conditions.

One frequently occurring example is four pipelines entering a common junction. Table IV-1 lists the recommended graphs and assumptions for this condition.

At all junctions where a change of pressure occurs, a loss of total head must occur whether the pressure change is positive or negative. This basic fact may be used to check pressure results.

General Instructions for Use of Design Charts

Several operations are common to use of the design charts for various types of junctions. Instructions for performing these recurring procedures are consolidated in the following General Instructions. In the detailed instructions for use of the individual charts, references to these General Instructions are made by number (Gen. Instr.1, etc.). The general instructions are as follows:

1. Determine and tabulate the elevation of the outfall pipe pressure line at the branch point or inlet center (refer to Figure IV-11). This elevation is obtained by adding the pipe friction loss to the

elevation of the pressure line at the preceding structure downstream.

2. Calculate the mean velocity head of the flow in the outfall pipe.

$$\frac{V_O^2}{2g} \qquad \text{Eq. IV-2}$$

3. Calculate the required flow rate and size ratios. Examples:

   Q_U/Q_O, Q_L/Q_U, Q_G/Q_O, etc.
   D_U/D_O, D_L/D_O, B/D_O, etc.

4. Estimate the depth of water, d, in a manhole with flow into the manhole from a top inlet, either alone or combining with flow from an upstream pipe.

   d = total depth of water, in feet.
   h = (outfall pressure line elevation minus inlet bottom elevation)+ (K) $V_O^2/2g$.
   K = the pressure change coefficient for the inlet water depth. (This is estimated as detailed for each type of manhole. Such estimates are not necessary for manholes with in-line or offset opposed laterals).

5. Use the coefficients K from the charts for manholes with square-edged entrance to the outfall pipe (entrance flush with box side, with square edges).

6. Use reduced coefficients K, where applicable, for a rounded entrance to the outfall pipe (rounded on 1/4 circle arc of approximate radius 1/8 D_O) or for an entrance formed by the socket end of a standard tongue-and-groove concrete pipe.

   Chart IV-A - insignificant effect, make no reduction
   Chart IV-B - read directly from the chart
   Chart IV-C - reduce K_U by 0.1 for usual proportions of inlet flow; by 0.2 for Q_G about 0.5 Q
   Chart IV-D - reduce K_U and K_L in same manner as Chart IV-C
   Chart IV-E - insignificant effect, make no reduction
   Chart IV-F - insignificant effect, make no reduction
   Chart IV-G, H, I - see specific instructions for each case.
   Chart IV-J - make no reductions.

7. Calculate pressure change. To calculate the change of pressure at a manhole, working upstream from the outfall pipe to an upstream pipe, the design chart applying to the type of junction involved is selected. The pressure change coefficient for a specific upstream pipe is read from the chart for the particular flow rate and size ratios already calculated. The pressure change is calculated from

$$h = K \times \frac{V_0^{\,2}}{2g} \qquad \text{Eq. IV-3}$$

The coefficient is a dimensionless number, and therefore, the change of pressure will be in feet.

8. Apply the pressure change. The pressure change, in feet, for each upstream pipe is added to the outfall pipe pressure line elevation at the branch point to obtain the elevation of each pressure line for further calculations upstream along the pipe. In some cases, the upstream pressure line at the branch point will be at a lower elevation than the downstream pressure line. Where this less common situation may occur with a particular type of junction, it is mentioned in the instructions for use of the specific chart.

9. Determine the elevation of the water surface. The elevation of the water surface in a manhole (with or without inlet flow) receiving flow from a pipe or pipes will correspond to that of the upstream in-line pipe pressure line. At a junction with offset opposed laterals, the water surface will correspond to the elevation of the far lateral pipe pressure line. At a junction with in-line opposed laterals, the water surface will correspond to the elevation of the pressure line of the higher-velocity lateral pipe.

Verify that the water surface is above the crown elevation of all pipe connections to the structures that are being analyzed. Small pipes, such as laterals to inlets, which carry a small portion of the total flow, may reasonably be constructed to affect a manhole in the same way as inlet flow from the ground surface.

The various cases are summarized below:

TABLE IV-1

SUMMARY OF DESIGN CHART/MANHOLE CONFIGURATION APPLICATION

Case	Design Chart
Catch Basin With Inlet Flow Only	IV-A
Flow Straight Through Any Manhole	IV-B
Rectangular Manhole, Through Pipe and Inlet Flow	IV-C
Rectangular Manhole With In-Line Upstream Main and 90° Lateral Pipe (with or without inlet flow)	IV-D
Rectangular Manhole With In-Line Opposed Lateral Pipes each at 90° to Outfall (with or without inlet flow)	IV-E
Rectangular Manhole With Offset Opposed Lateral Pipes Each at 90° to Outfall (with or without inlet flow)	IV-F
Square Manhole at 90° Deflection	IV-G
Round Manhole at 90° Deflection	IV-G
Deflectors in Square or Round Manholes at 90° Deflection	IV-G
Square Manhole on Through Pipeline at Junction of a 90° Lateral Pipe (large size laterals: $D_L/D_O > 0.6$)	IV-G, IV-H
Round Manhole on Through Pipeline at Junction of a 90° Lateral Pipe (large size lateral: $D_L/D_O > 0.6$)	IV-G, IV-H
Deflectors in Square or Round Manholes on Through Pipelines at Junction of a 90° Lateral Pipe (large size laterals: $D_L/D_O > 0.6$)	IV-G, IV-H
Square or Round Manhole on Through Pipeline at Junction of a 90° Lateral Pipe (smaller size laterals: $D_L/D_O < 0.6$) or laterals with no manhole.	IV-I
Sewer Bends with or Without Manhole	IV-J
Square or Round Manhole on Through Pipeline With Two Laterals ($D_u/D_o > 0.5$ or $Q_U/Q_O > 0.3$), Consider Smallest Lateral as Grate Flows	IV-D
Square or Round Manhole on Through Pipeline with Two laterals ($D_u/D_o < 0.5$ or $Q_U/Q_O < 0.3$), Consider Upstream Pipe as Grate Flows	IV-E, IV-F

Catch Basin With Inlet Flow Only - Chart IV-A

Pressure change coefficients are presented in this Chart for use in determining the elevation of the water surface in a catch basin with all inflow entering through an inlet. Separate curves are included for the outfall pipe connected at the box end (short dimension) and the box side (long dimension). The coefficient K_G depends on the pipe position and the depth of water in the inlet.

To use the Chart:

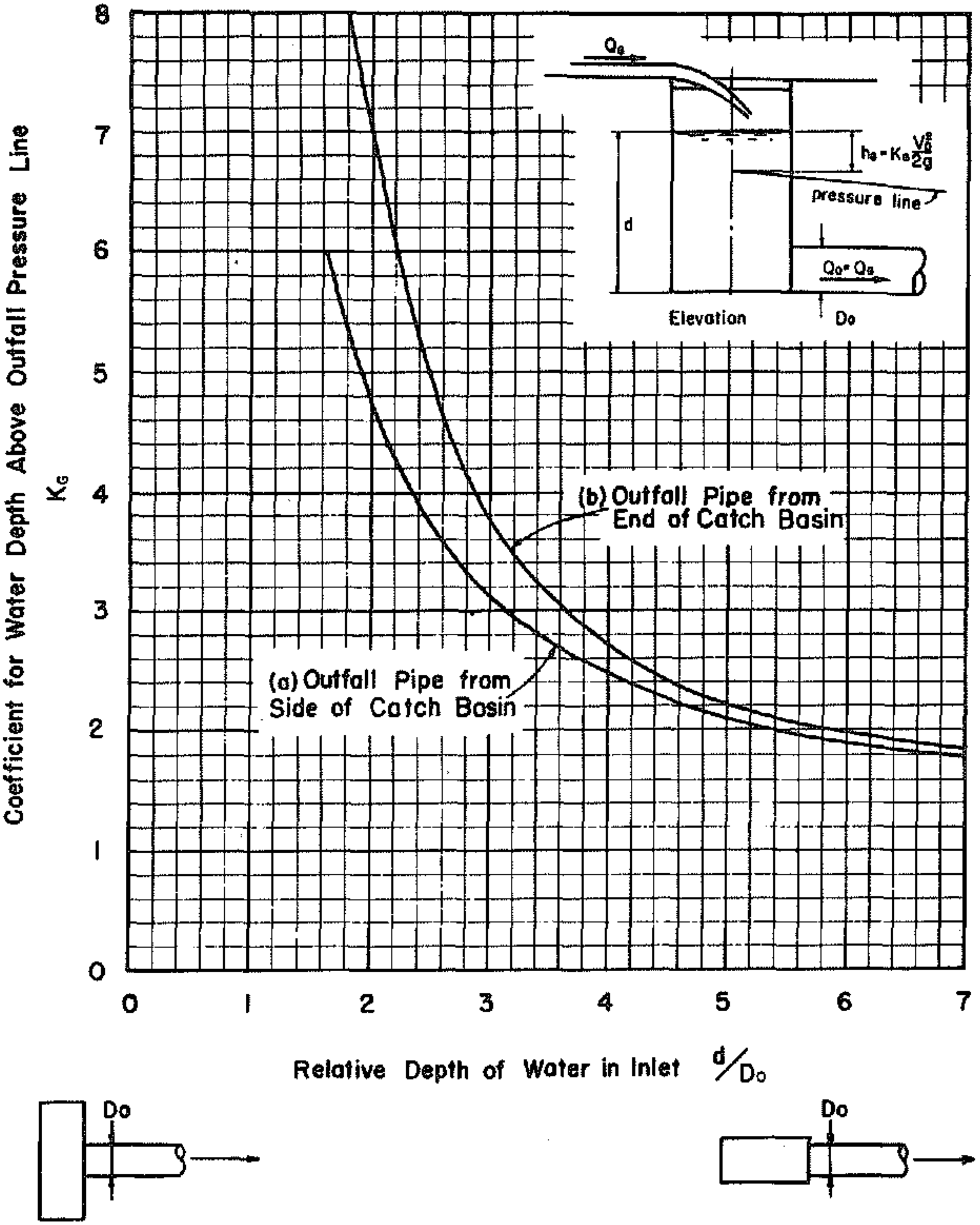
1. Note whether outlet is at end or side.
2. Determine outfall pipe pressure line elevation - Gen. Instr. 1.
3. Calculate outfall velocity head - Gen. Instr. 2.
4. Estimate a value for water depth d.
   - o outfall pressure line elevation minus inlet bottom elevation plus h_G equals d, where

     $$h_G = \frac{K_G V_O^2}{2g}$$

   - o estimate K_G as follows:

     7.0 for end outlet, 5.0 for side outlet--pressure line to bottom not over 2 pipe diameters.

     4.0 for end outlet, 3.0 for side outlet--for higher pressure lines.
5. Calculate the estimated relative water depth d/D_O.
6. Enter Chart IV-A at this depth d/D_O and read K_G from the curve for the particular outfall pipe location.
7. Calculate h_G as indicated on the diagram on the chart and by Gen. Inst. 7.
8. Add h_G to the elevation of the outfall pressure line at the inlet center to obtain the water surface elevation in the inlet.



(a) Box Side Flow

(b) Box End Flow

CHART IV-A CATCH BASIN WITH INLET FLOW ONLY

(University of Missouri)

9. From this water surface elevation subtract the elevation of the inlet bottom to obtain a more precise value for the water depth d.
10. Repeat the above procedure with the improved value of d from Step 9 if necessary. Such repetition may not be necessary if the estimated d/D_O of Step 5 was reasonably accurate.
11. Check to be sure that the inlet water elevation is below the gutter elevation at the inlet so that inflow may be admitted.

Flow Straight Through Any Manhole - Chart IV-B

Pressure change coefficients are presented in the Chart for use in determining the elevation of the pressure line of an upstream in-line pipe relative to that of the outfall. The pipe centerlines must be parallel and not offset more than would permit the area of the smaller pipe to fall entirely within that of the larger if projected across the junction box along the pipe axis. The shape of the junction in plan is not significant in determining the pressure change. The effect of junction size and outfall pipe entrance conditions are included in the chart. Negative pressure changes occur with an upstream pipe smaller than the outfall pipe. That is, at the junction center, the upstream pressure line is below the outfall pressure line for this case. No flow other than that from the upstream in-line pipe may be involved where this Chart applies.

To use the Chart:

1. Determine the outfall pipe pressure line elevation - Gen. Instr. 1.
2. Calculate the velocity head in the outfall - Gen. Instr. 2.
3. Calculate the size ratios $D_U D_O$ and A/D_U - Gen. Instr. 3.
4. Note whether the outfall pipe entrance is to be square-edged or rounded smooth (note Gen. Instr. 6).
5. Enter Chart IV-B at the pipe size ratio D_U/D_O and read K_U at the curve for the proper value of A/D_U for a square-edged entrance condition, or at the dashed curve for a rounded entrance.
6. Calculate h_U (positive or negative) as indicated on the diagrams on the Figure and by Gen. Instr. 7.

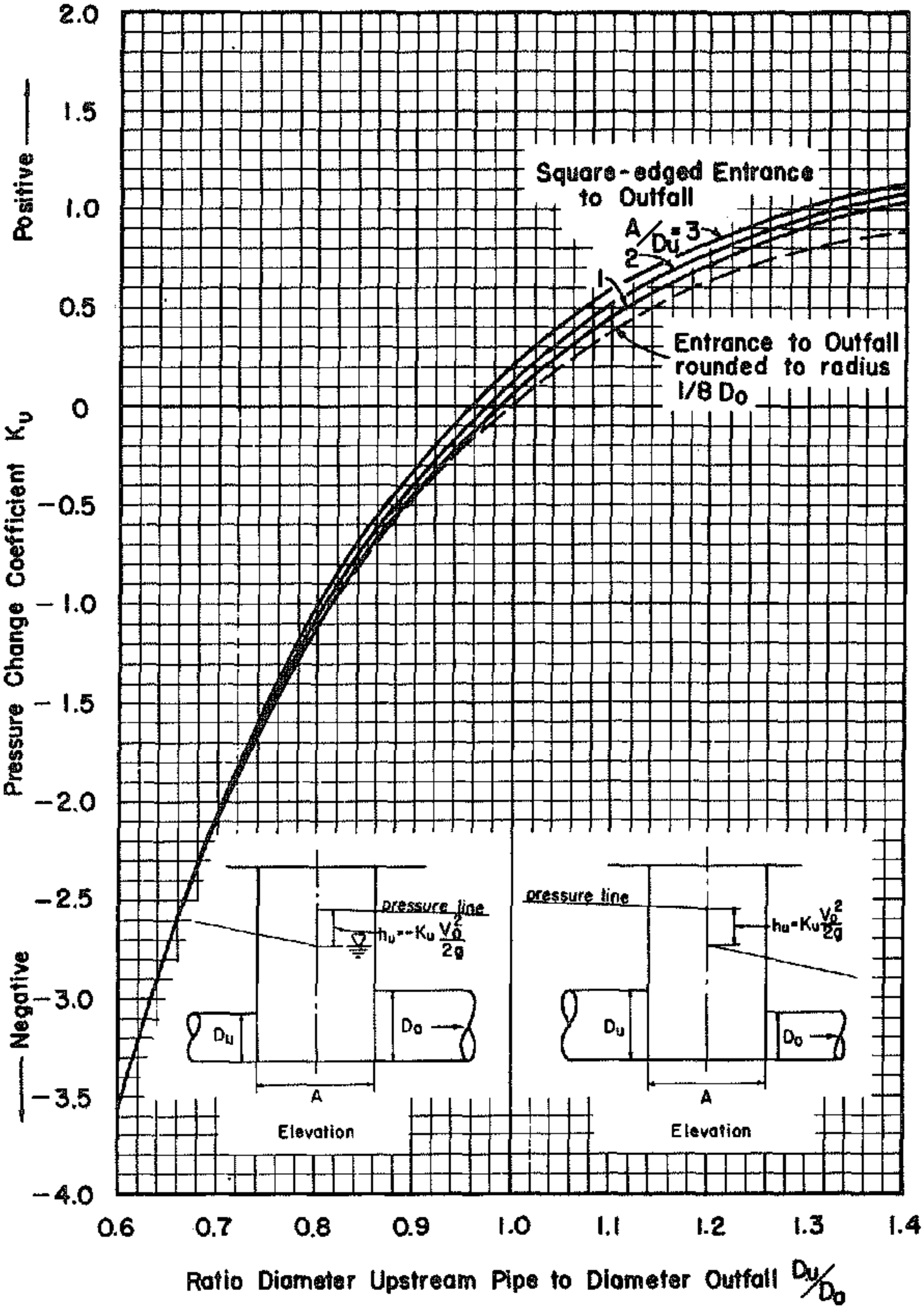


CHART IV-B FLOW STRAIGHT THROUGH ANY MANHOLE
(University of Missouri)

7. Add a positive h_U to (or subtract a negative h_U from) the elevation of the outfall pressure line at the junction center to obtain the elevation of the upstream pipe pressure line at the same location.
8. The water surface elevation in the junction corresponds to that of the upstream pipe, whether above or below the outfall pressure line.
9. Check to be sure the water surface elevation in the junction is below the top of the junction box so that overflow may not occur.

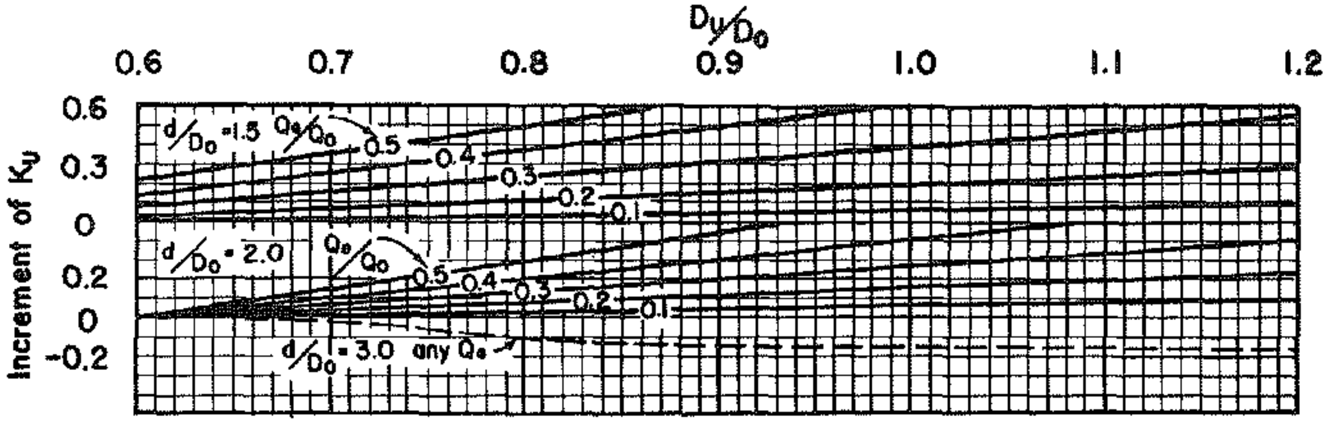
Comments: For a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U = 1$. K_U increases for distances A/D_U greater than 3, but such values are not usual in storm drain construction. For rounded entrances, the curve shown will apply with sufficient accuracy for all values of A/D_U up to 3.

Rectangular Manhole - Through Pipeline - Lateral Pipeline - Chart IV-C

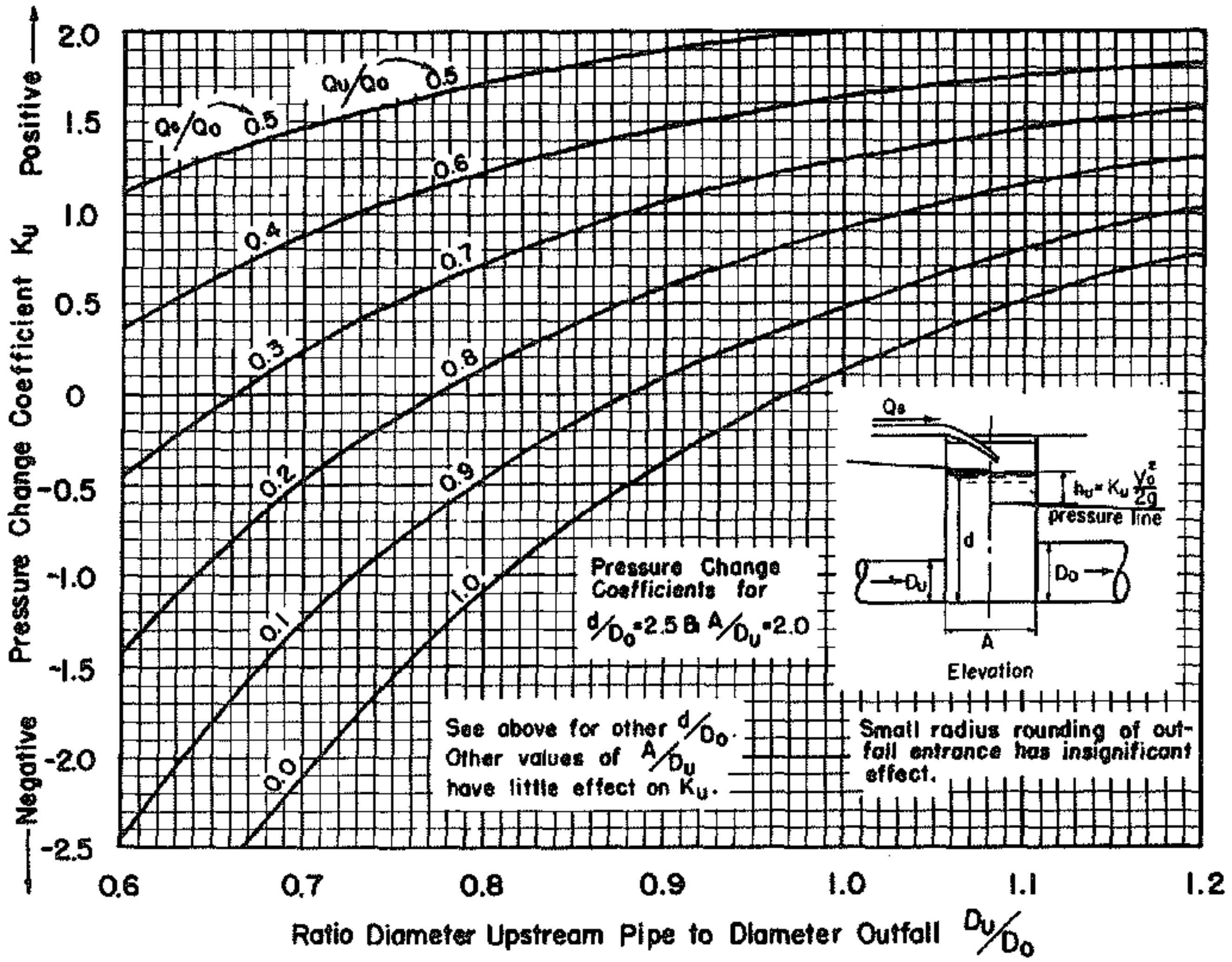
Pressure changes coefficients are presented in this Chart for use in determining the common elevation of the upstream in-line pipe pressure line and the water surface in the manhole. The in-line pipes connect at the manhole sides (long dimension) and must meet the alignment requirement stated for Chart IV-C. As much as half the total flow may enter through a top inlet. The main graph of Chart IV-C includes effects of various portions of grate flow for a relative water depth d/DZO of 2.5. Increments of K_U for othe relative water depths are shown in the supplemental graphs; positive increments for d/D_O less than 2.5 and negative for greater depths.

To use the Chart:

1. Determine the outfall pipe pressure line elevation - Gen. Instr. 2.
   Calculate velocity head in the outfall - Gen. Instr. 2.
3. Calculate the ratios D_U/D_O and Q_U/Q_O - Gen. Instr. 3. (The inlet flow ratio $Q_G/Q_O = 1 - Q_U/Q_O$).
4. Estimate a value for the water depth d.



Supplementary Chart for Modification of K_u for Depth in Inlet other than $2.5 D_o$



Pressure Change Coefficients for $d/D_o = 2.5$ & $A/D_u = 2.0$

See above for other d/D_o. Other values of A/D_u have little effect on K_u.

Small radius rounding of outfall entrance has insignificant effect.

CHART IV-C RECTANGULAR MANHOLE WITH THROUGH PIPELINE AND INLET FLOW

$\left(\frac{Q_s}{Q_o} \leq 0.5\right)$

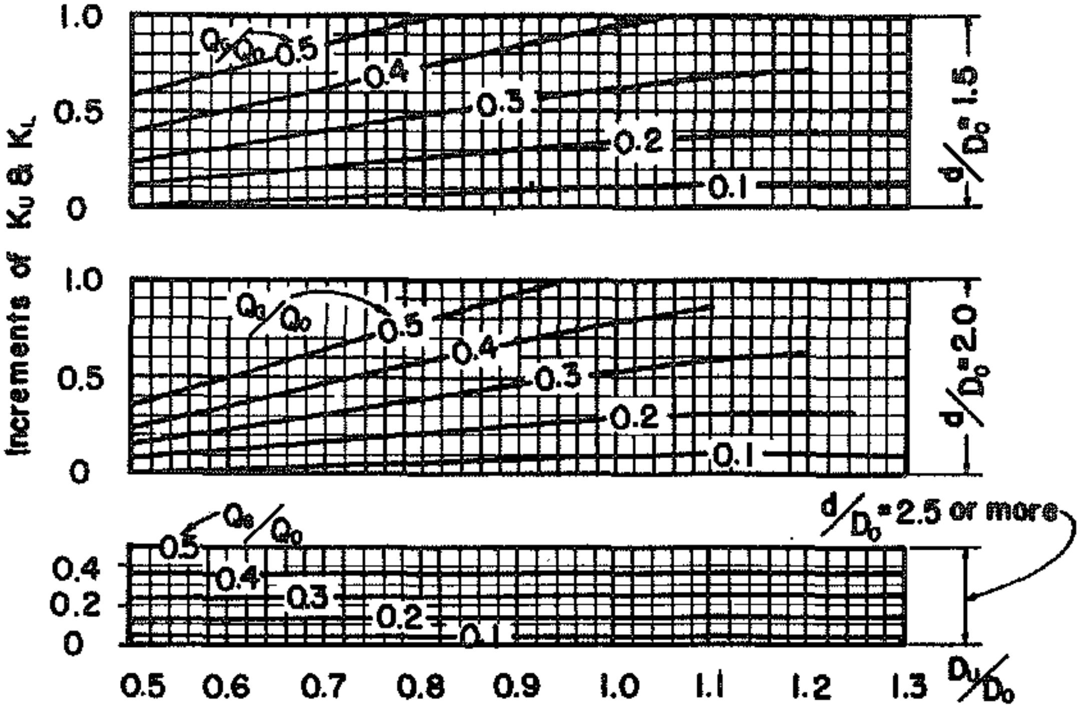
(University of Missouri)

- o Follow Gen. Instr. 4.
- o Estimate $K = 3\ Q_G/Q_O$.

5. Calculate the corresponding relative water depth d/D_O.
6. If the estimated d/D_O is approximately 2.5, enter the lower graph on Chart IV-C at the pipe size ratio D_U/D_O and read K_U at the curve of interpolated curve for Q_U/Q_O: then proceed as in Step 9.
7. If the estimated d/D_O is other than 2.5, follow Step 6, then enter the upper graph on Chart IV-C at the given D_U/D_O and determine the increment of K_U required to account for the effects of the estimated relative water depth d/D_O.
8. Add K_U from Step 6 and the increment from Step 7 to determine the total value of K_U. Note that negative values of K_U may occur.
9. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U according to Gen. Inst. 6.
10. Calculate h_U as indicated on the diagram on the Chart and by Gen. Instruction 7.
11. Add h_U to the elevation of the outfall pressure line at the inlet center to obtain a more precise value for the water depth d.
12. Repeat the above procedure with the improved value of d from Step 12. if necessary. Such repetition may not be necessary if the original estimated d/D_O of Step 5 was reasonably accurate.
13. Check to be be sure the water elevation is below the gutter elevation at the inlet so that inflow may be admitted.

Rectangular Manhole - Upstream Main and 90° Lateral pipe - With or Without Grate Flow - Chart IV-D

Pressure change coefficients are presented in this Chart for use in determining the common elevation of the two upstream pipe pressure lines and the water surface in the manhole. Flow into the combination inlet and junction box is supplied by an upstream main, in-line with the outfall and flowing through the short dimension of the manhole, and a 90° lateral pipe connected at one end of the box, supplemented by flow



Supplementary Chart for Modification of K_U & K_L for Grate Flow

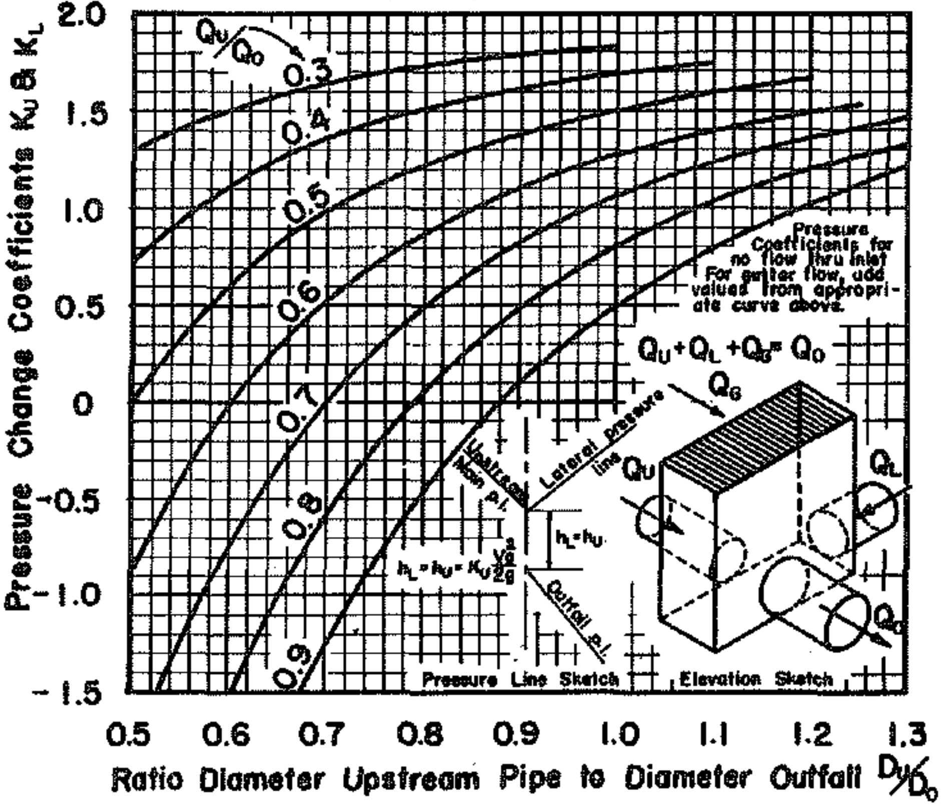


CHART IV-D RECTANGULAR MANHOLE WITH IN-LINE UPSTREAM MAIN & 90° LATERAL PIPE (WITH OR WITHOUT INLET FLOW)

(University of Missouri)

through a top inlet. The main graph of Chart IV-D applies directly for no flow into the manhole through the inlet. Increments of K_U and K_L for inlet flow conditions are shown in the supplementary graphs of the upper portion of the chart.

To use the Chart:

1. Determine the outfall pipe pressure line elevation - Gen. Instr. 1.
2. Calculate the velocity head in the outfall - Gen. Instr. 2.
3. Calculate the ratios D_U/D_O, Q_U/Q_O - Gen. Instr. 3.
4. If no inlet flow is involved, enter the lower graph on Chart at the pipe size ratio D_U/D_O and read K_U (or K_L) at the curve or interpolated curve for Q_U/Q_O, then proceed as in Step 10.
5. With inlet flow, estimate a value for the water depth d.
   - Follow Gen. Instr. 4
   - Estimate K = 1.5

   Calculate the corresponding relative water depth d/D_O.
7. Enter the lower graph and obtain K_U (or K_L) as in Step 4, this value applying for $Q_G/Q_O = 0$.
8. Enter the appropriate upper graph on Chart IV-D for the particular d/D_O nearest that estimated in Step 6 at the given D_U/D_O and determine the increment of K_U (or K_L) at the curve for Q_G/Q_O. *This increment accounts for the effects of inlet flow* and is always a positive value, even when K_U of Step 7 is negative.
9. Add K_U from Step 7 and the increment from Step 8 to obtain the total value of K_U. Note that in unusual cases the total value of K_U may be negative.
10. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U and K_L according to Gen. Instr. 6.
11. Calculate h_U (also equal to h_L) as indicated by the diagram on the Chart and by Gen. Instr. 7.
12. Add h_U to the elevation of the outfall pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure

line at this point. The elevtions of the upstream main pressure line, and lateral pipe pressure line and the water surface in the inlet will correspond.

13. From this water surface elevation, subtract the elevation of the inlet bottom to obtain a more precise value for the water depth d.
14. Repeat the above procedure with the improved value of d from Step 13 if necessary. Such repetition may not be necessary if the original estimated d/D_0 of Step 6 was reasonably accurate.
15. Check to be sure the inlet water surface elevation is below the top of the inlet so that inflow may be admitted.

Rectangular Manhole - In-Line Opposed Laterals With or Without Inlet Flow Chart IV-E

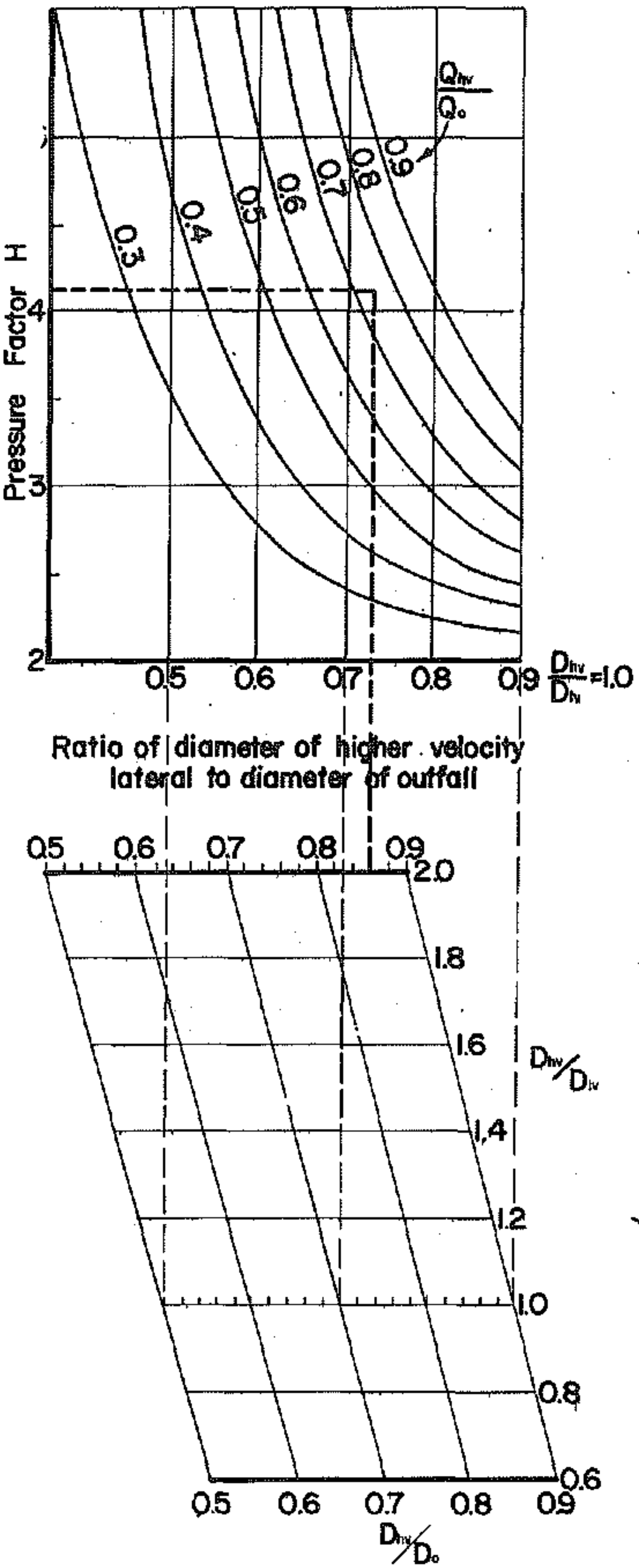
Pressure change coefficients are presented in this Chart for use in determining the elevation of the pressure line of the lateral carrying the lower-velocity flow of two in-line opposed lateral pipes supplying a combination junction and inlet box. The pressure change coefficient for the higher-velocity lateral is a constant and so is not read from the C-Chart. An inlet of this type may be used at a low point of street grade where lateral pipes supply flow from up-grade inlets in both directions, and the outfall pipe is located at right angles to the two lateral lines.

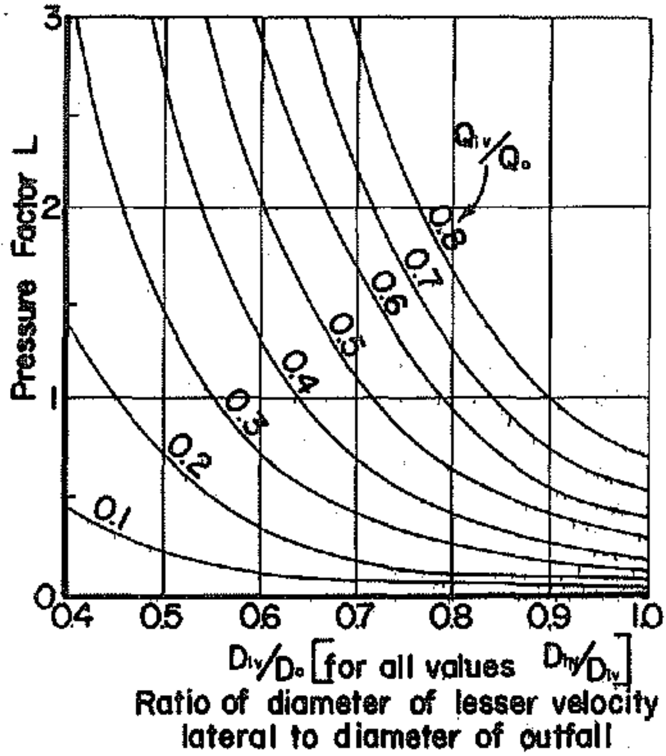
The Chart may be used for cases with all probable ratios of flow rates in the two laterals, with or without inlet flow. For this type of inlet and junction, the pressure changes are not modified by the depth of water in the inlet. The water surface elevation here will correspond to the pressure line of the higher-velocity lateral.

To use the Chart:

1. Determine the outfall pipe pressure line elevation - Gen. Instr. 1.
2. Calculate the velocity head in the outfall - Gen. Instr. 2.
3. Calculate the velocities in each of the laterals to determine which is the higher-velocity and which the lower-velocity lateral.

4. Calculate the ratios Q_G/Q_O, $Q_{HV}Q_O$, Q_{LV}/Q_O, D_{HV}/D_O, D_{LV}/D_O and D_{HV}/D_{LV} - Gen. Instr. 3
5. Determine H from the left-hand graph on Chart IV-E. Enter the figure at the pipe size ratio D_{HV}/D_O (note the relevant scale) and read H at the curve or interpolated curve for Q_{HV}/Q_O. In entering the graph, note that unequal size laterals (D_{HV}/D_{LV} not equal to 1.0 effect an offset of the scale for D_{HV}/D_O.).
6. Determine L from the right-hand graph on Chart IV-E. Enter the graph at the pipe size ratio D_{LV}/D_O (note only one scale is involved) and read L at the curve or interpolated curve of Q_{LV}/Q_O.
7. Calculate $K_{LV} = H - L$ with inlet flow involved. With no inlet flow, $K_{LV} = (H - L) - 0.2$.
8. $K_{HV} = 1.8$ with inlet flow involved. With no inlet flow, $K_{HV} = 1.6$.
9. Calculate $h_{LV} = K_{LV} (V_O{}^2/2g)$ and $h_{HV} = K_{HV} (V_O{}^2/2g)$.
10. Add h_{LV} to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the lower-velocity lateral pressure line at this point, similarly, add h_{HV} to the outfall pipe pressure line elevation to obtain the elevation of the higher-velocity lateral pressure line at the branch point.
11. Determine the water surface elevation in the inlet, which is equal to the lower of the two lateral presure line elevations that of the higher-velocity lateral).
12. Check to be sure the inlet water surface elevation is below the top of the inlet so that inflow may be admitted.

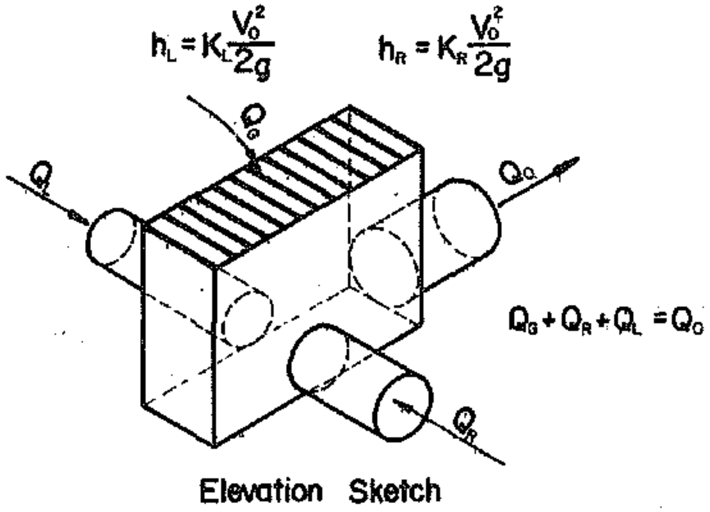




To find K_R or K_L for the right or left lateral pipe with flow at a lesser velocity than the other lateral, read H for the higher velocity lateral D and Q, then read L for the lower velocity lateral D and Q; then:

$$K_R \text{ (or } K_L) = H - L$$

K_R or K_L for the lateral pipe with higher velocity flow is always 1.8



Elevation Sketch

CHART IV-E RECTANGULAR INLET WITH IN-LINE OPPOSED LATERAL PIPES EACH AT 90° TO OUTFALL [WITH OR WITHOUT GRATE FLOW]

(University of Missouri)

Rectangular Manhole - Offset Opposed Laterals - With or Without Inlet Flow - Chart IV-F

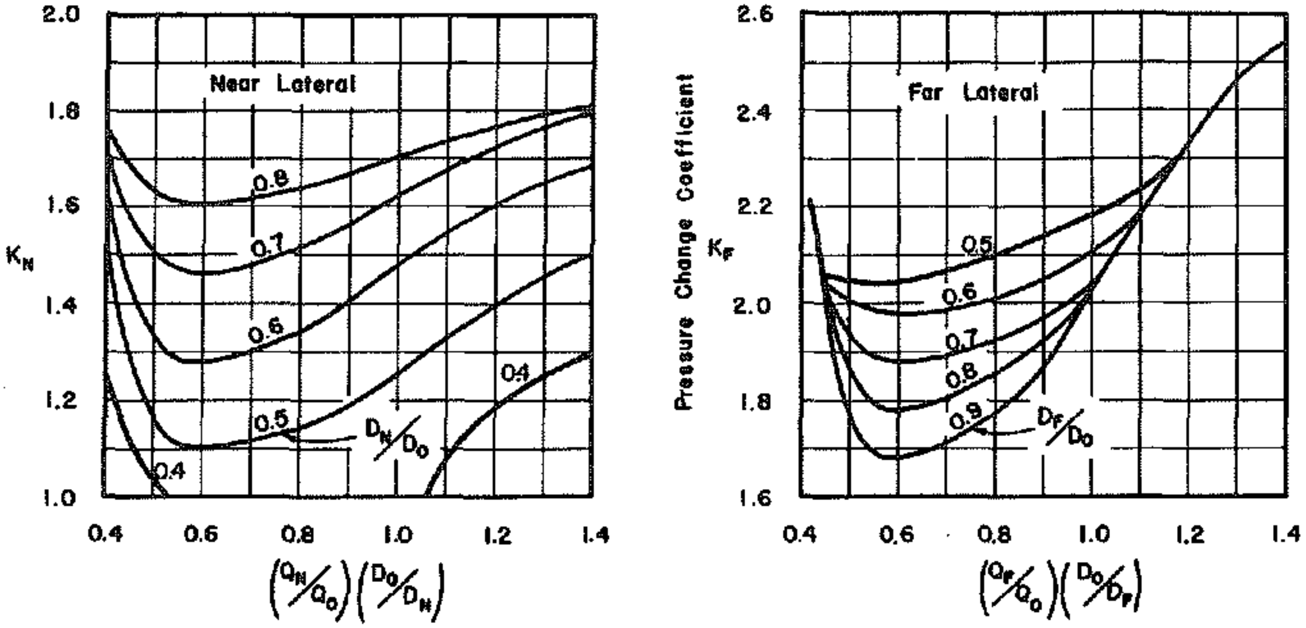
Pressure change coefficients are presented in this Chart for use in determining the elevations of the pressure lines of each of the two horizontally offset opposed lateral pipes supplying a combination junction and inlet box. The inlet is used in the same situations as those to which Chart IV-F applies, but the pressure rise of the lower-velocity lateral is restricted by locating the lateral pipes to enter opposite sides of the inlet box with their centerlines horizontally offset a distance not less than the sum of the two lateral pipe diameters. One lateral enters one side of the box near the outfall pipe end, and one, designated the far lateral, enters the opposite side near the other end.

This Chart is used for all probable ratios of flow rates in the two laterals, with or without inlet flow. For this type of junction the pressure changes are not modified by the depth of water in the manhole. The water surface elevation here will correspond to the pressure line of the far lateral.

Top use this Chart:

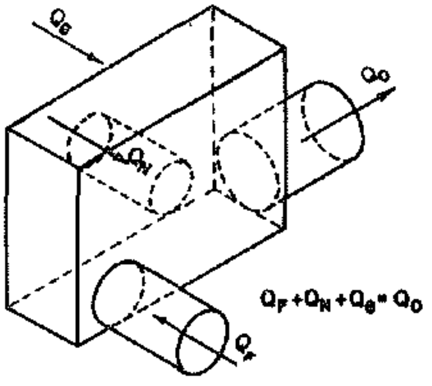
1. Determine the horizontal distance between the centerlines of the opposed flow laterals at the inlet; if more than the sum of the pipe diameters, this Chart will apply.
2. Determine the outfall pipe pressure line elevation at the branch points - Gen. Instr. 1. An average elevation applicable to both is sufficiently precise.
3. Calculate the velocity head in the outfall - Gen. Instr. 2.
4. Calculate the ratios Q_F/Q_O, Q_N/Q_O, D_F/D_O, and D_N/D_O, observing the nomenclature of Figure IV-5 - Gen. Instr. 3.
5. Calculate the factors $\frac{Q_F}{Q_O} \times \frac{D_O}{D_F}$ and $\frac{Q_N}{Q_O} \times \frac{D_O}{D_N}$.

6. For the far lateral, enter the right-hand graph of Chart IV-F at the abscissa value from Step (5) and read K_F at the curve or interpolated curve for D_F/D_O.
7. For the near lateral, obtain K_N from the left-hand graph by a similar procedure.
8. For a manhole with inlet flow, calculate h_F and h_N by multiplying the *outfall* velocity head by the corresponding coefficient K_F or K_N.
9. For a junction without inlet flow, calculate h_F and h_N by multiplying the outfall velocity head by the corresponding reduced coefficients $(K_F - 0.2)$ or $(K_N - 0.2)$.
10. Add h_F and h_N to the elevation of the downstream (outfall pipe) pressure line to obtain the elevations of the presure lines of the two laterals at their branch points.
11. Determine the water surface elevation in the inlet, which is equal to the far lateral pressure line elevation.
12. Check to be sure the inlet water surface elevation is below the top of the inlet so that inflow may be admitted.



$h_N = K_N \frac{V_O^2}{2g}$

$h_F = K_F \frac{V_O^2}{2g}$



Elevation Sketch

CHART IV-F RECTANGULAR MANHOLE WITH OFFSET OPPOSED LATERAL PIPES EACH AT 90° TO OUTFALL (WITH OR WITHOUT INLET FLOW)

(From University of Missouri)

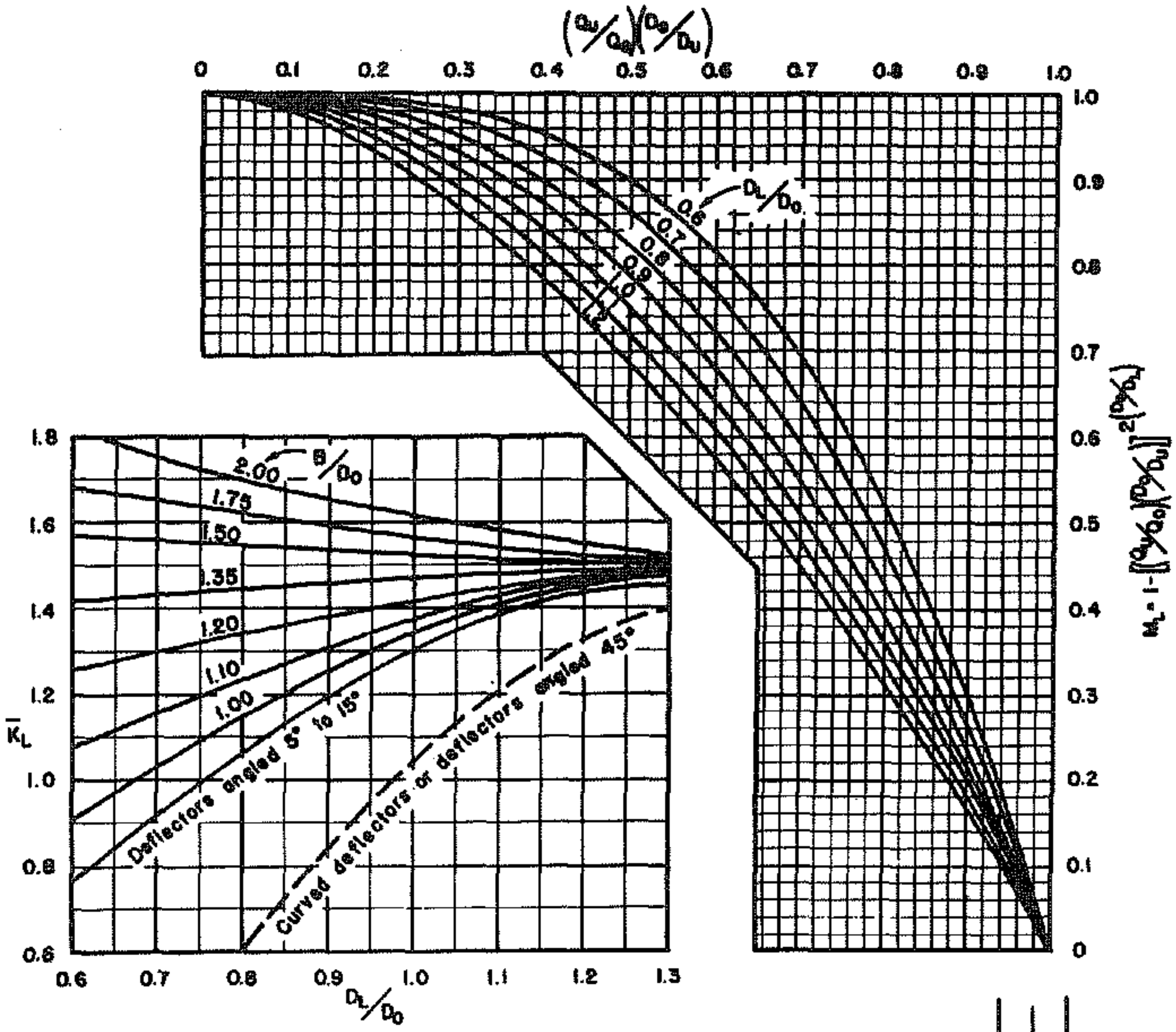
Square Manhole - 90° Deflection - Chart IV-G

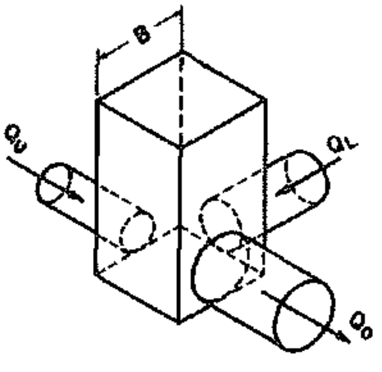
Pressure change coefficients are presented in this Chart for use in determining the elevation of the pressure line of an upstream pipe connected by means of a square manhole to an outfall pipe at a 90° angle. The manhole conditions covered by this Chart do not involve an upstream pipe in-line with the outfall pipe. For this and other manhole figures, the lateral pipe is designated by the subscript L irrespective of its right-hand or left-hand position. The coefficients given by the Chart apply directly to manholes having a square-edged entrance to the outfall pipe. Coefficients for a rounded entrance are obtained by reduction of the Chart values as shown in Table IV-2. The design of manholes with deflector devices is discussed separately.

To use the Chart:

1. Determine the outfall pipe pressure line elevation - Gen. Instr. 1.
2. Calculate the velocity head in the outfall - Gen. Instr. 2.
3. Calculate the ratios D_L/D_O and B/D_O - Gen. Instr. 3.
4. Enter the lower graph of Chart IV-G at the pipe size ratio D_L/D_O and $\bar{K}_L$ at the curve or interpolated curve for the manhole size ration B/D_O. For all flow from a lateral $K_L = \bar{K}_L$.
5. For a rounded outfall pipe entrance or one formed by a pipe socket reduce the figure value of $\bar{K}_L$ by 0.3 as defined by Gen. Instr. 6.
6. Calculate the change of pressure $h_L = K_L \times \frac{V_O{}^2}{2g}$

   (always positive for 90° deflections).
7. Add h_L to the elevation of the outfall pressure line at the branch point to obtain the elevation of the lateral pipe pressure line at this point.
8. The water surface elevation in the manhole will be above the lateral pipe pressure line. To determine the water-surface elevation, use Chart IV-H, as instructed in Steps 12 through 18 of the instructions for a square manhole at the junction of a 90° lateral with a through main.





Elevation Sketch

To find K_L for the lateral pipe, first read $\bar{K}_L$ from the lower graph. Next determine M_L. Then

$$K_L = \bar{K}_L \times M_L$$

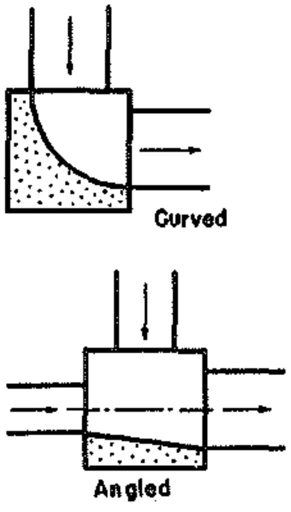
Dashed curve for curved or 45° angle deflectors applies only to manholes without upstream in-line pipe.

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\bar{K}_L$ by 0.2 for combining flow.

For $Q_u/Q_o \times D_o/D_u > 1$ use Chart IV-1.

For $D_L/D_o < 0.6$ use Chart IV-1.



Plan of Deflectors

$$h_L = K_L \frac{V_o^2}{2g}$$

CHART IV-G MANHOLE AT 90° DEFLECTION OR ON THROUGH PIPELINE AT JUNCTION OF 90° LATERAL PIPE (LATERAL COEFFICIENT).

(From University of Missouri)

9. Check to be sure the water surface elevation is above the pipe crowns to justify using these charts and that it is sufficiently below the top of the manhole to indicate safety from overflow.

Round Manhole - 90° Deflection - Chart IV-G

Pressure change coefficients may also be obtained from this Chart for use in determining the elevation of the pressure line of an upstream pipe connected by means of a round manhole to an outfall pipe at a 90° angle.

To use the Chart:

1. Proceed as instructed in Steps 1 through 4 for a square manhole at 90° deflection to obtain a base value of $\bar{K}_L$ for the particular values of D_L/D_O and B/D_O.
2. To provide for the effects of the round manhole cross section, reduce $\bar{K}_L$ in accordance with the following table:

TABLE IV-2

REDUCTIONS FOR $\bar{K}_L$ - MANHOLE WITH ROUNDED ENTRANCE

Reductions for $\bar{K}_L$

D_L/D_O B/D_O	0.6	0.8	1.0	1.2
1.75	0.4	0.3	0.2	0.0
1.33	0.3	0.2	0.1	0.0
1.10	0.2	0.1	0.0	0.0

The reduced values apply for a sharp-edged entrance to the outfall pipe.

3. With a well-rounded entrance to the outfall pipe from a round manhole, reduce $\bar{K}_L$ of Step 1 by 0.3 with no further reduction for manhole cross section shape.
4. Follow Steps 6 through 9 as detailed for square manholes at a 90° deflection.

Square or Round Manhole - 90° Deflection With Deflectors - Chart IV-G

Pressure change coefficients are presented in this Chart for use in determining the evaluation of the pressure line of an upstream pipe connected to an outfall pipe at a 90° angle by means of a square or round manhole modified by flow deflectors. Deflectors in a manhole effectively eliminate the effects related to the shape of the manhole. The basic types of deflector walls which may be constructed in square or round manholes to effect a reduction of the pressure loss are detailed and described in the main text and Figure IV-3.

The deflectors which are more easily constructed and are as effective as more complex types provide a vertical wall to guide the flow toward the outfall pipe diameter and must fill in that part of the manhole opposite the lateral pipe exit so that it is flush with the side of the outfall pipe. Three basic types of such deflector walls are possible and are included in the curves of Chart IV-G. These three are (1) walls parallel to the outfall pipe centerline or 0° walls, (2) inclined walls, limited to an angle of about 15° to the outfall centerline if an upstream in-line pipe is to be used, and (3) walls at 45° to both the lateral and outfall pipes, or walls curved on a radius of about the manhole dimension extending from lateral to outfall, and therefore to be used only when no upstream in-line pipe is involved. Rounding of the corner formed between the deflector wall and the manhole floor is not required, and may be detrimental in some cases.

To use the Chart:

1. Determine the outfall pipe pressure line elevation - Gen. Instr. 1.
2. Calculate the velocity head in the outfall - Gen. Instr. 2.
3. Classify the type of deflector used:
   - o Parallel wall - 0°
   - o Inclined wall - 5° to 15°
   - o 45° or curved wall
4. Calculate the ratios D_L/D_O and B/D_O. No distinction between square and round manholes is necessary.

5. If B/D_O is 1.5 or less, enter the lower graph of the chart at the ratio D_L/D_O and read $\bar{K}_L$ at the curve for the appropriate deflector type. In the case of a parallel wall, use the curve for B/D_O = 1.00.
6. If B/D_O is more than 1.5 and less than 2.0, use the same dashed curve for 45° or curved deflectors, use the curve for B/D_O = 1.10 for 5° to 15° angle deflectors, and use the curve for B/D_O = 1.20 for 0° angle deflectors.
7. A rounded entrance to the outfall pipe or one formed by a pipe socket is less effective in reducing the pressure change with deflectors than when deflectors are not used. A reduction of $\bar{K}_L$ by 0.1 may be justified.
8. Calculate the change of pressure

$$h_L = K_L \frac{V_O^2}{2g} \quad (\text{for } Q_L, = Q_O, \ K_L = \bar{K}_L).$$

9. Add h_L to the elevation of the outfall pressure line at the branch point to obtain the elevation of the lateral pipe pressure line at this point.
10. The water-surface elevation in the manhole will be above the lateral pipe pressure line. To determine the water surface elevation, use Chart IV-H, *as instructed in Steps 2 through 8 for deflectors in a* manhole at the junction of a 90° lateral with a through main.
11. Check to be sure the water surface elevation is above the pipe crowns to justify using these charts and that it is sufficiently below the top of the manhole to indicate safety from overflow.

Square Manhole - Upstream Pipe and Lateral - Charts IV-G and IV-H

Pressure change coefficients for use in determining the elevation of the pressure line of the 90° lateral pipe are obtained from Chart IV-G and the coefficients for the upstream in-line pipe are obtained from Chart IV-H. The diameter of the lateral pipe must be at least 0.6 of the diameter of the outfall pipe to permit use of these figures. Pressure changes at junctions of smaller laterals may be obtained through use of Chart IV-I. The coefficients given by the charts apply directly to a square-edged entrance to the outfall pipe. Coefficients for a rounded entrance are obtained by reduction of the chart values as stated below. *The design of manholes with deflector devices is discussed separately.*

To use the Charts:

1. Determine the outfall pressure line elevation - Gen. Instr. 1.
2. Calculate the velocity head in the outfall - Gen. Instr. 2.
3. Calculate the ratios Q_U/Q_O, D_U/D_O, and D_L/D_O. If D_L/D_O is less than 0.6, use Chart IV-I instead of Charts IV-G and IV-H.
4. Calculate the ratio B/D_O and note if the outfall entrance is rounded.
5. Calculate the factor $(Q_U/Q_O)x(D_O/D_U)$; if this is greater than 1.00, use Chart IV-I, instead of Charts IV-G and IV-H.

For Lateral Pipe:

6. Enter the lower graph of Chart IV-G at the ratio of D_L/D_O and read $\bar{K}_L$ at the curve or interpolated curve for the ratio B/D_O.
7. For a rounded outfall pipe entrance or one formed by a pipe socket as defined by Gen. Instr. 6, reduce the chart values of $\bar{K}_L$ by 0.2.
8. Determine the factor M_L by entering the upper graph of Chart IV-G at the value of the factor (Q_U/Q_O) x $(D_O/D_U$ and at the curve or interpolated curve for D_L/D_O.
9. Calculate $K_L = M_L \times \bar{K}_L$.
10. Calculate the lateral pipe pressure change

$$K_U \times \frac{V_O{}^2}{{}^2g}$$

11. Add h_L to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the lateral pipe pressure line at this point.

For Upstream In-Line Pipe:

12. Enter the lower graph of Chart IV-H at the ratio of D_L/D_O and read $\bar{K}_U$ at the curve or interpolated curve for B/D_O.
13. For a rounded entrance to the outfall pipe or one formed by a pipe socket, reduce $\bar{K}_U$ by 0.2.
14. Determine the factor M_U from the upper graph of Chart IV-H.
15. Calculate $K_U = M_U \times \bar{K}_U$.
16. Calculate the upstream in-line pipe pressure change

$$h_U = K_U \times \frac{V_O^2}{2g}$$

17. Add h_U to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point.

For Water Surface:

18. The water-surface elevation in the manhole will correspond to the upstream in-line pipe pressure line at the branch point.
19. Check to be sure that the water surface elevation is above the pipe crowns to justify using these charts and that it is sufficiently below the top of the manhole to indicate safety from overflow.

Round Manhole - Upstream Pipe and Lateral - Charts IV-G and IV-H

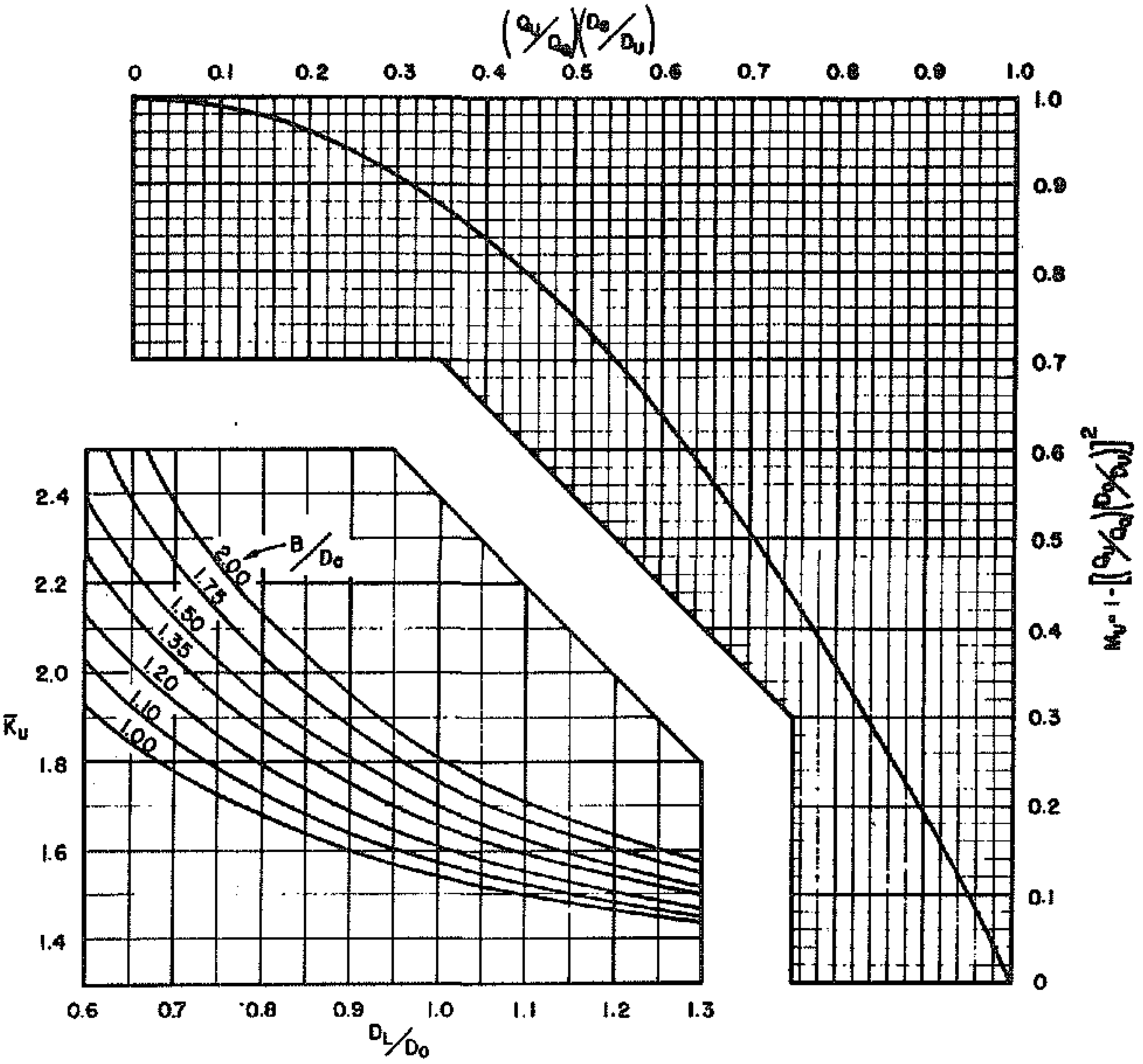
Pressure change coefficients may also be obtained from Charts IV-G and IV-H for use in determining the elevations of the pressure lines of the 90° lateral pipe and the upstream in-line pipe connected by a round manhole to an outfall pipe.

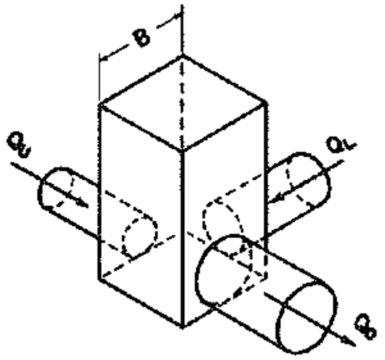
To use the Chart:

1. Proceed as instructed by Steps 1 through 6 for a square manhole at a similar junction to obtain a base value of $\bar{K}_L$.

For Lateral Pipes:

2. To provide for the effects of the round manhole cross sections, reduce $\bar{K}_L$ in accordance with the following table:





Elevation Sketch

To find K_U for the upstream main, first read $\overline{K}_U$ from the lower graph. Next determine M_U. Then

$$K_U = \overline{K}_U \times M_U$$

For manholes with deflectors at 0° to 15°, read $\overline{K}_U$ on curve for $B/D_o = 1.0$

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\overline{K}_U$ by 0.2 for combining flow.

For deflectors refer to sketches on Chart IV-G

For $Q_U/Q_o \times D_o/D_U > 1$ use Chart IV-I

For $D_L/D_o < 0.6$ use Chart IV-I

$$h_U = K_U \frac{V_o^2}{2g}$$

CHART IV-H MANHOLE ON THROUGH PIPELINE AT JUNCTION OF A 90° LATERAL PIPE (IN-LINE PIPE COEFFICIENT)

(From University of Missouri)

TABLE IV-3

REDUCTIONS FOR K_L FOR ROUND MANHOLES

B/D_O \ D_L/D_O	0.6	0.8	1.0	1.2
1.75	0.4	0.3	0.2	0.0
1.33	0.3	0.3	0.1	0.0
1.10	0.2	0.1	0.0	0.0

The reduced values apply for a square-edged entrance to the outfall pipe.

3. With a well-rounded entrance to the outfall pipe from a round manhole, reduce K_L obtained in Step 2 by 0.1.
4. Determine the factor M_L from the upper graph of Chart IV-G and proceed as instructed in Steps 8 through 11 for a square manhole to complete the determination of the elevation of the lateral pipe pressure line.

Upstream In-Line Pipe:

5. Proceed as instructed in Steps 12 through 17 for a square manhole at a similar junction to obtain the elevation of the upstream in-line pipe pressure line. Note that no reduction of K_U is to be made for effects of the round manhole cross section.

For Water Surface:

6. Proceed as instructed by Steps 18 and 19 for a square manhole at a similar junction.

Square or Round Manhole - Upstream Pipe and Lateral - Deflector - Charts IV-G and IV-H

Pressure change coefficients are also presented in Charts IV-G and IV-H for use in determining the elevations of the pressure lines of the lateral and in-line pipes at a junction of this type, with either a square of a round manhole modified by flow deflectors. Deflectors in a manhole effectively eliminate the effects related to the shape of the manhole. Deflector types are described in the instructions for use of Chart IV-G for a manhole with deflectors at a 90° defection of a storm drain. The curved and 45° deflectors cannot be used in a manhole on a through pipeline because of the space required for through in-line flow.

To Use the Chart:

1. Proceed as instructed in Steps 1 through 9 for deflectors in a manhole at a 90° deflection, disregarding the references to 45° or curved walls. Through use of Chart IV-G these steps will give the elevation of the lateral pipe pressure line at the branch point. As noted in the instructions for a manhole of this type without deflectors, Chart IV-I must be used when $D_L/D_O < 0.6$ or

$$\left(\frac{Q_U}{Q_O} \times \frac{D_O}{D_U}\right) > 1.00$$

For Upstream In-Line Pipe:

2. Enter the lower graph of Chart IV-H at the ratio of D_L/D_O and read $\bar{K}_U$ for all manhole sizes and any deflector wall angle from 0° to 15° at the curve for $B/D_O = 1.00$.
3. For a rounded entrance to the outfall pipe or one formed by a pipe socket, reduce $\bar{K}_U$ by 0.1.
4. Determine the factor M_U from the upper graph of Chart IV-H.
5. Calculate $K_U = M_U \times \bar{K}_U$.
6. Calculate the upstream in-line pipe pressure change

$$h_U = K_U \times \frac{V_O^2}{2g}$$

7. Add h_U to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point.

For Water Surface:

8. The water-surface elevation in the manhole will correspond to the upstream in-line pipe pressure line at the branch point.
9. Check to be sure that the water-surface elevation is above the pipe crowns to justify using these charts and that it is sufficiently below the top of the manhole to indicate safety from overflow.

Square or Round Manhole - Upstream Pipe with Small Lateral or Lateral Connecting With no Manhole - Chart IV-I

Pressure change coefficients are presented in Chart IV-I for use in determining the common elevation of the pressure lines of the lateral and in-line pipes at a junction of this type for cases of pipe sizes or flow divisions outside the range over which Charts IV-G and IV-H may be applied. *Charts IV-G and IV-H are more reliable within their range* and should be used if possible. Neither manhole shape nor size nor relative size of lateral pipe modify the coefficients of Chart IV-I. The chart may also be used for direct connection of a 90° lateral to a main without use of a manhole. The coefficients of the chart apply directly to a square-edged entrance to the outfall pipe. Coefficients for a rounded entrance are obtained by reduction of the Chart values as stated below. Deflectors in the manhole are not effective in the ranges covered by Chart IV-I and therefore need not be used.

To use the Chart:

1. Determine the outfall pipe pressure line elevation - Gen. Instr. 1.
2. Calculate the velocity head in the outfall - Gen. Instr. 2.
3. Calculate the ratios D_L/D_O, D_U/D_O, and Q_U/Q_O. Note that use of Charts IV-G and IV-H is advisable if the size and flow factors are within their range. Chart IV-I should not be used for $Q_U/Q_O \leq 0.7$ if other solutions are possible.
4. Note whether the outfall entrance is to be rounded or formed by a pipe socket as defined by Gen. Instr. 6.
5. Enter Chart IV-I at the ratio D_U/D_O and read K_U (also equal to K_L) at the curve or interpolate curve for Q_U/Q_O.
6. If $\frac{Q_U}{Q_O} \times \frac{D_O}{D_U}$ was found to be greater than 1.00 in an attempt to

   use Charts IV-G and IV-H, K_U of Step 5 will be negative in sign, thus providing a check on proper use of the figures.
7. For rounded entrance from the manhole to the outfall pipe use the reduced values from the Figure.

8. Calculate the change of pressure

$$h_U = h_L = K_U \times \frac{V_O^2}{2g}$$

h_U and h_L are positive or negative depending on the sign of K_U as read from the figure.

9. Add a positive h_U to or subtract a negative h_U from the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point. The elevation of the lateral pipe pressure line at the branch point and the water surface elevation in the manhole will correspond to the upstream in-line pipe pressure line elevation found in Step 9.
10. Check to be sure that the water-surface elevation is above the pipe crowns to justify using these charts and that it is sufficiently below the top of the manhole to indicate safety from overflow.

Flow Straight Through a Deflection - Chart IV-J

Pressure change coefficients are presented in the Chart for use in determining the elevation of the pressure line of an upstream in-line pipe relative to that of the outfall. The cases to which the Chart may be applied are shown on the Figure. No flow other than that from the upstream pipe may be involved where this Chart is applied.

To use the Chart:

1. Determine the outfall pipe pressure line elevation - Gen. Instr. 1
2. Calculate the velocity head in the outfall - Gen. Instr. 2.
3. Determine the deflection angle α.
4. Enter Chart IV-J at the particular deflection angle to the proper curve and read the appropriate loss coefficient.
5. Calculate h_U - Gen. Instr. 7.
6. Add a positive h_V to the elevation of the outfall pressure line at the manhole center to obtain the elevation of the upstream pipe pressure line at the same location.

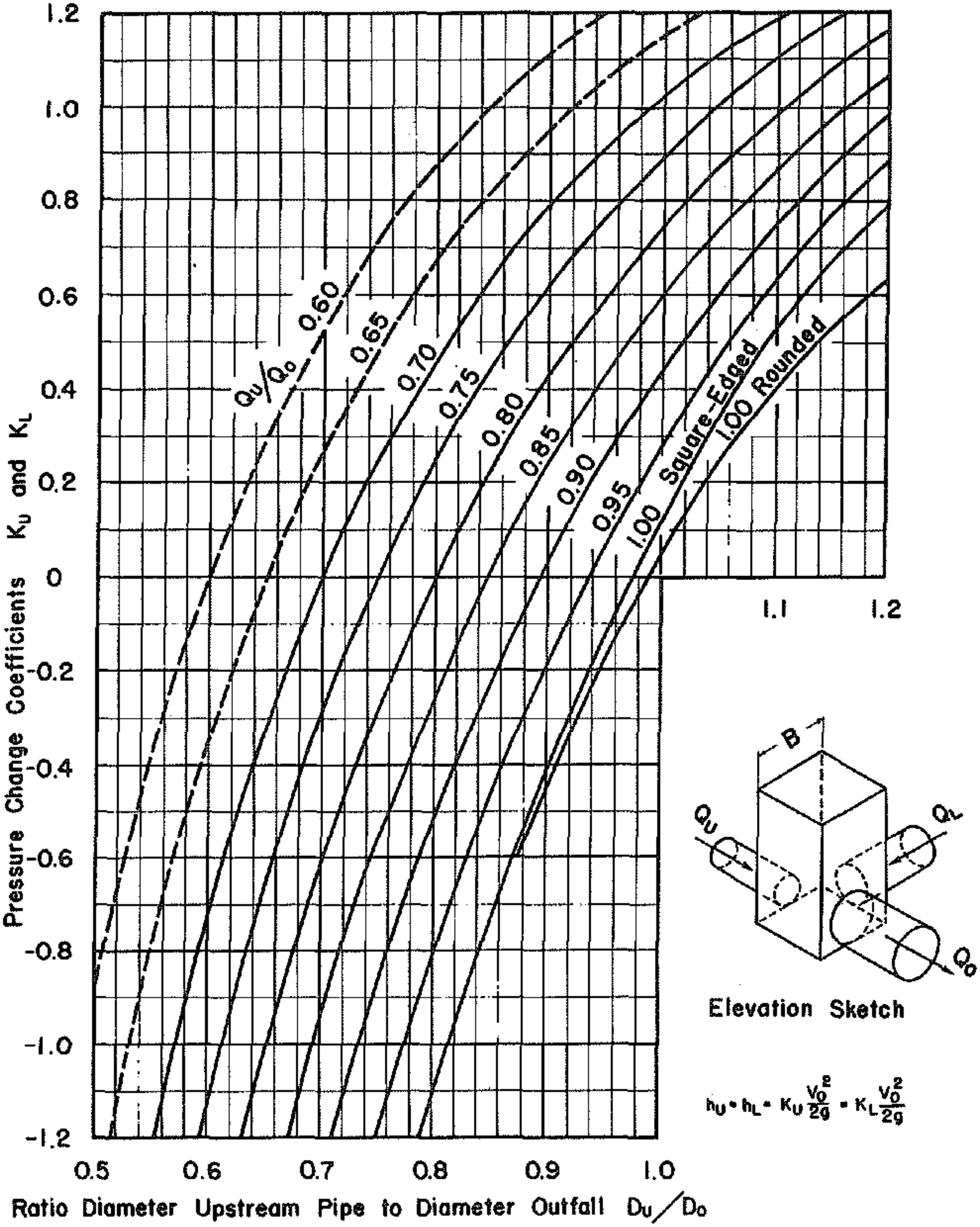


CHART IV-1 MANHOLE ON THROUGH PIPELINE AT JUNCTION OF A 90° LATERAL PIPE
(FOR CONDITIONS OUTSIDE RANGE OF CHARTS IV-G & IV-H
PROVIDED $Q_U/Q_O < 0.7$) *(From University of Missouri)*

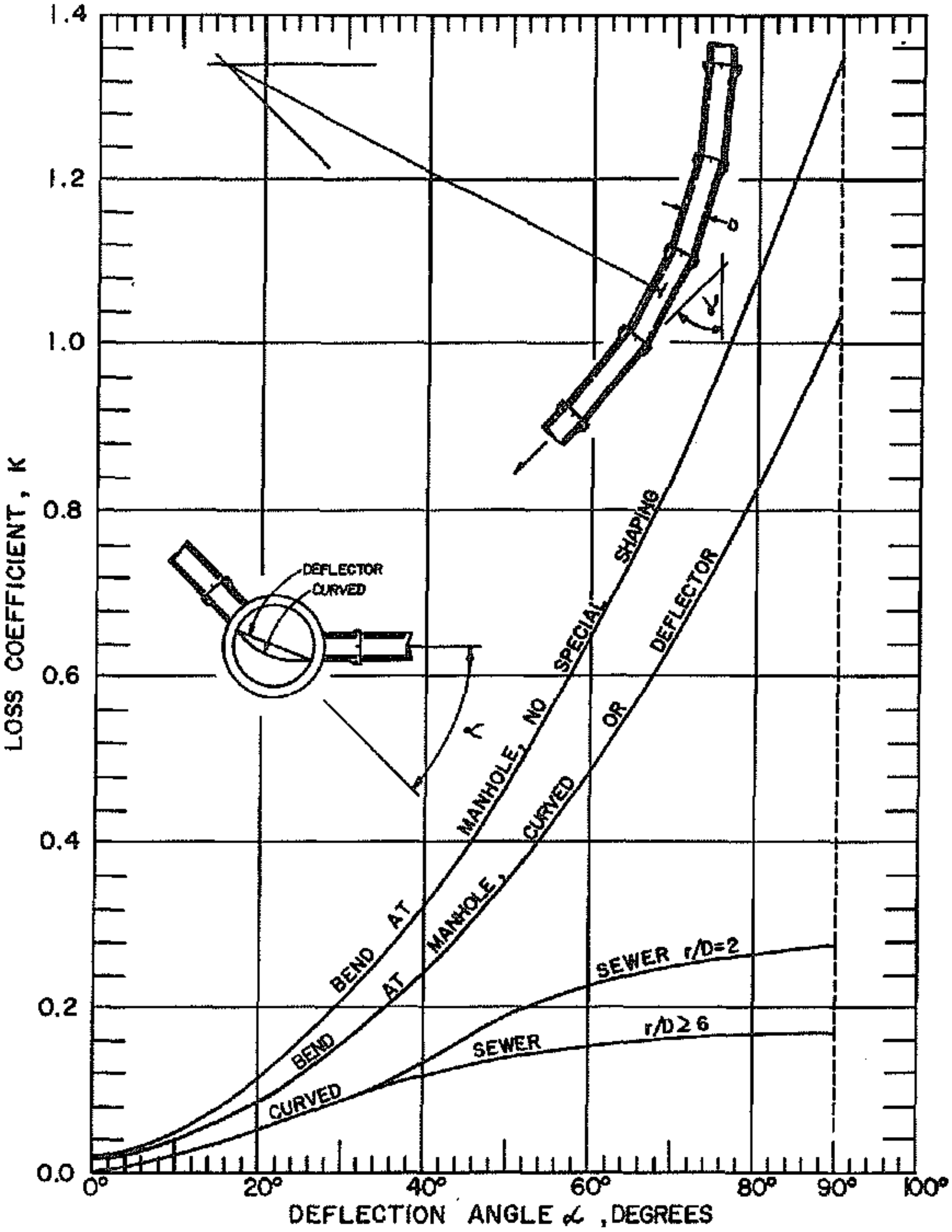


CHART IV-J SEWER BEND LOSS COEFFICIENT

(From University of Missouri)

7. The water surface elevation in the manhole corresponds to that of the upstream pipe.
8. Check to be sure the water surface elevation in the junction is below the top of the manhole so that overflow may not occur.

DESIGN METHODOLOGY FOR OPEN CHANNEL FLOW

HGL in upstream pipe in-line with outlet will seek normal depth when the slope of the pipe is greater than the slope required for full flow. Should the slope of the pipe be less than that required to flow full, the HGL will be at an elevation greater than the crown of the pipe. Drawdown effects will be observed near the outfall from the pipe. In this case, the depth will pass through critical depth at or near the point of outfall. Backwater or drawdown calculations for large diameter pipes should be made along the length of the pipe to determine whether normal depth or pressure flow is attained before the next manhole (see Chapter V, Part II).

For the size of pipes normally encountered in storm sewer design, it is reasonable to assume a straight water surface. It is also assumed that the energy grade line is a parallel to the pipe grade, and that any losses other than pipe friction may be accounted for by assuming point losses at each manhole.

The basic approach to design of open channel flow in storm sewers should be to calculate the energy grade line along the system. Once the discharge has been determined and a pipe size and slope assumed for a given section, the d/D and v/V full ratios can be determined from a graph of Hydraulic Elements for Circular Conduits (see Figures IV-4 through IV-8).

The next step is to calculate the energy grade line:

$$H = Z + d + (V^2/2g). \qquad \text{Eq. IV-4}$$

At each manhole the energy grade line of all pipes should coincide, allowing for reasonable values of head loss to the junction. Under certain conditions, this would indicate an upstream invert lower than the downstream invert. Inverts should be set at the same elevations under such circumstances.

The usual method of stating head losses at manholes is in terms of a constant K times the velocity head of the conduit in question,

$$h_e = (K)V^2/2g. \qquad \text{Eq. IV-5}$$

A difficulty in design of systems is the determination of the value of K.

Simple Transitions in Pipe Size

Simple transitions in conduit size in a manhole with straight through flow may be analyzed by the following equation:

$$h_e = K\Delta \ (V^2/2g) \qquad \text{Eq. IV-6}$$

The term$\Delta(V^2/2g)$ refers to the change in velocity head in the upstream and downstream conduits. The value of K varies from 0.1 for increasing velocity to 0.2 for decreasing velocity transitions if flow is sub-critical. For super-critical flow, greater values of K are probable, but have not been determined.

Bends

Reliable headloss coefficients through bends in open channel flow are almost entirely lacking. Reasonable assumptions may be made by conduits utilizing existing information available on losses in bends and pressure conduits.

Junctions

Values for head loss coefficients at junctions on storm sewers flowing as open channels are not readily available. Complicated methods for calculating head loss at certain types of junctions are available and are justified for certain situations.

Unless unusual conditions exist, the figures and procedures for pressure conduits should be used. Energy grade lines should be matched to insure continuity; that is, the upstream energy grade line equals the downstream energy grade line plus head loss.

Storm Water Inlets

As can be noted in Chart IV-A, the depth of water in an inlet has a profound effect on the energy losses in a catch basin. The shallower the depth, the greater is the head loss. Normal culvert design aids are not applicable to this condition. The water falling into the inlet causes significant turbulance and energy losses.

For this condition and for significant grate flow into any junction, the applicable curves for the pressure conduit analysis should be used.

OUTLETS

The outlet of the storm sewer system deserves special discussion. New storm sewers are constructed to serve areas because drainage problems exist or to serve newly developing areas.

In the former situation, unplanned ponding in urban areas is eliminated by the later situation, the runoff rate and volume are increased over the pre-development condition. Unless special ponding provisions are undertaken as a part of the new storm sewer or development program, the resulting conditions below the outfall can be disasterous.

Damage may result from overland flooding and/or stream degradation or aggradation. In fact, due to increased runoff from urbanization, major drainage facilities may be needed where, prior to upstream development (as storm sewer construction), runoff related problems are slight or non-existant.

The designer should be aware of the potential liability aerated by the previously described conditions and is referred to Chapter III of Part I, "Legal Aspects." To avoid these types of problems, careful examination of the outlet conditions is warranted.

Outlet Location

Cases in which the major drainageway is readily accessible by the storm sewer are easily solved. However, when it is not readily apparent that the storm sewer will discharge into a previously delineated major drainageway, the question of an acceptable outlet point becomes important.

It is often possible, in a developing area, to terminate a storm sewer in an open channel which flows to a major drainageway. Final development of the area may require that the channel be replaced with a storm sewer. The channel shall be designed to convey the runoff just as would any other open channel, but with the approach that it will only be temporary. In any case, it is necessary that the outfall facilities be modified to eliminate potential harm caused by storm sewer construction and/or development activities.

Hydraulic Design

The actual hydraulic design of an outlet can only proceed after the location has been approved.

The normal water level in the receiving major drainageway should be determined for the design storm frequency. If this elevation is above the crown of the sewer, it is less likely that special outlet control devices will be necessary to prevent erosion. However, the outlet should be reviewed for possible erosion tendencies if the major drainageway is flowing at less than design depth.

Erosion control measures must be taken when the possibility exists of affecting the outfall channel. These may vary from involved stilling basins to simple riprap. Chapter V, Major Drainage, contains a discussion of various types of outlet structures. In particular, impact basins are useful for application to storm sewer design.

Junctions of large sewers with major drainageways must receive thorough investigation. If design methods are not available which will adequately analyze the situation, model testing should be initiated.

SUGGESTED DESIGN STANDARDS

The following Chapter covers specific requirements of final construction drawings and specifications for storm sewer systems. The items are generally applicable to all drainage facilities, i.e., major drainageway or conduits, but are intended to apply directly to storm sewers.

Reference Data

A complete review of all utilities, property locations, and other items which may affect construction of the sewer must be initiated including, but not limited to, the following:

Property Data. Subdivision plats, section lines, and corners, utility easements, highway rights-of-way, and any other property data.

Street and Highway. As-built drawings of existing streets or highways shall be obtained wherever they would affect design or construction. Final grades, street geometries, types of construction, and all other street details relative to the design, construction, or operation of the storm sewer system must be available to the designer, or he must have control over their final establishment to insure proper functioning of the total drainage system which includes both the streets and the storm sewers.

Existing Utilities. Records of all existing utilities, pipelines, and structures both above and below ground must be obtained. Plans for future installation should be given due consideration to see if possible conflicts may be eliminated. Data that is incomplete or questionable should be checked by field survey.

- Water Lines -- size, type of pipe, depth of cover, valves, fittings, alternate supply routes.

- Sanitary Sewers -- size, construction material, invert elevations, area served, and type of users (last two items necessary to evaluate probable flow and problems in handling flows if pipe affected by construction.)

- Storm Sewers -- size, construction, material, invert elevations, area served and type of development.

- Other Utilities -- steam, gas, electric, telephone, traffic signal, etc.

Field Data and Surveys. Test holes should be located along the alignment as necessary to establish soil types. If the sewer is to receive groundwater, a sufficient number of holes must be involved to establish the groundwater table for underdrain design purposes.

Field surveys may be necessary to supplement design maps with reference to utilities, test hole locations, and other items which are not accurately located on the maps.

Regulations. All city, county, state, or other regulations or standards which apply to the proposed sewer must be reviewed. Examples would be street cut permits or city standards for sewer line locations.

Design Maps

Mapping for use in final design shall be of sufficient accuracy to enable sewer lengths to be set within 0.1 foot and elevations within 0.01 foot. The scale will be 1 inch equals 50 feet unless otherwise specified by the City.

Elevation datum shall be U.S. Geological Survey.

Layout

The layout of the system should conform to the following requirements. Any deviation should be discussed with the governing body which will review the final plans prior to final design.

Location Requirements

- Main Location -- The location of storm sewers shall be cleared with, and approved by the City of Stillwater.

- Alignment -- Storm sewers shall be straight between manholes insofar as possible. Where long radius curves are necessary to conform to street layout, the radius of curvature divided by the pipe diameter shall be at least 6.0. Radius of curvature specified should coincide with standard curves available in the type material utilized wherever possible. Specially fabricated bends will be permissible as long as their effect is included in the final hydraulic design.

- Crossings -- Crossings with other underground utilities except at intersections shall be avoided. Crossings, if necessary, should be at an angle greater than 45 degrees.

  The storm sewer main and/or the utility must be structurally reinforced if insufficient vertical clearance is available. Standard allowable clearance without reinforcing between storm and sanitary sewers is 24 inches.

Manholes

- Spacing -- Spacing of manholes shall conform to the following table.

TABLE IV-4

MANHOLE SPACING

Pipe Size	Maximum Spacing
15" or less	600 feet
18" to 36"	600 feet
42" or greater	800 feet

- Direction Changes -- Short radius bends may be used on 24" and larger pipes when flow must undergo a direction change at a junction or bend. Reductions in headloss at manholes may be realized in this way. A manhole shall always be located at the end of such short radius bends.

- Manhole Geometry -- Except as may be needed to induce head loss, the manhole bases shall be shaped as indicated in Figure IV-9, with the deflector height being equal to the crown of the outlet pipe. Deflections greater than 18-inches in height shall have toe pockets.

Grade. Except for slotted drains (see Chapter III, "Inlets"), storm sewer grades should be such that a minimum of 3'-0" cover over the crown of the pipe is maintained. Uniform slopes shall be maintained between manholes unless specifically approved otherwise.

Final grades shall be set with full consideration to capacity required, sedimentation problems, and other design parameters, but the minimum slopes shall be that capable of producing the cleansing velocity as determined from Figure IV-4. The grade will depend upon the geometry and roughness of the conduit.

Materials of Construction

Storm sewers may be constructed of any suitable material acceptable to the governing body, as long as it is capable of matching requirements set forth in this Manual. Soils tests shall be conducted when there is a possibility that conditions exist which would cause premature failure of certain materials. Structural calculations must be carried out on any material to verify that it is acceptable.

When alternate types of materials are acceptable for bidding purposes, hydraulic designs must be completed for each material to verify that both materials will be acceptable. The minimum line diameter will be 12 inches.

Hydraulic Design

Final hydraulic design shall be according to the methods set forth in this Chapter.

Inlets. Inlets shall be designed according to the Storm Water Inlets Chapter of this Manual.

Connector Pipes. Connector pipes shall be hydraulically designed. Connector pipes shall enter the main at manholes or in specially fabricated ties. The minimum size for connector pipes, or any other sewer, shall be 12 inches.

Construction Drawings. Standards for construction drawings shall meet the standards of the City of Stillwater.

Specifications

Complete specifications shall be furnished with all projects. Specifications shall be in sufficient detail to guarantee first class material and installation and shall meet the requirements of the City.

Easements

Unless parallelled by an existing utility easement, the minimum width of easement for installation of a storm sewer should be the pipe diameter plus 18 feet. With a parallel existing utility easement, the minimum width of easement shall be the pipe diameter plus 9 feet.

DESIGN EXAMPLE

A hypothetical pipe layout is analyzed to demonstrate the method of application of the design charts and to provide an overview of the final hydraulic design procedure. Figures IV-12 and IV-13 which show the system is used in the design example.

Each inlet is numbered, e.g. 1-4, and the design rate of flow into each is shown. The accumulated design rate of flow in each pipeline between inlets is given, together with the pipe diameter in inches and length in feet from center to center of inlets. The pipe slope is not stated, but appears on the profiles at the end of the design discussion. Manholes are designated M.H.-1. The pipe arrangement at each manhole and inlet is evident from the plan, and serves to identify the design chart which is to be used for the determination of the corresponding pressure change

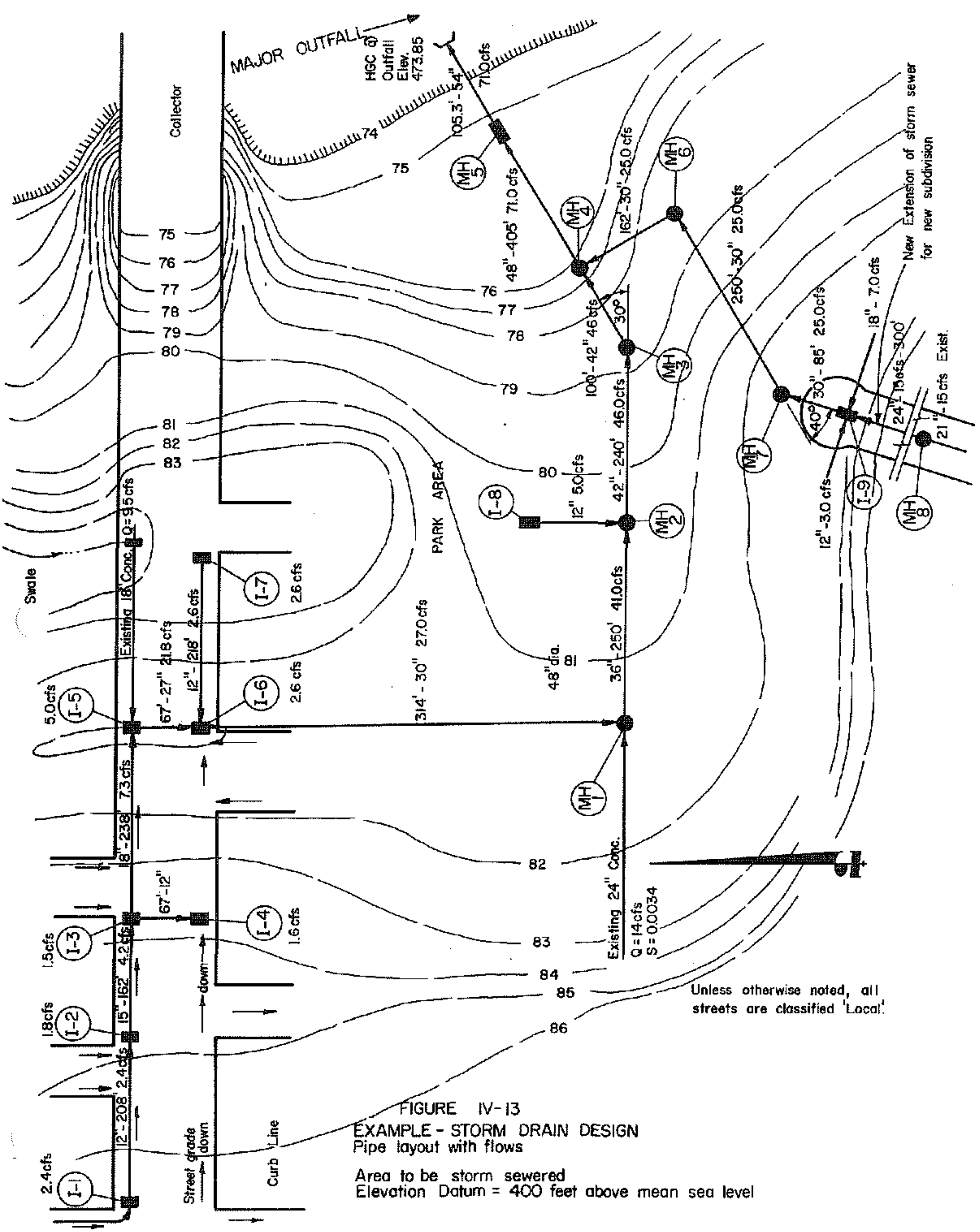
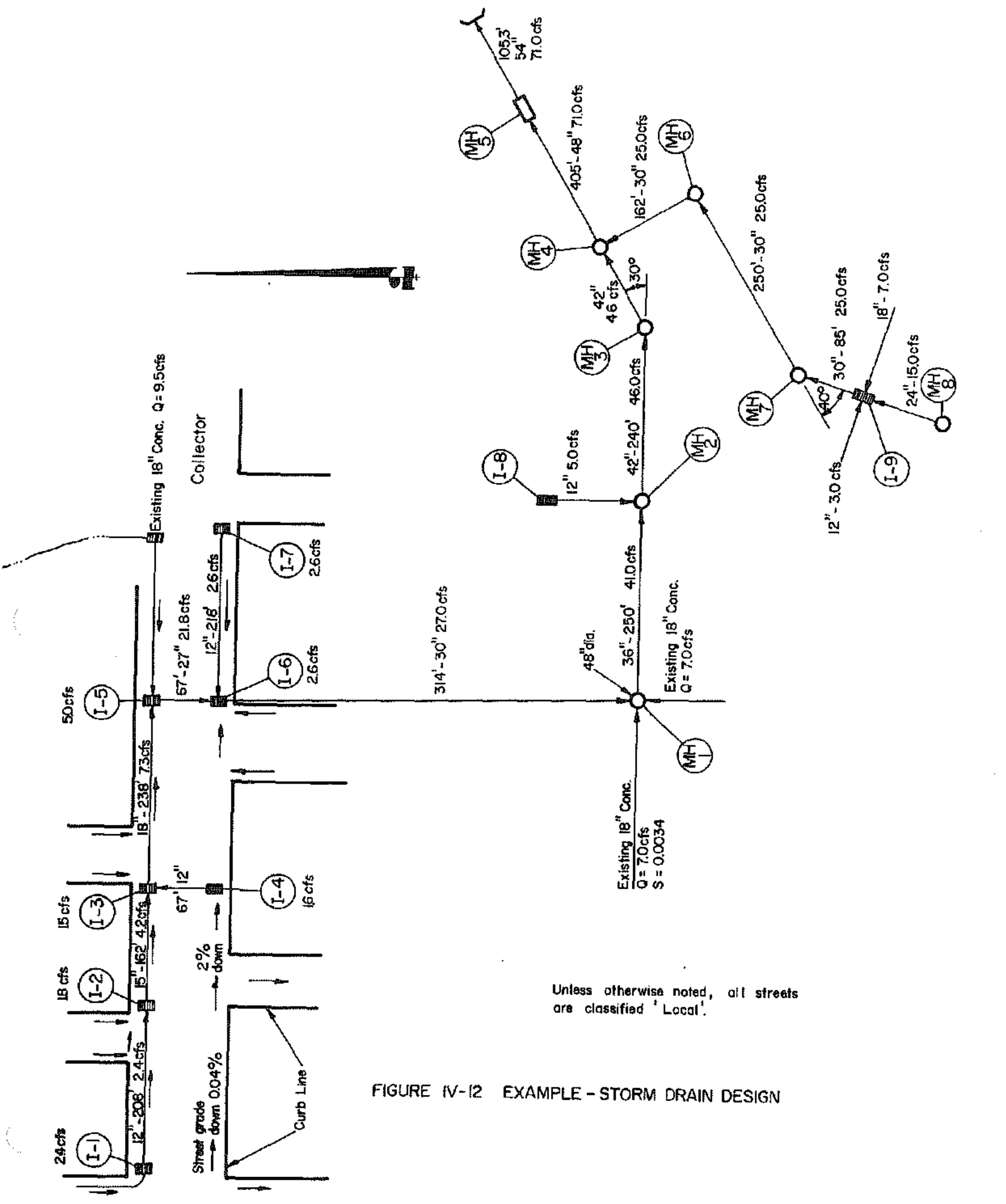


FIGURE IV-13
EXAMPLE - STORM DRAIN DESIGN
Pipe layout with flows

Area to be storm sewered
Elevation Datum = 400 feet above mean sea level



Unless otherwise noted, all streets are classified 'Local'.

FIGURE IV-12 EXAMPLE - STORM DRAIN DESIGN coefficients. These, in turn, are to be used to calculate the pressure change in feet for each upstream pipe.

The system has been laid out during the preliminary design phase, with all inlets located, the rate of inflow to each determined* and the preliminary pipeline sizes selected, and a preliminary profile established. Proceeding from the outfall, the design moves to the next junction upstream by adding the friction loss in the pipeline to the hydraulic grade line at the outfall. The value obtained is the downstream hydraulic grade line for the junction, which needs to be checked to verify pressure conduit or open channel.

If it is less than 80 percent of depth in the downstream pipe and if the normal pipeline depth is less than 80 percent of the pipe vertical height, then the downstream water surface is set at normal depth.

The design example in this Chapter illustrates how junction losses are computed for both pressure conduits and open channel flow and was developed to illustrate at least one condition for each of the design charts. The design then proceeds upstream from junction to junction.

A word of caution is needed to prevent the loss of significant design time. The designer should examine the conditions at each junction to try to determine whether the main line, a lateral(s) or a nearby inlet (usually with a high rate of inflow) is most likely to be more critical in regard to whether or not the preceding pipeline design may need to revised. The designer should keep in mind that the final hydraulic design procedure is iterative, and adjustments will probably be necessary

*Note: Due to the differing times of concentration, the rate of inlet flow for sizing of the storm sewer pipeline may be different than the flow for sizing the inlets.

to raise or lower the hydraulic grade lines for the design runoff event.

The design is carried from junction to junction with an explanation of the use of the applicable graph. Pipeline computations on each junction computation sheet are for the preceding or downstream pipeline. It is not recommended that junction computation sheets as elaborate as those included in this discussion be used. A simple hand sketch is usually sufficient.

In the design example, the accuracy of the computations is shown to 0.01 feet; however, in actual design, the needed accuracy is usually sufficient to 0.10 feet for hydraulic grade line computations. The pipeline inverts are to be designed to 0.01 feet.

For the design example the pipeline is assumed to be reinforced concrete pipe without rubber gasket joints; therefore, the roughness factor "n" was assumed to be 0.013. In the design example, it was assumed that the inverts of the manholes and inverts were known, in many instances, due to utility conflicts or due to the desire to control the hydraulic grade, line, the depth of hydraulic structures may be varied during final hydraulic align. A profile for each inlet connector pipe must be prepared where conflicting utilities may exist to allow for optimum hydraulic design.

The mainstem of the design example is shown in the profile, Figure IV-29 at the end of the design example. The profile includes the pipeline crown and invert, manholes and inlets, energy grade line and hydraulic grade line.

EXAMPLE CALCULATIONS

Manhole No. M.H.-5 to Outlet

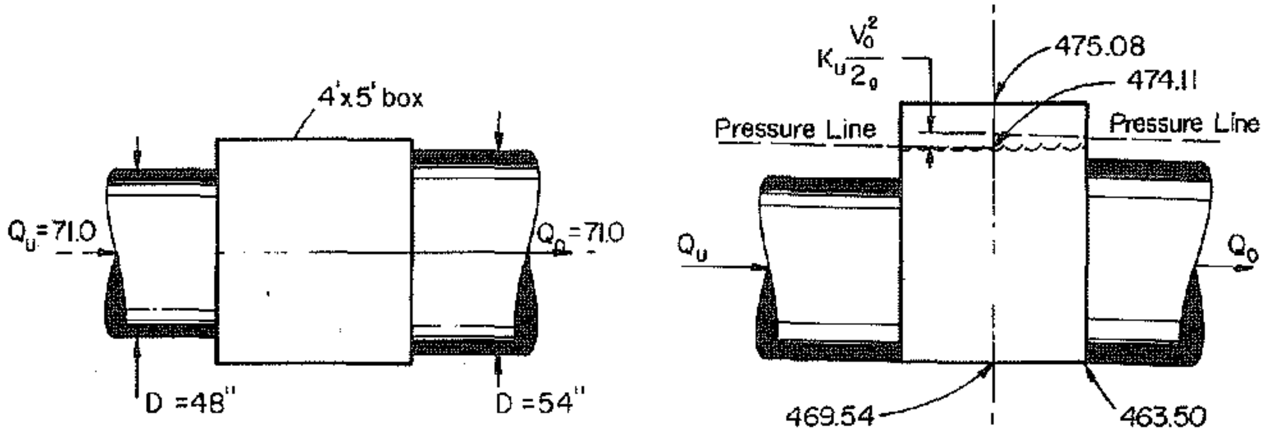
Preliminary surveys have shown that the tailwater elevation at the outlet is 473.82 feet. The top of the 54-inch pipe is at elevation of 473.89 feet, and the outlet is essentially submerged. The outfall exit loss of one velocity head should be added to the flowline elevation to establish the starting elevation of 474.13 feet. The friction loss in the pipe from the outlet to Manhole No. M.H. -5 is added to this elevation to establish the downstream pressure line at Manhole No. M.H.-5.

Manhole No. M.H.-5

This manhole illustrates a junction to which Chart IV-B applies. The use of this chart is restricted to cases where the pipe centerlines are parallel and not offset more than would permit the area of the smaller pipe to fall entirely within that of the larger if projected across the junction box along the pipe axis. If grate flow enters the junction, the designer should use Chart IV-C.

Known quantities are the gutter elevation, the inlet bottom elevation, the pipe flow rates and diameters, the inlet size, and the elevation of the downstream pressure line at the junction center. From these data, the velocity head of the outfall flow, the ratios D_U/D_O and A/D_U can be calculated. Next K_U is read from the chart and multiplied by the velocity head in the outfall to obtain h_U, the change of pressure. The h_U is subtracted (or added) to the outfall pressure line elevation to obtain the elevation of the upstream in-line pipe pressure line to which the water surface in the inlet corresponds. The clearance of the water below the gutter is checked.

EXAMPLE STORM DRAIN DESIGN MANHOLE NO. 5



PLAN

ELEVATION

FIGURE IV-14 USE CHART IV-B

Item	M.H.-5	
Gutter Elevation	475.08	
Inlet Bottom Elevation	469.50	
Flow Rate Q_U	71.0	
D_U/D_O	0.89	
A/D_U	1.0	
Outfall Velocity Head $\frac{V^2}{2g}$	0.31	
Downstream Pressure Elevation	474.27	
Chart IV-G: sq.-edged entrance to outfall, K_U	-0.50	
Pressure Rise, $K_U \times \frac{V^2}{2g}$	-0.16	
Upstream Pressure Elevation	474.11	
Water Surface Elevation	474.11	
Distance Below Grate, ft.	1.29	
Distance Above Invert, ft.	4.61	Pressure Conduit

USHGL @ Outlet = 474.13

Pipeline Data:

Downstream:

Q = 71 cfs

Length = 105.3 ft.

ϕ = 54 inches

s = 0.0010 ft./ft.

V = 4.46 fps.

$\frac{V^2}{2g}$ = 0.31 ft.

s_f = 0.0013 ft./ft.

h_f = 0.14 ft.

Upstream:

Q = 71.0 cfs

ϕ = 48"

Manhole No. M.H.-4

This manhole is typical of round manholes to which Charts IV-G and IV-H IVapply. Calculations for the determination of the pressure changes at this manhole are presented in Figure IV-15.

Known data are the elevations of the top and bottom of the manhole, the manhole diameter, the rates of flow in each pipe, the pipe diameters, and the elevation of the downstream (outfall pipe) pressure line at the branch point.

From these data the ratios D_L/D_O, D_U/D_O, Q_U/Q_O, and B/D_O, the chart factor

$$\frac{Q_U}{Q_O} \times \frac{D_O}{D_U}$$

and the outfall velocity head may be calculated. The values of D_L/D_O and

$$\frac{Q_U}{Q_O} \times \frac{D_O}{D_U}$$

indicate that Charts IV-G and IV-H are applicable in this case.

Chart IV-G (for square manholes) is used to obtain the pressure change coefficient K_L for the lateral pipe even though Manhole No. M.H.-1 is a round manhole. First $\bar{K}_L$ for a square manhole is read from the lower graph of the chart which may be reduced by 0.2 for the round manhole in accordance with the table of Step (2) of the instructions for use of Chart IV-G for round manholes at the junction of a 90° lateral with a through pipeline. The outfall pipe entrance is sharpedged in this case, so no further reduction is made. The upper graph of the chart is used to obtain the multiplying factor M_L, then K_L is obtained by multiplying $\bar{K}_L$ by M_L. Next K_L is multiplied by the outfall velocity head to obtain h_L, the change in pressure (or pressure rise) at the manhole. Finally, h_L is added to the outfall pressure line elevation to obtain the elevation of the lateral pipe pressure line at the branch point.

Chart IV-H, for square or round manholes, is used to obtain the pressure change coefficient K_U for both the upstream in-line pipe and the water depth in the manhole. First $\bar{K}_U$ for all flow from the lateral is read from the lower graph of the chart and is used without modification since the outfall entrance is square-edged in this case. Note that no reduction is to be made for the round manhole cross section.

Next M_U is read from the upper graph of the chart and K_U is obtained by multiplying $\bar{K}_U$ by M_U. Then K_U is multiplied by the velocity head in the outfall to obtain h_U, the change of pressure. Next, h_U is added to the outfall pressure line elevation to obtain the elevation of the upstream in-line pipe pressure line at the branch point. The water surface elevation in the manhole is the same as the pressure line for the in-line pipe. Finally, the clearance of the water surface below the top of the manhole is checked and found to be ample.

Note that for a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U=1$. For an enlargement of pipe size, as in this case, the pressure change across the junction is negative, even though there is a loss in total energy.

<u>Manhole No. M.H.-3</u>

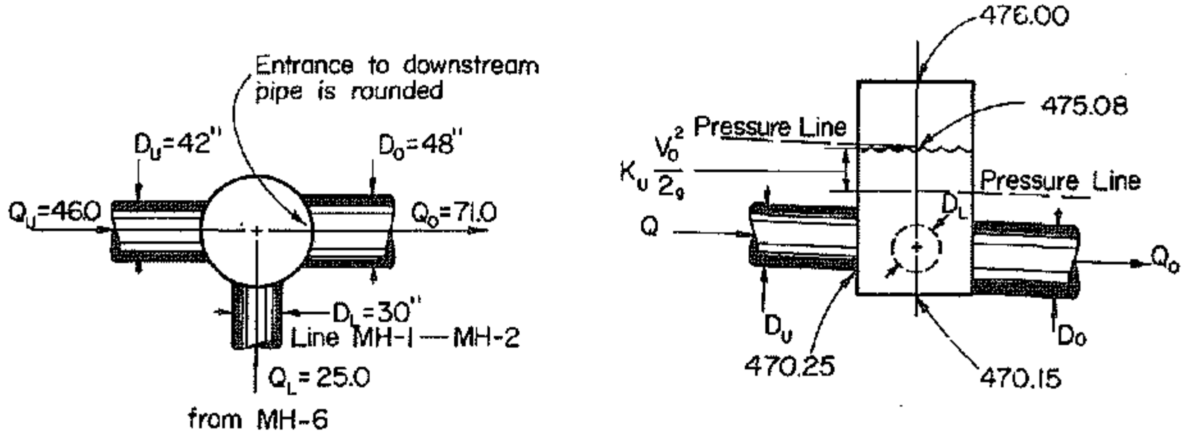
This manhole is typical of junctions to which Chart IV-J applies.

Known quantities are the gutter elevations, the manhole bottom elevation, the flow rate, the pipe diameter, the deflection angle and characteristics, and the elevation of the downstream pressure line. From these data, the velocity head of the outfall flow may be determined. The loss coefficient K is read from Chart IV-J and is multiplied by the outfall velocity head to obtain the rise of the water surface above the downstream pressure line elevation. This corresponds to the upstream pressure line elevation. The clearance of the water surface below the gutter should be checked.

<u>Manhole No. M.H.-2.</u> This manhole illustrates a junction to which Chart IV-I applies.

Known quantities are the gutter elevation, the inlet bottom elevation, the pipe inflow rates, the outfall flow rate, the pipe diameters, and the

EXAMPLE STORM DRAIN DESIGN MANHOLE NO. 4



PLAN ELEVATION

FIGURE IV-15 USE CHART IV-G & H

Item	M.H. -4
Top of M.H. Elevation	476.00
Bottom of M.H. Elevation	470.15
Lateral Flow Q_L cfs	25.0
Upstream In-Line Flow Q_U cfs	46.0
Outfall Flow $Q_O$cfs	71.0
Lateral Pipe Ratio D_L/D_O	0.63
In-Line Pipe Ratio D_U/D_O	0.88
Chart Factor $Q_U/Q_O \times D_O/D_U$	0.74
Manhole Diameter B in.	48.00
M.H. Size Ratio B/D_O	1.00
Outfall Velocity Head $V_O^2/2g$ft.	0.50
Downstream Pressure Elevation	475.08
Lateral Pressure Rise Coefficient (sq. edge entr.)	
Chart IV-G $\bar{K}_L$ for sq. edged M.H.	0.93
K_L for rd. edged M.H. (less 0.2)*	0.75
Chart IV-G M_L	0.61
$K_L = \bar{K}_L \times M_L$	0.57

*The use of rounded entrance from manhole to outlet pipe is usually not economically justified when $V_O^2/2g < 1.0$.

elevation of the downstream (outfall pipe) pressure line at the inlet center. From these data, the velocity head of the outfall flow, the ratios D_L/D_O, D_U/D_O, and Q_U/Q_O can be calculated. Charts IV-G and IV-H should be used if the size and flow factors are within their range. Chart IV-I should not be used for $D_L/D_O < 0.6$ if other solutions are possible. Enter Chart IV-I at the ratio D_U/D_O and read K_U (also equal K_L) at the curve or interpolated curve for Q_U/Q_O. If the factor ($Q_U/Q_O \times D_O/D_U$) was found to be greater than 1.00 when checking the applicability of Charts IV-G and IV-H K_U will be negative in sign. This provides a check on proper use of the charts. Neither manhole shape nor size nor relative size of the lateral pipe will modify the coefficients of Chart IV-I. Next, K_U is multiplied by the outfall velocity head to obtain h_U, the change of outfall velocity head to obtain h_U, the change of pressure at the manhole. Finally, h_U or (h_L) is subtracted or added to the outfall pressure line elevation to obtain the elevation of the upstream in-line pipe pressure at the manhole center. The lateral pipe pressure line and water surface elevation will correspond to the upstream in-line pipe pressure line elevation.

Item	M.H.-4	
Lateral Pressure Rise, $K_L \times V_O^2/2g$	0.28	
Lateral Upstream Pressure Elevation	475.36	
Upstream Pipe Pressure Rise Coefficients		
Chart IV-H $\bar{K}_U$ for sq. or rd. M.H.	1.86	
M_U	0.45	
$K_U = \bar{K}_U \times M_U$	0.84	
In-Line Upstream Pressure Elevation	475.50	
Water Surface Elevation	475.50	
Clearance, Water Below Top ft.	0.50	
Distance Above Invert, ft.	5.35	Pressure Conduit

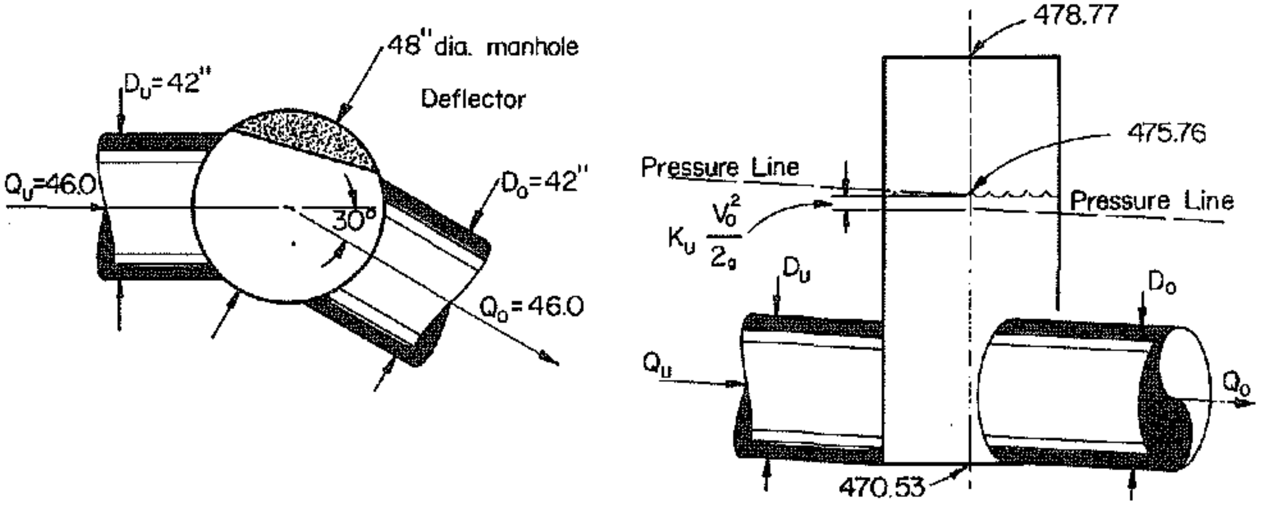
USHGL @ M.H.-5 = 474.11

Pipeline Data:

Downstream:	Upstream:	Lateral:
Q = 71.0 cfs	Q = 46.0 cfs	Q = 2 cfs
Length = 405 ft.		
ø = 48"		
s = 0.0015 ft./ft.		
V = 5.65 fps		
$\frac{V^2}{2g} = 0.50$		
$s_f = 0.0024$ ft./ft.		
$h_f = 0.97$		

* This value will be used later in the computations to start the lateral pipe computations from M.H.-4 to M.H. -6

EXAMPLE STORM DRAIN DESIGN MANHOLE NO 3



PLAN ELEVATION

FIGURE IV-16 USE CHART IV-J

Item	M.H.-3
Gutter Elevation	478.77
Manhole Bottom Elevation	470.53
Upstream = Downstream Flow cfs	46.0
Upstream Pipe Diameter in.	42
Downstream Pipe Diameter in.	42
Outfall Velocity Head $V_0^2/2g$ ft.	0.36
D_U/D_O	1.0
(This implies that there is no contraction or expansion headloss)	
Deflection Angle	30°
Downstream Pressure Elevation	475.71
Chart IV-K, K (with Deflector)	0.15
Upstream Pressure Rise = K x $V_0^2/2g$	0.05
Upstream Pressure Elevation, W.S.E.	475.76
Clearance, Water Below Top ft.	3.01
Distance Above Invert, ft.	5.23 Pressure Conduit

USHGL @ MH-4 = 475.50

Pipeline Data:

Downstream:
Q = 46.0 cfs
Length = 100.0 ft.
∅ = 42"
s = 0.0028 ft./ft.
V = 4.78 fps
$V^2/2g$ = 0.35 ft.
s_f = 0.0021 ft./ft.
h_f = 0.21

Upstream:
Q = 46.0 cfs
∅ = 42"

EXAMPLE STORM DRAIN DESIGN MANHOLE NO. 2

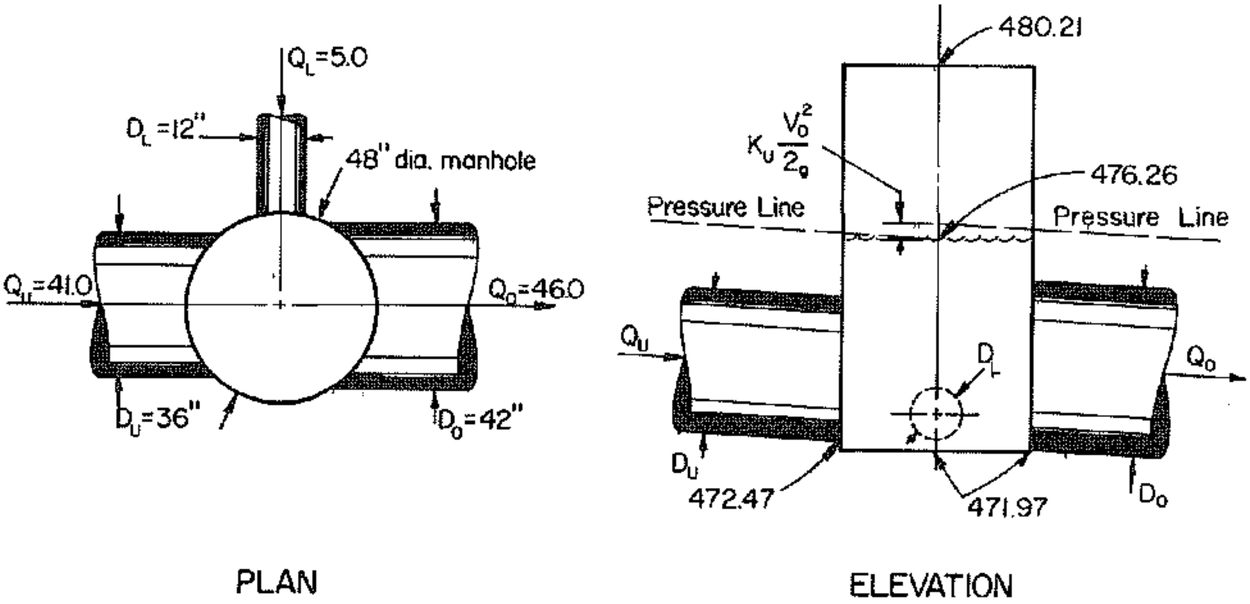


FIGURE IV-17 USE CHART IV-I

Item	M.H.-2
Top of Manhole Elevation	480.21
Bottom of Manhole Elevation	471.97
Lateral Flow Q_L cfs	5.0
Upstream In-line Flow Q_U cfs	41.0
Outfall Flow Q_O cfs	46.0
Flow Ratio Q_U/Q_O	0.89 > 0.7
Lateral Pipe Ratio D_L/D_O	0.29 < 0.6
In-Line Pipe Ratio D_U/D_O	0.86
Outfall Velocity Head $V_O^2/2g$ ft.	0.35
Factor $Q_U/Q_O \times D_O/D_U$	1.04
Downstream Pressure Elevation	476.26
Assume Square-edged Entrance, Chart IV-I	
Chart IV-K; K_U and K_L	-0.18
Upstream Pressure Rise = 0.18 x 0.35 ft.	-0.06
Upstream Pressure Elevation and WSE	476.20
Clearance, Water Below Top, ft.	3.85
Distance Above Invert ft.	4.23

USHGL @ MH-3 = 475.76 ft.

Pipeline Data:

Downstream:	Upstream	Lateral:
Q = 46.0 cfs	Q = 41.0 cfs	Q = 5.0 cfs
Length = 240'	Q= 36"	Q = 12"
Ø = 42"		
s = 0.0060 ft./ft.		
V = 4.78 fps		
$V^2/2g$ = 0.35 ft.		
s_f = 0.0021 ft./ft.		
h_f = 0.50 ft.		

A check should be made to ensure that the water surface is above the pipe crowns to justify using these charts and that it is sufficiently below the top of the manhole to indicate safety from overflow.

Deflectors in the manhole are not effective in the ranges covered by Chart IV-I, and, therefore, need not be used.

Inlet I-8 is similar to Inlets I-1 and I-7 and is not analyzed in this example.

Manhole No. M.H.-1

This manhole has four laterals and has been included to illustrate the use of the design charts for a condition not specifically covered by the charts. The following parameters are needed to determine which charts to use (see Table IV-1):

$$\frac{Q_U}{Q_O} = 0.17 < 0.3 \ , \qquad \frac{D_U}{D_O} = 0.50$$

Charts IV-E or IV-F are to be used and the upstream in-line pipe is analyzed as grate flow. For this analysis, it is assumed that the laterals are in-line.

For this manhole, assuming in-line laterals, the known data are the gutter elevation, the elevation of the inlet bottom, the lateral pipe and the grate inflow rates, their total--the outfall flow rate, the pipe diameters, and the elevation of the downstream (outfall pipe) pressure line. From the lateral pipe flow rates and sizes the velocity in each of the laterals is determined, and the two laterals are identified as higher velocity and lower velocity. In this case, the existing line has the higher-velocity. From the known data and the above determination, the ratios Q_G/Q_O, Q_{HV}/Q_O, Q_{LV}/Q_O, D_{HV}/D_O, D_{LV}/D_O, and D_{HV}/D_{LV} are calculated. Next the velocity head of the outfall flow is calculated. Then the elevation of the downstream pressure line is tabulated for convenience in adding the pressure rise, which is now calculated by use of Chart IV-E. The pressure factors H and L are read from the charts, and identified by the lateral to which the D and Q of the two graphs apply. The difference between H and L (3.7 - 0.5 = 3.2) is the pressure change coefficient $K_R = K_{LV}$ for existing lower velocity lateral, which is also to be applied to the upstream existing pipe in-line with the outlet pipe. The constant coefficient $K_L = K_{hv}$ is 1.8 because grate flow is involved. Each coefficient is multiplied by the velocity head of the outfall flow to obtain the pressure changes h_{LV} and h_{HV} for the laterals. The pressure change is always positive, that is, producing a rise in pressure upstream, for junctions of this type. Thus h_{LV}, the pressure rise, is added to the elevation of the outfall pipe (downstream) pressure line to obtain the elevation of the pressure line in the lower velocity lateral at the branch point. Similarly, h_{HV} is used to obtain the elevation of the pressure line of the higher-velocity flow in the existing line. The water surface elevation in the inlet corresponds to the latter pressure line. Finally, the clearance of the water surface below the gutter is checked.

Inlet No. 6. This inlet illustrates inlets to which Chart IV-D applies. Inlet No. 6 involves the basic through pipeline and lateral pipe arrangement, and also has flow through a top grate. It might appear that Inlet No. 6 does not meet the requirements for Chart IV-D, in that the in-line flow is through the length of the box rather than across its short dimension; however, this deviation from the more usual arrangement (see Inlet No. 3) is not sufficient to effect a significant change in the hydraulic performance.

EXAMPLE STORM DRAIN DESIGN MANHOLE NO. 1

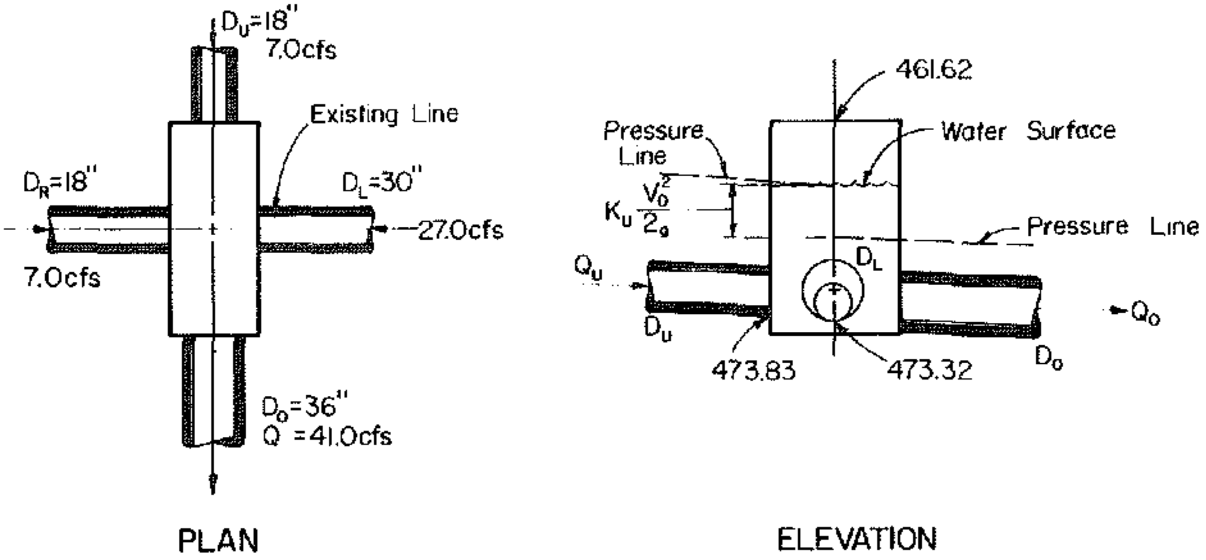


FIGURE IV-18 USE CHART IV-E

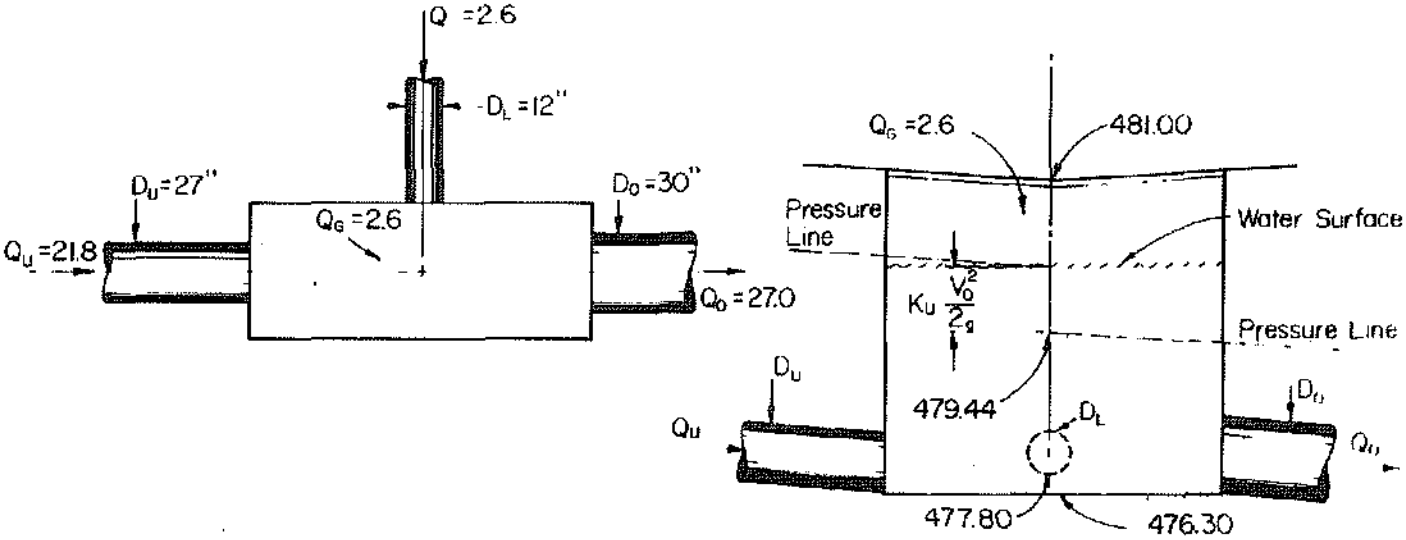
Item	M.H.-1
Gutter Elevation	481.62
Inlet Bottom Elevation	473.32
Flow Ratios Q_C/Q_O	0.17
Q_{HV}/Q_O	0.66
Q_{LV}/Q_O	0.17
Pipe Size Ratios D_{LV}/D_O	0.50
D_{HV}/D_O	0.83
D_{HV}/D_{LV}	
Velocity Head $V_0^2/2g$, ft.	0.52
Downstream Pressure Elevation	477.15
Chart IV-E: Factor H for exist. lat.	3.7
Factor L for exist. lat.	0.5
K_L = for new lat.	1.8
K_R = H-L, for lat.	3.2
Pressure Rise exist. Lat. 3.2 x 0.52	1.66
New 30" Lat. 1.8 x 0.52	0.94
Upstream Pressure Elevation	
Exist Lateral & in-line	478.81
New 30"	478.09
Water Surface Elevation in Inlet	478.09
Clearance, Gutter to Water, ft.	3.53
Distance Above Invert (ck 30")	4.77 Pressure Conduit

USHGL @ M.H.-2 = 476.2

Pipeline Data:

Downstream:	Left Lateral:	Right Lateral:
Q = 41.0 cfs	Q = 27.0 cfs	Q = 7.0 cfs
Length = 250 ft.	Ø = 30"	Ø = 18"
Ø = 36"	V = 5.5 fps	V = 3.96 fps
s = 0.0060 ft./ft.	High Velocity	Low Velocity
V = 5.80 fps		
$\frac{V^2}{2g}$ = 0.52 ft.		
s_f = 0.0038 ft./ft.		
h_f = 0.95 ft.		

EXAMPLE STORM DRAIN DESIGN INLET NO. 6



PLAN ELEVATION

FIGURE IV-19
USE CHART IV-D

Item	Inlet 6
Gutter Elevation	481.00
Inlet Bottom Elevation	476.30
Grate Inflow, Q_G cfs	2.6
Upstream In-Line Flow, Q_U	21.8
Left Lateral Flow, Q_L	2.6
Outfall Flow, Q_O cfs	27.0
Outfall Pipe Diameter D_O in	30.0
Outfall Velocity Head $V_O^2/2g$ ft.	0.47
Flow Ratios Q_U/Q_O	0.81
Q_G/Q_O	0.10
Pipe Size Ratio D_U/D_O	0.90
Downstream Pressure Elevation	479.44
Pressure Elevation Above Bottom ft.	3.14
Estimated d/D_O	1.4
Pressure Rise Coefficient for U.S. main and Lateral	
Chart IV-D: $K_U = 0.45 + 0.10$	0.55
Pressure Rise 0.55 x $V_O^2/2g$ ft.	0.26
Upstream Pressure Line Elevation for Main and Lateral and Water Surface Elevation	479.70
Check d/D_O	1.36
Clearance, Gutter to Water ft.	1.30

USHGL @ M.H.-1 = 478.09

Pipeline Data:

Downstream:

$Q = 27.0$ cfs

Length = 314 ft.

$\phi = 30''$

$s = 0.0079$ ft./ft.

$V = 5.5$ fps

$\frac{V^2}{2g} = 0.47$ ft.

$s_f = 0.0043$ ft./ft.

$h_f = 1.35$

Upstream:

$Q = 21.8$ cfs

$\phi = 27''$

$V = 5.48$ fps

Lateral:

$Q = 2.6$ cfs

$\phi = 12''$

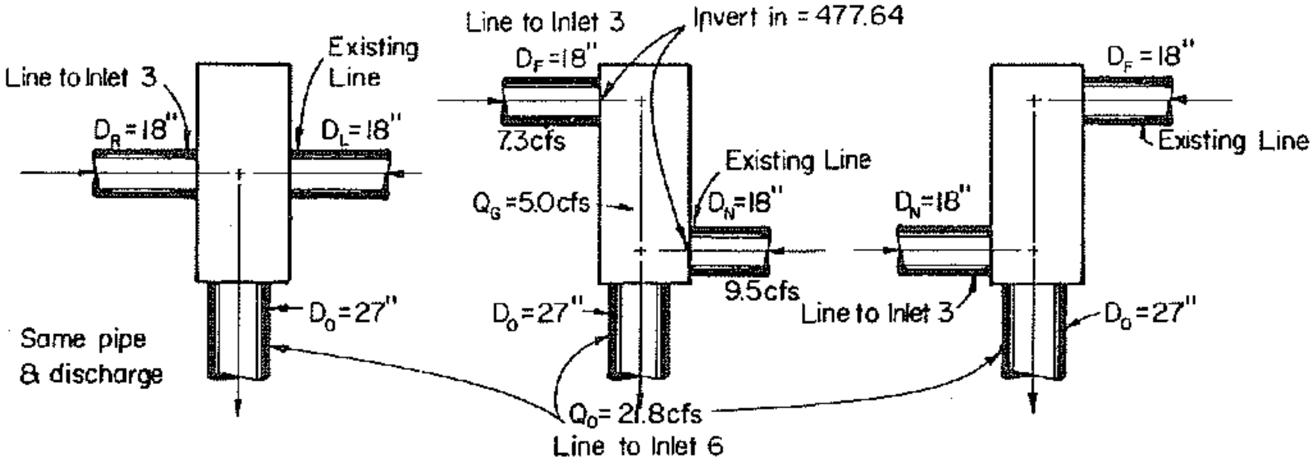
$V = 3.31$ fps

Inlet No. 5

This inlet is a typical inlet to which the design methods of Chart IV-E apply. The calculations of pressure changes at this inlet with in-line opposed laterals are presented in the left-hand column of the tabulation in Figure IV-20.

For this inlet with in-line laterals, the known data are the gutter elevation, the elevation of the inlet bottom, the lateral pipe and the grate inflow rates, their total--the outfall flow rate, the pipe diameters, and the elevation of the downstream (outfall pipe) pressure line. From the lateral pipe flow rates and sizes the velocity in each of the laterals is determined, and the two laterals are identified as higher-velocity and lower-velocity. In this case, the existing line has the higher-velocity. From the known data and the above determination, the ratios Q_G/Q_O, Q_{HV}/Q_O, Q_{LV}/Q_O, D_{HV}/D_O, D_{LV}/D_O, and D_{HV}/D_{lv} (1.0 in this case) are calculated. Next the velocity head of the outfall flow is calculated. Then the elevation of the downstream pressure line is tabulated for convenience in adding the pressure rise, which is now calculated by use of Chart IV-E. The pressure factors H and L are read from the chart, and identified by the lateral to which the D and Q of the two graphs apply. The difference between H and L (3.7 - 0.5 = 3.2) is the pressure change coefficient $K_R = K_{LV}$ for the new lateral to inlet 3, the lower-velocity lateral. The constant coefficient $K_L = K_{HV}$ is 1.8 because grate flow is involved. Each coefficient is multiplied by the velocity head of the outfall flow to obtain the pressure changes h_{LV} and h_{HV} for the laterals. The pressure change is always positive, that is, producing a rise in pressure upstream, for inlets of this type. Thus h_{LV}, the pressure rise, is added to the elevation of the outfall pipe (downstream) pressure line to obtain the elevation of the pressure line in the lower-velocity lateral at the branch point. Similarly, h_{HV} is used to obtain the elevation of the pressure line of the higher-velocity flow in the existing line. The water surface elevation in the inlet corresponds to the latter pressure line. Finally, the clearance of the water surface below the gutter is checked.

EXAMPLE STORM DRAIN DESIGN INLET NO. 5



IN-LINE LATERALS — OFFSET LATERALS NEW LINE=FAR LATERAL — OFFSET LATERALS EXISTING LINE=FAR LATERAL

FIGURE IV-20 USE CHARTS IV-E & F

Item	In-Line Laterals	New Line Far Lat.	Exist. Line Far Lat.
Gutter Elevation	481.50	481.50	481.50
Inlet Bottom Elevation	476.50	476.50	476.50
Flow Ratios Q_G/Q_O	0.23	0.23	0.23
Q_F/Q_O		0.33	0.44
Q_N/Q_O		0.44	0.33
Q_{HV}/Q_O	0.44		
Q_{LV}/Q_O	0.33		
Pipe Size Ratios $D_{LV}/D_O = D_{HV}/D_O$	0.67		
$D_F/D_O = D_N/D_O$		0.67	0.67
Factor $Q_F/Q_O \times D_O/D_F$		0.50	0.66
$Q_N/Q_O \times D_O/D_N$		0.66	0.50
Velocity Head $V_O^2/2g$ ft.		0.47	0.47
Downstream Pressure Elevation	480.04	480.04	480.04
Chart IV-E: Factor H for exist. lat.	3.0		
Factor L for new lat.	0.6		
K_R = H-L, new lat.	2.4		
K_L exist. lat.	1.8		
Pressure Rise new lat. to Inlet 3			
2.4 x 0.47	1.13		
Exist lat. 1.8 x 0.47	0.85		

		Offset	
Item	In-Line Laterals	3-5 New Lat.	Exist. Line Far Lat.
Chart IV-F: K for 3-5		$K_F = 2.0$	$K_N = 1.4$
K for exist.		$K_N = 1.4$	$K_F = 1.9$
Pressure Rise, new lat. 2.0 x 0.47		0.94	
Pressure Rise, exist., 1.4 x 0.47		0.66	
Pressure Rise, new lat. 1.4 x 0.47			0.66
Pressure Rise, exist., 1.9 x 0.47			0.89
Upstream Pressure Elevation			
New Line to Inlet 3	481.17	480.98	480.70
Existing lateral	480.89	480.70	480.93
Water Surface Elevation in Inlet	480.89	480.98	480.93
Clearance, gutter to water, ft.	0.61	0.52	0.56
Depth of Water in Inlet, ft.	4.39	4.48	4.43 Pressure Conduit

USHGL @ Inlet I-6 = 479.70

Pipeline Data:

Downstream:	New Line To Inlet 3:	Existing Line:
Q = 21.8 cfs	Q = 7.3 cfs	Q = 9.5 cfs
Length = 67.0 ft.	ø = 18 inches	ø = 18 inches
ø = 27 inches	V = 4.13 fps	V = 5.38 fps
s = 0.0030 ft./ft.		
V = 5.48 fps		
$\frac{V^2}{2g} = 0.47$ ft.		
$s_f = 0.0050$ ft./ft.		
$h_f = 0.34$ ft.		

An alternate arrangement of the lateral pipes at Inlet No. 5 can be effected to produce an inlet with offset opposed laterals of the type to which Chart IV-F will apply. Two different arrangements are possible, each placing one of the laterals in the far position. The pipe arrangement is shown in Figure IV-20, and the calculations of pressure changes are shown in the center and right-hand columns of the tabulation in the Figure.

With either placement of the laterals in the offset arrangement, the known data are the gutter and inlet bottom elevations, the flow rates, pipe diameters, and elevation of the downstream pressure line, all of which are the same as for the in-line lateral arrangement. From these data and using the designation of the laterals as far and near in position, the ratios Q_G/Q_O,Q_F/Q_O, Q_N/Q_O, D_F/D_O, and D_N/D_O are calculated. Then the factors composed of the flow ratio times the reciprocal of the pipe size ratio are calculated. Next the velocity head of the outfall flow is calculated and the downstream pressure elevation is entered in the tabulations.

Chart IV-F is used to determine the pressure change coefficients, working with each lateral arrangement separately to avoid confusion. Considering the lateral to Inlet 3 as the far lateral, as shown in the center column of the tabulations, K_F for the new lateral is found to be 2.0 and K_N for the existing line is found to be 1.4. Each coefficient is multiplied by the outfall velocity head to obtain the pressure rises h_F and h_N for the corresponding laterals. Then each pressure rise is added to the elevation of the downstream (outfall pipe) pressure line to obtain the elevation of each lateral pipe pressure line at its branch point. The water surface elevation in the inlet will correspond to the far lateral pressure line; that is, the new lateral pipe in this case. Finally, the clearance of the water surface elevation below the gutter is checked. It will be noted that the pressure line of the existing line in the near position is at a lower elevation than that of the lateral to Inlet 3 in the far position.

The alternate offset lateral arrangement, with the existing lines in the far position, is examined in the right-hand column of the tabulations. The position for use of Chart IV-F is similar to that shown in the center column. In this case, the existing line is found to have the higher pressure line elevation. Although the pressure difference at Inlet No. 5 is not large, it is significant in this case because the existing pipeline has the larger discharge rate and consequently the greater friction slope for its flow. Since the pressure line in this pipe is steeper in this example than in the new pipe to Inlet 3, it is advisable to select the arrangement at the inlet which places the existing pipe in the near position; that is, the design shown by the center column in Figure IV-20.

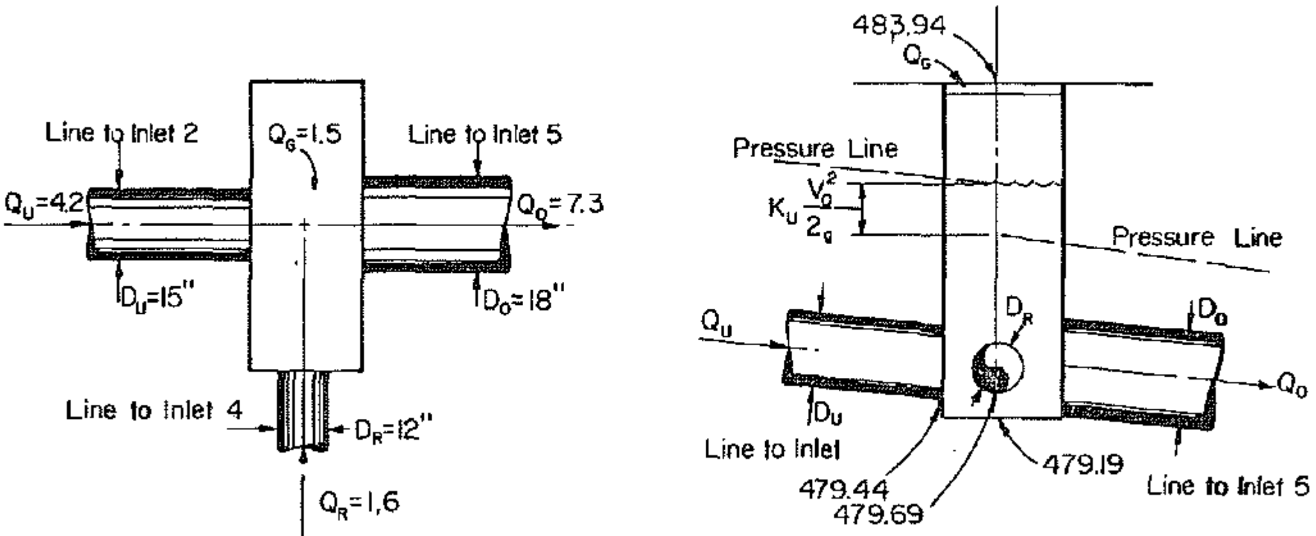
It is worthy to note that the water surface elevation in Inlet No. 5 is for all intents and purposes, at the level of the gutter. This provides an "automatic valve" in the system.

This fact will prevent extra runoff entering the system and causing unforeseen problems at other locations in the pipe network.

Inlet No. 3

Known data in this case are the gutter elevation, the inlet bottom elevation, the pipe and grate inflow rates, the outfall flow rate, the pipe diameters, and the elevation of the downstream (outfall pipe) pressure line at the branch point. From these data, the velocity head of the outfall flow, the ratios D_U/D_O, Q_U/Q_O and Q_G/Q_O, and the distance from the downstream pressure line to the inlet bottom may be calculated. Next d/D_O is estimated, including an allowance for h_U. Next K_U is obtained from Chart IV-D, using a base value from the lower graph and adding an increment from the upper graph for $d/D_O = 2$. The total for K is multiplied by the velocity head in the outfall to obtain h_U, the change of pressure. Then h_U is added to the outfall pressure line elevation to obtain the elevation of the upstream in-line pipe pressure line at the branch point. The pressure line of the lateral pipe and the water surface in the inlet will correspond to this upstream in-line pipe pressure elevation. Finally d/D_O is recomputed to check the estimate made initially, and the clearance of the water surface below the gutter is checked.

EXAMPLE STORM DRAIN DESIGN INLET NO. 3



PLAN ELEVATION

FIGURE IV-21 USE CHART IV-D

Item	Inlet 3
Gutter Elevation	483.94
Inlet Bottom Elevation	479.19
Grate Inflow Q_G cfs	1.5
Upstream In-line Flow Q_R	4.2
Right Lateral Flow Q_R	1.6
Outfall Flow Q_O cfs	7.3
Outfall Pipe Diameter D_O in.	18
Outfall Velocity Head $V_O^2/2g$ ft.	0.26
Flow Ratios Q_U/Q_O	0.58
Q_G/Q_O	0.21
Pipe Size Ratio D_U/D_O	0.83
Downstream Pressure Elevation	482.12
Pressure Elevation Above Bottom	2.93
Estimated d/D_O	2.1
Pressure Rise Coefficient for U.S. Main and Lateral Chart IV-D K_U = 1.07 + 0.28	1.35
Pressure Rise 1.35 $V_O^2/2g$ ft.	0.35
Upstream Pressure Line Elevation for Main and Lateral and Water Surface Elevation	482.47
Check d/D_O	2.18
Clearance, gutter to water ft.	1.47
Depth of Water in Inlet ft.	3.28 Pressure Conduit

USHGL @ Inlet 5 = 480.98

Pipeline Data:

Downstream:	Upstream:	Lateral:
Q = 7.3 cfs	Q = 4.2 cfs	Q = 1.6 cfs
Length 238 ft	ø = 15 inches	ø = 12 inches
ø = 18 inches	V = 3,42 fps	V = 2.04 fps
s = .0065 ft./ft.		
V = 4.13 fps		
$\frac{V^2}{2g}$ =0.26 ft.		
s_f = 0.0048 ft./ft.		
h_f = 1.14 ft.		

Inlet No. 2

This inlet illustrates an inlet to which Chart IV-C applies.

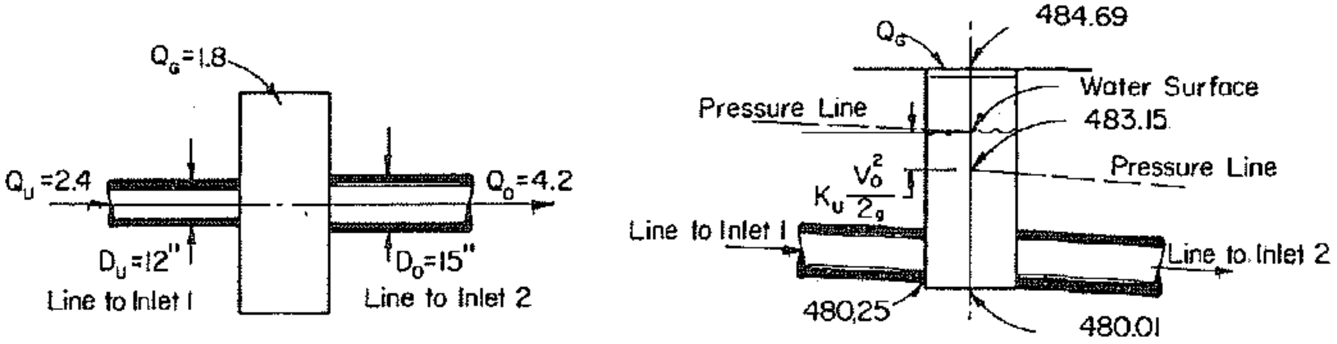
Known quantities are the gutter elevation, the inlet bottom elevation, the pipe and grate inflow rates, the outfall flow rate, the pipe diameters, and the elevation of the downstream (outfall pipe) pressure line at the inlet center. From these data the velocity head of the outfall flow, the ratios D_U/D_O and Q_U/Q_O, and the distance from the downstream pressure line to the inlet bottom may be calculated. Next d/D_O is estimated, including an allowance for h_U. Next K_U is read from Chart IV-C (the lower graph in this case) and multiplied by the velocity head in the outfall to obtain h_U, the change or pressure. Then h_U is added to the outfall pressure line elevation to obtain the elevation of the upstream in-line pipe pressure line, to which the water surface in the inlet corresponds. Finally d/D_O is recomputed to check the estimate made initially, and the clearance of the water surface below the gutter is checked.

Inlet No. 1

This inlet illustrates inlets to which Chart IV-A applies for box-side outfall. The determination of the water surface elevation in the inlet proceeds in the same manner in either case.

Known quantities are the gutter elevation, the inlet bottom elevation, the inflow rate, the outfall pipe diameter, and the elevation of the downstream (outfall pipe) pressure line. From these data, the velocity head of the outfall flow and the depth from the downstream pressure line to the inlet bottom may be calculated. Then d/DO is estimated, including an allowance of h_G. Then K_G is read from Chart IV-A and multiplied by the velocity head in the outfall to obtain h_G, the rise of the water surface elevation above the pressure line. Finally h_G is added to the outfall pressure line elevation to obtain the water surface elevation, d/D_O is recomputed to determine the accuracy of the estimate made initially, and the clearance of the water surface below the gutter is checked.

EXAMPLE STORM DRAIN DESIGN INLET NO. 2



PLAN ELEVATION

FIGURE IV-22 USE CHART IV-C

Item	Inlet 2	
Gutter Elevation	484.69	
Inlet Bottom Elevation	480.01	
Q_G cfs	1.8	
Q_U cfs	2.4	
Q_O cfs	4.2	
D_O in.	15	
Outfall Velocity Head $V_O^2/2g$ ft.	0.18	
Downstream Pressure Elevation	483.15	
Pipe Size Ratio D_U/D_O	0.80	
Flow Ratio Q_U/Q_O	0.57	
Pressure Elevation Above Bottom ft.	3.14	
Estimated d/D_O	2.7	
Chart IV-C: $K_U = K_G$	1.40	
Pressure Rise $K_G \times V_O^2/2g$ ft.	0.25	
Upstream Pressure Line and Water Surface Elevation	483.40	
Check d/D_O	2.72	
Clearance, Gutter to Water ft.	1.29	
Depth to Water to Inlet, ft.	3.39	Pressure Conduit

USHGL @ Inlet 3 = 482.47

Pipeline Data:

Downstream:

Q = 4.2 cfs

Length = 162 ft.

Ø = 15 inches

s = .0035 ft./ft.

V = 3.42 fps

$\frac{V^2}{2g}$ = 0.18 ft.

s_f = 0.0042 ft./ft.

h_f = 0.68 ft.

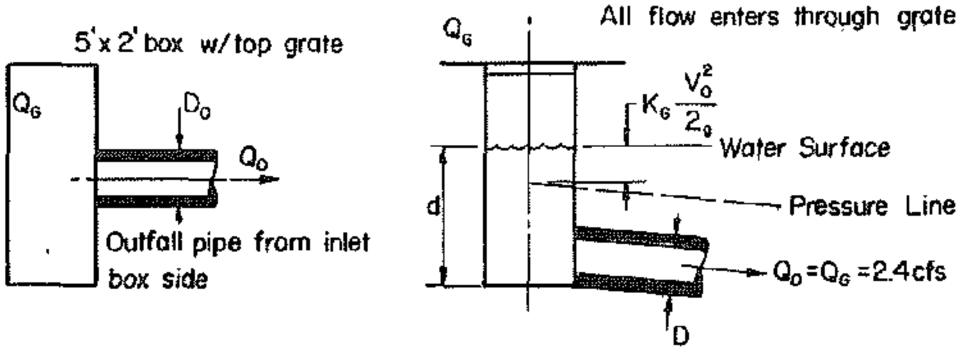
Upstream

Q = 2.4 cfs

Ø = 12 ft.

V = 3.06 fps

EXAMPLE STORM DRAIN DESIGN INLET NO. 1



PLAN ELEVATION

FIGURE IV-23 USE CHART A

Item	Inlet 1
Gutter Elevation	486.28
Inlet Bottom Elevation	482.00
$Q_G = Q_O$ cfs	2.4
D_O in.	12
Outfall Velocity Head $V_O^2/2g$ ft.	0.14
Downstream Pressure Elevation	484.34
Pressure Elevation above Bottom Ft.	2.34
Estimated d/D_O	2.8
Water Depth Over Pressure Line	
Chart IV-A, K_G	3.3
Rise, K_G x $V_O^2/2g$ ft.	0.46
Water Surface Elevation	484.80
Check d/D_O	2.80
Clearance, Gutter to Water ft.	1.48
Depth of Water in Inlet ft.	2.80 Pressure Conduit

USHGL @ Inlet 2 = 483.40

Pipeline Data:

$Q = 2.4$ cfs

Length = 208 ft.

$\phi = 12$ inches

$s = .0084$ ft./ft.

$V = 3.06$ fps

$\frac{V^2}{2g} = 0.14$ ft.

$s_f = 0.0045$ ft./ft.

$h_f = 0.94$ ft.

Inlet No. I completes the main line computations. It is now necessary to compute the hydraulic grade line in the laterals. Some inlets which are repetative in procedure have not been included in the example computations.

Manhole No. M.H.-6

This type of manhole is a typical round manhole to which Chart IV-G applies. Calculations for determination of the pressure change as this manhole are presented in Figure IV-24.

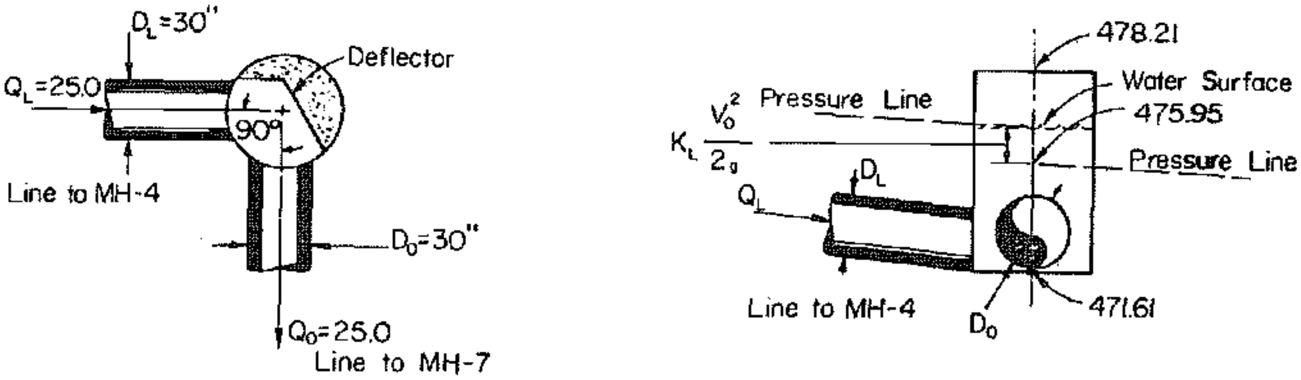
Known data are the elevations of the top and bottom (flowline) of the manhole, the manhole diameter, the rate of flow, the two pipe diameters (equal in this case), and the elevation of the downstream (outfall pipe) pressure line at the branch point.

From these data the ratios D_L/D_O and B/D_O, and the velocity head of the outfall flow may be calculated.

Chart IV-G (for square manholes) is used to obtain the pressure change coefficient K_L even though Manhole No. M.H.-6 is round (where $Q_L = Q_O$, $K_L = \bar{K}_L$). First K_L for a square manhole is read from the chart at $D_L/D_O = 1.00$ and $B/D_O = 1.33$, and this value is reduced by 0.1 for the round manhole in accordance with the table contained in Step (2) of the instructions for the use of Chart IV-G for round manholes. The outfall pipe entrance is sharp-edged in this case, so no further reduction is made. Next $\bar{K}_L$ is multiplied by the outfall velocity head to obtain h_L, the change of pressure (or pressure rise) at the manhole. Finally h_L is added to the outfall pressure line elevation to obtain the elevation of the lateral pipe pressure line at the branch point, or manhole center.

Chart IV-H is used to obtain the water-surface elevation in the manhole. In this case of example Manhole No. M.H.-6, the coefficient $\bar{K}_U$ an upstream in-line pipe with no flow ($Q_L/Q_O = 1.00$) is found from Chart IV-G to be 1.73. This coefficient will define the depth of water above the downstream pressure line for no in-line flow, whether or not the in-line pipe is actually present. Then $\bar{K}_U$ is multiplied by the

EXAMPLE STORM DRAIN DESIGN MANHOLE NO. 6



PLAN

ELEVATION

FIGURE IV-24 USE CHART IV-G

Item	M.H.-6	
Top of M.H. elevation	478.21	
Bottom M.H. Elevation	471.61	
Lateral Flow Q_L cfs	25.0	
Outfall Flow Q_O cfs	25.0	
Outfall Pipe Diameter D_O in	30	
Pipe Size Ratio D_L/D_O	1.00	
Manhole Diameter B in	48	
M.H. Size Ratio B/D_O	1.60	
Outfall Velocity Head $V_O{}^2/2g$ ft.	0.40	
Downstream Pressure Elevation	475.96	
Pressure Rise Coefficient (sq. edge entr.)		
Chart IV-G K_L for sq. M.H.	1.55	
k_L for rd. (less 0.1)	1.45	
Pressure Rise 1.4 x 0.40 ft.	0.58	
Upstream Pressure Line Elevation	476.54	
Water Surface		
Chart IV-H $\bar{K}_U$	1.73	
Water Depth Over Outfall Pressure - 1.73 x 0.40 ft.	0.69	
Water Surface Elevation	476.65	
Clearance, Water Below Top ft.	1.56	
Depth of Water in Manhole ft.	5.04	Pressure Conduit

USHGL @ M.H. -4 = 475.36

Pipeline Data:

Downstream	Upstream
Q = 25 cfs	Q = 25 cfs
Length = 162 ft.	Q = 30 inches
Ø = 30 inches	
s = 0.0084 ft./ft.	
V = 5.09 cfs	
$\frac{V^2}{2g}$ = 0.40 ft.	
s_f = 0.0037 ft./ft.	
h_f = 0.60 ft.	

outfall velocity head to obtain h_U, the depth of water over the outfall pressure line. Next h_U is added to the elevation of the outfall pressure line at the branch point to obtain the elevation of the water surface in the manhole. Finally, the clearance of the water surface below the manhole top is checked.

Manhole No. M.H.-7

This type of manhole is one to which Chart IV-J applies.

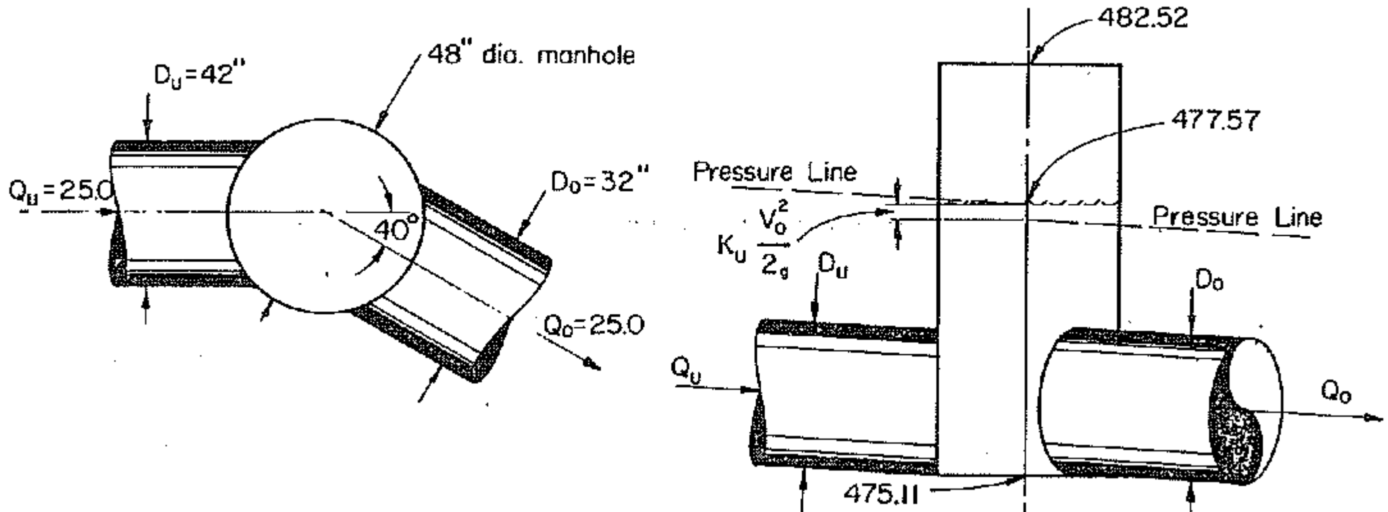
Known quantities are the gutter elevations, the manhole bottom elevation, the flow rate, the pipe diameter, the deflection angle and characteristics, and the elevation of the downstream pressure line. From these data the velocity head of the outfall flow may be determined. The loss coefficient K is read from Chart IV-J and is multiplied by the outfall velocity head to obtain the rise of the water surface above the downstream pressure line elevation. This corresponds to the upstream pressure line elevation. The clearance of the water surface below the gutter should be checked.

Inlet No. 9

This inlet is an example of the type of inlet to which Chart IV-E applies. The ground above Inlet No. 9 rises very sharply and for the sake of economy, the upstream in-line pipe is raised 3.66 feet above the inlet bottom. As will be seen in the worked example, the water surface elevation in the inlet is below the invert of the upstream pipe. This implies that flow will be open-channel in the upstream pipe, at least in its lower section. The flow from the upstream pipe will be treated as grate flow to the inlet. The designer should check upstream junctions to identify the type of flow at each and design accordingly.

The known data are the gutter elevation, the elevation of the inlet bottom, the lateral pipe inflow rates, the outfall flow rates, the pipe diameters and the elevation of the downstream pressure line. From the lateral pipe flow rates and sizes the velocity in each of the laterals is determined, and the two laterals are identified as higher-velocity and lower-velocity. In this case, the right lateral, looking downstream, is the higher velocity lateral. From the given data and the above

EXAMPLE STORM DRAIN DESIGN MANHOLE NO. 7



PLAN

ELEVATION

FIGURE IV-25 USE CHART IV-J

Item	M.H.-7	
Gutter Elevation	482.52	
Manhole Bottom Elevation	475.11	
Upstream = Downstream flow cfs	25.0	
Upstream Pipe Diameter in.	30	
Downstream Pipe Diameter	30	
Outfall Velocity Head $V_0^2/2g$ ft.	0.40	
D_U/D_O	1.0	
(This implies that there is no contraction or expansion headloss)		
Deflection angle	40°	
Downstream Pressure Elevation	477.47	
Chart IV-K, K	0.25	
Upstream Pressure Rise = K x $V_0^2/2g$	0.10	
Upstream Pressure Elevation, W.S.E.	477.57	
Clearance, Water Below Top ft.	4.96	
Depth of Water in Manhole	2.46	Assuming flowing full

USHGL @ M.H.-6 476.54

Pipeline Data:

$Q = 25.0$ cfs

Length = 250 ft.

$\not{0} = 30''$

$s = 0.0140$ ft./ft.

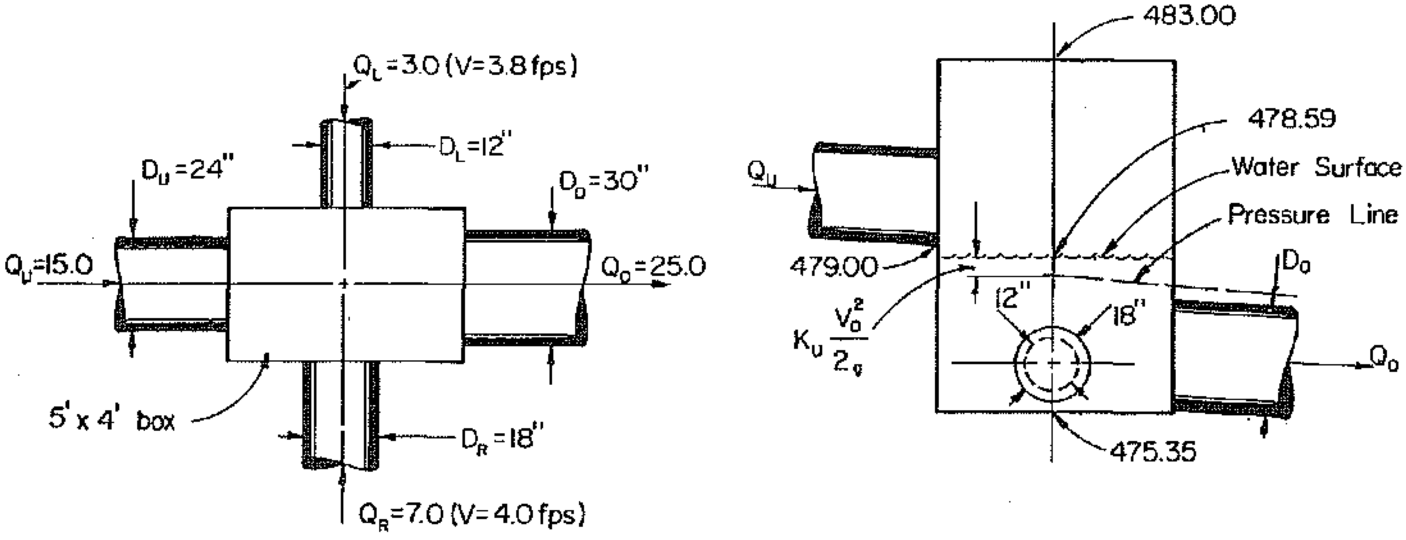
$V = 5.09$ fps

$\frac{V^2}{2g} = 0.40$ ft.

$s_f = 0.0037$ ft./ft.

$h_f = 0.93$ ft.

EXAMPLE STORM DRAIN DESIGN INLET NO. 9



PLAN

ELEVATION

FIGURE IV-26 USE CHART IV-E

Item	Inlet 9
Gutter Elevation	483.00
Inlet Bottom Elevation	475.35
Flow Ratios Q_G/Q_O	0.60
Q_{hv}/Q_O	0.28
Q_{lv}/Q_O	0.12
Pipe Size Ratios D_{lv}/D_O	0.40
D_{hv}/D_O	0.60
D_{hv}/D_{lv}	1.50
Velocity Head $V_O^2/2g$	0.40
Downstream Pressure Elevation	477.88
Chart IV-E Factor H	3.1
Factor L	0.6
K_L = H-L	2.5
K_R =	1.8 (high vel. lat)
Pressure Rise Left Lateral = 2.5 x 0.40 ft.	1.0
Pressure Rise Right Lateral = 1.8 x 0.40 ft.	0.72
Upstream Pressure Elevation Left Lateral	478.88
Upstream Pressure Elevation Right Lateral	478.60
Water Surface Elevation	478.60
Clearance, Gutter to Water ft.	4.41
Depth of Water in Inlet ft.	3.24 Pressure Conduit Downstream & Lats.

USHGL @ MH -7 = 477.57

Pipeline Data:

Downstream:	Upstream:	L. Lateral:	R. Lateral;
Q = 25.0 cfs	Q = 15.0 cfs	Q = 3.0 cfs	Q = 7.0 cfs
Length = 85.0 ft.	Ø = 24 in.	Ø = 12 in.	Ø = 18 in.
Ø = 30 inches		V = 3.82 fps	V = 3.96 fps
s = .0028 ft./ft.			
V = 5.09 fps			
$\frac{V^2}{2g}$ = 0.40' ft.			
s_f = 0.0037 ft./ft.			
h_f = 0.0031 ft.			

determination, the ratios Q_G/Q_O, Q_{hv}/Q_O, $Q_{lv},/Q_O$, D_{hv}/D_O, D_{lv}/D_O, and D_{hv}/D_{lv} (=1.50) are calculated. Next the velocity head in the outfall pipe is determined. The pressure factors H and L are read from Chart IV-E, and identified by the lateral to which the D and Q of the two graphs apply. The difference between H and L (3.1 - 0.6 = 2.5) is the pressure change coefficient $K_L = K_{lv}$, the lower velocity lateral. The constant coefficient $K_R = K_{hv}$ is 1.8 because the upstream flow is treated as grate flow entering an inlet. Each coefficient is multiplied by the velocity head of the outfall flow to obtain the pressure changes, h_{lv} and h_{hv}, for the laterals. The pressure change is always positive and so produces a rise in pressure upstream. The pressure rise, h_{lv}, is used to obtain the pressure line elevation on the higher velocity lateral. The water surface elevation corresponds to the latter pressure line. The clearance of the water surface below the gutter should be checked.

Manhole No. M.H.-8

This manhole has been included to illustrate junction energy loss computations with open channel flow. The following data apply:

USHGL @ Inlet I-9-Must be computed

$Q = 15.0$ cfs

$\phi = 24''$ Downstream

$\phi = 21''$ Upstream

Length to MH-8 = 300 feet

$s = 0.012^1/ft.$ Downstream

$s = 0.014^1/ft.$ Upstream

No Deflection in Manhole

Computations:

Downstream Pipe:

$Q_f = 25.0$ cfs

$\frac{Q}{Q_f} = 0.60$

$\frac{d}{D} = 0.63$ (Figure IV-4)

d =1.26 feet = Normal Depth

$V_f = 7.96$ fps

$\frac{V}{V_f} = 0.90$ (Figure IV-4)

$V = 7.16$ fps

$V^2/2g = 0.80$ ft.

Downstream Invert Elevation = 482.60

Downstream Energy Grade Line Elevation = 484.66

Upstream Pipe

$Q_f = 19$ cfs

$\frac{Q}{Q_f} = 0.79$

$\frac{d}{D} = 0.75$ (Figure IV-4)

d = 1.31 ft. = normal depth

$V_f = 6.24$ fps

$\frac{V}{V_f} = 0.98$ (Figure IV-4)

$V = 6.11$ fps

$V^2/2g = 0.58$ ft.

Head Losses for Expansion = 0.2 (△ hv) = 0.2 (0.80 - 0.58) = 0.04 ft.

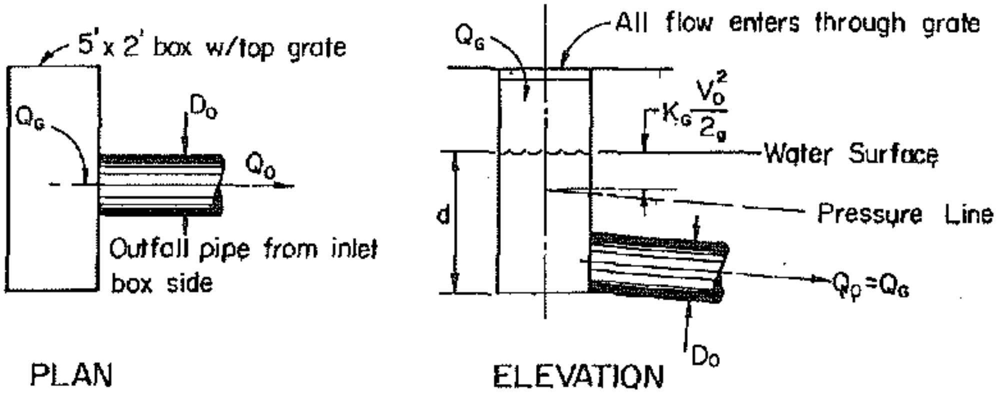
Downstream Energy Grade Line Elev.	484.66
+ Loss for Expansion	+0.04
Upstream Energy Grade Line Elev.	484.70
- Upstream Velocity Head	0.58
- Upstream Depth of Flow	1.31
Upstream Invert Elevation	482.81

Inlets 4 and 7

Inlet 7 illustrates the type to which Chart IV-A applies for box-side outfall. Inlet 4 illustrates an inlet to which the chart applies for box-end outfall. The determination of the water surface elevation in the inlet proceeds in the same manner in either case.

Known quantities are the gutter elevation, the inlet bottom elevation, the inflow rate, the outfall pipe diameter, and the elevacion of the downstream (outfall pipe) pressure line. From these data, the velocity head of the outfall flow and the depth from the downstream pressure line to the inlet bottom may be calculated. Then d/D_O is estimated, including an allowance of h_G. Then K_G is read from Chart IV-A and multiplied by the velocity head in the outfall to obtain h_G, the rise of the water surface elevation above the pressure line. Finally h_g is added to the outfall pressure line elevation to obtain the water surface elevation, d/D_O is recomputed to determine the accuracy of the estimate made initially, and the clearance of the water surface below the gutter is checked.

EXAMPLE STORM DRAIN DESIGN INLET NO. 7



PLAN

ELEVATION

FIGURE IV-27 USE CHART IV-A

Item	Inlet 7
Gutter Elevation	483.77
Inlet Bottom Elevation	479.00
$Q_G = Q_O$ cfs	2.6
D_O in.	12
Outfall Velocity Head $V_O^2/2g$ ft.	0.17
Downstream Pressure Elevation	480.86
Pressure Elevation Above Bottom ft.	1.86
Estimated d/D_O	2.5
Water Depth Over Pressure Line	
Chart IV-A, K_G	3.7
Rise, $K_G \times V_O^2/2g$ ft.	0.63
Water Surface Elevation	481.49
Check d/D_O	2.49
Clearance, Gutter to Water ft.	2.28
Depth of Water in Inlet ft.	2.49

USHGL @ Inlet 6 = 479.70

Pipeline Data:

$Q = 2.6$ cfs

Length = 218 ft.

$\emptyset = 12$ inches

$s = 0.0055$ ft./ft.

$V = 3.31$ fps

$\frac{V^2}{2g} = 0.17$ ft.

$s_f = 0.0053$ ft./ft.

$h_f = 1.16$ ft.

EXAMPLE STORM DRAIN DESIGN INLET NO. 4

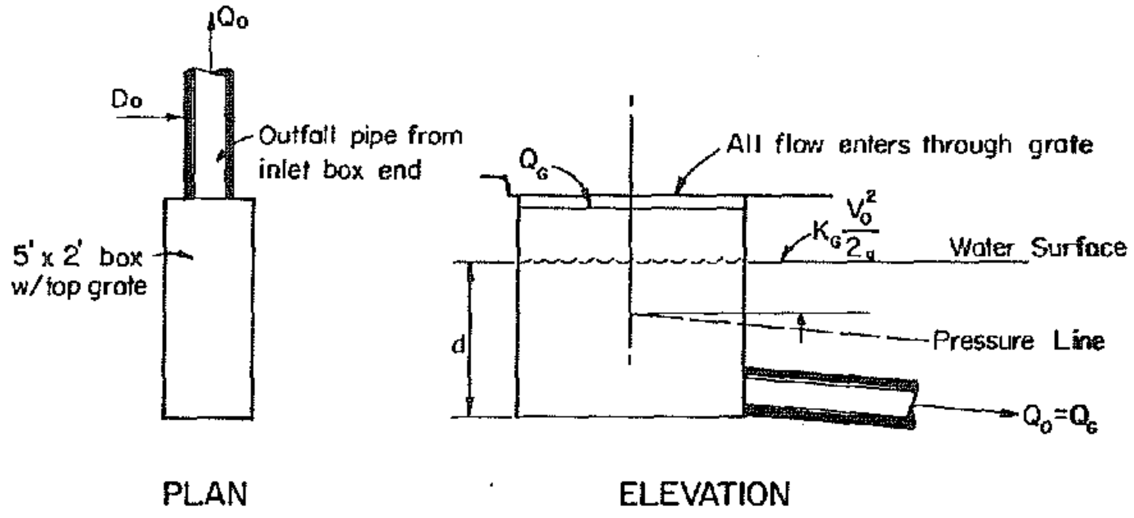


FIGURE IV-28 USE CHART IV-A

Item	Inlet 4
Gutter Elevation	483.94
Inlet Bottom Elevation	480.50
$Q_G = Q_O$ cfs	1.6
D_O in	12
Outfall Velocity Head $V_O^2/2g$	0.06
Downstream Pressure Elevation ft.	482.60
Press. Elevation Above Bottom ft.	2.10
Estimated d/D_O	2.5
Water Depth Over Pressure Line	
Chart IV-A K_G	5.0
Rise, $K_G \times V_O^2/2g$ ft.	0.30
Water Surface Elevation	482.90
Check d/D_O	22.40
Clearance, Gutter to Water ft.	1.04
Depth of Water in Inlet, ft.	2.40

USHGL @ Inlet 3 = 482.47

Pipeline Data:

$Q = 1.6$ cfs

Length = 67 ft.

$\emptyset = 12$ in.

$s = 0.0121$ ft./ft.

$V = 2.04$ fps

$\frac{V^2}{2g} = 0.06$ ft.

$s_f = 0.0020$ ft./ft.

$h_f = 0.13$ ft.

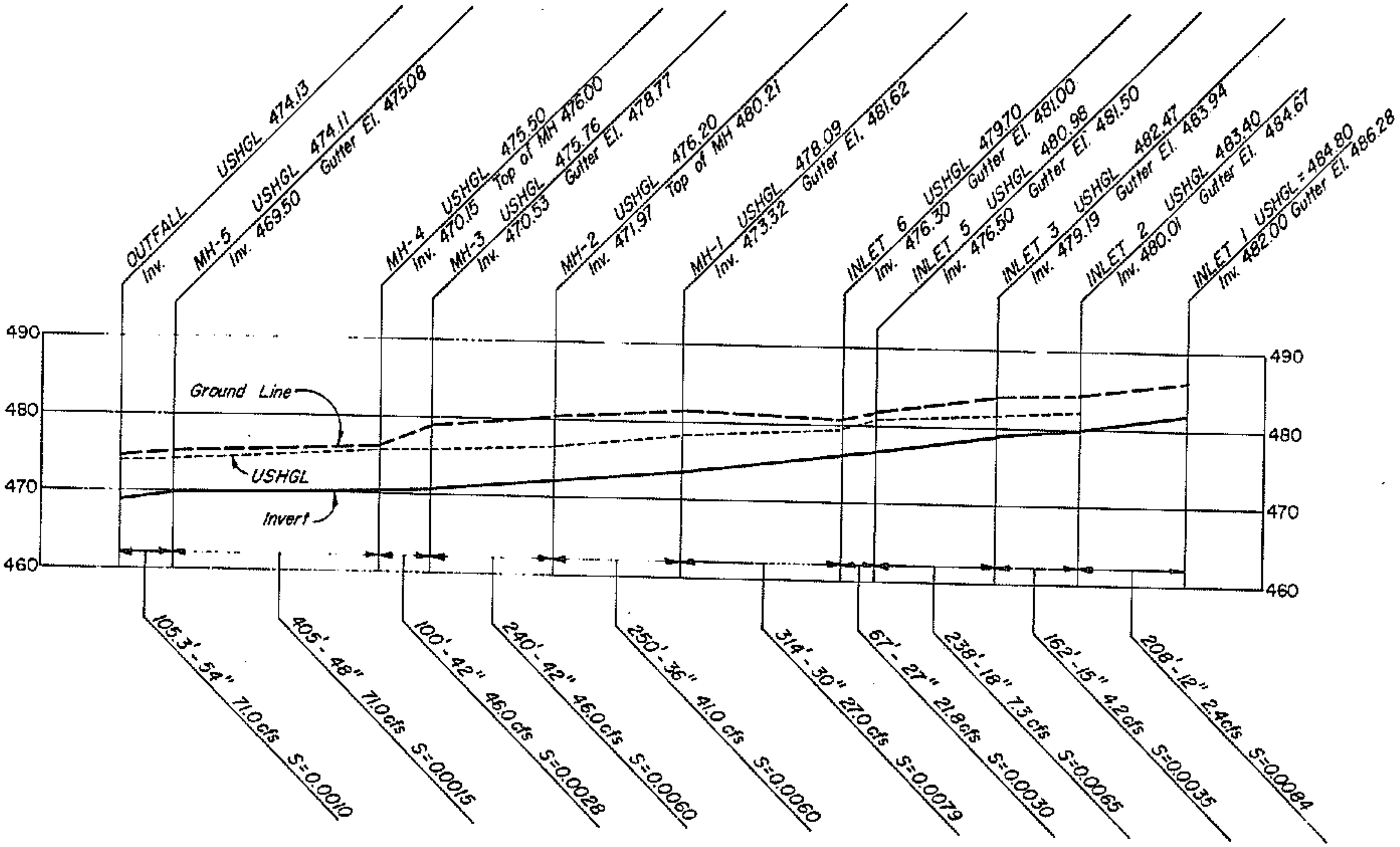


FIGURE IV-29
PROFILE OF EXAMPLE PROBLEM SEWER SHOWING HYDRAULIC PROPERTIES

APPENDIX IV-A

RATIONAL METHOD FOR SIZING STORM SEWER SYSTEM

The purpose of this Appendix is to describe the manner in which the "summation of discharges" form of the rational formula is to be utilized.

This method is a part of preliminary design and represents the hydrology portion of the final design. That is, it established the estimated flows which need to be carried in the system. An example is also contained in this Appendix which develops the discharges used in the hydraulic (final) design example contained in the main body of Chapter IV, Part II. After the preliminary minor system design is completed and checked for its interaction with the major runoff, reviews made of alternatives, hydrological assumptions verified, new computations made and final data obtained on street grades and elevations, the engineer should proceed with final hydraulic design of the system.

The following step-by-step procedure should be used in conjunction with Figure IV-30. The procedure is for the average situation and variations will often be necessary to fit actual field conditions.

Column 1 - Determine design point location and list. This design point should correspond to the subbasin illustrated on the preliminary layout map.

Column 2 -List basins contributing runoff to this point which have not previously been analyzed.

Column 3 - Enter length of flow path between previous design point and design point under consideration.

Column 4 - Determine the inlet time for the particular design point. For the first design point on a system, the inlet time will be equal to the time of concentration. For subsequent design points, inlet time should also be tabulated to determine if it may be of greater magnitude than the accumulated time of concentration from upstream basins. If the inlet time exceeds the time of concentration from the upstream basin, and the area tributary to the inlet is of sufficient magnitude, the longer inlet time should be substituted for time of concentration and used for this and subsequent basins. See the Hydrology Chapter of Part II of this Manual for methods of determining inlet time.

Column 5 - Enter the appropriate flow time between the previous design point and the design point under consideration. The flow time of the street should be used if a significant portion of the flow from the above basin is carried in the street.

Column 6 - Pipe flow time should generally be used unless there is significant carry-over from above basins in the street.

Column 7 - The time of concentration is the summation of the previous design point time of concentration and the intervening flow time.

Column 8 - Rational Method Runoff Coefficient, "C," for the basins listed in Column 2 should be determined and listed. The "C" value should be weighted if the basins contain areas with different "C" values.

Column 9 - The intensity to be applied to the basins under consideration is obtained from the time-intensity-frequency curve developed for the specific area under consideration based upon the depth-duration-frequency curves in the Hydrology Chapter of this Manual. The intensity is determined from the time of concentration and the design frequency for this particular design point.

Column 10 - The area in acres of the basins listed in Column 2 is tabulated here. Subtract ponding areas which do not contribute to direct runoff such as rooftop and parking lot ponding areas.

Column 11 - Direct runoff from the tributary basins listed in Column 2 is calculated and tabulated here by multiplying Columns 8, 9, and 10 together.

Column 12 - Runoff from other sources, such as controlled releases from rooftops, parking lots, base flows from groundwater, and any other source, is listed here.

Column 13 - The total of runoff from the previous design point summation plus the incremental runoff listed in Columns 11 and 12 is listed here.

Column 14 - The proposed street slope is listed in this column.

Column 15 - The allowable capacity for the street is listed in this column. Allowable capacities should be calculated in accordance with procedures set forth in the Streets Chapter of this Manual.

Column 16 - List the proposed pipe grade.

Column 17 - List the required pipe size to convey the quantity of flow necessary in the pipe.

Column 18 - List the capacity of the pipe flowing full with the slope expressed in Column 16.

Column 19 - Tabulate the quantity of flow to be carried in the street.

Column 20 - List the actual velocity of flow for the volume of runoff to be carried in the street.

Column 21 - List the quantity of flow determined to be carried in the pipe.

Column 22 - Tabulate the actual velocity of flow in the pipe for design.

Column 23 - Include any remarks or comments which may affect or explain the design. The allowable quantity of carry-over across the street intersections should often be listed for the minor design storm. When routing the major storm through the system, required elevations for adjacent buildings can often be listed in this column.

DESIGN EXAMPLE

The data contained in Figure IV-31 is intended to supplement the information shown in Figure IV-12 and IV-13 for the hydraulic design example. Because junction losses are ignored in this analysis, the pipe roughness coefficient factor is to be increased by 25 percent.

The designer should be aware that pipe diameters may change in final design from preliminary design; however, these effects are generally cancelling and can be ignored. When there is a net change of pipe diameters (non-cancelling) in 20 percent of the pipe length, the designer should redo this analysis to insure system integrity (higher discharges and shorter time of concentration) or eliminate wasted investment (lower discharges and lower times of concentration). If a different type of pipe with a large difference in roughness factors is used (RCP vs CMP). The system must be designed using both materials.

After the system has been designed according to the summation method of the Rational Method, the 100 year runoff is routed though the storm sewer system to insure that structural flooding criteria are met for that event. Because the analysis is similar, it is not shown here. The designer should be aware that the "C" factors must be increased by 25 percent for the 100 year event in the Rational Formula (maximum = 1.0) and the flow time from point to point is determined by the length and velocity of gutter flow (as compared to the time of flow in pipes used to size the system).

All basic data are shown in Figure IV-31. The entire area is assumed to be residential and the streets are either local or collector. The design frequency is 2 years, and the points of beginning for the storm sewer have been assumed. The purpose of this analysis is to demonstrate the use of the Rational Method for preliminary design of storm sewers. Unless computed in the Figure IV-31, times of concentration are assumed. Both the method of arriving at T_c and the rainfall-intensity-duration curve are contained in Chapter I of Part II, "Hydrology".

Of special note is the Hydraulic (final) design of the lateral which enters Manhole MH-4 from Basins 9 and 10. The hydraulic design example was done for the design conditions for the overall system. In reality, the system must also be hydraulically designed for the higher discharge (26.9 cfs) eminating from Basins 9 and 10. This is true of all laterals and inlet connector pipes.

The methodology is to determine the flow from the mainstem when the peak flow from Basins 9 and 10 occurs. This is a trial and error process starting at some point between I-1 and MH-4. This can be approximately determined by progressively subtracting the flow times (proceding upstream) obtained in the preliminary design of the main trunk. The new starting point will be when the T_c at a design point equals the T_c after subtracting flow time to that point. Using the pipes as originally determined, new discharges are computed at the point in question to obtain the discharge in the mainstem when the higher lateral flow enters.

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This Chapter is to cover the criteria for various elements normally associated with major drainage facilities, that is, open channels, large closed conduits, bridges, culverts, and entrance and exit structures. It should be noted that these facilities may occasionally also apply to minor drainage. When this situation does occur, the same criteria contained herein will apply.

The delineation between major and minor drainage facilities is not always clear. The method of determining where major drainage facilities begin is articulated in Chapter IV of Part I "Design Procedures." When it is determined that major drainage facilities are required, the size of facilities dictate that careful selection of structure type is required and the magnitude of hazard requires careful attention to the major drainage criteria contained in this Chapter.

While the nature of this Chapter dictates that it be heavily structure oriented, the designer is reminded that preventive measures are less costly. The means of predicting the expected extent of flooding is also contained in this Chapter and is necessary for choosing both corrective and preventive measures.

OPEN CHANNELS

Open channels for use in the major drainage system have significant advantages in regard to cost, capacity, multiple use for recreational and aesthetic purposes, and potential for channel detention storage. Disadvantages include right-of-way needs and maintenance costs. Careful planning and design are needed to minimize the disadvantages, and to increase the benefits.

The ideal channel is a natural one carved by nature over a long period of time. The benefits of such a channel are that:

- Velocities are usually low, resulting in longer concentration times and lower downstream peak flows.
- Channel storage tends to decrease peak flows.
- Maintenance needs are usually low because the channel is somewhat stabilized.
- The channel provides a desirable greenbelt and recreational area, adding significant social benefits.

The closer the character of an artificial channel can be made to that of a natural channel, the better the artifical channel will be in regard to stability, maintenance, and the hydrologic characteristics downstream.

In many areas about to be urbanized, the runoff has been so minimal that natural channels do not exist. However, depressions or thalwegs nearly always exist which provide an excellent basis for location and construction of channels. Good land planning should reflect even these minimal thalwegs and natural channels to reduce development costs and minimize drainage problems. In some cases, the utilization of natural water routes wisely in the development of a major drainage system will obviate the need for an underground storm sewer system.

Channel stability is a well recognized problem in urban hydrology because of the significant increase in low flows and peak storm runoff flows following urbanization. A natural channel must be studied to determine what measures are needed so as to avoid future bottom scour and bank cutting. Erosion control measures can be taken which will preserve the natural appearance, not be costly, and function properly.

Choice of Channel

The choices of channel available to the designer are almost infinite, depending only upon good hydraulic practice, environmental design, The Manning Formula is:

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \qquad \text{Eq. V-1}$$

Where: Q = Discharge in cubic feet per second
n = Roughness coefficient
A = Area in square feet
R = Hydraulic radius, A/P
P = Wetted perimeter, feet
S_o = Channel bottom slope in feet per foot

Computations can be simplified by use of Figure V-1 (which is self-explanatory). The various design cases are rectangular, trapezoidal, and circular cross sections.

Because of variable channel cross sections and channel properties, uniform flow computations are rarely used. Normally, a designer will use these values for conceptual level decisions. Decisions relative to preliminary and final design requirements should be made through the use of backwater determinations.

For natural channels and for compound artificial channels, it will normally be necessary to apply Manning's equation (Equation V-1) to sections of the channel which have similar properties.

Critical Flow. Critical flow in an open channel or covered conduit with a free water surface is characterized by several conditions. They are:

The specific energy is a minimum for a given discharge.
The discharge is a maximum for a given specific energy.
The specific force is a minimum for a given discharge.
The velocity head is equal to half the hydraulic depth in a channel of small slope.
The Froude number is equal to 1.0.
The velocity of flow in a channel of small slope is equal to the celerity of small gravity waves in shallow water.

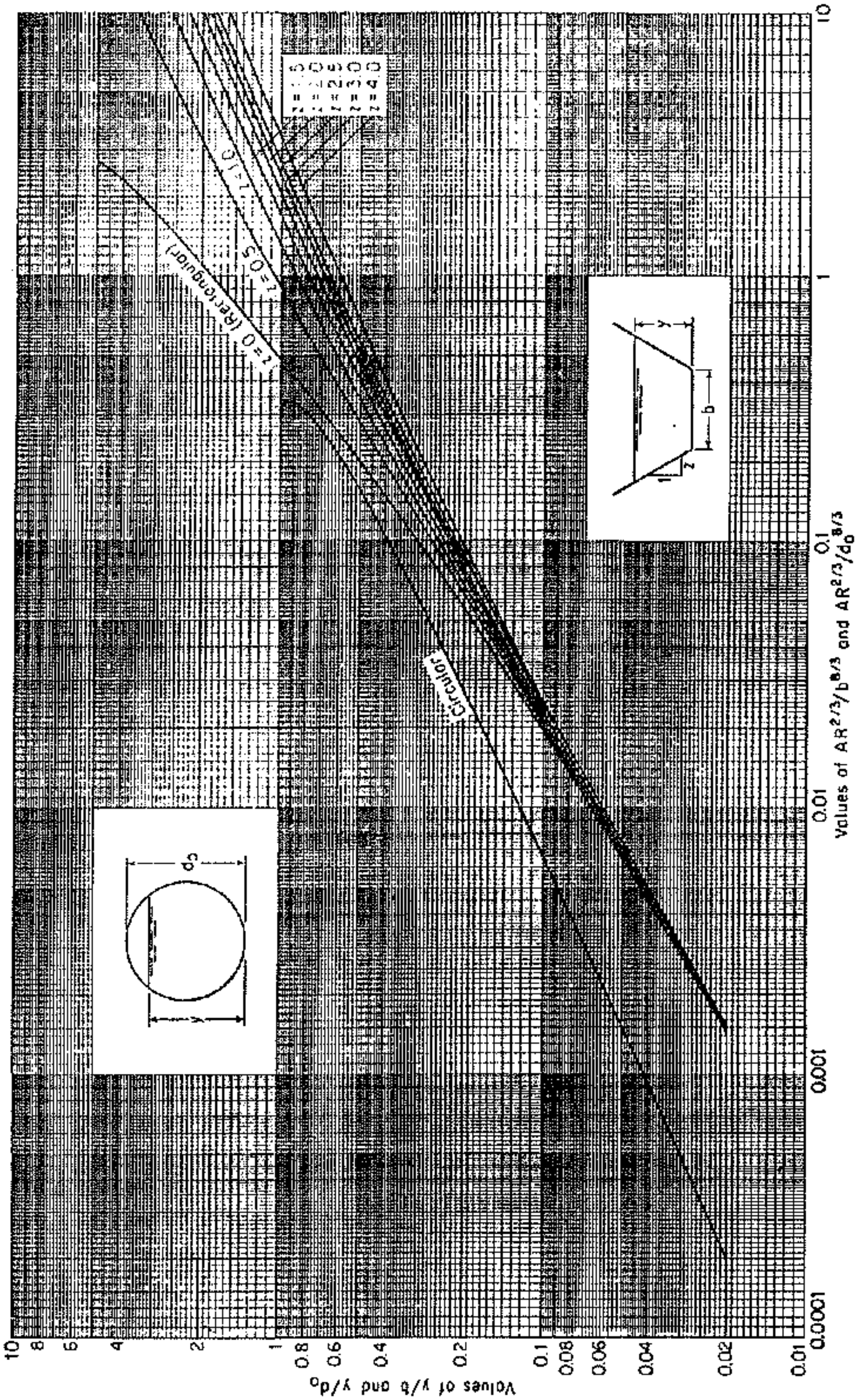


FIGURE V-1
CURVES FOR DETERMINING THE NORMAL DEPTH
(from V.T. Chow)

If the critical state of flow exists throughout an entire reach, the channel flow is critical flow 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 will cause supercritical flow. A flow at or near the critical state is not stable. In the design, if the depth is found to be at or near critical, the shape or slope should be changed to achieve greater hydraulic stability. For critical flow, $\frac{v^2}{2g} = \frac{y_c}{2}$ and if Q/A is substituted for v, the equation

may be written as: $$F = \frac{V}{\sqrt{gy}} \qquad \text{Eq. V-2}$$
where F represents $Ay_c^{0.5}$, the section factor for critical flow computation.

Since F is a function of depth, the equation indicates there is only one possible critical depth for maintaining a given discharge in a given channel.

Equation V-2 is a useful tool for the computation and analysis of critical flow in an open channel. When the discharge is known, the equation gives the critical section factor F_c and, hence, the critical depth y_c.

To simplify the computation of critical flow, dimensionless curves showing the relation between depth and the section factor F have been given for rectangular, trapezoidal, and circular channels in Figure V-2.

Except in rare cases, velocities will not exceed the critical velocity in natural, earth, grass-lined or riprapped channels. When the flow conditions reach the unstable critical flow, the channel roughness increases and prohibits greater velocities than the critical value. It is at this point where channel sediment transport capacity (and erosion) is the greatest; therefore, artificial channels (except concrete-lined) should be designed with a Froude number less than 0.8.

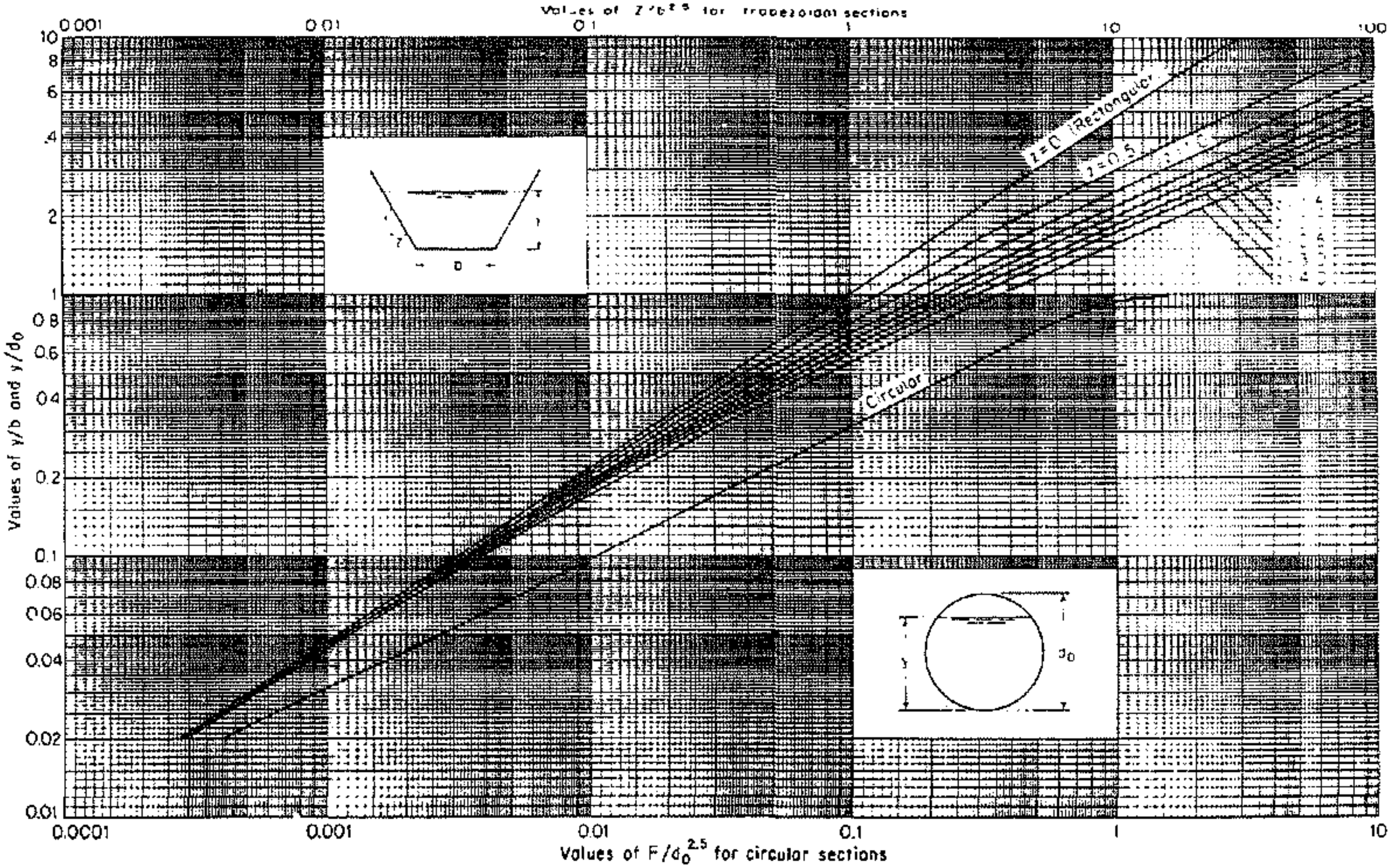


FIGURE V-2

CURVES FOR DETERMINING THE CRITICAL DEPTH IN OPEN CHANNEL (5)

(from V.T. Chow)

When using Manning's equation on natural channels, the Froude number should be checked to determine if it exceeds 1.0. If so, in most cases, the normal depth can be determined by computing the critical depth.

Roughness Coefficients. Roughness coefficients (n) for use in Manning's equation vary considerably according to type of material, depth of flow, and quality of workmanship. Table V-1 lists roughness coefficients for pipes and for various artificial channels.

TABLE V-1

ROUGHNESS COEFFICIENTS FOR MANNING'S EQUATIONS

Coefficients for Channel Design Capacity

Concrete	n
Trowel Finish	0.013
Float Finish	0.015
Unfinished	0.017
Shotcrete, trowelled, not wavy	0.018
Shotcrete, trowelled, wavy	0.020
Shotcrete, unfinished	0.022

	Roughness Coefficient When Depth of Flow Equal	
Grass-lined Channels	0.7-1.5 ft.	3.0 - 4.0 ft.
Bermuda grass, Buffalo grass, Kentucky Bluegrass		
o Mowed to 2 inches	0.035	0.030
o Length 4 - 6 inches	0.040	0.030
Good stand any grass		
o Length of 12 inches	0.070	0.035
o Length of 24 inches	0.100	0.035
Fair stand any grass		
o Length of 12 inches	0.060	0.035
o Length of 24 inches	0.070	0.035

For additional information on roughness coefficients, the reader is referred to Geological Survey Water Supply Paper 1849.

Table V-2 lists roughness coefficients for earthen and natural channels.

The designer should be aware that roughness greater than that assumed will cause the same discharge to flow at a greater depth, or conversely that flow at the computer depth will result in less discharge. In addition, it should be realized that obstructions in the channel cause an increase in depth above normal and must be taken into account.

Design of Concrete-Lined Channels

Where the project requires a waterway for storm runoff to be concrete lined due to constricted right-of-way, concrete lining is usually chosen. Whether the flow will be supercritical or subcritical, the lining must be designed to withstand the various forces which act on the channel. Supercritical flow offers substantial challenge for the designer, and without prior approval of the City Engineer, supercritical channels will not be used.

Supercritical flow in an open channel in an urbanized area creates certain hazards which the designer must take into consideration. From a practical standpoint, it is generally not possible to have any curvature in such a channel. When the channel must be deflected, careful attention must be taken to insure against excessive oscillatory waves which may extend down the entire length of the channel from only minor obstructions upstream. Imperfections at joints may rapidly cause a deterioration of the joints, in which case, a complete failure of the channel can readily occur. In addition, high velocity flow entering cracks or joints creates an uplift force by the conversion of velocity head to pressure head which can damage the channel lining. It is evident that when designing a lined channel with supercritical flow the designer must use utmost care and consider all relevant factors.

TABLE V-2

VALUES OF THE ROUGHNESS COEFFICIENT n

From V. T. Chow

Type of Channel and Description	Minimum	Normal	Maximum
Excavated or Dredged:			
Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
Channels not maintained, weeds and Brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
Natural Streams			
Minor streams (top width at flood state < 100')			
Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045

TABLE V-2 (Continued)

Type of Channel and Description	Minimum	Normal	Maximum
4. Same as above, but some weeds and stones	0.035	0.040	0.050
5. Same as above, lower states, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4 but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways w/heavy stand of timber and underbrush	0.075	0.100	0.150
Mountain streams, no vegetation in channel, banks, usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles w/large boulders	0.040	0.050	0.070
Floodplains			
Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land w/tree stumps, no sprouts	0.030	0.040	0 050
3. Same as 2, but w/heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timer, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
Major streams (top width at flood state > 100') The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
Regular section with no boulders or brush	0.025	—	0.060
Irregular and rough section	0.035	--	0.100

All channels carrying supercritical flow shall be lined with continuously reinforced concrete, the reinforcing being continuous both longitudinally and laterally. There shall be no diminution of wetted area cross section at bridges or culverts. Freeboard shall be adequate to provide a suitable safety margin, the safety margin being at least 2 feet or an additional capacity of approximately one-third of the design flow. Bridges or other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load which might be imposed upon the structure in the event of major trash plugging.

Concrete-lined channels must be protected from hydrostatic uplift forces by the use of underdrains and weepholes, which are often created by a high water table of momentary inflow behind the lining from localized flooding. Often a perforated underdrain pipe will be required under the lining, the underdrain designed to be free-draining. With supercritical flows, minor downstream obstructions do not create any backwater effect. Backwater computation methods are applicable for computing the water surface profile or the energy gradient in channels having a supercritical flow; however, the computations must proceed in a downstream direction for supercritical flow. The designer must take care to insure against the possibility of unanticipated hydraulic jumps forming in the channel.

Because of field construction limitations, the designer should not use a Manning n roughness coefficient any lower than 0.013 for a well-troweled concrete finish. The freeboard should equal the velocity head plus 1.0 feet.

Grass-Lined Channels (Artificial)

Because of their similarity to natural channels, a well-designed grass-lined channel is considered to be the most desirable artificial channel. The channel storage, the lower velocities, and the sociological benefits obtainable create significant advantages over other channel types. The design must give full consideration to aesthetics, to sediment deposition, and to scour, as well as hydrualics.

Design Criteria. Design criteria must be established in the perliminary design stage and layout work. Any design which has parameters which vary significantly from the following criteria should be carefully reviewed for adequacy. Unless prior approval is received from the City Engineer, Bermuda Grass will be used for all grass lined channels.

- Design Velocity. For an irrigated or non-irrigated Bermuda Grass lining, the maximum velocity for the major storm design runoff of 8.0 feet per second should be used. This permits an economical cross section and yet keeps scour problems within reasonable limits. Without a satisfactory grass cover established, however, the annual flows will cause serious channel cutting and bank cutting at bends.

- Design Depths. The maximum design depth of flow is 5.0 feet, though 4.0 feet is preferable. Erosion is a function of velocity, depth, and time. Urban runoff peaks are generally short-lived, which makes velocity and depth key design parameters. For channels with design capacities greater than 4,000 cubic feet per second, greater depths can be considered.
- Design Slopes. Grass-lined channels, to function well, normally have slopes of from 0.2 to 0.6 percent. Where the natural topography is steeper than desirable, drops should be utilized.
- Curvature. The less sharp the curves, the better the channel functioning will be. In general, centerline curves should not have a radius of less than about twice the design flow top width, but not less than 100 feet.
- Design Discharge Freeboard Bridge deck bottoms and sanitary sewer often control the freeboard along the channel banks in urban areas. Where they do not control, the allowance for freeboard should be equal to the velocity head plus 1 foot. Where appropriate floodplain zoning is used localized overflow in certain areas may be desirable because of ponding benefits. In general, the freeboard may range from 1.0 to 2.0 feet. Except as may be specified by the City Engineer, all channels will be designed for a freeboard of 18-inches for the design storm.

Grass. The grass chosen needs to be sturdy and have a thick root structure to obviate unsightly weed growth, and to resist erosion. Native grasses will be found necessary if irrigation is not contemplated.

Newly constructed channels often need a protective cover immediately such as sod. Sometimes it is possible to seed the permanent grass. If so, a mulch cover will be applied to help hold moisture in the soil surface, improve infiltration, and help prevent erosion while the grass is getting started.

Temporary covers may be necessary for a variety of reasons. During the growing season, temporary grasses may be necessary to protect completed earthwork while waiting for other improvements to be constructed. During the non-growing season, temporary mulch covers will be necessary.

o Temporary Grass Cover

Small grains like oats, rye and wheat and sudans and sorghums are the most feasible temporary vegetation to control erosion for the Stillwater area. This practice is effective for areas where soil is left exposed for a period of 6 to 12 months. The time period may be shorter during periods of erosion rainfall.

1. Prior to seeding, needed erosion control practices such as diversions, grade stabilization structures, berms, dikes, etc. shall be installed.
2. Temporary vegetative practice is usually applied prior to the completion of final grading of the site.
3. If the area to be seeded has been recently loosened to the extent that an adequate seedbed exists, no additional treatment is required. However, if the area to be seeded is packed, crusted and hard, the top layer of soil shall be loosened by other suitable means.

4. Fertilizer shall be applied at a rate of 600 pounds per acre <u>or</u> 15 pounds per 1000 square foot using 10-20-10 or equivalent.

5. Soils known to be highly acidic shall be lime treated.

6. Seeding requirements shall be as specificed in the following:

TABLE V-3

SEEDING REQUIREMENTS FOR TEMPORARY COVER

PLANT	PER ACRE	PER 1000 SQ. FT.	PLANTING DATE	DEPTH OF SEEDING
Annual Ryegrass	40 lbs.	0.9 lbs	9/15 - 11/30	1/4 inch
Elbon Rye	2 bu.	3.0 lbs.	8/15 - 11/30	2 inches
Wheat	2 bu.	3.0 lbs.	8/15 - 11/30	2 inches
Oats	3 bu.	2.5 lbs	8/15 - 11/30	2 inches
Sorghum	60 bu.	1.4 lbs.	3/1 - 9/15	2 inches
Sudan Grass	40 lbs.	0.9 lbs.	4/1 - 9/15	2 inches

7. Seeds shall be drilled uniformly.

8. Seeding implements should be used at right angles to the general slope to minimize erosion.

9. After 2 to 3 months of planting, the seeded site shall be top dressed with 8 pounds per 1000 square foot or 30 pounds per acres of 33-0-0.

10. Areas that are not well covered shall be replanted.

11. The seeded area shall be watered when feasible and needed.

- <u>Temporary Mulch Cover</u>

  Temporary mulch covers shall meet the requirements of the Soil Conservation Service Standard Number 443.

  Native prairie, K.R. bluestem, Caucasian bluestem, or weeping lovegrass hay shall be used as mulch material. The hay mulch shall be of reasonably good quality. It shall be pliable and of sufficient length to permit anchoring without breaking. Rotten or molded hay that would deteriorate rapidly will not be acceptable. Hay having more than a trace of Johnson-grass or annual threeawn grass will not be accepted. The material must be free of field bindweed.

  Acceptable weight certificates for all hay mulch will be provided at the time of delivery. The hay shall be properly stacked to facilitate handling and checking, and covered to prevent damage during inclement weather prior to application. Weight certificates will be used as a cross-check on the area treated.

  1. The machine for spreading hay mulch will be provided with a blower discharge pipe to evenly distribute the hay mulch. The machine shall be equipped with mechanisms to prevent mulch from being applied in clumps or chunks without materially shortening or pulverizing the hay mulch.
  2. Equipment for anchoring the hay mulch shall be a disc-type implement such as the Imco Landscape Soil Erosion Mulch Tiller, which is needed to securely anchor the hay mulch. Where discs are used they shall be straight and about one-fourth (1/4) inch thick and spaced not more than eight (8) inches apart.
  3. Unless otherwise approved, an anchor truck or tractor will be required to hold anchoring equipment on slopes.
  4. The hay mulch shall be applied in a reasonably continuous unbroken cover of uniform thickness at the rate of one (1) pound of air-dry hay per square yard (4,840 pounds per acre). The mulch shall be secured by treading or cutting the hay into the soil. Care must be

exercised not to cover an excessive amount of hay. The operation shall be on the approximate contour. The hay shall be anchored a minimum of two inches in the soil in disc rows or rows of other treading devices. Sufficient mulch shall remain on the surface to prevent excessive runoff and soil loss. On small areas not suitable for use of equipment, spreading and anchoring may be done by hand.

5. All materials or items resulting from the mulching operation that would hamper growth of vegetation or maintenance operations shall be removed from the area.

o <u>Permanent Practices</u>

Bermuda grass shall be effectively used for permanent coverings in grass-lined channels. This grass shall be installed according to Soil Conservation Service Standard Number 443 for Bermuda Grass mulch sodding.

1. Mulch sod shall consist of dense, well rooted, and vigorous runners (stonons), and root divisions (rhizomes) of common or select common bermuda grass. The Contractor shall notify the Engineer of the source of bermuda grass mulch-sod and secure approval of the source before any mulch-sod is harvested and delivered to the job site.
2. The Engineer shall have the right to require the Contractor to submit a schedule of the time and place that bermudagrass mulch-sod will be harvested and shall have the right to reject any mulch-sod that does not appear to be satisfactory.
3. To provide a bond between the mulch-sod and the soil, the area on which sod-mulch is to be placed shall be treated or tilled to a depth of 4 inches by chiseling (on 12" centers) or other suitable methods as approved by the City Engineer.
4. The fertilizer shall be a 1-1-1 ratio in a pelleted form and uniformly mixed. The rate per acre of application shall not be less than 52 pounds of nitrogen, 52 pounds of phosphate (P_2O_5) and 52 pounds of potash (K_2O). Labeling or analysis records shall be in accordance with the Oklahoma Fertilizer Law. After ground preparation and prior to placement of mulch-sod the fertilizer shall be broadcast evenly and uniformly in the specified quantity over the area to be treated.

5. The mulch-sod shall be placed on moist soil. Where soil moisture cannot be readily detected by visual and tactile examination the area to be sodded shall be moistened to a depth of 4 inches before the mulch sod is placed.

   Unless otherwise approved, after mulch-sod placement (including compacting) is completed, watering shall be required. The watering shall be applied immediately after placement requirements are completed. Application will be in a manner and at such a rate that provides uniform distribution without runoff, waste and erosion. A single application of water will normally be 3 to 4 gallons per square yard.

   Water shall be reasonably clean and free from oil, toxic amounts of salt and other substances harmful to plant growth.

6. The mulch-sod shall be spread uniformly on the area to be treated to a thickness of 4 inches before compaction. Compaction shall be accomplished by a rolling type soil packer or other equivalent methods. The interval of time between spreading and compaction shall not exceed 4 hours. When finished the planting area shall be smooth, free from stones, woody roots, or other undesirable foreign matter that would hamper grass growth or maintenance operations.

Channel Cross Sections. The channel shape may be almost any type suitable to the location and to the environmental conditions. Often the shape can be chosen to suit open space and recreational needs to create additional sociological benefits.

However, limitations within which design must fall for the major storm design flow include:

o Side Slopes. The flatter the side slope, the better. A normal minimum is 4:1. Under special conditions, the slopes may be as steep as 3:1 which is also the practical limit for mowing equipment.

- Depth. The maximum depth should be limited to 4.0 feet, though 5.0 feet is acceptable where good maintenance can be expected and where durations of peak flows are short-lived.
- Bottom Width. The bottom width should be at least 6 to 8 times the depth of flow. Twenty to 30 times the depth is common.
- Trickle Channel. Trickle channels or underdrain pipes are required on all urban grassed channels. Trickle channels are preferred because of maintenance.

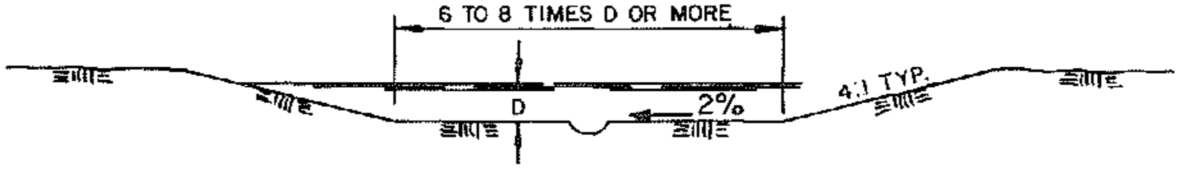
Typical cross sections suitable for grassed channels are given in Figure V-3.

Trickle Channels. The low flows, and sometimes base flows, from urban areas must be given specific attention. Waterways which are normally dry prior to urbanization will often have a continuous base flow after urbanization because of lawn irrigation return flow, both overland and from groundwater inflow. Continuous flow over grass will destroy a grass stand.

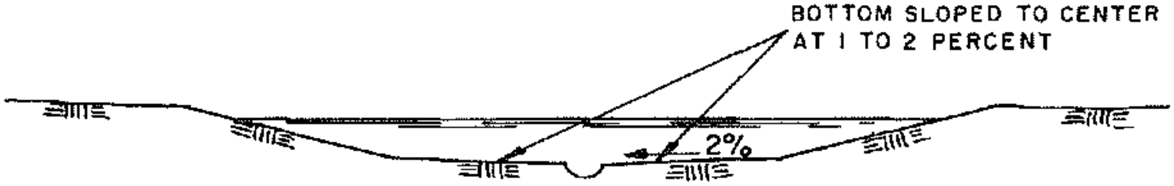
Low flows must be carried in a trickle channel, or in an underground conduit. A trickle facility capacity should be approximately 0.5 to 1.0 percent of the major design flow, the lower value being more applicable to the underdrain pipe.

A trickle channel is subject to erosion and must therefore be amply protected with appropriate erosion control devices when design velocities exceed 5.0 feet per second.

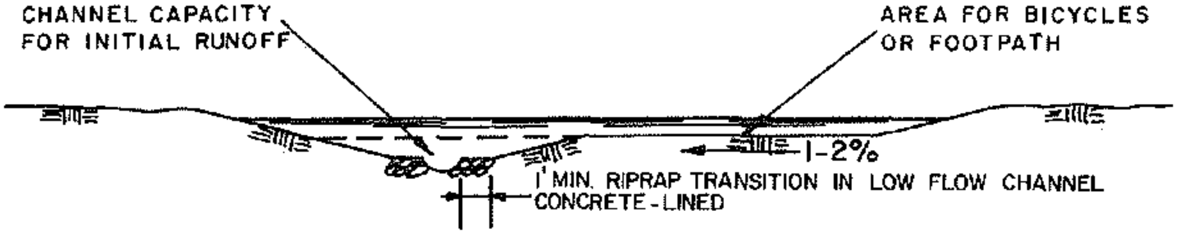
Care must be taken to insure that low flows enter the trickle channel without the attendant problem of the flow parelleling the trickle channel or bypassing the inlets.



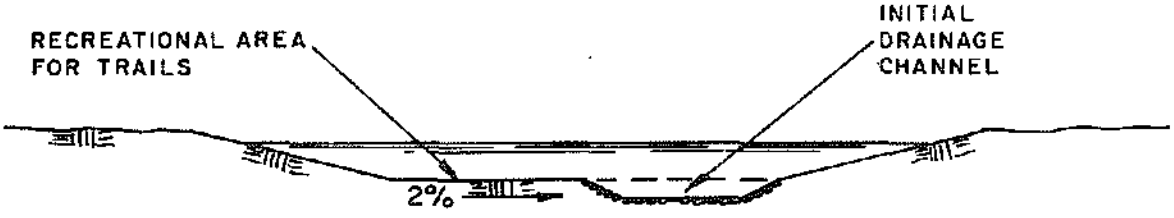
FLAT BOTTOM WITH TRICKLE CHANNEL



OVAL OR SLOPED BOTTOM WITH TRICKLE CHANNEL



INITIAL DRAINAGE CHANNEL WITH OVERFLOW AREA FOR MAJOR DRAINAGE RUNOFF



RIPRAPPED INITIAL DRAINAGE CHANNEL WITH OVERFLOW FOR MAJOR DRAINAGE RUNOFF

FIGURE V-3 TYPICAL GRASSED CHANNELS

Erosion Control. Grassed channels are erodable to some degree. Practice has shown that to design a grassed channel completely protected from erosion is uneconomical and costly. It is far better to provide reasonable erosion-free design with plans laid to take additional erosion control measures and corrective steps after the first year of operation. However, the use of erosion control cutoff walls at regular intervals in a grassed channel is desirable. Such cut-off walls will safe-guard a channel from serious erosion in case of a large runoff prior to the grass developing a good root system. Such cut-off walls are also useful in containing the trickle channel.

Erosion control cut-off walls are usually of reinforced concrete, approximately 8 inches thick and from 18 to 21 inches deep, extending across the entire bottom of the channel. They can be shaped to fit a slightly sloped bottom to help direct water to the trickle channel or to an inlet.

Often a concrete encased sewer passing under the channel bottom may be utilized in such a way as to serve the function of a cut-off wall.

Under bridges grass frequently will not grow, and therefore, the erosion tendency is large. A cut-off wall at the downstream edge of a bridge is good practice, or the designer might choose to soil-cement the entire bottom width under the bridge deck.

At bends in the channel, special erosion control measures may be taken; however, once a good growth of grass is established and if the design velocities, depths, and curvatures are adhered to, erosion at bends will normally not be a problem.

In maintaining the appropriate channel slope, the designer may find it necessary to use fequent drops. Erosion tends to occur at the edges and immediately downstream of a drop even though it may be only 6 to 18 inches high. Drops in excess of 3.0 feet should be avoided. Proper use of grouted riprap and/or timbers are necessary.

Grass located adjacent to concrete-lined flow channels may also scour when velocities exceed 5.0 feet per second. A band of riprap, grouted riprap or timber adjacent to the concrete should be used in these instances to prevent erosion.

EUROPEAN TYPE CHANNELS

This type of channel refers to artificial channels with grassed bottoms and concrete sides. The sides may be cast-in-place or precast and may have several different types of texture. The criteria listed previously for grass-lined channels shall apply.

Earth Channels Without Grass Cover

Earth channels of an artificial character, that is, either constructed channels or heavily modified natural channels, shall not be used for drainage because of the potential erosion and damage to those downstream.

NATURAL CHANNELS

Natural waterways are often in the form of steep banked gulches which have erodible banks and bottoms. On the other hand, many natural waterways that exist in urbanized and to-be-urbanized areas which have mild slopes, are reasonably stabilized, and are not obviously in a state of degradation. However, for either type of channel, if it is to be used for carrying storm runoff from an urbanized area, it can be assumed initially that the changed runoff regime will result in new and highly active erosional tendencies. Careful hydraulic analysis must be made of natural channels to counteract these new tendencies. In some cases, slight modification of the channel will be required to create a somewhat better stabilized condition for the channel.

The investigations necessary to insure that the natural channel will be adequate are different for every waterway; however, the designer will generally find it necessary to prepare cross sections of the channel for the major design runoff, to investigate the bed and bank material as to the particle size classification and to generally study the stability of the channel under future conditions of flow. It is called to the designer's attention that supercritical flow usually does not exist in natural channels and frequent checks should be made during the course of the backwater computations to insure that the computations do not reflect supercritical flow.

Because of the decided advantages which are available to a community from the sociological point of view by utilizing natural waterways for urban storm drainage purposes, the designer should most certainly consult with experts in related fields as to the methods of development. Nowhere in urban hydrology is it more important to convene an environmental design team to develop the best means for using a natural waterway. Very often it will be concluded that park and greenbelt areas should be incorporated into the channel works. In these cases, the usual rules of freeboard depth, curvature, and other rules applicable to artificial channels do not apply. For instance, there are significant advantages which may accrue if the designer 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 areas during the major runoff peak.

The entire hydrological approach to converting a natural waterway which has historically transported water from rural lands to an urban major drainage channel is so complex that applicable design criteria cannot be presented complete in this Manual. It will suffice here to state that the planning for use of such channels must be undertaken with the full benefit of engineers with adequate experience in open channel flow, together with experts in related fields.

The usual design criteria for artificial open channels do not apply to natural channels, but such criteria can be used to advantage in gaging the adequacy of a natural channel for future changes in runoff regime.

Utilization of natural channels requires that primary attention be given to erosive tendencies and carrying capacity adequacy. The floodplain of the waterway must be defined so that adequate zoning can take place to protect the waterway from encroachment to maintain its capacity and storage potential.

General criteria for analyzing the effectiveness of natural channels are:

- Channel and overbank capacity adequate for 100-year runoff.
- Velocities in natural channels do not exceed critical velocity for a particular section which is only rarely more than 10 fps.
- Define water surface limits so that floodplain can be zoned.
- Filling of the flood fringe reduces valuable storage capacity and tends to increase downstream runoff peaks. Filling should be discouraged in the urban waterways where hydrographs tend to rise and fall sharply. The specific policies of the City in regard to floodplain fill will be used.
- Use roughness factors (n) which are representative of unmaintained channel conditions.
- Construct drops or check dams to control water surface profile slope, particularly for the initial storm runoff.
- Prepare plans and profiles of floodplain. Make appropriate allowances for future bridges which will raise the water surface profile and cause the floodplain to be extended.
- Use a freeboard of a minimum of 18-inches.

Water Surface Profiles. Water surface profile computation has its greatest application to natural channels; however, in final design, all open channels and box culverts should have the design water surface profile determined.

The most frequently used tool is the HEC-2 backwater program developed by the U.S. Army Corps of Engineers (USACE). In lieu of this program, other standard programs may be used. While it is possible to compute water surface profiles by hand (normally, the Standard Step Procedure), it is not recommended that profiles be computed in this manner except for short reaches.

Although it is not necessary in the HEC-2 program, the general procedure is to start at a known water surface elevation and proceed upstream for subcritical flow, and downstream for supercritical flow. The channel cross-sections must be at no more than 500 feet and more often when channel properties change or if more accurate results are desired. The channel cross sections must be divided into sections with like properties and the appropriate "n" factor applied.

The primary difficulty with using the HEC-2 program is its applicability to structures, particularly bridges and culverts. The designer must be sure to check the results for reasonableness for any water surface program used. It may be necessary to compute structure hydrualics by hand.

For a more detailed description of the hand computation methods, see Open Channel Hydrualics (21) by Ven Te Chow. For computer programs, the designer should utilize the appropriate user's manual.

CLOSED CONDUITS

The use of box culverts and corrugated steel plate arch pipe for underground outfall conduits of larger capacity can have cost advantages over other types of diameter pipe. Furthermore, because box culverts are normally poured in place, advantages accrue in being able to incorporate conflicting utilities into the floor and roof of the structure. Box culverts as used in this Chapter refer to long box-like conduits similar to long pipes.

Major disadvantages of closed conduits for long distance conveyance are:

- The fact that the capacity drops significantly when the water surface reaches the roof. The drop is 20 percent for a square cross section, and more for a rectangular cross section where the width is greater than the height.
- Normal structural design, because of economics, usually does not permit any significant interior pressures, meaning that if the conduit reached full and the capacity dropped, there could be a failure due to interior pressure caused by a choking of the capacity.

It is apparent that the use of long closed conduits for outfall conduit purposes requires a high standard of planning and design involving complex hydraulic considerations.

Hydraulic Design

Box culverts are often considered to be covered free-flow conduit. They ar open channels with a cover.

Structural requirements and efficiency for sustaining external loads, rathe than hydraulic efficiency, usually control the shape of the box culvert.

Computational procedures for flow in closed conduits are essentailly th same as for canals and lined channels, except that special consideration i needed in regard to rapidly increasing flow resistance when the condui reaches full.

An obstruction, or even a confluence with another conduit, may cause th flow in a near full box culvert to strike the roof and cause a choking dow of the capacity. The capacity reduction may then cause the entire upstrea reach of the conduit to flow full with a resulting surge and pressure hea increase of sufficient magnitude to cause a structural failure. Thoroug design is required to overcome this inherent potential problem. Structura design must account for internal pressure if pressure will exist.

In urban drainage use, a closed conduit should have a nearly straight alignment, should not decrease in size in a downstream direction, and the slop should not decrease in a downstream direction. It is desirable to have slope which increases in a downstream direction as an added safety facto against it flowing full. This is particularly important for supercritica velocities. Because of sediment load normally associated with urban runoff the bottom of a box culvert should be lined with steel plates when the average velocity exceeds 20 fps.

Roughness coefficients should be chosen carefully because of the effect o the proper operation of the conduit (See Table V-4). Quality control is important during construction, with attention paid to grinding of projection and keeping good wall alignment.

For the flatter conduits, the sediment deposition problem must be considered, so as not to permit an inadvertent loss of capacity.

Bedding and covering on conduits are structural considerations, and specifications for bedding and covering are closely allied to the loads and forces used in the structural design.

TABLE V-4

ROUGHNESS COEFFICIENTS FOR LARGE CONDUITS

Concrete	Manning's n
Precast concrete pipe, good joint alignment	0.012
Precast concrete pipe, ordinary joint alignment	0.013
Poured in place, steel forms, projections 1/8" or less	0.013
Poured in place, smooth wood forms, projections 1/8" or less	0.013
Poured in place, ordinary work with steel forms	0.014
Poured in place, ordinary work with wood forms	0.015
Steel	
Structural plate corrugated , 2"x6" corrugations, 5' x 20' diameter	$\frac{0.0377}{D^{0.078}}$ Eq.V-3
Corrugated pipe 1"x3" corrugations, 3'x8' diameter	$\frac{0.0306}{D^{0.075}}$ Eq.V-4

Where D = Pipe Diameter

Entrance A large closed conduit is costly per square foot of cross sectional area. For this and other reasons, the hydraulic characteristics at the entrance are particularly important. A conduit which cannot flow at the design discharge because of an inadequate inlet represents wasted investment.

The entrances take on a special degree of importance for box culverts, however, because the flow must be limited to an extent to insure against overcharging of the conduit which otherwise might cause a failure as the water surface reaches the roof. Special maximum flow limiting entrances are often used with box culverts. These special entrances should reject flow over the design discharge so that if a runoff larger than the design flow occurs, the excess water will flow via other routes, often overland, which must be planned. A combined weir-orifice design is useful for this purpose. Model tests are needed for dependable design.

A second function of the entrance should be to accelerate the flow to the design velocity of the conduit, usually to meet the velocity requirements for normal depth of flow in the upstream reach of the conduit.

Ports for air are needed at the entrance to obviate both positive and negative pressures, and to permit released entrained air to readily escape from the conduit.

Internal Pressure. The allowable internal pressure in a box culvert is limited by structural design. If structural design has not been based on internal pressure, internal pressures are often limited to no more than 2 to 5 feet of head before structural failure will commence.

It is evident that surges or conduit capacity choking cannot normally be tolerated.

Curves and Bends. The analysis of curves in box culverts is critical from the point of view of insuring against the water reaching the roof of the conduit because of hydraulic losses. Superelevation of the water surface must also be studied and allowances made for a changing hydraulic radius, particularly in high velocity flow.

Dynamic loads created by the curves must be analyzed to insure structural integrity for the maximum flows.

Transitions. Transitions provide complex hydraulic problems and require specialized analyses.

Transitions, either contracting or expanding, are important with most larger outfall conduits because of usually high velocity flow. The development of shock waves which continue on downstream can create significant problems in regard to proper conduit functioning. The best way to study transitions is through model tests. Analytical procedures can only give approximate results. Poor transitions can cause upstream problems with both subcritical and supercritical flow, as well as unnecessary flooding.

Air Entrainment

In box culverts, as well as in pipes and open channels, flowing water will entrain air at higher velocities. Air usually becomes entrained at about velocities of 20 fps and higher. Besides velocity, however, other factors such as entrance condition, channel roughness, distance travelled, channel cross section, volume of discharge, etc., all have some bearing on air entrainment.

Entrained air causes a swell in the volume of water, and an increase in depth. This entrained air, resulting in greater depth than anticipated, could cause conduit flow to the full height of the roof with resulting loss of capacity. Hydraulic design must account for entrained air. Volume swell can be as high as 20 percent.

Major Inlets. Major inlets to a box culvert at conduit junctions or from large storm inlets should receive a rigorous hydraulic analysis to insure against mainstream conduit flow striking the top of the box culvert due to momentum changes in the main flow body as a result of the introduction of the additional flow. Model tests may be necessary.

Sedimentation. The conduit must be designed to eliminate sediment depositional problems during storm runoffs which have a frequency of occurrence of about twice a year.

Appurtenances

The appurtenances to a long box culvert are dictated by the individual needs of the particular project. All appurtenances are parts of the system and most have some effect upon the overall operation of the system. The designer must consider all effects.

Energy Dissipators. Long conduits usually have high exit velocities which must be slowed to avoid downstream problems and damage. Energy dissipators are nearly always required and are discussed later in this Chapter.

Access Manholes. A long box culvert should be easy to inspect, and therfore, access manholes are desirable at various locations. If a box culvert is situated under a curb, the access manholes may be combined with the storm inlets.

Access manholes and storm inlets are useful for permitting air to flow in and out of a box culvert as filling and emptying of the conduit occurs. They might also be considered as safety water ejection ports, should the conduit ever inadvertently flow full and cause a pile-up of water upstream. The availability of such ejection ports could very well save a box culvert from serious structural damage.

Vehicle Access Points. A large box culvert with a special entrance and an energy dissipator at the exit usually needs an access hole for vehicle use in case of major repair work being necessary. A vehicle access point might be a large grated opening just downstream from the entrance. This grated opening can also serve as an effective air breather for the conduit. Vehicles may be lowered into the conduit by a crane or A-frame.

Riprap

The large-scale use of riprap in Stillwater is unlikely, as the sources are few. Maintenance of any riprap is costly; however, when necessary, and approved by the City Engineer, the use of riprap will be authorized. The City has a preference for gabion-type riprap.

Riprap-lined channels are used infrequently as a lining for artificial channels. Cost is often prohibitive except in areas where rock is plentiful and concrete is not readily at hand. The most frequent use of riprap is for localized erosion protection.

There are different ways to prevent channel bottom and bank damage upstream and downstream from hydraulic structures, at bends, at bridges, and in other channel areas where erosive tendencies exist, but the primary method is by the use of riprap. One problem which the design often neglects, however, is the "erosive" effect of neighborhood children in urban areas on the riprap itself. It has been found by many designers that the riprap is almost completely lost within the first month or two of project completion. It is usually thrown into the water by the children, purely for the sake of causing splashes. This non-hydraulic problem as to the use of riprap should keep the designer from choosing ordinary riprap in urban areas except for unusal cases, and then the material should be large.

In lieu of ordinary riprap, the designer may consider grouted riprap or riprap enclosed in wire baskets, which is usually called gabions.

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 large pieces, the gradation of the mass, the thickness, the type of bedding under the riprap, and the slope of the riprap layer. Hydraulic forces affecting the riprap include the velocity, current direction, eddy action, and waves.

Experience has shown that the usual cause of riprap failure is undersized individual rocks in the maximum size range. Riprap should be lain on a gravel bedding to prevent piping failure, another common cause for riprap failure.

It has been established that a well-graded riprap layer containing about 40 percent of the rock pieces smaller than the required size is as stable or more stable than individual rocks of the required size. This is probably due to the interlocking benefits of graded riprap.

Design. Field experience has shown that a riprap layer, to work most effectively, should be about one and one-half times or more as thick as the

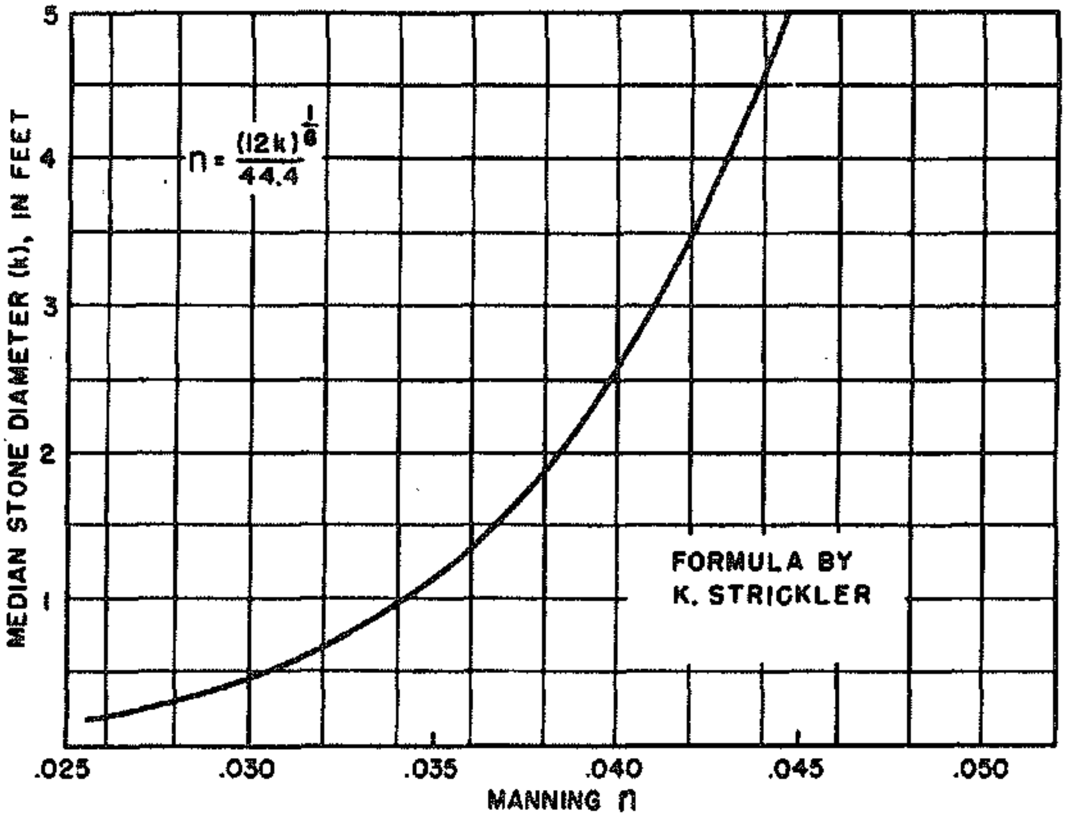


FIGURE V-4 RELATION OF MANNING n TO SIZE OF STONE
(from Bureau of Public Roads)

dimension of the large rocks and that the riprap should be placed over a gravel layer.

Figure V-4 is used for determination of n values in riprapped channel sections. Figure V-5 and Figure V-6 are used in sizing of riprap, utilizing a trial and error process:

1. Select a trial value of n from Figure V-4 corresponding to the estimated size of stone to be used. Figure V-4 applies to a stone lining on both sides and bottom of channel; when only the channel sides are lined, the n value might require weighting when the bottom width exceeds 4 times the depth of flow. The value of the Manning n also varies with the ratio of stone size to the hydraulic radius. The effect of this variation is generally minor in the determination of stone size.
2. Compute a size of channel, using the Manning equation, that will carry the design discharge.
3. Divide the assumed stone diameter (k) in feet, by the computed depth of flow in the channel (d) to obtain the k/d ratio.
4. Enter Figure V-5 with this ratio to obtain the V_s/V ratio.
5. Multiply the computed mean value of V by the V_s/V ratio from Figure V-5 to obtain the value of V_s.
6. Enter Figure V-6 with the value of the V_s and read the stone size in feet at the intersection of the V_s and the curve corresponding to the channel side slopes.
7. If the estimated stone size (Step 1) is small or much greater than the required size (Step 6), select a different size stone and repeat Steps 1 through 6, until the estimated size agrees with the required size.

A filter blanket is often needed beneath the stone lining to prevent the bank material from passing through the voids in the stone blanket and escaping. The loss of bank material leaves cavities behind the stone blanket and a failure of the blanket might result. In general, a filter ratio of 5 or less between successive layers will result in a stable condition. The filter ratio is defined as the ratio of the 15 percent particle size (D15) of the coarser layer to the 85 percent particle size (D85) of the finer layer. An additional requirement for stability is that the ratio of the 15 percent

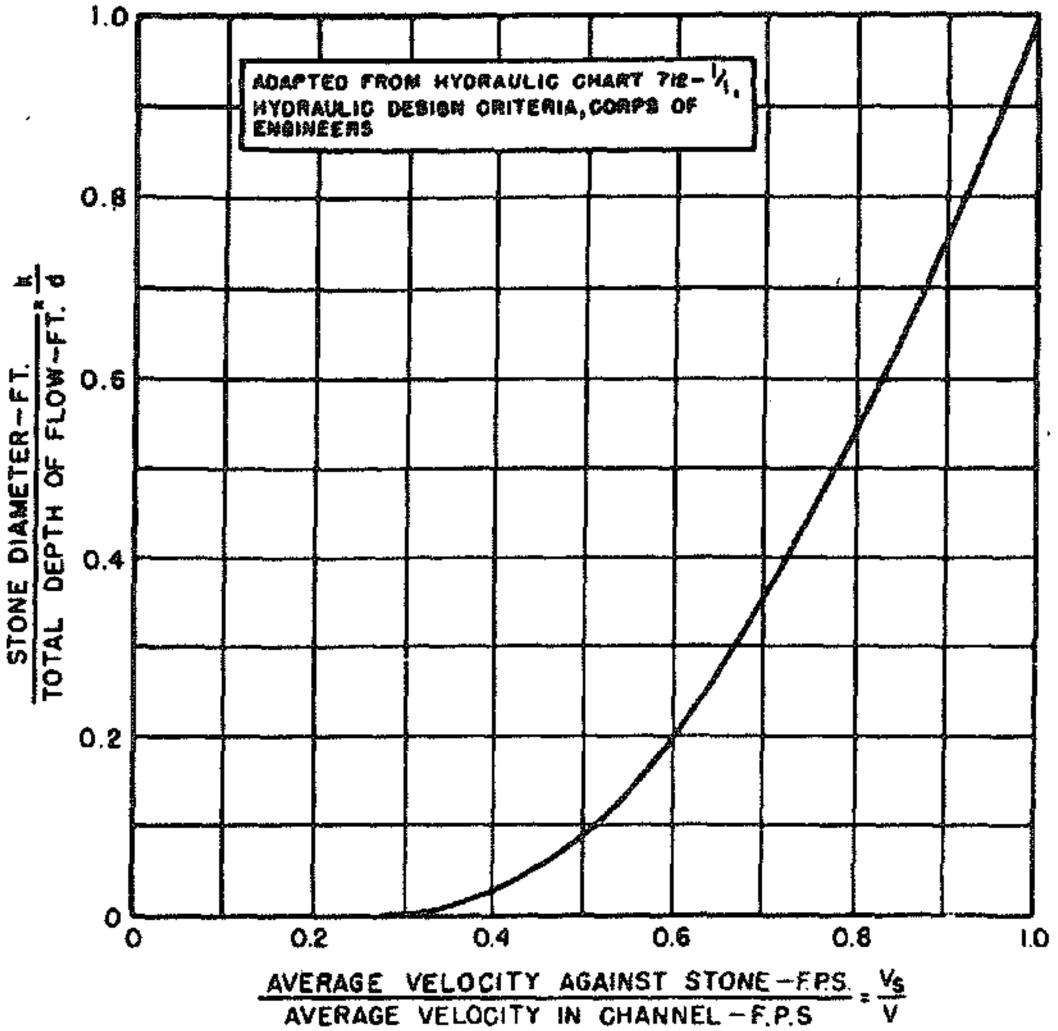


FIGURE V-5 AVERAGE VELOCITY AGAINST STONE ON CHANNEL BOTTOM
(from Bureau of Public Roads)

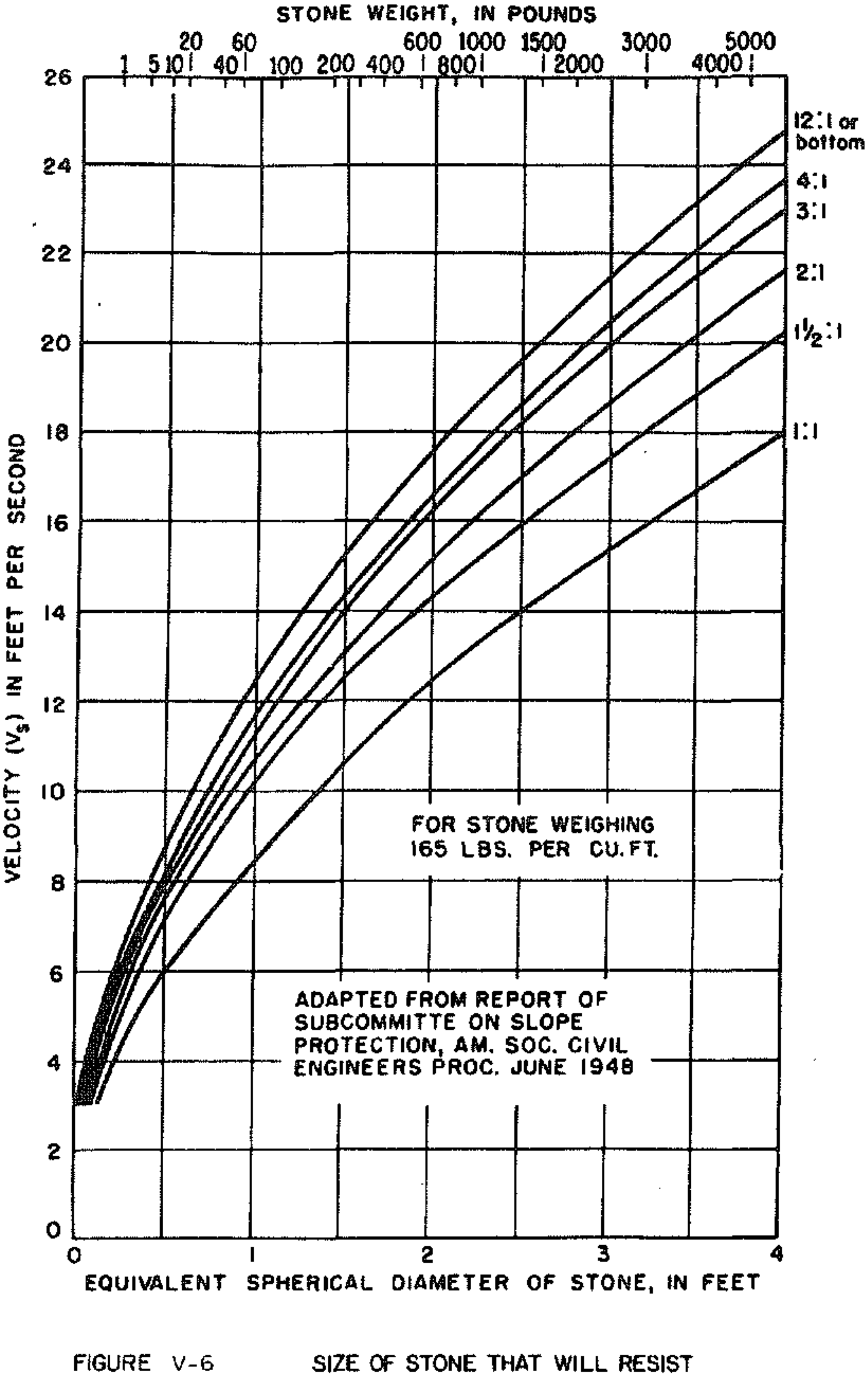


FIGURE V-6 SIZE OF STONE THAT WILL RESIST DISPLACEMENT FOR VARIOUS VELOCITIES AND SIDE SLOPES
(from Bureau of Public Roads)

particle size of the coarse material to the 15 percent particle size of the fine material should exceed 5 and be less than 40. The requirements can be stated as follows:

$$\frac{D_{15} \text{ (of courser layer)}}{D_{85} \text{ (of finer layer}} < 5 < \frac{D_{15} \text{ (of courser layer)}}{D_{15} \text{ (of courser layer)}} < 40 \qquad \text{Eq. V-5}$$

If a single layer of filter material will not satisfy the filter requirements, one or more additional layers of filter material should be used. The filter requirement applies between the bank material and the filter blanket, between successive layers of filter blanket material, if more than one layer is used, and between the filter blanket and the stone lining. In addition to the filter requirements, the grain size curves for the various layers should be approximately parallel to minimize the infiltration of the fine material into the coarse material. The filter material should contain not more than 5 percent of material passing the No. 200 sieve.

The thickness of the filter blanket ranges from 6 to 12 inches for a single layer, or from 4 to 8 inches for individual layers of a multiple layer blanket. The thicker layer is used where the gradation curves of adjacent layers are not approximately parallel.

Grouted Riprap. Grouted riprap is particularly useful in Stillwater in that it ties the individual rock pieces together, providing a somewhat monolithic mass which precludes unwanted plant growth, and it also permits the use of smaller sized rock. Care should be taken with grouting of riprap in urban areas, however, to insure a reasonably acceptable appearance. The grout may be a weak mix.

The grout should penetrate into the riprap mass, it being important to not just create a veneer with the top few inches of the riprap. It is generally more effective both hydraulically and from the appearance standpoint to have a rough surface with portions of the rock particles projecting out from the grout surface. After completing the placement of the grout it is usually desirable to clean off the projecting rocks with a wet broom. Cracking of the grouted riprap will occur with settlement and frost; however, this does not affect the appearance nor its function.

Gabions. Gabions, in addition to being more resistant to vandalism, provide a dependable erosion-resistant bank or bottom and permit the use of smaller sized rocks because the wire basket tends to make the entire basket act monolithically. Besides providing protection against scour, gabions are very useful in urban drainage work as drops with either vertical or stepped faces. The hydraulic roughness of gabions is usually about 0.035; however, for use in drops, larger stones may be used at the surface to increase the roughness to dissipate additional hydrualic energy.

In designing gabion erosion control protection or hydraulic structures, normal good hydrualic practices should be followed. Side slopes of 1:1 are satisfactory for channel banks. In regard to drops, the gabions should be keyed into both banks to prevent flanking, and downstream cutting should be considered. Gabion baskets should be laid on a gravel filter.

Final Design

Before proceeding to final design, it is well to remember that the preliminary planning and conceptual design of an outfall conduit or channel are the most important portion of the engineer's job, and have the greatest effect on the performance and cost of the works. Imagination and general hydraulic experience are the most important tools of the engineer in the preliminary planning stage.

The character of an outfall often changes from reach to reach to account for neighborhood needs and environmental requirements. A major conduit or channel has an impact upon an urban area. Much depends upon its hydraulic and environmental functions which need to be addresssed both in preliminary and final design.

A water surface profile must be computed for all channels and conduits in preliminary and final design (after completion of the design) and clearly shown on a copy of the final drawings. Computation of the water surface profile should utilize standard backwater methods, taking into consideration all losses due to changes in velocity, drops, bridge openings, and other obstructions. Other than supercritical concrete lined facilities, computations begin at a known point and extend in a upstream direction for subcritical flow. It is for this reason that the channel should be designed from a downstream direction to an upstream direction. It is necessary to show the energy gradient on all preliminary drawings to help insure against errors. Whether or not the energy gradient line is shown on the final drawings is optional.

It must be remembered by the designer that open channel flow in urban drainage is usually non-uniform because of bridge openings, curves, and structures. This necessitates the use of backwater computations for all final channel design work.

HYDRAULIC STRUCTURES

Hydraulic structures are used in storm runoff drainage works to control water. Flowing water does not readily change direction, accelerate, or slow down without help, and water will flow faster than it should if a thalweg is too steep, causing uncontrolled erosion.

Hydraulic structures increase the cost of drainage facilities, and their use should be limited by careful and thorough hydraulic engineering practices to those locations and functions justified by prudent planning.

On the other hand, use of hydraulic structures can reduce initial and future maintenance costs by changing the character of the flow to fit the project needs, and by reducing the size and cost of related facilities.

Hydrualic structures include energy dissipators, channel drops or checks, bridges, acceleration chutes, bends, baffle chutes, and many other specific drainage works. Their shape, size, and other features vary widely from job to job, depending upon the function to be served. Hydraulic design procedures, and sometimes model testing must govern the final design of all structures.

Types of Structures

The following descriptions are general and describe the various structures and their applications.

Energy Dissipators. Energy dissipators are often necessary at the end of outfall sewers or channels. Stilling basins, a type of energy dissipator, are useful at locations where the designer wants to convert supercritical flow to subcritical flow to permit placid water in a pool area downstream from a high velocity channel.

Drops. The use of drops is a convenient and economical way to reduce the effective slope of a natural or artificial channel. In general, the vertical height of the drop should be kept minimal so as to reduce erosion and turbulence problems. With natural channels, the use of check dams is often preferable. Similar principles are involved as with drops.

Bridges. The use of bridges provides for the crossing of the channel with a roadway, as against a culvert, which permits a channel to cross under a roadway. Bridges should not unduly restrict or adversely affect the flow character of the channel. Adequate hydraulic opening area should be allowed (1).

Acceleration Chutes. Acceleration chutes can be used to maximize the use of limited downstream right-of-way, and to reduce downstream channel and pipe costs. Chutes should, of course, be used only where good environmental design concepts permit the use of high velocity flow. Generally, in urban drainage design, open channels should have slow flow.

Bends. Hydrualic structures at bends are seldom needed; however, on supercritical flow channels, a bend may be required occasionally. In these cases, they should be chosen only after all other alternatives have been tried. The structure should be used to insure that the flowing water remains in the channel, rather than having the water flowing uncontrolled outside the channel.

ENERGY DISSIPATORS

Energy dissipators consist of three important features -- the approach channel, the dissipator, and the tailwater channel. Each one depends upon the other for proper functioning. The reader is referred to excellent publications by the U.S. Bureau of Reclamation as given in the Reference at the end of this section.

Care must be exercised in the design of energy dissipators to avoid overdesign and to insure performance over a wide range of discharges. The most common energy dissipators were designed for sustained operation over a narrow range of discharges. The design runoff event for energy dissipators can be considered a theoretical abstraction in that the exact design discharge as a peak flow will probably never exactly occur during the life of the structure. When it does occur, usually due to runoff events larger than the design discharge, the design rate of runoff is experienced for only a short period of time.

On the other hand, energy dissipators cannot fail during lesser or greater runoff events, even though some damage is permissible.

A second factor is the exit velocities are seldom high enough to move beyond the intermediate Froude Number range (Froude Nos. > 2.5) and the conventional stilling basins are frequently quite long.

To shorten the energy dissipator and to improve performance of a range of discharges, it is recommended that consideration be given to forcing submergence of the exit structure.

Approach Channel

To improve the energy dissipatin characterisitcs, flow in the approach channel is supercritical with velocities ranging from 10 fps to 30 or 40 fps.

Design must insure against a premature hydraulic jump in the channel and the flow must remain at the supercritical stage throughout the channel. The flow might be uniform or decelerating, followed by accelerating flow in the steep drop leading to the downstream level. Flow at any point along the channel will depend upon the specific energy ($d + h_v$), available at that point. The velocities and/or depths of flow along the channel can be fixed by selecting the grade and the cross-sectional dimensions of the channel.

The velocities and depths of free surface flow in a channel, whether an open channel, a conduit, or a tunnel, conform to the principle of conservation of energy as expressed by Bernoulli, i.e., the absolute energy of flow at any cross section is equal to the absolute energy at a downstream section plus intervening losses of energy. As applied to Figure V-7, this relationship can be expressed as follows:

$$\Delta Z + d_1 + h_{v1} = d_2 + h_{v2} + \Delta h_L \qquad \text{Eq. V-6}$$

The coefficient of roughness, n, will depend on the nature of the channel surface. For conservative design, the frictional loss should be maximized when evaluating depths of flow and minimized when evaluating the energy content of the flow. For determining depths of flow in a concrete-lined channel, a value of n of about 0.016 should be assumed to account for air swell, wave action, etc. For determining specific energies of flow needed for designing the dissipating device, a value of n of about 0.010 should be assumed.

Open Channels. Sharp convex and concave vertical curves should be avoided to prevent unsatisfactory flows in the channel. Convex curves should be flat enough to maintain positive pressures and thus avoid the tendency for separation of the flow from the floor. Concave curves should have a

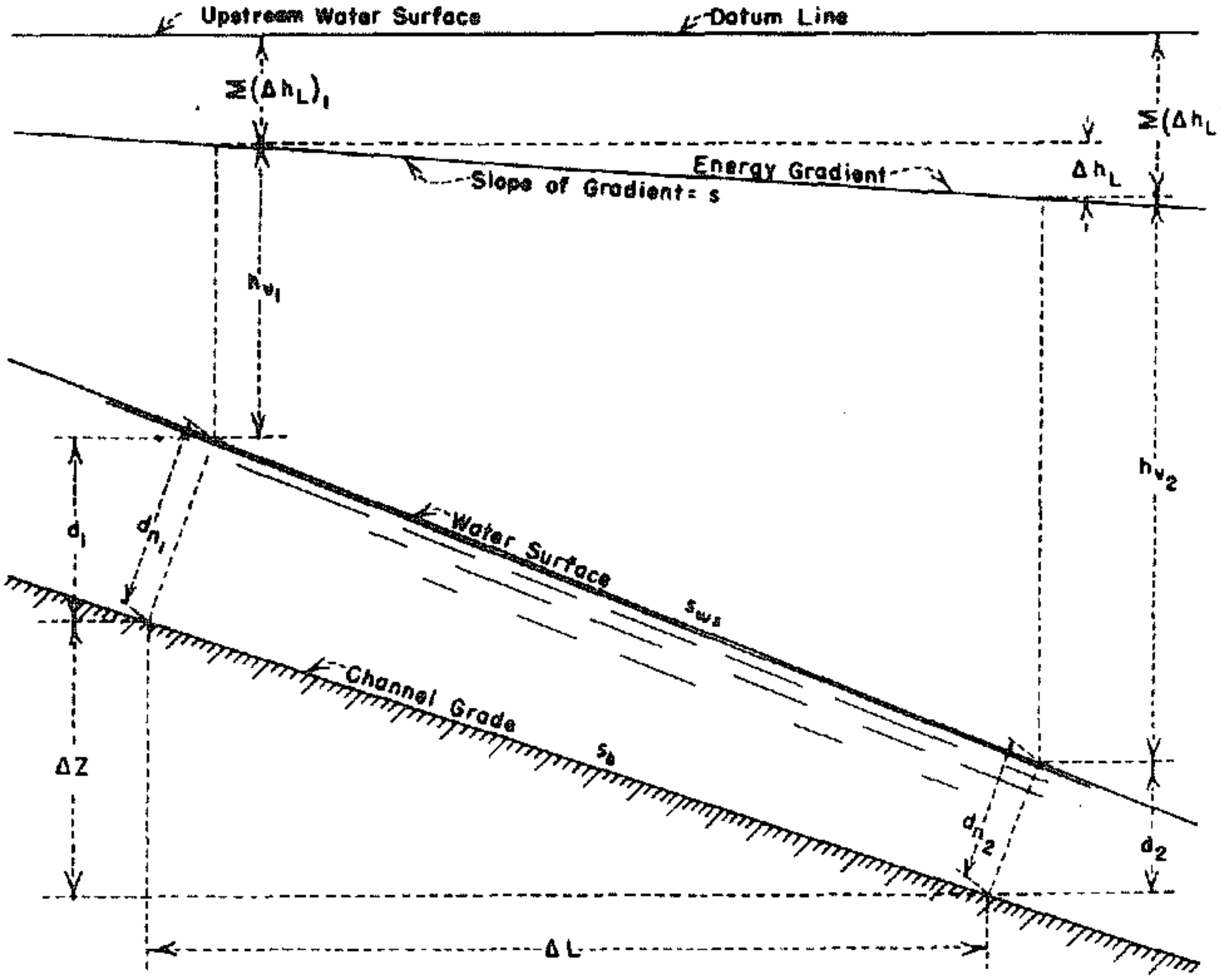


FIGURE V- 7
FLOW IN OPEN CHANNELS

sufficiently long radius of curvature to minimize the dynamic forces on the floor brought about by the centrifugal force which results from a change in the direction of flow (5).

The best hydraulic performance in a discharge channel is obtained when the confining sidewalls are parallel and the distribution of flow across the channel is maintained uniform. However, economy may dictate a channel section narrower or wider than either the crest or the terminal structure, thus requiring converging or diverging transitions to fit the various components together. Sidewall convergence must be made gradual to avoid cross waves, "ride ups" on the walls, and uneven distribution of flow across the channel. Similarly, the rate of divergence of the sidewalls must be limited or else the flow will not spread to occupy the entire width of the channel uniformly, which will result in undesirable flow conditions at the terminal structure.

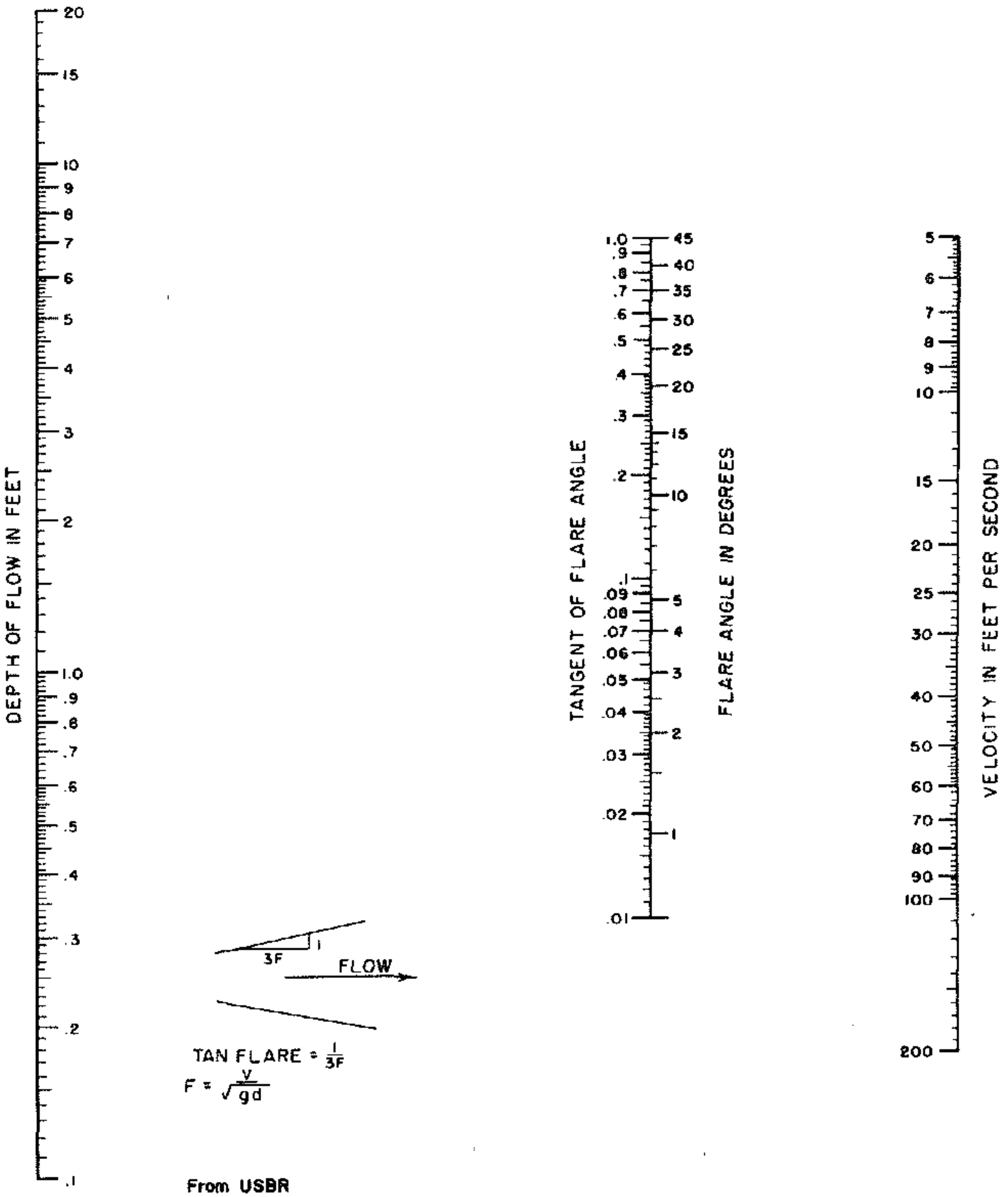
The inertial and gravitational forces of streamlined kinetic flow in a channel can be expressed by the Froude number:

$$F_r = \frac{v}{\sqrt{gd}}$$

Variations from streamlined flow due to outside interferences which cause an expansion or a contraction of the flow also can be related to this parameter. Experiments have shown that an angular variation of the flow boundaries not exceeding that produced by the equation,

$$\tan\alpha = \frac{1}{3F_r} \qquad \text{Eq. V-8}$$

will provide an acceptable transition for either a contracting or an expending channel. In this equation F_r is the Froude number defined above and α is the angular variation of the sidewall with respect to the channel centerline; v and d are the averages of the velocities and depths at the beginning and at the end of the transition. Figure V-8 is a nomograph from



From USBR

FIGURE V-8

FLARE ANGLES FOR DIVERGENT FLOW (2)

which the tangent of the flare angle or the flare angle in degrees may be obtained for known values of depth and velocity of flow.

Channel Freeboard. In a channel conducting flow at supercritical state, the surface roughness, wave action, air bulking and splash and spray are related to the velocity and energy content of the flow. The energy per foot of width, qh_v, expressed in terms of v and d is (13):

$$qh_v = \frac{{}^3v\ d}{2g} \qquad \text{Eq. V-9}$$

Therefore, the relationship of velocity and depth to the flow energy also can be expressed in terms of v and $d^{1/3}$. An empirical expression based on this relationship which gives a reasonable indication of desirable freeboard values is as follows (13):

$$\text{Freeboard (in feet)} = 2.0 + 0.025\frac{v}{3}\sqrt[3]{d} \qquad \text{Eq. V-10}$$

Stilling Basins The type of stilling basin chosen should be based upon hydraulic requirements, available space, and costs. The hydraulic jump which occurs in a stilling basin has distinctive characteristics and assumes a definite form, depending on the energy of flow which must be dissipated in relation to the depth of the flow. A comprehensive series of tests has been performed by the Bureau of Reclamation for determining the properties of the hydraulic jump. The jump form and the flow characteristics can be related to the kinetic flow factor (13),

$$\frac{v^2}{gd} \qquad \text{Eq. V-11}$$

of the discharge entering the basin; to the critical depth of flow, d_c; or to the Froude number,

$$F_r = \frac{v}{\sqrt{gd}} \qquad \text{Eq. V-12}$$

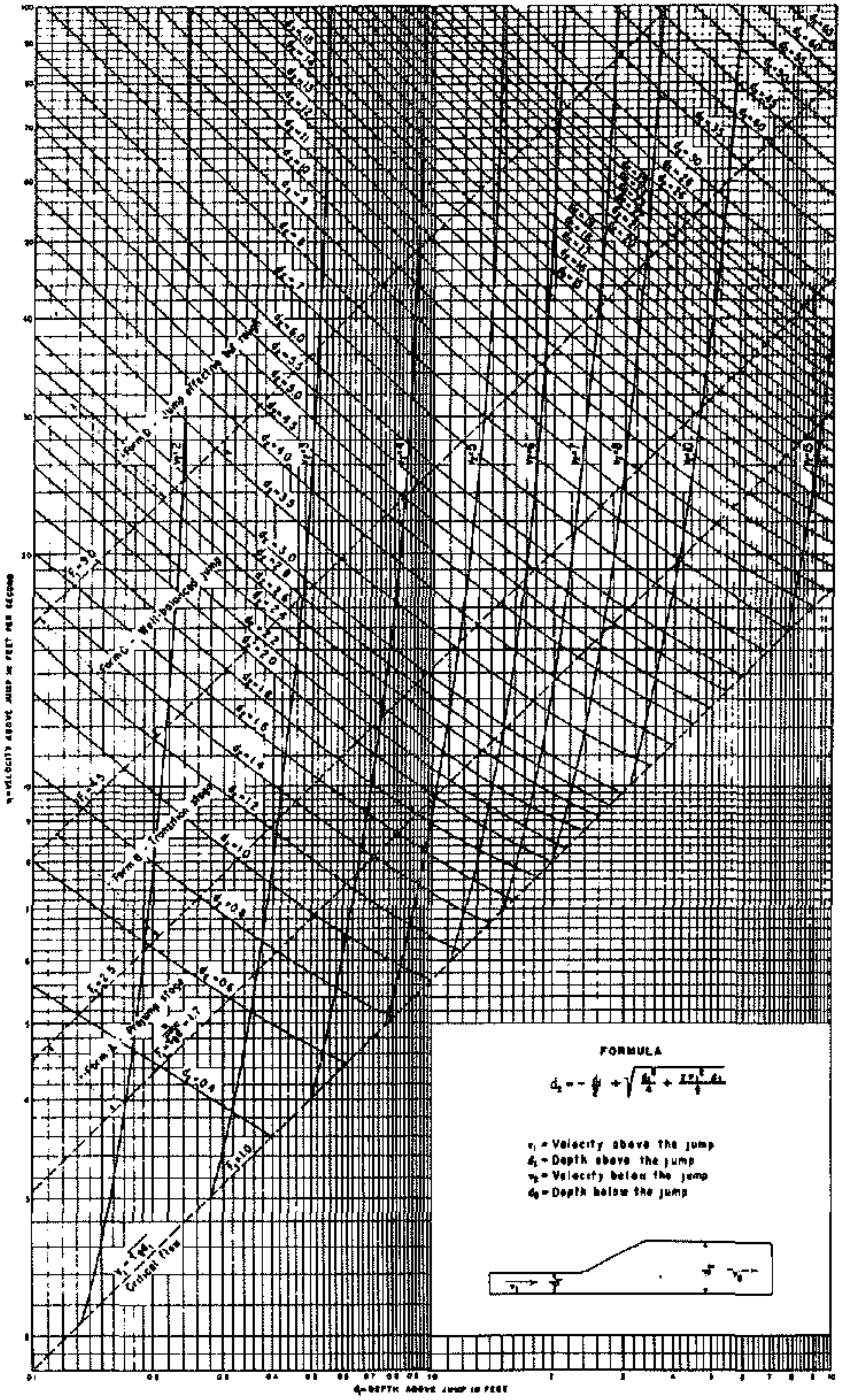
When the Froude number of the incoming flow is equal to 1.0, the flow is at critical depth and a hydraulic jump cannot form. For Froude numbers from 1.0 up to about 1.7, the incoming flow is only slightly below critical depth, and the change from this low stage to the high stage flow is gradual and manifests itself only by a slightly ruffled water surface. As the Froude number approaches 1.7, a series of small rollers begins to develop on the surface which become more intense with increasingly higher values of the number. Other than the surface roller phenomena, relatively smooth flows prevail throughout the Froude number range up to about 2.5.

For Froude numbers between 2.5 and 4.5 an oscillating form of jump occurs, the entering jet intermittently flowing near the bottom and then along the surface of the downstream channel. This oscillating flow causes objectionable surface waves which carry considerably beyond the end of the basin.

Figure V-9 plots relationships of conjugate depths and velocities for the hydraulic jump in a rectangular channel. Also indicated on the figure are the ranges for the various forms of jump described above.

Low Froude Number Basins. For a Froude number of 1.7, the conjugate depth d_2 is about twice the incoming depth, or about 40 percent greater than the critical depth. The exit velocity v_2 is about one-half the incoming velocity, or 30 percent less than the critical velocity. No special stilling basin is needed to still flows where the incoming flow Froude number is less than 1.7 except that the channel lengths beyond the point where the depth starts to change should be not less than about $4d_2$. No baffles or other dissipating devices are needed. The length of such basins is often too long, and thus too costly, and therefore baffle blocks and/or forced submergence may well be utilized to shorten the basin.

Medium Froude Number Basins. Flow phenomena for basins where the incoming flow is in the Froude number range between 1.7 and 2.5 will be in the form designated as the prejump stage. Since such flows are not attended by



From USBR

FIGURE V-9

RELATIONS BETWEEN VARIABLES IN HYDRAULIC JUMP FOR RECTANGULAR CHANNEL (2) active turbulence, baffles or sills are not required. The basin should be sufficiently long to contain the flow prism while it is undergoing retardation. Baffles and sills will often help to shorten the basin, though the nature of these appurtenances should be analyzed carefully, perhaps with the use of models.

Higher Froude Number Basins. Jump phenomena where the incoming flow factors are in the Froude number range between 2.5 and 4.5 are designated as transition flow stage, since a true hydraulic jump does not fully develop. Stilling basins to accommodate these flows are the least effective in providing satisfactory dissipation, since the attendant wave action ordinarily cannot be controlled by the usual basin devices. Waves generated by the flow phenomena will persist beyond the end of the basin and must often be dampened by means apart from the basin.

Where a stilling device must be provided to dissipate flows for this range of Froude number, the basin shown on Figure V-10, which is designated as type 1 basin, has proved to be relatively effective for dissipating the bulk of the energy of flow. However, the wave action propagated by the oscillating flow cannot be entirely dampened. Auxiliary wave dampeners or wave suppressors must sometimes be employed to provide smooth surface flow downstream.

Because of the tendency of the jump to sweep out and as an aid in suppressing wave action, the water depths in the basin should be about 10 percent greater than the computed conjugate depth.

Often the need for utilizing this type of basin in design can be avoided by selecting stilling basin dimensions which will provide flow conditions which fall outside the range of transition flow. For example, with an 800-second-foot capacity spillway where the specific energy at the upstream end of the basin is about 15 feet and the velocity into the basin is about 30 feet per second, the Froude number will be 3.2 for a basin width of 10 feet. The Froude number can be raised to 4.6 by widening the basin to 20 feet. The selection of basin width then becomes a matter of economics as well as hydraulic performance.

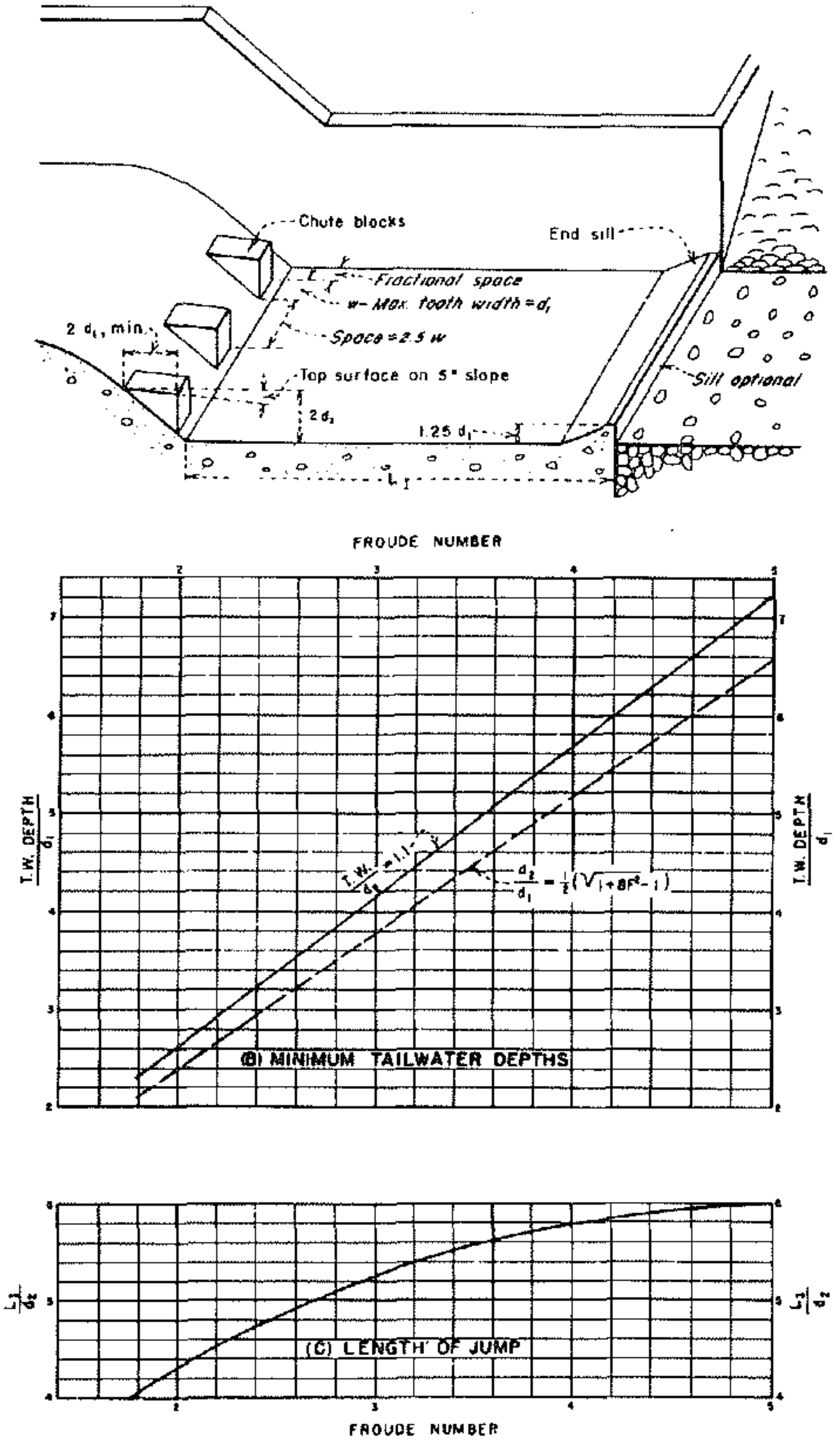


FIGURE V-10

STILLING BASIN CHARACTERISTICS
F_R = 2.5 TO 4.5.(2)

Variations in the design are possible, though changes must be based upon careful hydraulic analysis. Variations might include the addition of baffle blocks and a dentated sill at the downstream end. The use of baffle blocks is illustrated in Figure V-11.

Impact Stilling Basin. A lower cost stilling basin may be utilized for lower discharge magnitudes. Generally, this type of basin lends itself to use with pipes and has good application to the outlets from storm sewers. An impact type of energy dissipator has been developed which is an effective stilling device even with deficient tailwater where the discharge is relatively small. This basin can be used with either an open chute or a closed conduit structure. The design shown on Figure V-12 has been used for discharges up to about 400 second-feet; for larger discharges multiple basins could be placed side by side.

The general arrangement of the basin and the dimensional requirements for various discharges are shown on Figure V-12. This type of basin is subjected to large dynamic forces and turbulences which must be considered in the structural design. The structure must be made sufficiently stable to resist sliding against the impact load on the baffle wall. The entire structure must resist the severe vibrations inherent in this type of device, and the individual structural members must be sufficiently strong to withstand the large dynamic loads. Other types of impact stilling basins available are Design of Small Canal Structures published by the U.S. Bureau of Reclamation.

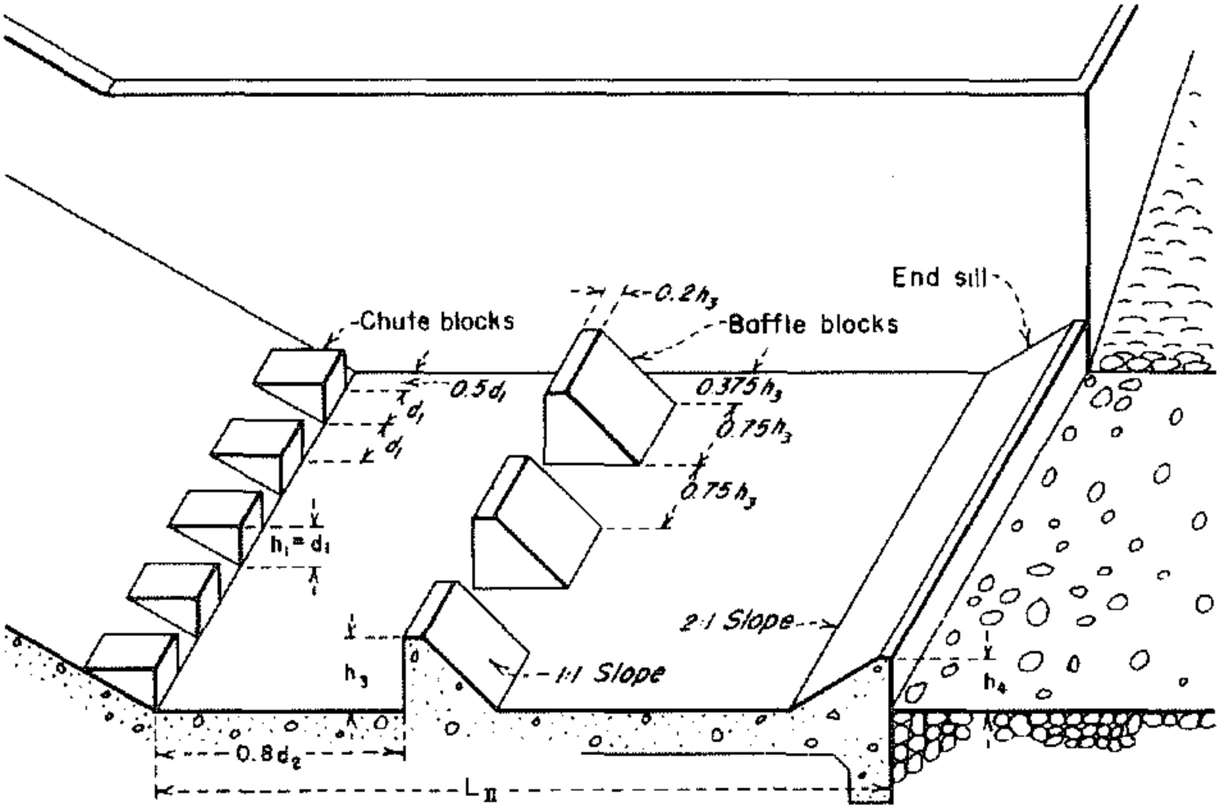
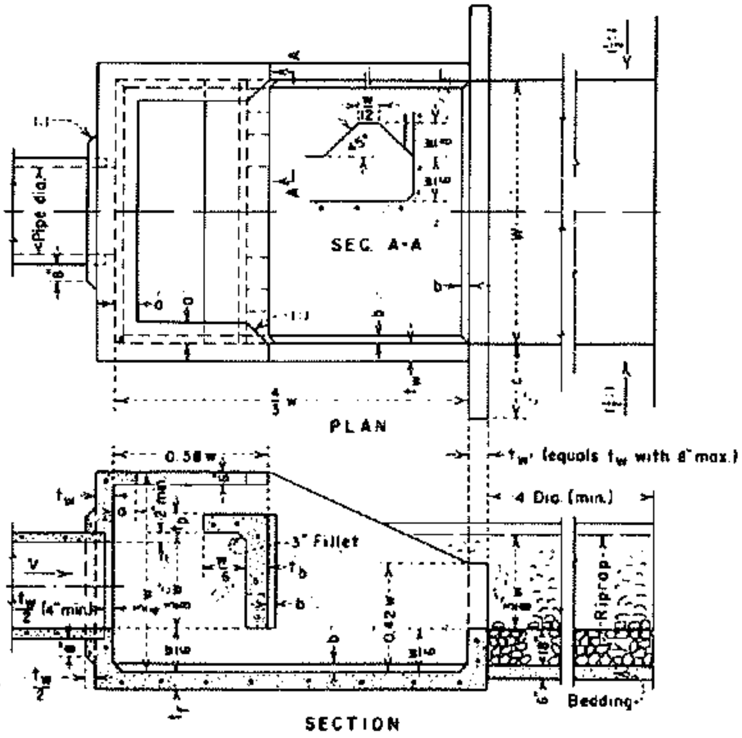
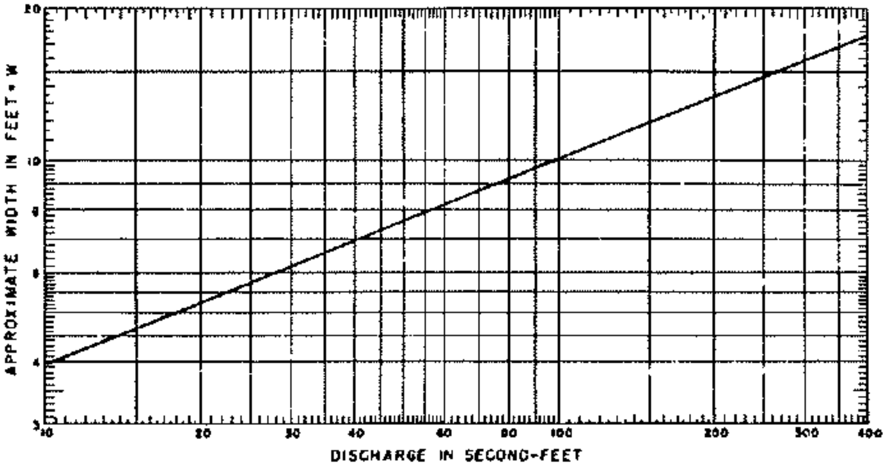


FIGURE V-11
BAFFLE BLOCK ARRANGEMENT (2)



Q	a	b	c	t_w	t_f	t_b	t_p
100	9"	3"	3'-0"	8"	6"	9"	6"
200	12"	4"	3'-0"	10"	11"	10"	6"
300	14"	6"	3'-0"	12"	12"	12"	8"
400	16"	6"	3'-0"	12"	13"	12"	6"
Suggested minimum thickness of concrete is 6"							

SUGGESTED CONCRETE DIMENSIONS



HYDRAULIC DIMENSIONS

FIGURE V-12

DIMENSIONAL CRITERIA FOR IMPACT TYPE STILLING BASINS (2)

Riprapping should be provided along the bottom and sides adjacent to the structure to avoid the tendency for scour of the outlet channel downstream from the end sill when a shallow tailwater exists. Downstream wingwalls placed at 45^o may also be effective in reducing scouring tendencies and flow concentrations downstream.

Plunge Pools. An unusual, but interesting energy dissipator device is the plunge pool. This is a free falling overflow which drops vertically into a pool. Design should follow model testing of the pool because of the serious problems which could occur with an improperly designed pool.

The pool must be heavily protected with large grouted riprap or reinforced concrete. The approximate pool depth is given by the following equation:

$$d_s = 1.32\ H_T^{0.225} q^{0.54} \qquad \text{Eq. V-13}$$

where:

d_s = the maximum depth of scour below tailwater level in feet,

H_T = the head from the reservoir to tailwater levels in feet, and

q = the unit discharge in second-feet per foot of width.

A plunge pool may only be used with a continuous low flow in the channel because of stagnated water.

Other Energy Dissipators. Other forms of energy dissipators are available which lend themselves to use in urban drainage works. However, the designer is cautioned against choosing one which functions in a questionable manner because of high operation and maintenance costs that may result, including significant downstream channel damage.

Downstream Channel. Care in planning and design is necessary for the downstream channel. Submerged jets of water can erode a downstream channel badly during only one runoff. On the other hand, currents not parallel with the channel centerline will also have erosive tendencies downstream as they move from one bank to the other.

Suitably low downstream average velocities and properly dissipated currents are important to avoid excessive channel damage.

CHANNEL DROPS

The use of channel drops permits adjustment of a thalweg which is too steep for the design conditions. In urban drainage work, it is often desirable to use several low head drops in lieu of a few higher drops.

Vertical drops should be avoided to minimize turbulence and erosion problems. A drop with a sloped face of 2:1 or 4:1 is generally suitable. The face should be roughened so as to dissipate energy, at least for the lower and more frequent flows. The use of gabions provides excellent drop characteristics with built-in surface roughness.

In most cases, additional bank and bottom protection will be needed after the first runoff or two, when erosional tendencies are field-tested. For this reason, the engineer should allow in his estimates for funds to be spent during the first two years following construction completion.

Vertical Drops

The use of vertical drops should generally be avoided because of the cost of the structure and resulting turbulence. However, at times the vertical drop will be used and for that reason the following criteria are presented.

Hydraulic Analysis. The aerated free-falling nappe in a straight drop spillway will reverse its curvature and turn smoothly into supercritical flow on the apron (Fig. V-13). As a result, a hydraulic jump will usually form downstream. Chow (21) describes the geometry of the straight drop spillway with various functions of the drop number, which is defined as:

$$D_N = q^2/gh^3 \qquad \text{Eq. V-15}$$

Where q is the unit discharge per unit width of crest of overfall, g is the acceleration of gravity, and h is the height of the drop. The functions are:

$$L_d/h = 4.30\ D_N^{0.27} \qquad \text{Eq. V-16}$$

$$d_p/h = 1.00\ D_N^{0.22} \qquad \text{Eq. V-17}$$

$$d_1/h = 1.66\ D_N^{0.425} \qquad \text{Eq.-18}$$

$$d_1/h = 1.66\ D_N^{0.27} \qquad \text{Eq. V-19}$$

where L_d is the drop length (the distance from the drop wall to the position of the depth d_1), y_p is the pool depth under the nappe, d_1 is the depth at the toe of the nappe or the beginning of the hydraulic jump, and d_2 is the tailwater depth sequent to d_1. L is the length of the hydraulic jump and may be determined as outlined for stilling basins.

From the above equations, the drop length and design tailwater depth may be determined. The above discussion is continued upon the length of the spillway crest being approximately the same width as the approach channel.

Practical Modifications. The actual application of the vertical drop would include generous and well placed grouted riprap and/or gabions at the side, upstream and downstream. The use of large boulders just downstream of the d_1 will cause some backwater at d_1, and decrease the needed length of protected channel downstream. The boulders may be natural types with dimensions of 3 to 4 feet, firmly grouted to the channel bottom.

Sloped Drops

The use of sloped drops will generally result in lower cost installations. Slope drops can be designed to fit the channel topography needs with little difficulty.

Slope drops should have faces from 2:1 to 4:1, have roughened faces, be adequately protected from scour, and should not cause an upstream water surface drop which would result in high velocities upstream. Side cutting just downstream from the drop is a common problem which must be protected against.

A typical 1.5 foot drop is shown in Figure V-14. Here, grouted riprap is shown, though the use of gabions would probably result in a structure with lower cost. Gabions would also tend to readjust themselves to take care of some minor erosion.

The length L will depend upon the hydraulic characteristics of the channel and drop. For a design q of 30 cfs/ft, L would be about 15 feet; that is, about 1/2 of the q value. The L should not be less than 10 feet, even for low q values. In addition, followup riprapping will often be necessary at most drops to more fully protect the banks and channel bottom. The criteria given is minimal, based on the philosophy that it is less costly to initially underprotect with riprap, and then to place additional protection later after erosional tendencies are determined in the field.

BRIDGES

Bridges are required across nearly all open urban channels sooner or later and, therefore, sizing the bridge openings is of paramount importance. When large culverts are used in lieu of bridges, the design approach often differs. For culverts, the reader is referred to the part of the Chapter on Culverts.

Open channels with improperly designed bridges will either have excessive scour, or deposition, or not be able to carry the design flow.

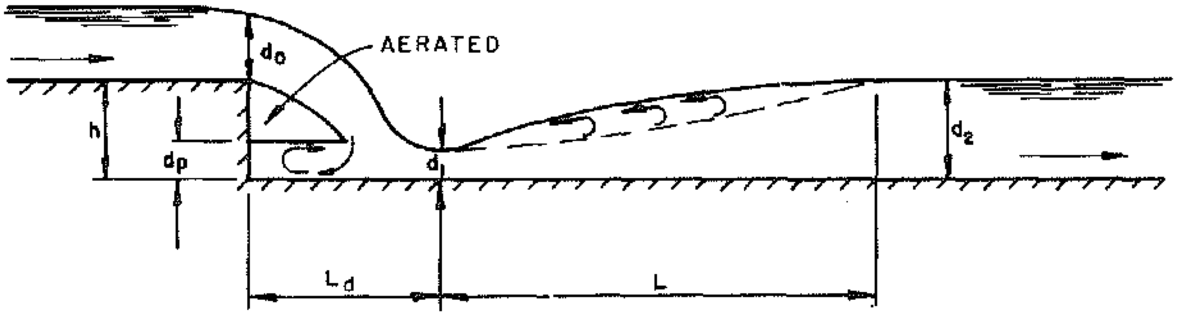


FIGURE V-13

FLOW GEOMETRY OF A STRAIGHT DROP SPILLWAY

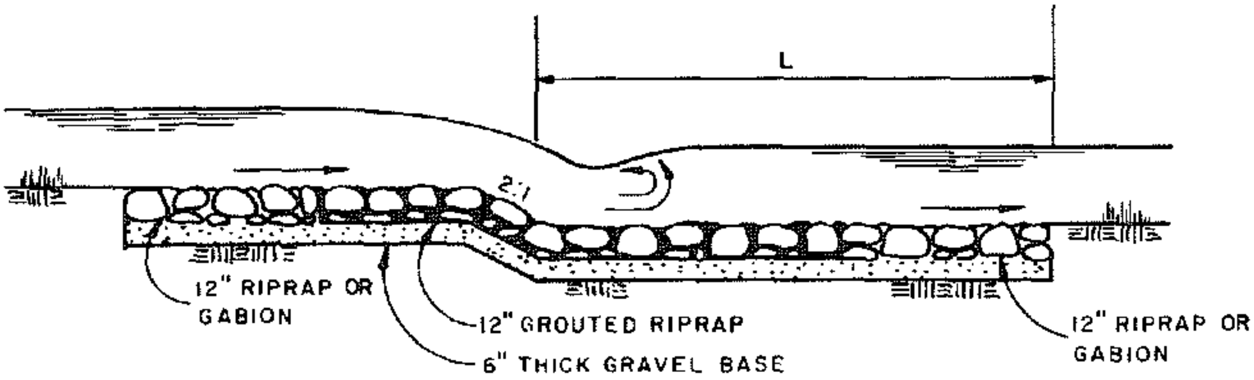
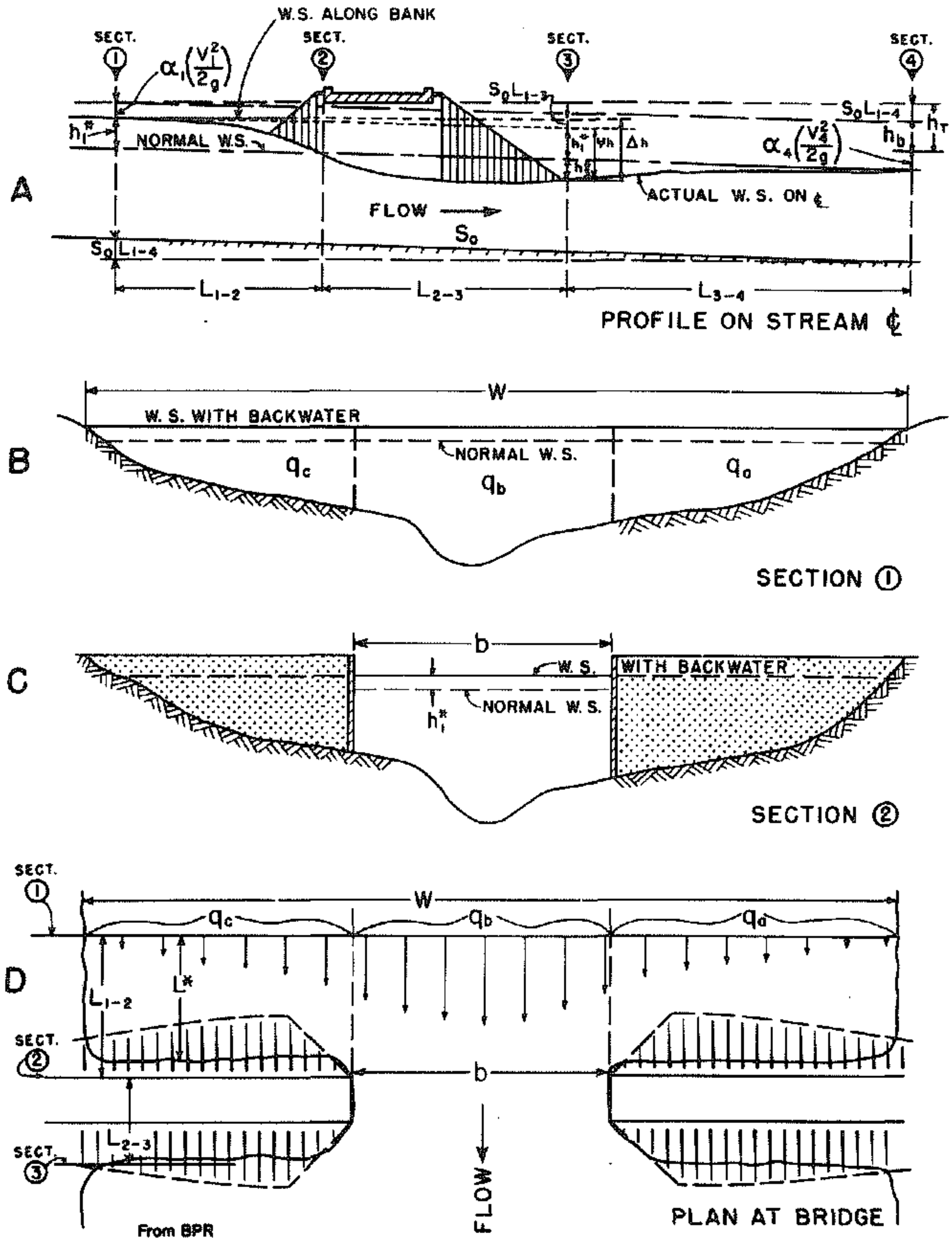


FIGURE V-14

TYPICAL SLOPED CHANNEL DROP



From BPR

FIGURE V-15

NORMAL BRIDGE CROSSING DESIGNATION (I)

Where

h^*_1 = total backwater (fts.).

K^* = total backwater coefficient.

α 1 = $\frac{(qv^2)}{QV_1{}^2}$ = kinetic energy coefficient

A_{n2} = gross water area in constriction measured below normal stage (sq. ft.).

V_{n2} = average velocity in constriction or $Q/A_n{}^2$ (f.p.s.) The velocity V_{n2} is not an actual measurable but represents a reference velocity readily computed for both model and field structures.

A_4 = water area at Section 4 where normal stage is reestablished (sq. ft.).

A_1 = total water area at Section 1 including that produced by the backwater (sq. ft.).

To compute backwater by expression 4-1, it is necessary to obtain the approximate value of h^* by using the first part of the expression:

$$H^*_1 = K^* \frac{V_{n2}{}^2}{2g} \qquad \text{Eq. V-21}$$

The Value of A_1 in the second part of expression 4-1, which depends on h^*_1, can then be determined:

$$\alpha 1 \left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2\right] \frac{V_{n2}{}^2}{2g} \qquad \text{Eq. V-22}$$

This part of the expression represents the difference in kinetic energy between sections 4 and 1, expressed in terms of the velocity head $V_{n2}^2/2g$. Expression 4-1 may appear cumbersome, but it was set up as shown to permit omission of the second part when the difference in kinetic energy between Sections 4 and 1 is small enough to be insignificant in the final result.

To permit the designer to readily recognize cases in which the kinetic energy term may be ignored, the following guides are provided:

$$M > 0.7;$$

$$V_{n2} < 7 \text{ f.p.s.; and}$$

$$K^* \frac{V^2{}_{n2}}{2g} < 0.5 \text{ foot}$$

If values in the problem at hand meet all three conditions, the backwater obtained from expression V-21 can be considered sufficiently accurate. Should one or more of the values not meet the conditions set forth, it is advisable to use expression V-20 in its entirety. The use of the guides is further demonstrated in the examples given in the reference (1) which should be used in all bridge design work.

Backwater Coefficient. The value of the overall backwater coefficient K^*, which was determined experimentally, varies with:

1. Stream constriction as measure by bridge opening ratio M;
2. Type of bridge abutment -- wingwall, spillthrough, etc.;
3. Number, size, shape, and orientation of piers in the constriction;
4. Eccentricity, or asymmetric position of bridge with the floodplains; and
5. Skew (bridge crosses floodplain at other than 90° angle).

The overall backwater coefficient K^* consists of a base curve coefficient K_b, to which is added incremental coefficients to account for the effect of piers, eccentricity, and skew. The value of K^* is primarily dependent on the degree of constriction of the flow but also changes to a limited degree with the other factors.

Effect of M and Abutment Shape (Base Curves). Figure V-16 shows the base curve for backwater coefficient K_b, plotted with respect to the opening ratio M, for several wingwall abutments and a vertical wall type. Note how the coefficient K_b increases with channel constriction. The several curves represent different angles of wingwall as can be identified by the accompanying sketches; the lower curves, of course, represent the better hydraulic shapes.

Figure V-15 shows the relation between the backwater coefficient K_b and M, for spillthrough abutments, for three embankment slopes. A comparison of the three curves indicates that the coefficient is little affected by embankment slope. Figures V-16 and V-17 will be designated "base curves" and K_b will be referred to as the "base curve coefficient." The base curve coefficients apply to normal crossings for specific abutment shapes, but do not include the effect of piers, eccentricity, or skew.

Effect of Piers (Normal Crossings). The effect produces on the backwater by introduction of piers in a bridge constriction has been treated as an incremental backwater coefficient designated ΔK_p, which is added to the base curve coefficient when piers are a factor. The value of the incremental backwater coefficient ΔK_p is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening, the type of piers (or piling in the case of pile bents), the value of the bridge opening ratio M, and the angularity of the piers with the direction of flood flow. The ratio of the water area occupied by piers A_p to the gross water area of the constriction A_{n2}, both based on the normal water surface, has been

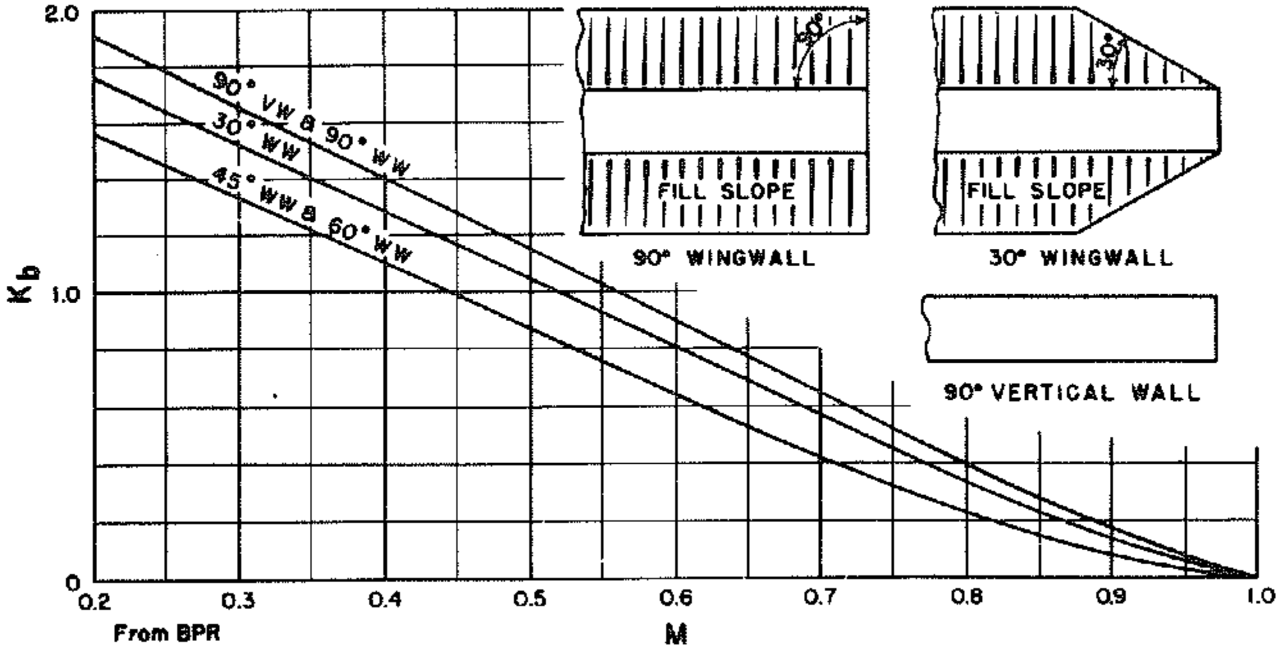


FIGURE V-16
BASE CURVES FOR WINGWALL ABUTMENTS

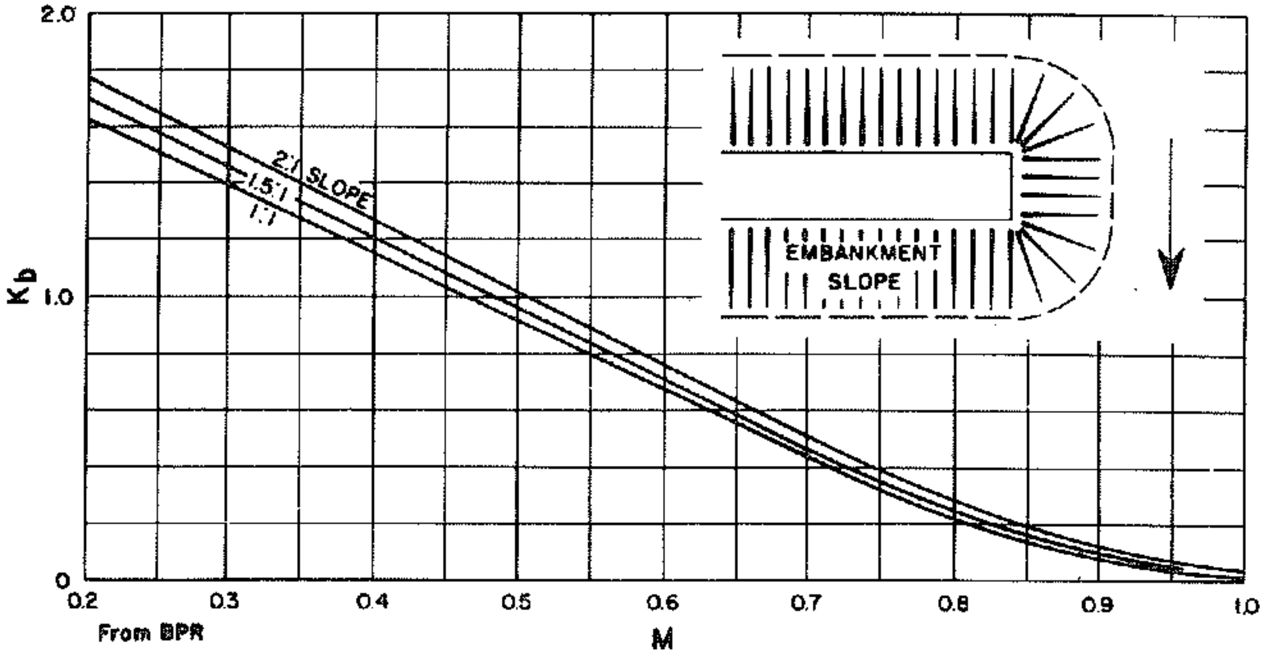


FIGURE V-17
BASE CURVES FOR SPILLTHROUGH ABUTMENTS (I)

assigned the letter J. In computing the grosswater area A_{n2}, the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from Figure V-18. The procedure is to enter Chart A, Figure V-18 with the proper value of J and read ΔK and obtain the correction factor from Chart B, Figure V-18, for opening ratios other than unity. The incremental backwater coefficient is then:

$$\Delta K_p = \Delta K \sigma \qquad \text{Eq. V-23}$$

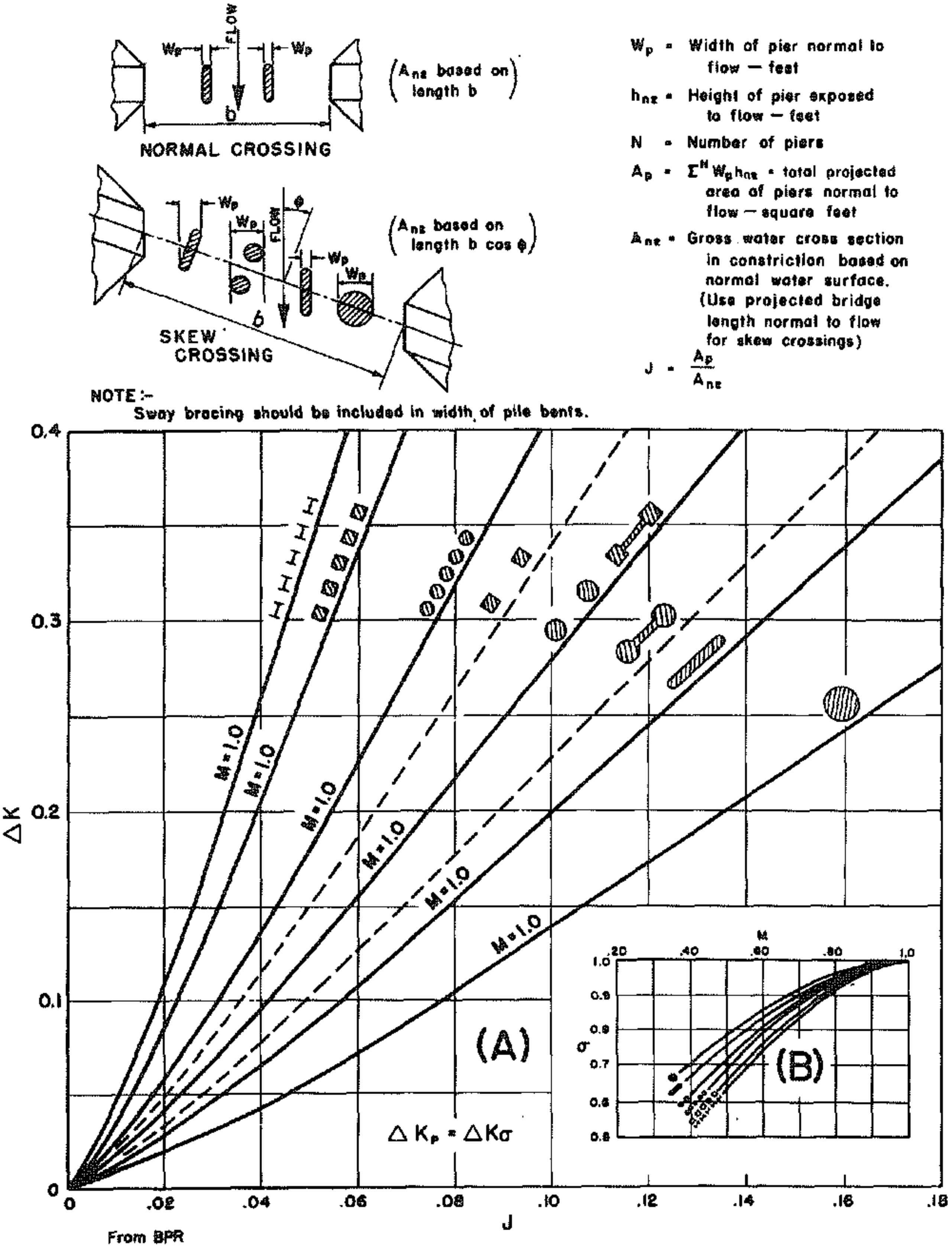
The incremental backwater coefficients for piers can, for all practical purposes, be considered independent of diameter, width, or spacing, but should be increased if there are more than 5 piles in a bent. A bent with 10 piles should be given a value of ΔK_p about 20 percent higher than those shown from bents with 5 piles. If there is a good possibility of trash collecting on the piers, it is advisable to use a value greater than the pier width to include the trash. For a normal crossing with piers, the total backwater coefficient becomes:

$$K^* = K_b \text{ (Figs. V-16 or V-17)} + \Delta K_p \text{ (Fig. V-18)} \qquad \text{Eq. V-24}$$

Design Procedure

The following is a brief step-by-step outline for determination of backwater produced by a bridge constriction:

1. Determine the magnitude and frequency of the discharge for which the bridge is to be designed.
2. Determine the stage of the stream at the bridge site for the design discharge.
3. Plot representative cross section of stream for design discharge at Section 1, if not already done under Step 2. If stream channel is essentially straight and cross section substantially uniform in the vicinity of the bridge, the natural cross section of the bridge site may be used for this purpose.



W_p = Width of pier normal to flow — feet

h_{nz} = Height of pier exposed to flow — feet

N = Number of piers

A_p = $\Sigma^N W_p h_{nz}$ = total projected area of piers normal to flow — square feet

A_{nz} = Gross water cross section in constriction based on normal water surface. (Use projected bridge length normal to flow for skew crossings)

$J = \frac{A_p}{A_{nz}}$

NOTE:-
Sway bracing should be included in width of pile bents.

From BPR

FIGURE V-18

INCREMENT BACKWATER COEFFICIENT FOR PIER (I)

4. Subdivide above cross section according to marked changes in depth of flow and roughness. Assign values of Manning roughness coefficient n to each subsection. Careful judgment is necessary in selecting these values.
5. Compute conveyance and then discharge in each subsection.
6. Determine value of kinetic energy coefficient.
7. Plot natural cross section under proposed bridge based on normal water surface for design discharge, and computer gross water area (including area occupied by piers).
8. Compute bridge opening ratio M, observing modified procedure for skewed cross crossings.
9. Obtain value of K_b from appropriate base curve.
10. If piers are involved, compute value of J and obtain incremental coefficient ΔK_p.
11. If eccentricity is severe, compute value of eccentricity and obtain incremental coefficient ΔK_e. (19).
12. If a skewed crossing is involved, observe proper procedure in previous steps, then obtain incremental coefficient ΔK_s for proper abutment type.
13. Determine total backwater coefficient K^* by adding incremental coefficients to base curve coefficient K_b.
14. Compute backwater by expression V-20.
15. Determine distance upstream to where the backwater effect is negligible.

Detailed steps illustrated by examples are presented in "Hydraulics of Bridge Waterways" (19).

Inadequate Openings

The engineer will often encounter existing bridges and culverts which have been designed for runoff having return periods significantly less than 100 years. In addition, bridges will be encountered which have been improperly designed. Culverts may be analyzed using the information in Chapter VIII.

Often the use of the orifice formula will provide a quick determination of the adequacy or inadequacy of a bridge opening:

$$Q = C\ A\ \sqrt{2gH} \qquad \text{Eq. V-25}$$

or

$$H = .04\left(\frac{Q}{A}\right)^2 \qquad \text{Eq. V-26}$$

where: Q = the major storm discharge in cfs
C = the bridge opening coefficient (0.6 assumed in equation V-26)
A = the area of the bridge opening
H = the head, that is the vertical distance from the bridge opening centerpoint to the upstream water surface about 10H upstream from the bridge. It is approximately the difference between the upstream and downstream water surfaces where the lower end of the bridge is submerged.

These expressions are valid when the water surface is above the top of the bridge opening.

ACCELERATION CHUTES

Acceleration chutes, whether leading into box culverts, pipes, or high velocity open channels, are often used to permit reduced downstream cross sections and resulting reduced costs. Chute spillways may be used in connection with both off-stream and on-stream detention reservoirs for a control structure and/or a spillway.

Acceleration chutes are potentially hazardous if inadequately planned and designed(8,17). High velocity flow can wash out channels and structures downstream in short order, resulting in property damage and uncontrolled flow.

The three references listed address acceleration chutes in detail for greater than can be discussed in this Manual. The designer is referenced to these publications for detailed analysis. In particular, the availability of the Soil Conservation Services to Stillwater makes the use of the reference Chute Spillways (2) advisable.

Hydraulics

Chutes have four component parts:

- Inlet
- Vertical Curve Section
- Concrete, Steeply Sloped Channel
- Outlet

Several types of inlets can be incorporated depending on the physical conditions and the type of control desired, particularly in regard to the use of chute spillways for off-stream detention facilities. The types of inlets which should be considered are:

- Straight Inlet
- Box Inlet
- Side-Channel Inlet
- Culvert Inlet
- Drop Inlet

Normally, the flow must remain at supercritical through the length of the chute and into the channel or conduit downstream. Care must be exercised in the design to insure against an unwanted hydrualic jump in the downstream channel or conduit. The analysis must include computation of the energy gradient through the chute and in the downstream channel or conduit.

BAFFLE CHUTES

Baffle chutes are used to dissipate the energy in the flow at a larger drop. They require no tailwater to be effective (17).

They are particularly useful where the water surface upstream is held at a higher elevation to provide head for filling a side storage pond during peak flows.

Baffle chutes are used in channels where water is to be lowered from one level to another. The baffle piers prevent undue acceleration of the flow as it passes down the chute. Since the flow velocities entering the downstream channel are low, no stilling basin is needed. The chute, on a 2:1

slope or flatter, may be designed to discharge up to 60 cfs per foot of width, and the drop may be as high as structurally feasible. The lower end of the chute is constructed to below streambed level and back-filled as necessary. Degradation of the streambed does not, therefore, adversely affect the performance of the structure. In urban drainage design the lower end should be protected from the scouring action.

Design Procedure

The baffled apron should be designed for the full design discharge.

The unit design discharge $q = Q/W$ may be as high as 60 cfs per foot of chute width, W. Less severe flow conditions at the base of the chute exist for 35 cfs and a relatively mild condition occurs for unit discharges of 20 cfs and less. Referring to Figure V-19, it will be noted that the entrance velocity, V_1, should be as low as practical. Ideal conditions exist when

$$V_1 = \sqrt[3]{gq} - 5 \qquad \text{Eq.V-27}$$

Flow conditions are not acceptable when

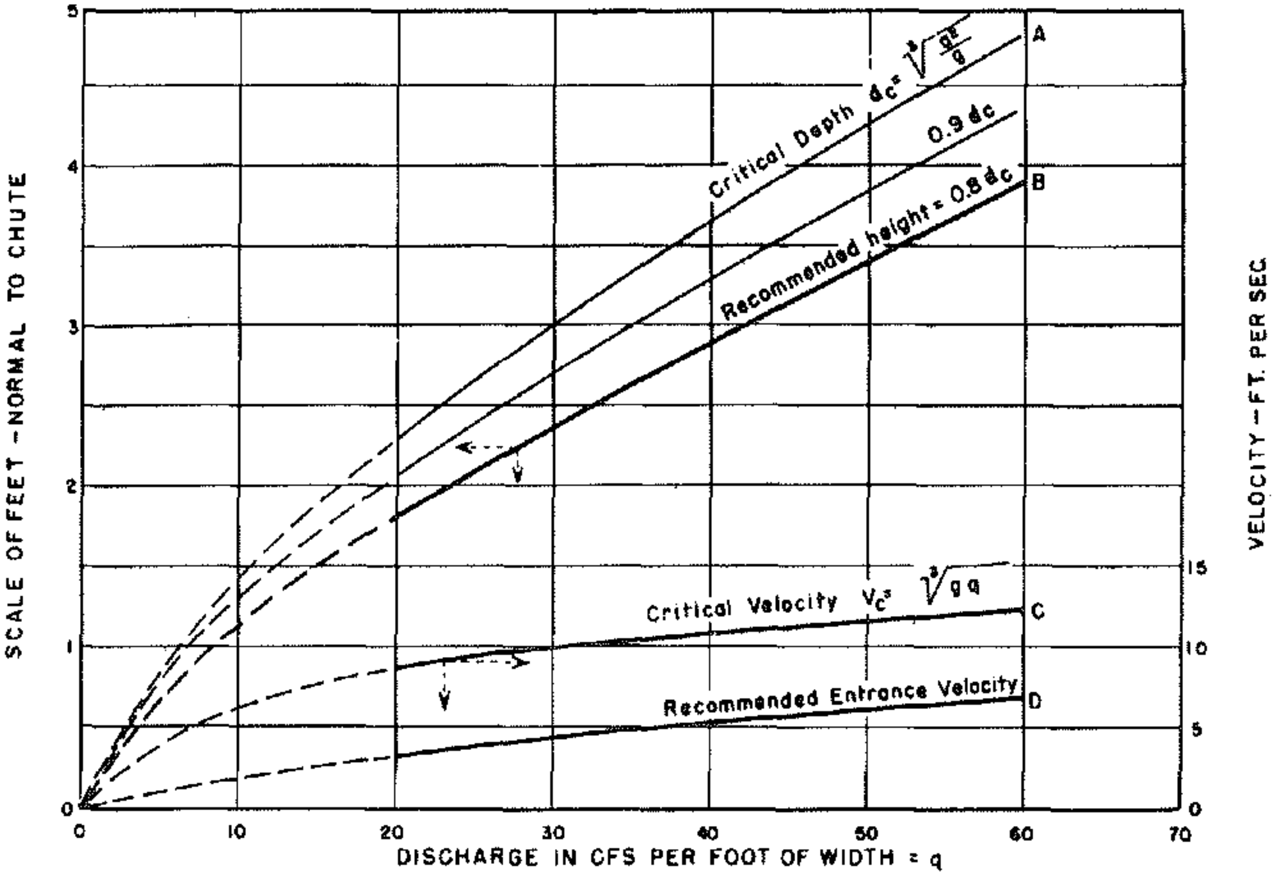
$$V_1 = \sqrt[3]{gq} \qquad \text{Eq.V-28}$$

The vertical offset between the approach channel floor, Figure V-20, and the chute is used to create a stilling pool or desirable V_1 and will vary in individual installations. Place the first row of baffle piers close to the top of the chute no more than 12 inches in elevation below the crest.

The baffle pier height, H, should be about 0.0 d_c, Curve B, Figure V-19. The critical depth on the rectangular chute is given by Curve A as:

$$Y_c = \sqrt[3]{\frac{q^2}{g}} \qquad \text{Eq. V-29}$$

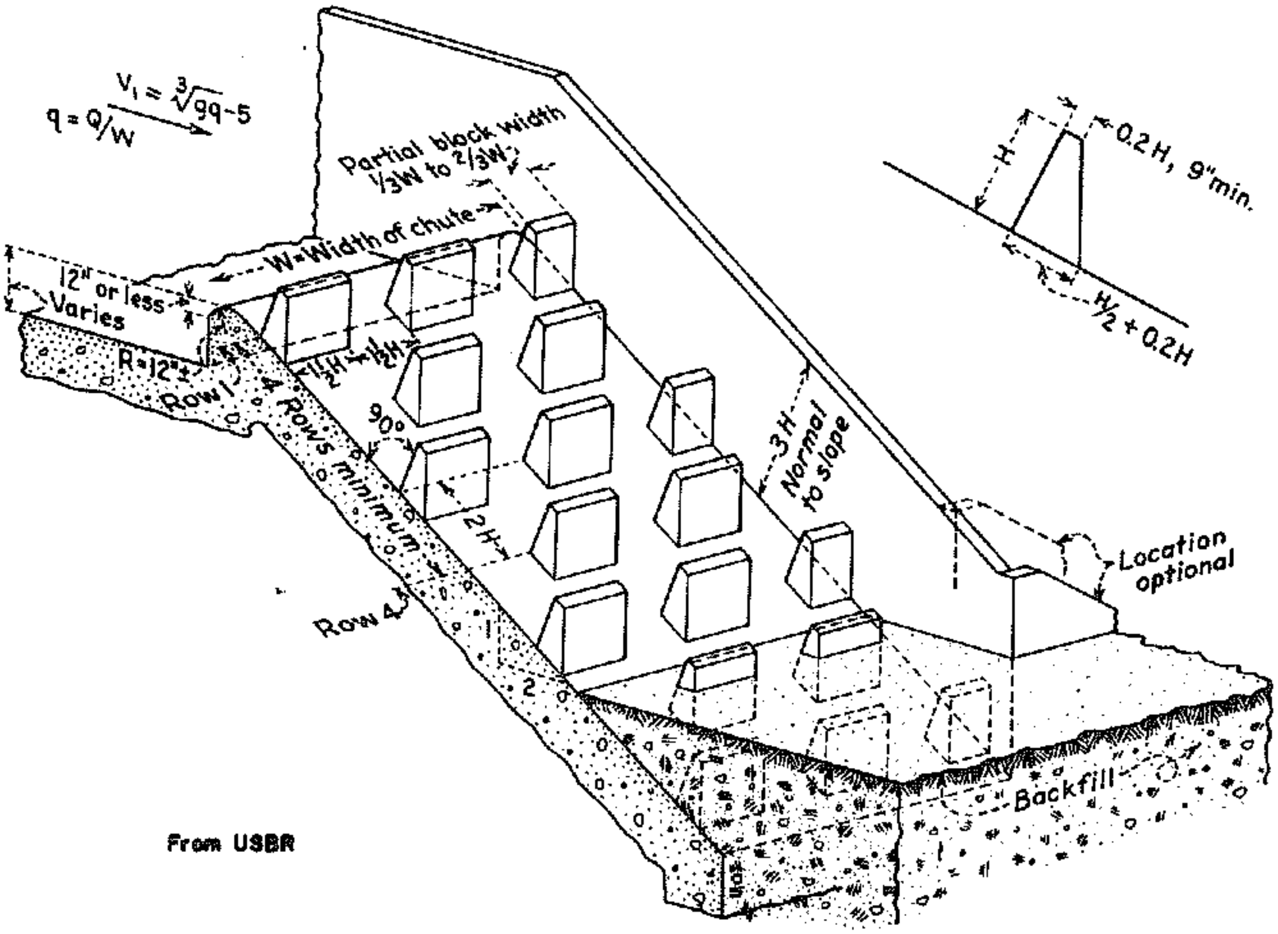
Baffle pier widths and spaces should be equal, preferably about 3/2 H. Partial blocks, width 1/3 to H to 3/2 H, should be placed against the training walls in Rows 1, 3, 5, 7, etc., alternating with spaces of the same width in Rows 2, 4, 6, etc.



From USBR

FIGURE V-19

BAFFLE CHUTE RECOMMENDED BAFFLE PIER HEIGHTS AND ALLOWABLE VELOCITIES (3)



From USBR

FIGURE V- 20

BASIC PROPORTIONS OF A BAFFLE CHUTE (3)

The slope distance (along a 2:1 slope) between rows of baffle piers should be twice the baffle height H. When the baffle height is less than 3 feet, the row spacing may be greater than 2 H but should not exceed 6 feet. For slopes flatter than 2:1, the row spacing may be increased to provide the same vertical differential between rows as expressed by the spacing for a 2:1 slope.

The baffle piers are usually constructed with their upstream faces normal to the chute surface; however, piers with vertical faces may be used.

Four rows of baffle piers are required to establish full control of the flow, although fewer rows have operated successfully. Additional rows beyond the fourth maintain the control established upstream, and as many rows may be constructed as is necessary. The chute should be extended to below the normal downstream channel elevation, and at least one row of baffles should be buried in the backfill.

The chute training walls should be up to three times as high as the baffle piers (measured normal to the chute floor) to contain the main flow of water and splash. It is impractical to increase the wall heights to contain all the splash.

Riprap consisting of 6- to 12-inch stones should be placed at the downstream ends of the training walls to prevent eddies from working behind the chute.

The reader is referred to reference (17) Hydraulic Design of Stilling Basins and Energy Dissipators for the design procedure for baffle chutes.

BENDS

Structures are generally unnecessary in subcritical flow channels unless the bend is of small radius. Structures for supercritical flows are complex and require careful hydraulic design to control the flow.

Supercritical Flow

Bends are normally not used in supercritical flow channels because of the costs involved and the hazards introduced. It is possible to utilize banking, easement curves, and diagonal sills (7). Sometimes outside bank rollover structures might even be considered. All of these, however, are generally out of place in urban drainage works.

When a bend is necessary, and it is not practical to first take the flow into subcritical flow, the designer will generally conclude that the channel should be placed in the closed conduit for the entire reach of the bend, and downstream far enough to eliminate the main oscillations. A model test is usually required on such structures. Furthermore, the forces exerted on the structure are large and must be analyzed.

Hydraulic Forces. The forces involved with hydraulic structures are large, and their analyses are often complex. The forces created can cause substantial damage if provisions are not made for their control (24).

In regard to bends, forces are usually larger than one would intuitively assume.

Newton's third law of motion: "For every force acting on a body there is a corresponding force exerted by the body; these two forces are equal in magnitude but opposite in direction," describes the basic fundamentals. See Figure V-21.

For time (t) of one second,

$$F = M\ (v_1 - v_2) \qquad \text{Eq. V-26}$$

where: F = force
M = mass
v = velocity12v

The force due to pressure on the bend should also be calculated when conduits flow under pressure. The total force exerted on the bend by the water, the total of momentum and pressure forces, must be counteracted by external forces. Allowable soil bearing should be determined using soil tests if necessary. Forces which cannot be handled by the pipe bearing on the soil must be compensated for by additional thrust blocks or other structures.

Example Computing Horizontal Forces

As an example of computer horizontal forces, assume the conditions as shown in Figure V-21 where the flow is 1920 cfs, the velocity is 30 fps, and the bend is in a box culvert with inside dimensions of 8' x 8'.

The magnitude and direction of the resultant force on the wall is needed. F is the total force of the wall on the water, and it can be broken into components Fx and Fy. Fx is a force in the direction indicated which decelerates the water from 30 fps to zero velocity. Fy is a force in the direction indicated which accelerates the water from zero to 30 fps. Then,

F = Mass x acceleration, or

$$Fx = 8' \times 8' \times 30 \ \frac{(62.4)}{(32.2)} \times (0 - 30) = 115{,}000 \text{ lbs.}$$

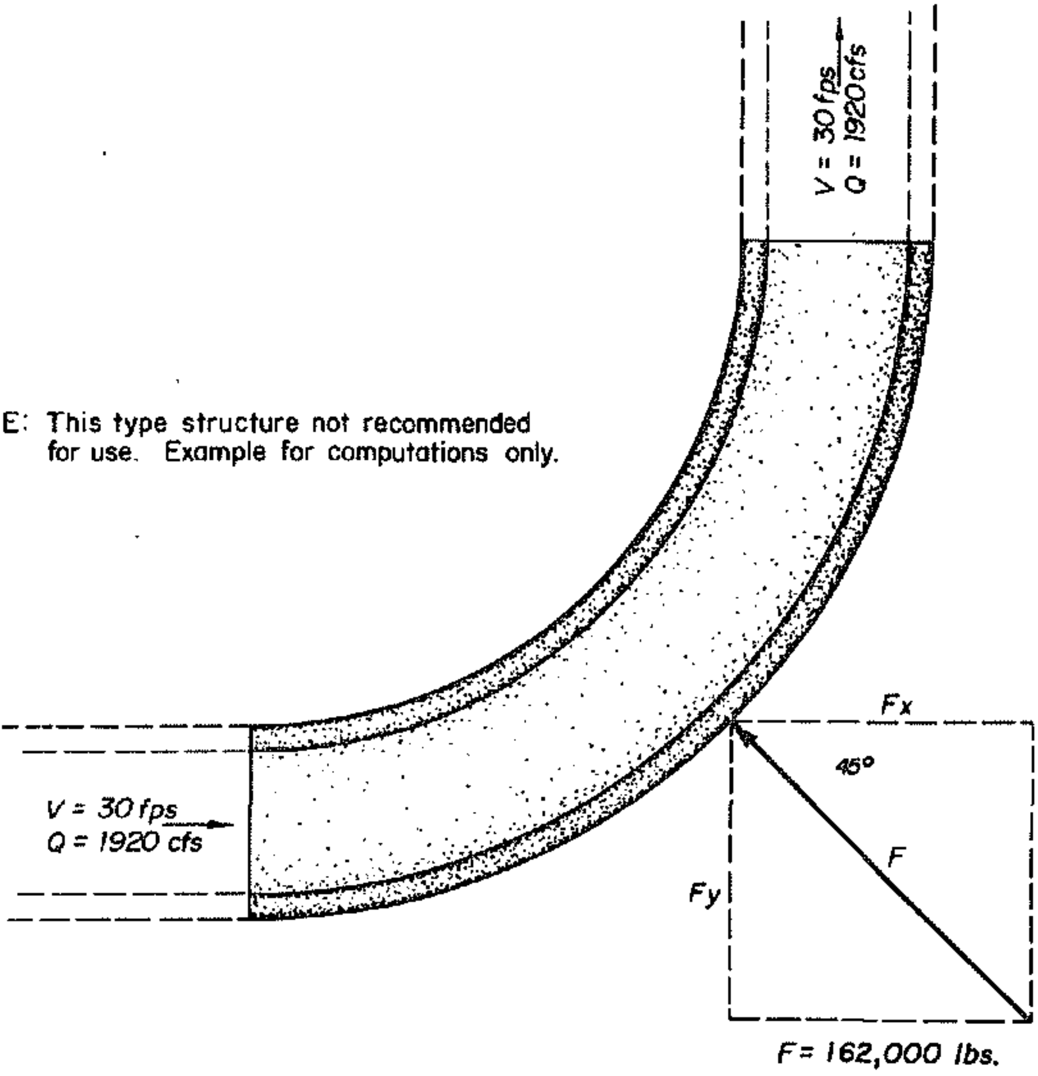
$$Fy = 8' \times 8' \times 30 \ \frac{(62.4)}{(32.2)} \times (30 - 0) = + 115{,}000 \text{ lbs.}$$

The magnitude of F is

$$F = \sqrt{Fx^2 + Fy^2} = 162{,}000 \text{ lbs.}$$

The resultant force of 162,000 lbs. acts at 45 degrees with the original flow direction.

The force due to pressure on the bend should also be calculated when conduits flow under pressure. The total force exerted on the bend by the water, the total of momentum and pressure forces, must be counteracted by external forces. Allowable soil bearing should be determined using soil tests if necessary. Forces which cannot be handled by the pipe bearing on the soil must be compensated for by additional thrust blocks or other structures.



NOTE: This type structure not recommended for use. Example for computations only.

FIGURE V-21
DYNAMIC FORCES AT BEND

STRUCTURE AESTHETICS

The use of hydraulic structures in urban drainage planning and design requires an environmental approach not normally encountered in the design of such structures. The appearance of these structures is very important.

The treatment of the exterior should not be considered of minor importance where cosmetic type work is done after design. Appearance must be an integral part of design.

It must be remembered that structures are often the only above-ground indication of the underground works involved in an expensive project. Furthermore, parks and green belts may later be developed in the area in which the structure will play a dominant environmental role.

Play Areas

An additional consideration is that drainage structures offer excellent opportunities for neighborhood children to play. It is almost impossible to make drainage works inaccessible to children, and therefore what is constructed should be made as safe and attractive as is reasonably possible. Safety hazards should be minimized and vertical drops protected with decorative fencing or rails.

Concrete Surface Treatment

The use of bushhammered concrete presents a pleasing appearance and removes form marks. The exposure of the aggregate may require special control of the aggregate used in the concrete.

Rails and Fences

The use of rails and fencing along concrete walls provides a pleasing topping to an otherwise stark wall, and yet gives a degree of protection against someone inadvertently falling over the wall.

High velocity channels baffle chutes and similar structures require fencing. It has been found that neighborhood type chain link fencing about 42 inches high is a satisfactory compromise between the safety aspect and the aesthetic need. In a lined channel, escape ladders are recommended for safety reasons, and as an aid to maintenance crews.

SYMBOLS

A = Area (subscripts as shown in figures)
A_p = Total projected area of piers normal to flow in square feet
D_n = Drop number as applied to drop structure
d = Depth of flow
D_c = Critical depth
F = Force
Fr = Froude number
g = Acceleration due to gravity
H_t = Total head
H_L = Head loss
H_v = Velocity head
H^*_1 = Total backwater in feet
J = A_p/A_{n2}
K = Backwater coefficient for bridges
K^* = Total backwater coefficient
K_b = Base curve coefficient, part of K^*
ΔK_p = Incremental backwater coefficient
L = Length (subscripts as shown in figures)
M = Bridge opening ratio, flow which can pass unimpeded through constriction to total flow in channel
Q = Discharge in cfs
q = Discharge per unit width in cfs/ft
S = Slope
V = Velocity in feet per second
W = Width
Z = Vertical distance
α = Angular variation of sidewall with respect to channel centerline
α 1 = Kinetic energy coefficient
σ = Correction factor for K

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MAN MADE STORAGE

Creation or allocation of planned storage space is a means to offset increased storm runoff resulting from urbanization.

Stormwater management is a time related, space allocation problem. Water cannot be compressed. If natural storage is reduced by urban or other land use practices without appropriate compensatory measures, additional space will be claimed by the floodwaters at some other location.

Storage space also provides better opportunities for reuse and pollution control efforts because of the storm water being more manageable and less erosive.

The most costly space for storm runoff is in small diameter storm sewer collection systems as the transient peak runoff is being transported to a discharge point. Thus, planned storage has important economic ramifications. It can help reduce urbanization costs.

NEED

Urbanization has an adverse effect upon the natural storage. Typically, development will reduce the water held by vegetation, in the soil, in soil depressions, and in ponds and lakes. Usually, urbanization also reduces the transient storage capability of wide slow flow flood plains. Man made storage should compensate for these reductions. Further, man made storage should be used to compensate for increasing rates of runoff resulting from shorter times of concentration of the runoff.

In summary, the effect of urbanization is generally to increase the rate and volume of runoff response due to faster hydraulic conditions that exist in paved areas versus vegetated areas. An objective of storage is to slow this rate of response. By using slow-flow channels, revegetation, and planned storage, the effects of urbanization are minimized. Planned positive results can occur.

POTENTIAL OF MAN MADE STORAGE

Man made storage can exist anywhere in the basin. The closer it is to the point of rainfall occurrence, the better.

Storage can exist on principal creeks and drainageways, and it can be on rooftops and parking lots. It is best to have storage dispersed to best achieve the objectives of this Manual.

In a newly developed area, man made storage is a preventive measure. It is a corrective measure when applied to solving existing problems.

Storage facilities can be managed to provide multiple benefits. Such benefits include water quality improvement, sediment control, water supply, and recreational opportunities. In some instances, valley configurations are conducive to the development of storage sites. Such storage can be achieved by a road fill across a valley. Excavations for aggregate or fill can also provide storage opportunities.

Man-made storage must be viewed as only one of the possible measures to be considered in a drainage program. It must be coordinated with efforts to maintain and possibly enhance the natural storage.

Man-made storage should be planned initially in terms of drainage requirements. Aesthetics and/or recreational considerations must be subordinated to that purpose. Such storage should be evaluated in regard to economic feasibility and physical practicality.

When provision of storage is being considered, the designer must verify that the attenuation of the peak runoff will not undesirably aggravate any potential downstream peaking conditions for a range of flood frequencies.

Consideration must also be given to the effect of the prolongation of flows. Assessment of these aspects must not be limited to the immediate watercourse or watercourses under consideration, but must extend to any other watercourse along which the floodwaters are conveyed. In some instances this may necessitate routing of storms of durations critical for each reach of the watercourses under consideration through the whole of the drainage system upstream of the reach. In other instances, only a superficial assessment based on experienced judgment may suffice.

The greater the number of storages in a system, the more complex is the analysis of the interaction of the various outflows. Also, for such storages to function in accordance with their design intent during a given event, they must be regularly and effectively maintained. This factor must be taken into account by the designer.

The most effective man-made storage would be that which duplicates natural storage. In that way, the natural hydrologic regime would more nearly be maintained.

LOCATION OF MAN-MADE STORAGE

Man-made storage can be located throughout a catchment. To be effective, the storage must be related to the area to be protected. With respect to location, man made storage can be located upstream of the area to be protected, within (dispersed) the areas to be protected, and downstream from the area to be protected. The location selected will be determined by the nature and source of the flood problem.

Upstream Storage

This storage takes place upstream from the area to be protected. Its purpose is to store runoff which originates upstream or beyond the area to be protected.

Localized Storage

This storage takes place within the area to be protected. It can be dispersed throughout the area. Its primary purpose is to provide storage for the increased runoff which results from the urban development. Frequently such storage is provided at the development sites.

Downstream Storage

This storage takes place downstream from the area to be protected. In general, downstream storage manages the runoff from the protected area and mitigates many downstream effects that may be associated with development in the protected area.

TYPES OF STORAGE

Storage facilities can be classified into three basic types. These types provide a range of management opportunities to the designer. Within a given catchment, a plan may use a mix of the three types of storage.

Retention Storage

Retention storage is provided in a basin in which the runoff from a given flood event is stored and is not discharged into the downstream drainage system during the flood event. This type of stored water may be used for beneficial purposes such as irrigation or low-flow augmentation or be allowed to evaporate or seep into the ground. To be totally effective, the stored water in the flood control part of the basin must be used or lost before the next flood event occurs. A permanent conservation pool can be designed into a retention storage facility. When this is done, the facility may be referred to as wet storage.

Detention Storage

Detention Storage is short-term storage which attenuates the peak flow by reducing the peak outflow to a rate less than the peak inflow and thereby lengthens the time base of the hydrograph. The total volume of water discharged is the same, it is simply distributed over a longer duration. The detention basin usually drains completely in less than a day. The area is normally dry and can be used for recreational purposes. On rare occasions, the storage of runoff may conflict with the planned recreational use of the site.

Conveyance Storage

During the period that channels, floodplains, drains, and storm sewers are filling with runoff, the waters are being stored in transient form. This type of storage is known as conveyance storage. Construction of slow velocity channels with large cross sectional areas assists in the accomplishment of such storage. In a 25-foot wide channel, a 1-foot increase in water level will provide approximately 9 acre-feet of storage in a 1-mile reach.

GENERAL REQUIREMENTS

All man-made storage should be planned to meet the following general requirements to provide safe facilities that will help to achive the goals and objectives of the City of Stillwater.

- Facilities should be coordinated with the development goals and objectives and the existing land use.
- Facilities should be designed to protect against failure that would increase the potential for downstream flood loss and must meet the standards of the Oklahoma Water Resources Board.
- Facilities should be evaluated with consideration of normal flow conditions, frequent events, less frequent intense events such as the 100-year frequency rainfall event, and maximum probable events. The evaluation of such considerations will ensure that the storage does not worsen downstream flood conditions.

- Facilities should be designed with careful attention to a particular design event. A design rainfall probability of 1 percent should normally be used unless specific minor facilities are being evaluated.
- Facilities should be planned with respect to the topography, soil, and geology.
- Facilities should be planned to reduce the degree of operation, maintenance, and administrative needs.
- Provisions should be made to ensure the maintenance of the facilities over their design life.
- Floodplains should be regulated downstream of new storage facilities to prevent new encroachment into the area protected by the storage. A storage facility should not encourage creation of new flood hazards or set the stage for larger disasters than formerly.

METHODS OF STORAGE

The potential methods of developing man-made storage are presented in Table VI-1. This list of methods is illustrative of the range of choices. Storage must be site specific. The means to be applied will reflect the proposed specific development and the site conditions.

These techniques are usually controlled by the planner in the early stages of the development. However, the architect, engineer, home-builder, land developer, and government officials all have a responsibility to work toward the concept of storage which fits within the framework of a comprehensive, basin-wide strategic drainage plan. Using this approach, the concepts described in this section can effectively reduce urban costs through miltipurpose use for drainage, parking, recreation, and open space, both downstream of the development and within the development.

TABLE VI-1

EXAMPLES OF STORAGE METHODS

Residential Lots

1. Driveway storage
2. Infiltration
3. Irrigation

Subdivisons, Office Complexes, Industrial Parks

1. On-site ponds
2. Parking lot storage
3. Slow-flow drainage patterns
4. Open space storage

Commercial and Industrial Parcels

1. Rooftop storage
2. Parking lot storage
3. Cistern/infiltration
4. Cistern/irrigation
5. On-site ponds

Catchment

1. Retention reservoirs
2. Detention reservoirs
3. Gravel pits and quarries
4. Open space storage

Storage can be planned for an individual residential, commercial, or industrial parcel; an entire subdivision, office complex, or industrial park; and an entire catchment. These storage methods are briefly discussed in the following text.

Driveway Storage

Driveways can be constructed so that runoff from the lot and/or roof is routed to a depressed section of driveway. The design of the outlet system will regulate the small discharge into the drainage system.

Infiltration

Runoff from the lot and particularly from roofs can be routed to a buried tank of adequate volume and with emergency overflow. Depending upon the subsurface soils and geologic conditions, the water can be infiltrated at a later time.

Irrigation

Alternatively to the above method, the water in the tank can be used for an irrigation water supply or discharged into the storm drain system. In areas of rolling terrain, the irrigation water may be distributed by a spreader pipe by gravity.

Rooftop Storage

Storage of water on flat commercial or industrial roofs can be economically achieved. Roofs are usually designed to be able to support adequate loads. A special drainage outlet is provided to regulate the release of water.

A typical rooftop storage ring is shown in Figure VI-1. The ring is placed around the standard roof drain outlet specified in the building code. Bottom holes permit small flows to reach the roof drain unimpeded. The ring and spacing of the upper orifice in the ring are designed to allow the reduced 1 percent flow to proceed unimpeded to the roof drain. Any larger volume of storage will overtop the ring unimpeded and flow normally to the roof drain. Maximum average depth of controlled ponding is usually not more than 3 inches, representing a hydraulic load of 15.6 pounds per square foot. Structural engineers should verify roof loading capability on existing buildings and should include roof storage loads when designing new buildings.

Parking Lot Storage

Grading of parking lots for storage is one of the least troublesome and most effective means of reducing runoff. Grading routes runoff to storage areas where controlled outlets are placed. Outlets are either grated storm inlets sized to restrain the outflow or cuts in surrounding low berms or concrete retaining walls sized to regulate the design flow.

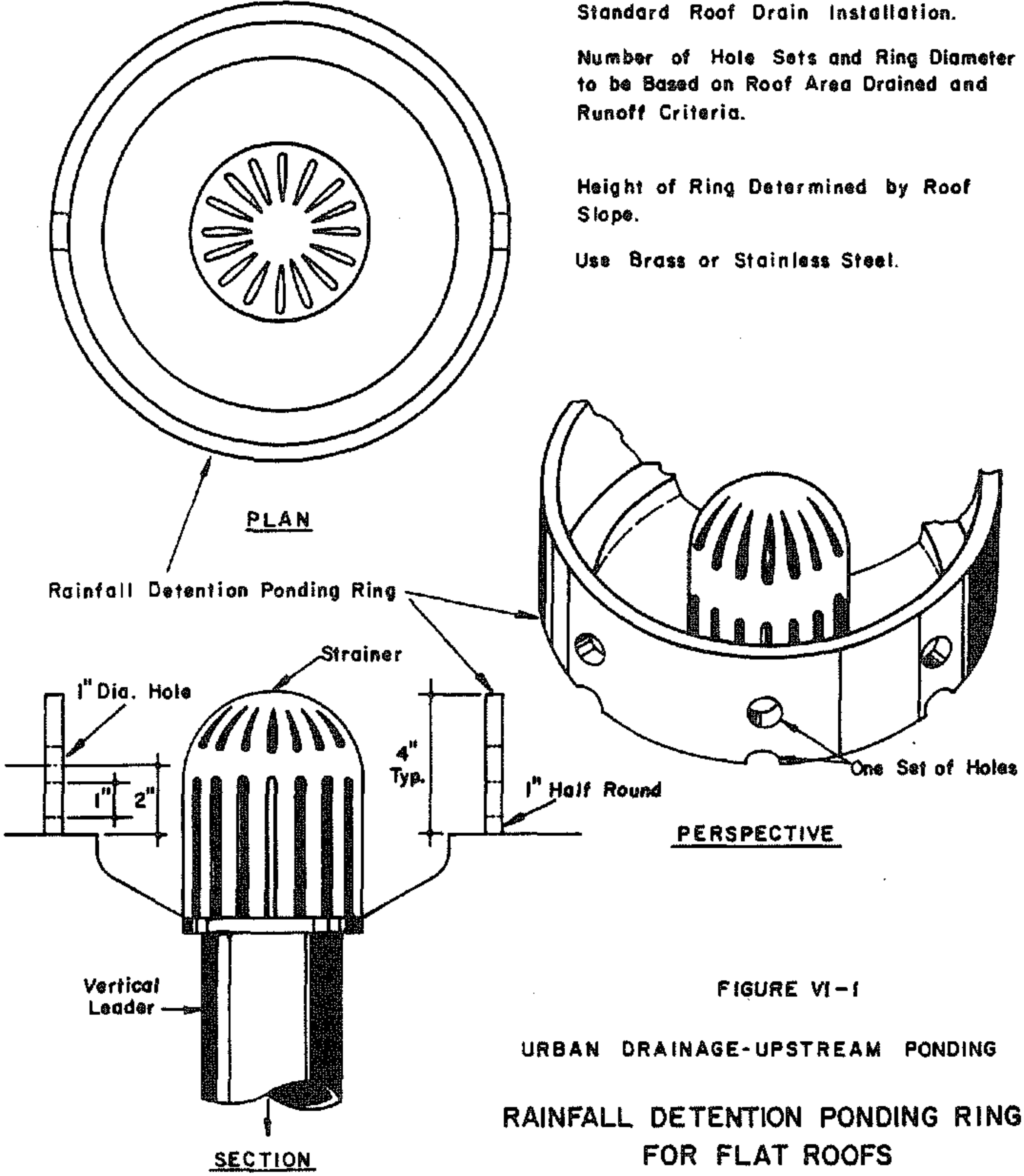
Grading of the pavement surface should be accomplished to minimize conflict between use of the lot and storage of storm runoff. However, storage of runoff is appreciable only several times each 10 years and even then, Notes:

Roof Drain Ring is Placed Around Standard Roof Drain Installation.

Number of Hole Sets and Ring Diameter to be Based on Roof Area Drained and Runoff Criteria.

Height of Ring Determined by Roof Slope.

Use Brass or Stainless Steel.



PLAN

SECTION

PERSPECTIVE

FIGURE VI-1

URBAN DRAINAGE-UPSTREAM PONDING

RAINFALL DETENTION PONDING RING FOR FLAT ROOFS storage may occur during times of non-use. Maximum depth of storage would occur only for a short time, about once each 100 years. Conflict in uses is not a significant problem.

On-Site Ponds

The construction of on-site ponds which have aesthetic and recreational benefits provides significant storm runoff detention benefits when properly planned and designed. Such ponds can be designed in common open space or incorporated as green median strips in a site development plan.

The hydraulic design of the storage function of permanent decorative ponds is based upon surcharge of the pond during precipitation events. Outflow of storm runoff is controlled by overflow weirs or orifices. This is effective ponding and can be implemented in high density areas.

Slow-Flow Drainage Patterns

This method can be used in specific instances. Subdivision planning requires adequate surface drainage away from buildings. The drainage plan might be designed in a manner that will cause temporary ponding by using grades which will create reduced water velocity.

The planner and engineer should insure that the neighborhood does not have expansive clays or shales underlying the surface which could affect building foundations. In such cases, water should not be ponded or percolated into the ground except in preselected locations. Use of subsurface drains at a shallow depth may be beneficial.

As an alternative to curbs and gutters, grassed roadside channels can be used to limit the effects of urbanization. In the case of planned development with integral open space, such depressions could be used as the primary means for transporting runoff. Also storage may be augmented by providing controls (weirs, checks, etc.) along the channels. In effect, a series of small linear reservoirs can be created, thereby providing storage volume.

Depending upon the extent of such controls and the volume provided, storage can be obtained along with possible increased infiltration of the runoff.

Open Space Storage

Storage can be combined with open space and recreation areas. Open space areas can be utilized for the temporary detention of the storm runoff with a minimum effect on their primary function.

Recreational areas, such as baseball or football fields, generally have a substantial area of grass cover which often has a good infiltration rate. Storm runoff from such fields is generally minimal. The multiple-use of such recreational fields can be made by providing for the ponding of runoff from adjacent areas.

Parks create little runoff of their own, but provide excellent storage potential. Using parks as storage areas can reduce the total urban system cost by combining capital requirements and maintenance requirements into multiple-purpose facilities.

If properly planned and constructed, utilization of parks for storage will cause little additional maintenance costs due to the storm drainage function and will often be nonconflicting with park purposes. If a permanent conservation pond is provided in the storage plan, recreational opportunities will be expanded. Also, a water supply could be developed for park irrigation.

Retention Reservoirs

Retention reservoirs in a catchment generally are major storage facilities. They are located in the valleys and have the ability to regulate the stream flow. Because they can have permanent conservation ponds or lakes, they can be integrated into the system of metropolitan parks or other large open areas. Water oriented recreational features can be incorporated into the planning of such reservoirs.

Detention Reservoirs

Detention reservoirs generally are located on streams. However, they are frequently located above the reaches where there is a continous flow. Thus, they may not have permanent ponds and may not provide opportunities for water oriented recreation. They can be designed as integral parts of a park and open space plan.

Gravel Pits and Quarries

Gravel pits and quarries can be designed to provide significant flood storage. Such storage is off-channel. A side channel spillway can be used to permit the peak of the hydrograph to spill into the storage area. Outflow from such storage areas is site specific.

HYDRAULIC DESIGN

The development and use of hydraulic design criteria for the retention and detention of storm runoff, both upstream and downstream, is the domain of the urban hydrologist. Generally, the hydrologist will find that a computer model is best for a basin-wide analyses. For a specific tract or smaller area, a hydrograph procedure is suitable. For small areas, the Rational Method is often used.

The objective of the storage of urban runoff should be the reduction of downstream peak flows and not the storage of all of the runoff.

Rational Method Analysis

Occasionally the designer will choose to use the Rational Method for sizing the storage, particularly in regard to upstream storage. Simplified techniques have been developed for computing the benefits of the retention and detention of storm runoff water, based upon the Rational Formula. The procedure is presented in "Airport Drainage," prepared by the Federal Aviation Agency (1) and should be limited to use on parcels of land under 20 acres.

Hydrograph Analysis

The hydrograph procedure provides a hydrograph which permits a dependable and straightforward approach to the analysis of the effect of storage of runoff water. It provides a degree of flexibility for use of judgment by the designer.

A storm runoff hydrograph is presented in Figure VI-2 which represents inflow to a reservoir. The analysis for the reservoir storage must take into consideraion the characteristic of the outlet pipe, the discharge of which is shown on Figure VI-2 as a solid line. The shape of the solid line reflects the carrying capacity of the outlet works with various headwater elevations. The higher the elevation of the water surface in the reservoir, the greater the discharge through the outlet works. The area between the solid line and the hydrograph of storm outflow can be planimetered to determine the volume of storage required to reduce channel flow from 200 cfs to 100 cfs because of the filling of the reservoir.

For offstream storage the basic approach to the analysis is presented in Figure VI-3. In this case, the peak of the storm hydrograph is routed over a side channel spillway into a ponding area adjacent to the channel. The water removed from the channel, represented by the shaded area of the hydrograph, provides for a reduction in the peak channel flow from 200 cfs to about 100 cfs.

The sizing of the outlet works for detention ponding is a matter of judgment depending upon the actual conditions for the specific case. The designer may approach the sizing of the outlet works on a trial and error basis with the objective being the optimum use of the available storage capacity of the ponding area. All ponding areas must be analyzed in regard to the major storm runoff conditions. In many cases, it will be found that the initial storm runoff should be routed through the outlet works with only minimal ponding. The storage would then be utilized to reduce the major runoff. If other provisions are made for the major runoff, the designer may be

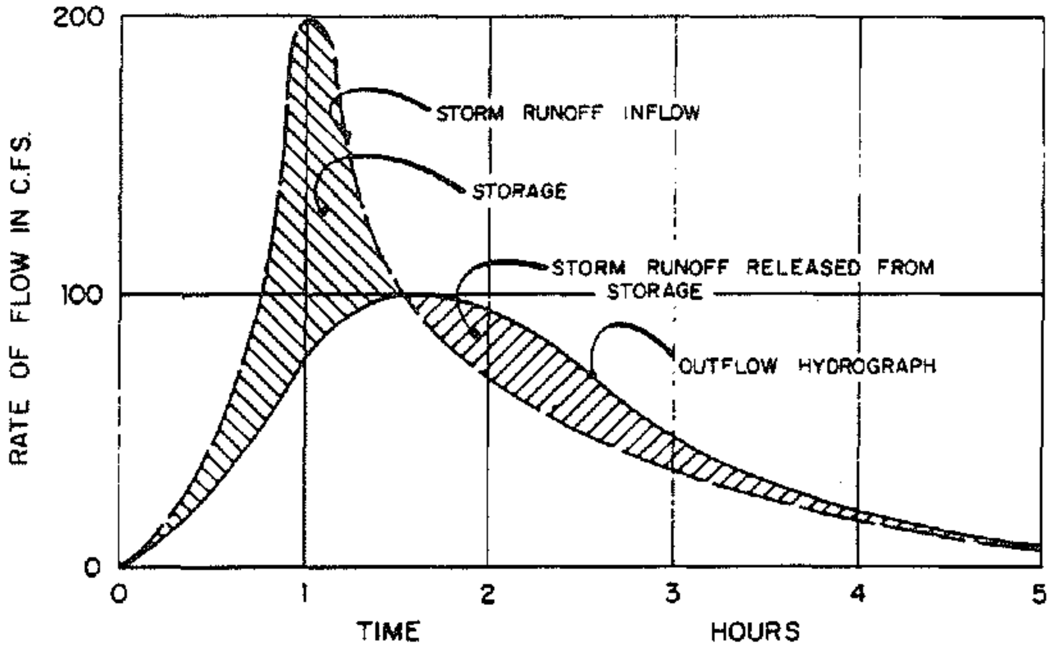


FIGURE VI-2 EFFECT OF ONSTREAM RESERVOIR ON STORM RUNOFF HYDROGRAPH

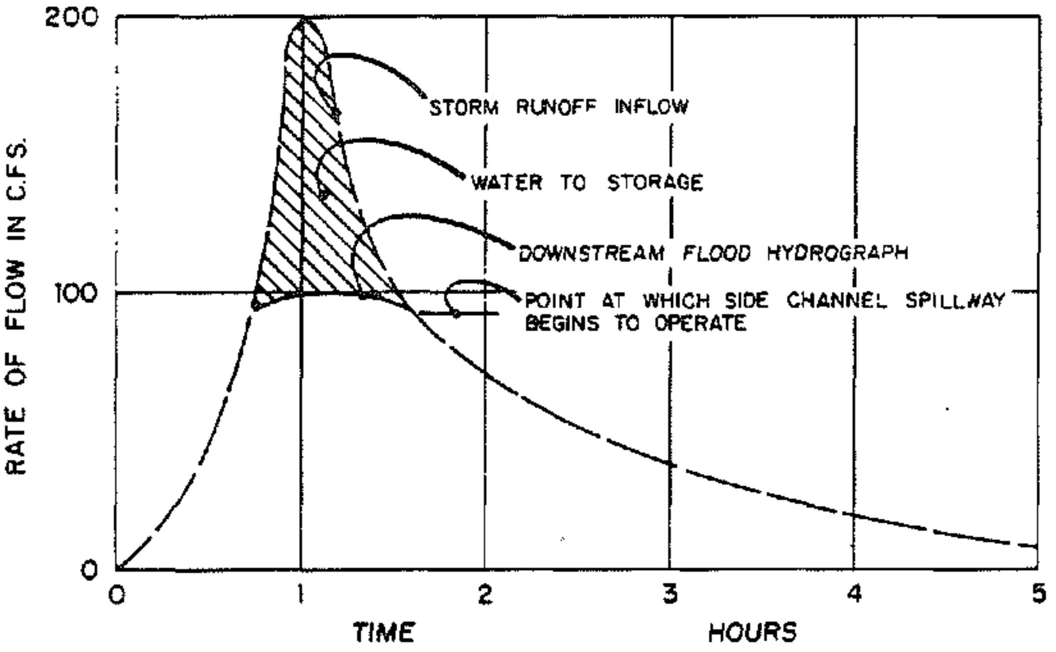


FIGURE VI-3 EFFECT OF OFFSTREAM RESERVOIR ON STORM RUNOFF HYDROGRAPH primarily concerned with the initial drainage system. The outlet capacity would then be substantially less than the inflow from the intial runoff. It cannot be overemphasized, however, that the use of downstream channel storage requires competent planning and design to obviate the creation of an unnecessary hazard which would result from haphazard planning. Spillway criteria must include use of the maximum probable storm runoff, which is much larger than the major runoff.

Reservoir Routing

For larger ponds and reservoirs, it is desirable to study the routing of the storm runoff inflow through the storage area in greater detail. For routing procedures and techniques, the reader is referred to publications describing the methods (3) (4).

DESIGN CRITERIA

Design criteria vary considerably, depending on the topography, soil characteristics, the degree and type of use of the area by the public, accessibility by the public, and the degree of maintenance desired. There are few set criteria applicable to every site; however, some guidelines are articulated in this Section to assist the designer. The various guidelines will be discussed by the general categories of hard surface or vegetated surface.

Drain Time

This is an important factor that varies widely depending on the use of the area by the public. Because man-made storage, as applicable to Stillwater drainage projects, is most effective on high-intensity rainfall events, the maximum drain time should be 24 hours, unless successive beneficial use of the storm water is part of the project. In this case, the effects of successive storms must be evaluated and the storage volume increased if the downstream hazard is significant.

Hard Surface Storage

This category refers to roofs, plazas, storage areas, and parking lots. The drain time for roofs, storage areas, and storage parking lots (i.e. parking for a car dealership) is less stringent than are plazas and parking lots frequented by the public. For the former category, a drain time of 2 to 4 hours is reasonable. For the latter category, a drain time from 1 to 2 hours is reasonable. The designer should consider that maximum depths are attained infrequently, usually an average of once each 100 years. Further, the hard surface detention facilities should be designed such that snow melt and storm runoff from small events does not pond.

Vegetated Surface Storage

This type of storage can range from open space and passive recreation areas to high intensity recreation areas. In the former case, the detention time can range up to 24 hours or longer if successive use of the storm water is desired. Further, there are greater opportunities to attenuate lower frequency runoff events as well as the design runoff event (usually the 100-year event) and this provision should be incorporated into the design.

For detention facilities where high intensity recreation uses are contemplated, the drain time can range up to 8 hours, although 4 hours is frequently used. To attain greater depths (covered later) and to increase detention time, passive recreation areas may be incorporated into a detention facility which is otherwise intended for high intensity recreation use.

Water Depth

Because some bottom slope is required to completely drain detention facilities and to pass lesser runoff events, storage facilities will rarely have uniform depth. The following guidelines refer to average depth, and the designers should arrange the detention facilities such that the minimum depth in the facility is near to where the public will have the most immediate access.

Hard Surface Storage. Table VI-2 lists the maximum and normal average depths for the various types of uses where detention ponding is normally located. The maximum depth of ponding refers to the depth of water at a low point, typically for draining the detention pond. Except for roof top ponding, in both instances it is assumed that a particular use area is not fully covered by stormwater thereby allowing movement through any area during and after a runoff event.

TABLE VI-2

MAXIMUM AND NORMAL DEPTHS OF PONDING HARD SURFACE DETENTION FACILITIES

Type of Area	Average	Maximum
Roofs	3	4*
Parking Lots	6	9
Storage Areas	6	12
Plazas	3	6
Storage Parking Lots	6	12

* Greater maximum ponding depths are possible, however, the structural design of the roof must account for the greater loading.

Vegetated Surface Storage. For passive recreational and open space areas, there are no limits as to depths which are more logically determined by topography and the storage volume required.

For high intensity recreation areas, the maximum allowable average depth is 5 feet. In instances where this criteria requires too much land be acquired to attain the required storage volume, it is recommended that terracing be used. The high intensity recreation activities can be located on the highest level where the maximum depth criteria can be met.

Side Slopes

This subject generally only applies to vegetated surface storage. In open space and passive recreation areas, the steepness of side slopes is governed by side slope stability as determined by soils investigation.

High intensity recreation areas require side slopes be no steeper than of 4:1 (4 horizontal to 1 vertical) where grass is to be maintained and 3:1 in non-grassed areas. In addition, both types of areas need one area no steeper than 10:1 to allow for the entrance and exit of maintenance vehicles.

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NATURAL STORAGE

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NATURAL STORAGE

An understanding of natural storage phenomena and mechanisms is of assistance to the planner and engineer when they are involved in man-made storage analyses for urban drainage purposes. This Chapter presents a brief description of some of the types of natural storage and how they function.

Natural storage of precipitation and runoff exists in all natural settings, and to different degrees. Some of the more important types of natural storage are:

- Interception of precipitation by vegetation,
- Infiltrtaion of precipitation through the soil surface where it is retained or detained,
- Retention in surface depressions,
- Land surface detention storage,
- Ponds and lakes,
- Floodplain storage.

The effects of urbanization result in a modification of the natural storage capability of each of the types, often eliminating its storage component entirely. Compensation for these effects is the usual purpose for creating man-made storage.

The effects of reducing, or eliminating natural storage in a drainage system can be drastic. For example, studies by the U.S. Corps of Engineers in an Oklahoma city indicated that a concrete-channel alternative rather than a slow moving natural channel would more than double peak flows downstream. Likewise, studies by private consultants on another stream in Oklahoma demonstrated that a similar channelization, limited to specific parts of the urbanizing basin, would result in peak flow increases on the order of 50%.

VEGETAL INTERCEPTION

The effect of vegetal interception of rainfall is more significant percentage-wise, for smaller storms than large storms. Nevertheless, vegetal interception plays an important role in detaining water at the point of rainfall occurence even for the more infrequent event. Interception by some types of cover amounts to considerable portion of the annual rainfall.

When the first drops of rain strike the leaves of vegetation, they are retained as droplets or as a thin film over the surface of the leaves. Only a small portion of the rain reaches a point on the ground until such time as all the leaves overhead have retained their maximum amount of stored water. Some portion of the water which does spatter earthward is evaporated before it reaches the ground. When a leaf has acquired its maximum surface storage,added water causes drops to form on the lower edge. Each of these drops grows in size, and when gravity overbalances surface-tension forces, the drops fall to the ground or to a lower leaf.

After the vegetation is completely saturated, the net interception would be zero, were it not for the fact that even during rain there is evaporation from the enormous wet surface of the foliage. Thus, after interception storage has been filled, the amount of water reaching the soil surface is equal to the rainfall less the evaporation from the vegetal cover. At the cessation of rain, the vegetation still retains the interception storage. This water is eventually returned directly to the atmosphere through evaporation.

Infiltration

Detailed data on infiltration characteristics of the soils in the Stillwater area are presented in the Hydrology Chapter of this Manual. This discussion is to provide a description of the mechanisms involved with storage of precipitation via infiltration through the surface of the ground.

While infiltration has an important influence on the rainfall-runoff relationship, it also plays an important role in determining soil moisture, evapotranspiration of vegetation, and ground water recharge. It is an important parameter for the flood hydrologist, but also to other disciplines.

Important factors affecting infiltration rates of soils are soil character and the suspended soilds in the water. There is a three step sequence with infiltration consisting of surface entry, percolation, and depletion of the water in the soil zone.

The surface of the soil may be sealed by the inwash of fines or other arrangements of particles that prevent or retard the entry of water into the soil. Soil having excellent underdrainage may be sealed at the surface and thereby have a low infiltration rate.

Water cannot continue to enter the soil more rapidly than it is transmitted downward. Conditions at the surface cannot increase infiltration unless the transmission capacity of the soil profile is adequate.

Transmission rates may vary for successive horizons of the soil profile. A surface horizon may become compacted by wet-weather traffic, or it may be naturally impermeable because of its texture and structure.

After saturation, the rate of infiltration is limited to the lowest transmittal rate encountered by the infiltrating water up to that time. This concept may be visualized by assuming a theoretical soil profile in which three master horizons, like layers of a cake, are identified by the letters, A, B, and C, in the order of the depth. The B-horizon has a rate of transmission lower than that of the A- and C-horizons and a surface-entry rate higher than the transmission rate of any horizon. Infiltration will then equal the transmission rate of the A-horizon until the available storage in the A-horizon is exhausted. Thereafter infiltration will be limited to the limited to the transmission rate of the B-horizon. The transmission rate of the C-horizon is not at its capacity, since the B-horizon of the theoretical soil profile is the least permeable layer.

If the surface-entry rate is slower than the transmission capacity of any horizon, infiltration is limited to the surface-entry rate throughout an entire storm.

The storage available in any horizon depends on porosity, thickness of the horizon, and the amount of moisture already present. Texture, structure, organic-matter content, biologic activity, root penetration, colloidal swelling, and many other factors determine the nature and magnitude of the porosity of the soil horizon. Total porosity, as well as the size and arrangement of pores, has a significant bearing upon the availability of storage. The infiltration that occurs in the early part of the storm will largely be controlled by the volume, sizes, and continuity of noncapillary or large pores, because such pores provide easy paths for the movement of percolating water.

Storage capacity may directly affect infiltration rates during the storm. When infiltration rates are controlled by transmission rates through soil strata, the infiltration rates will diminish as storage above a restricting stratum is exhausted.

Factors that affect infiltration are the characteristics of the permeable medium, soil, and the characteristics of the percolating fluid. The problem concerns itself largely with pore size and pore-size distribution, that is, the proportion of different sizes present, as well as their relative stability during storms, irrigations, or other applications of water. In sands, the pores are relatively stable, since the sand particles that form them are not readily disintegrated, nor do they swell upon wetting. During a storm, they may rearrange themselves into a more dense mix than formerly. However, this change in condition of the sand is relatively slow when compared with changes that occur in silts or clays.

Soils with appreciable amounts of silt or clay are subject during a storm to the disintegration of the crumbs or aggregates which in their dry state may provide relatively large pores. These soils also normally contain more or less colloidal material, which in most cases swells appreciably when wet. Thus a deterioration in permeability of the mass is much more readily accomplished than in sands. The impact of raindrops break down soil crumbs, there is a melting of aggregates, and the very small particles of silt and clay float across the surface and penetrate previously existing pores, thus clogging them and greatly reducing infiltration.

Vegetation is one of the most significant factors affecting surface entry of water. Vegetation, or mulches, protect the soil surface from rainfall impact. Massive root systems such as sods perforate the soil, keeping it unconsolidated and porous. The organic matter from crops promotes a crumb structure and improves permeability. On the other hand, vegetation, such as a row crop, gives less protection from impact, depending upon the stage of growth, and the root systems perforate only small portions of the soil profile and the accompanying tillage reduces permeability.

Forest litter, crop residues, and other humus materials also protect the soil surface. High biotic activity in and beneath such layers opens up the soil, resulting in high entrance capacities.

Improved infiltration is well demonstrated in many studies of forest hydrology also, since most forest soils are richer in organic matter than cultivated ones. A soil with an old established grass cover, like pasture land or prairie, is likewise more permeable, all other things being equal.

In brief, the characteristics of the permeable medium are affected primarily by the kind of soil, its texture, its structure, the amount and kind of clay and colloid that it contains, the depth and thickness of its more permeable layers, and its prior history of land use.

Depression Storage

Depression storage is an important factor affecting both rural and urban rainfall-runoff relationships, though it is not necessarily specifically identified by the hydrologist.

Depression storage is that precipitation or runoff which is retained in puddles, ditches, and other soil surface depressions. They are closed drainages ranging from very small microdepressions to flooded flat areas of several acres.

As soon as rainfall intensity at the soil surface exceeds the infiltration capacity, the rainfall excess begins to fill surface depressions. An understanding of the sequence of events which takes place after the beginning of rainfall excess requires recognititon of the following facts:

- Each depression has its own capacity or maximum depth.
- As each depression is filled to capacity, further inflow is balanced by outflow plus infiltration and evaporation.
- Depressions of various sizes are both superimposed and interconnected. In other words, every large depression encompasses many interconnected smaller ones.
- Each depression, until such time as it is filled, has a definite drainage area of its own.

Almost immediately after the beginning of rainfall excess, the smallest depressions become filled and overland flow begins. Most of this water in turn fills larger depressions, but some of it follows an unobstructed path to the stream channel. This chain of events continues, with successively larger portions of overland flow contributing water to streams, until such time as all depression storage within the basin is filled. Water held in depressions at the end of rain is either evaporated or absorbed by the soil through infiltration. The process is repeated each time a runoff-producing storm occurs.

Generally speaking, the capacity of depressions varies inversely with surface slope.

The rate at which depression storage is extracted from rainfall excess is a function of the volume of excess up to the time under consideration. The first small increment of rainfall excess is almost entirely lost to the depressions, while greater portions of later increments contribute to runoff.

Surface Detention Storage

Surface detention storage in the natural hydrologic regime is that component which the urban drainage planner can most readily emulate in his work. This is transient storage. It effects the time of concentration, i.e., the speed of runoff.

Detention Storage

Soon after rainfall excess begins, a thin sheet of water builds up over the soil surface and overland flow takes place. Water in temporary storage as a sheet over the basin, known as surface detention, is not to be confused with depression storage, which does not contribute to the flood hydrograph. Detention depths increase until discharge reaches equilibrium with the rate of supply to surface runoff. The type of flow occurring at any given point within an area of overland flow depends upon such factors as viscosity, discharge, and degree of roughness.

Any natural soil surface is uneven, and consequently detention depth and slope change from point to point. Flow is apparently always laminar close to the divide, but the portion of the area covered by turbulent flow increases downslope toward the channel because of increased depth and velocity. Part of the energy is expended on the vegetation, and the quantity so expended increases with detention depth until depth exceeds the height of the vegetation. Vegetation also reduces the effective cross-sectional area.

The longer this pattern of overland flow from the point of runoff occurence to the channel, the longer the time it will take for the water molecules to reach a given downstream point, and the less will be the rate of flow, or the flood peak.

Detention storage under natural conditions of a watershed, or under agricultural conditions, helps store significant volumes of water in the upper reaches of a drainage basin which stretches out the runoff hydrograph over a longer period of time.

Ponds and Lakes

The beneficial effect of existing ponds and lakes and runoff characteristics is well understood. Even though farm ponds are not natural storage, they pre-exist urbanization and therefore are considered when evaluating predevelopment runoff conditions.

Floodplain Storage

This type of natural storage is potentially the easiest to maintain during the urbanization process because of the existing legal and institutional mechanism in Stillwater.

Floodplain storage is transient storage. It represents overbank storage where the water flows slowly, thus increasing the time it takes for the water to reach a downstream point. As a result, the peak flow rate is less.

The benefit of floodplain storage is most apparent where the designer is also considering channelization of the stream and, therefore, he must study peak flows under each alternative. Often, the peak rate of flow for this design flood will be increased significantly for the channelization alternative over and above that for the floodplain alternative. The channel discharge rate may be double in some instances.

The economic benefit of floodplain storage becomes apparent when one estimates the cost of constructing new compensatory storage reservoirs. By channelizing a stream and eliminating the floodplain, a measurable flood storage volume is lost. The cost of constructing a storage reservoir of equal size provides a fiscal measuring stick for the floodplain storage lost to channelization. But then, the new facilities do not provide the same continuing benefits for larger floods, as does the floodplain.

Evaluations of Natural Storage

The various types of natural storage should not be destroyed in the urbanization process and then be replaced with only one man-made type of major storage facility. This is too much of an abrupt change in the hydrologic regime.

The urban drainage strategy should be based on the maintenance of as many of the natural storage components as reasonable, and then to replace those that are lost, or to replace the portions that are lost.

For any new development in Stillwater, whether it be typical single family subdivisions, apartments, commercial office parks, or industrial development, natural storage can be either maintained or duplicated if the land planner is sensitive and caring.

In those cases where the constraints are too severe, and only a portion of the natural storage volume can be maintained and/or duplicated, offsite compensating storage can be included in the form of overflow recreational parks, gravel pits, behind road fills, or sub-regional detention storage.

A drainage plan which is based on slow response time between rainfall and runoff has been given a good starting point. It will be one which has a much better chance of being a good plan than those done in the traditional manner of speeding urban runoff off of each lot and into the street or storm sewer.

SUMMARY

The description of the components of natural storage, and their beneficial effect on the rainfall runoff relationship has been described for the urban drainage planner and engineer. An understanding of natural storage functioning is important so that in planning for man-made storage of all types of the planner may have as his objectives the duplication of natural storage impacts on the storm runoff phenomena.

Perhaps the clearest message to come out of an understanding of natural storage is the need for upstream (on-site) detention near where the rainfall occurs, and the value of natural channel and floodplain flow characteristics.

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CULVERT DESIGN

Culverts have a wide range of application in urban drainage which ranges from road crossings of slight depressions or for roadside ditches to major crossings of drainage channels. The procedures and design aids contained herein are largely obtained from the Federal Highway Administration and its predecessor agencies. The reader is referred to these publications (1, 3, 8, 9, and 10) for more detailed discussions of this subject.

Corrugated metal pipe (CMP) culverts are to be used only for minor drainage facilities (i.e., under driveways crossing roadside channels) and for temporary installations for major drainage. In the former situation, the entrance and exits must have headwalls or end sections, or be beveled. CMP culverts are not to be used for permanent installations on major drainages.

Concrete culverts for major drainages must have end-sections, improved entrances (described later), or headwalls. Concrete culverts do not require headwalls or end-sections for driveway crossings of roadside channels.

HYDRAULICS

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 he is familiar with the following equations:

Manning $Q = \frac{1.49}{n} AR^{2/3}S^{1/2}$ Eq. VIII-1

Continuity $Q = V_1A_1 = V_2A_2$ Eq. VIII-2

Energy $\frac{V^2}{2g} + \frac{P}{w} + Z_c$ + losses = constant Eq. VIII-3

A culvert is defined as a conduit for the free passage of surface drainage water under a highway, railroad, canal, or other embankment.

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 which must be evaluated to determine the carrying capacity of the culverts consist of inlet losses, friction losses and exit losses.

Inlet Losses. For inlet losses, the governing equations are:

$$Q = CA \quad 2gh_e \qquad \text{Eq. VIII-4}$$

$$H_e = K_e \frac{V^2}{2g} \qquad \text{Eq. VIII-5}$$

where K_e is the entrance loss coefficient.

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.

Friction Losses. Friction loss for pipes flowing full can be determined from

$$H_f = f \frac{L}{D} \frac{V^2}{2g} \qquad \text{Eq. VIII-6}$$

where f = friction factor; L = Length of culvert; D = Diameter of culvert barrel; and $v^2/2g$ = velocity head of the flow in the pipe. 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.

Energy Gradient and Hydraulic Gradeline

Figures VIII-1 and VIII-2 illustrate the energy gradeline and hydraulic grade line and related terms.

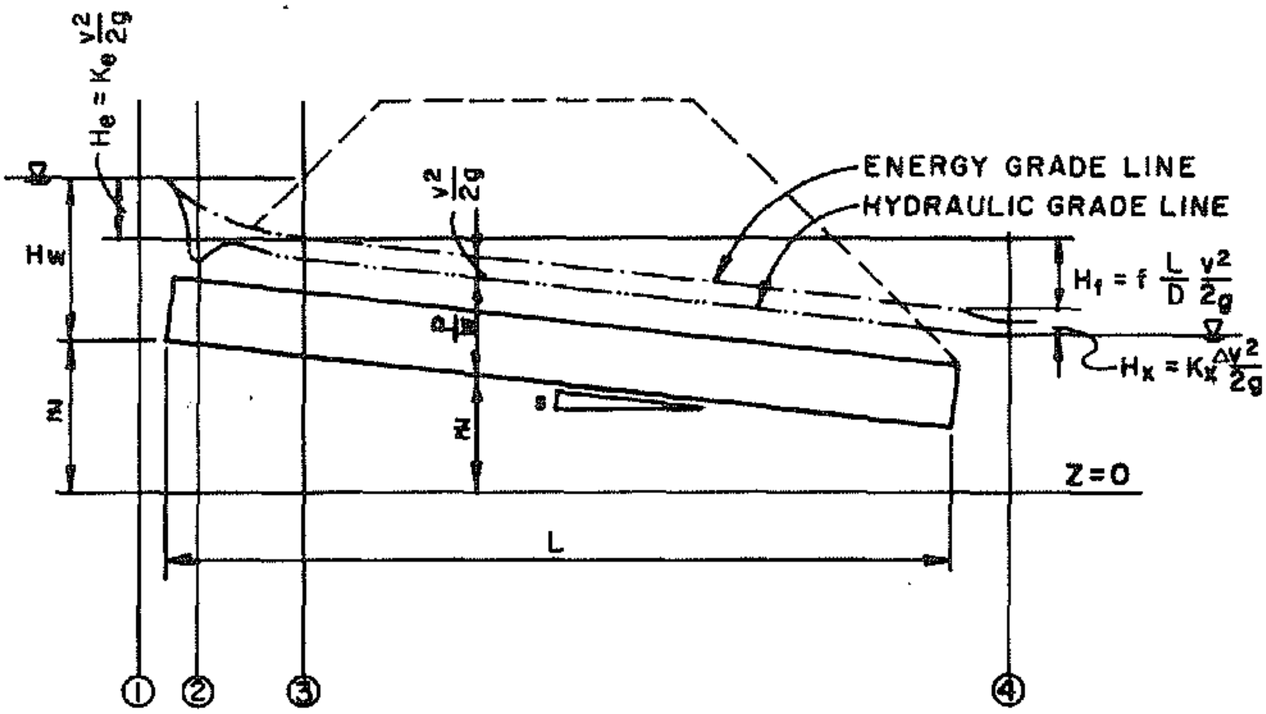


FIGURE VIII-1
DEFINITION OF FORMS FOR CLOSED CONDUIT FLOW

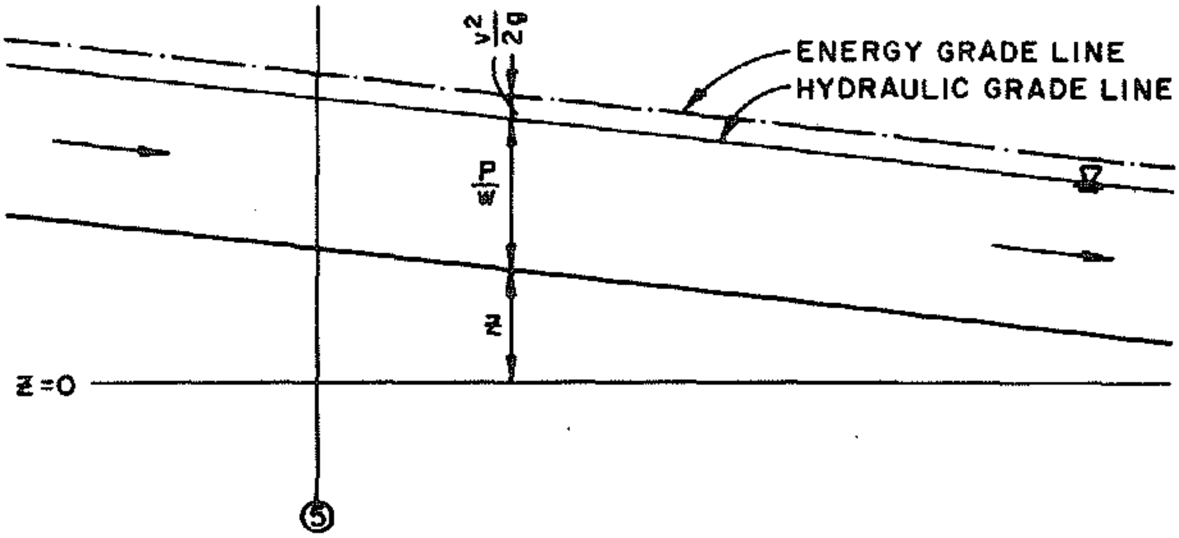


FIGURE VIII-2
DEFINITION OF FORMS FOR OPEN CHANNEL FLOW

The energy gradeline (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 H_L/L, where H_L equals the total energy loss over the distance L. The hydraulic gradeline (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 gradeline is the same as the water surface. For pressure flow in conduits P/w is the pressure head and the hydraulic gradeline falls above the top of the conduit as long as the pressure relative to atmospheric pressure is positive.

Hydraulics of Culverts

Approaching the entrance to a culvert as at point 1 of Figure VIII-1, the flow is essentially uniform and the hydraulic gradeline and energy gradelines 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 VIII-2, open channel flow is established and the hydraulic gradeline is the same as the water surface.

Inlet Control. 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 VIII-3.

The most common occurrence of inlet control is when the headwater submerges the top of the culvert, Figure VIII-4, and the pipe does not flow full. A culvert flowing under inlet control is defined as a hydraulically short culvert.

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. In outlet control, the discharge is a function of the inlet losses, the headwater depth, the culvert roughness, the culvert length, the barrel diameter, the culvert slope, and sometimes tailwater elevation.

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 sub-critical, Figure VIII-5. The most common condition exists when the culvert is flowing full, Figure VIII-6. A culvert flowing under outlet control is defined as a hydraulically long culvert.

Throat, Face, and Crest Control. These controls refer to areas of possible culvert control at the entrance for improved entrances. These will be discussed in detail later in this Chapter.

CULVERT INLETS

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. On large culverts, a significant savings may be possible by improving the entrance and reducing the barrel size.

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

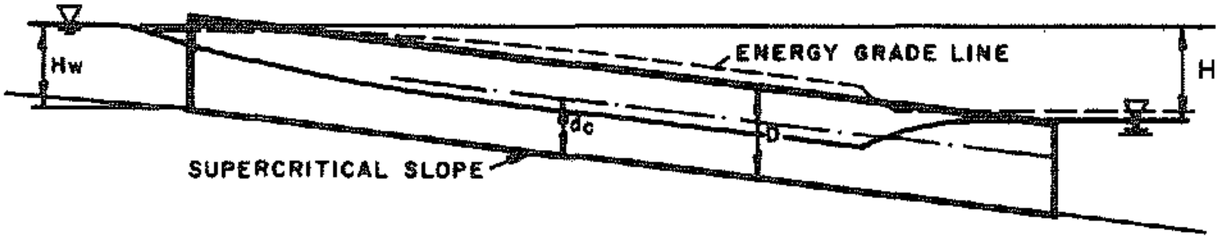
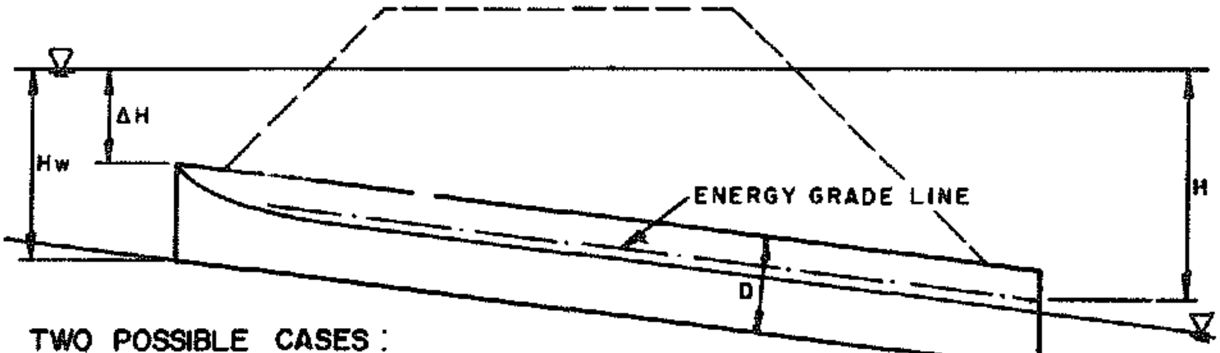


FIGURE VIII-3

INLET CONTROL UNSUBMERGED INLET



TWO POSSIBLE CASES:

a) SUPERCRITICAL SLOPE.

b) SUBCRITICAL SLOPE.

FIGURE VIII-4

INLET CONTROL SUBMERGED INLET

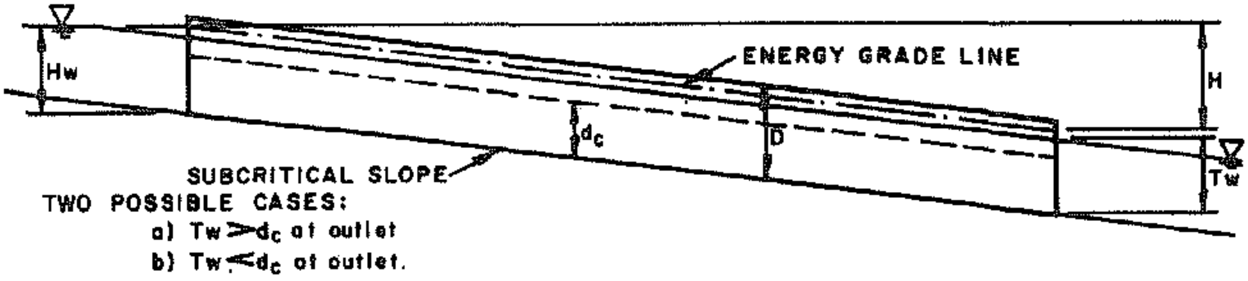


FIGURE VIII-5

OUTLET CONTROL-PARTIALLY FULL CONDUIT

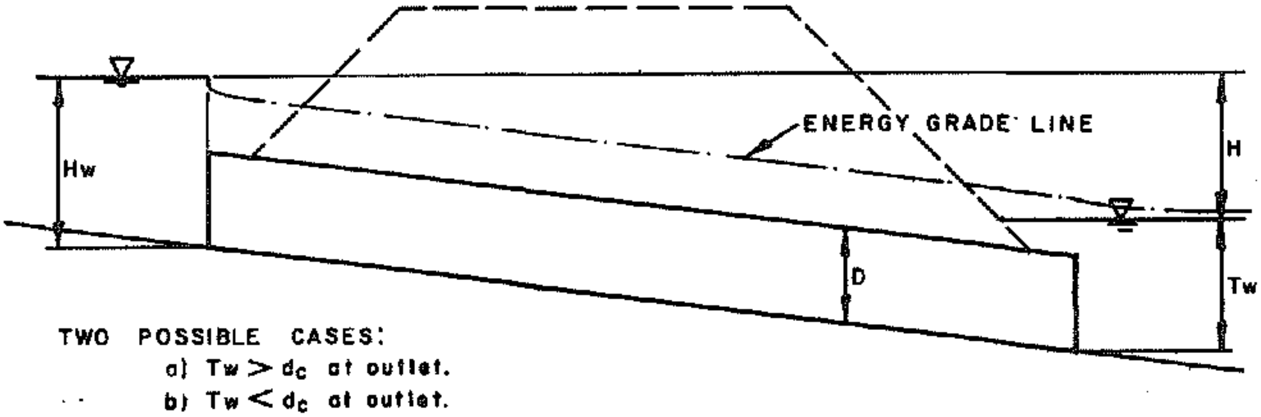


FIGURE VIII-6

OUTLET CONTROL-FULL CONDUIT

to reduce a storm runoff peak. For this case, an inefficient inlet may be the most desirable choice.

The inlet types described in this Chapter may be selected to fulfill either of the aforementioned requirements depending on the topography or conditions imposed by the designer. The entrance coefficient, Ke as defined by equation VIII-5 is a measure of the hydraulic efficiency at the inlet type, with lower valves indicating greater efficiency.

Inlet coefficients recommended for use are given in Table VIII-1.

Projecting Inlets

Projecting inlets vary greatly in hydraulic efficiency and adaptability to requirements with the type of pipe material used. Figure VIII-7 illustrates this type of inlet.

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.

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.

Discussion of Projecting Inlets. 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 vary with the type of material used.

Corrugated metal pipe 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

TABLE VIII-1

OUTLET CONTROL, FULL OR PARTLY FULL

Entrance head loss $H_e = k_e \frac{V^2}{2g}$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, sq. 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
Mitered 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
Mitered 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 manufactures. From limited hydraulic test 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. These latter sections can be designed using the information given for the beveled inlet.

crews to work around the inlet. Projecting inlets are not allowable on CMP culverts. 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 embankments 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.

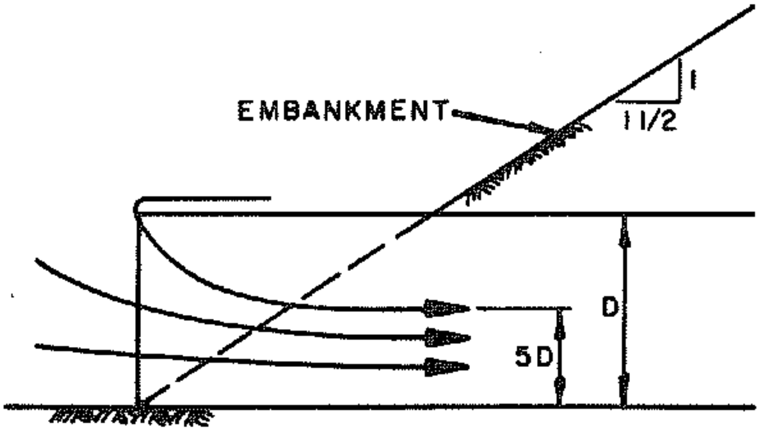
Inlets with Headwalls

Headwalls may be used for a variety of reasons, as 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 VIII-8 illustrates a headwall with wingwalls.

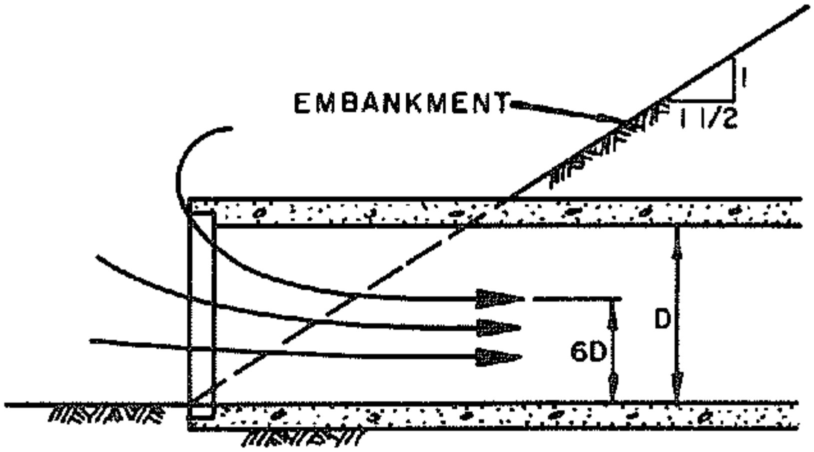
Corrugated Metal Pipe. Corrugated metal pipe in a headwall is essentially a square-edged entrance with an entrance coefficient of about 0.4. The entrance losses may be reduced by rounding the entrance. The entrance coefficient may be reduced to 0.15 for a rounded edge with a radius equal to 0.15 times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

Concrete Pipe. For tongue and groove, 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.

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 efficiency is realized with the use of wingwalls, regardless of the pipe material used and, therefore, the use should be justified for reasons other than an increase in hydraulic efficiency. Figure VIII-9 illustrates several cases where wingwalls are



PROJECTING THIN-WALL PIPE ENTRANCE



PROJECTING CONCRETE GROOVED END PIPE ENTRANCE

FIGURE VIII-7
COMMON PROJECTING CULVERT INLETS

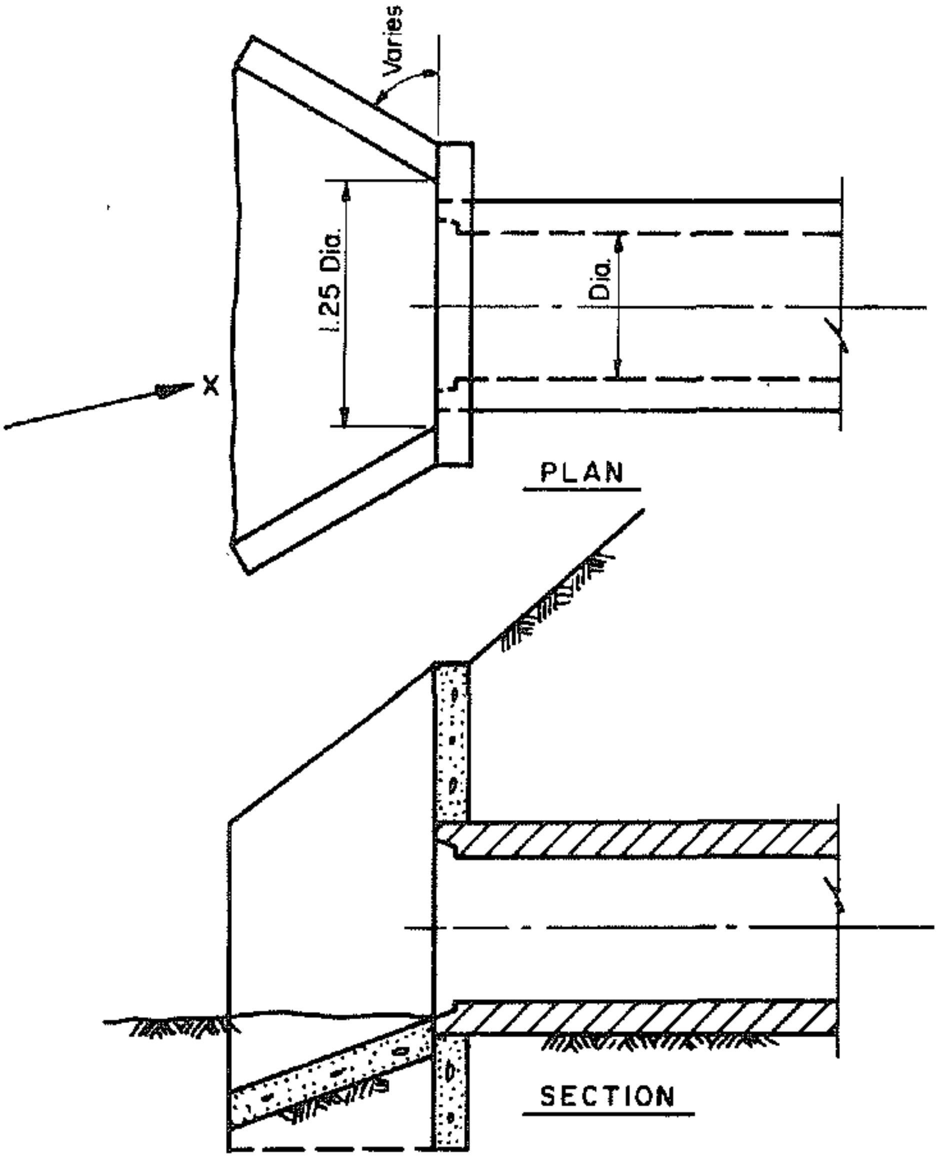
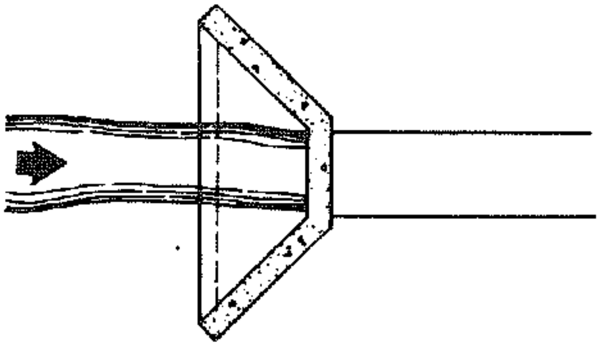
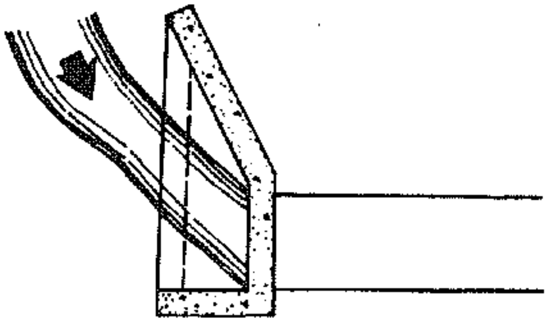


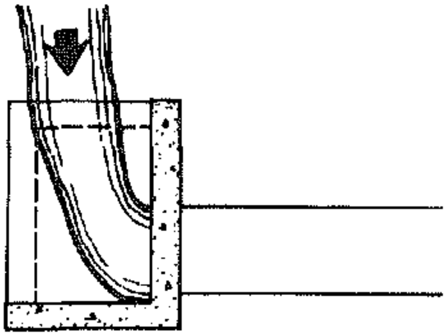
FIGURE VIII - 8
INLET WITH HEADWALL & WINGWALLS



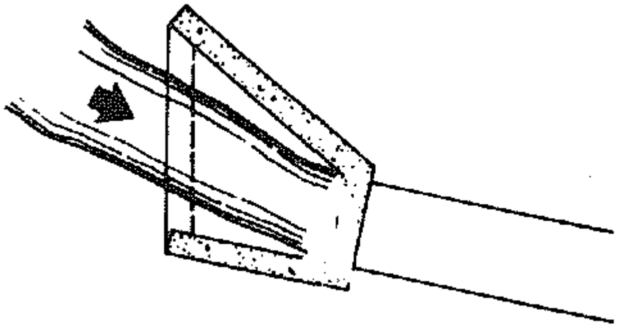
FLOW NORMAL TO EMBANKMENT



FLOW SKEWED TO EMBANKMENT



FLOW PARALLEL TO EMBANKMENT



FLOW AND CULVERT SKEWED TO EMBANKMENT

FIGURE VIII-9

TYPICAL HEADWALL WINGWALL CONFIGURATIONS used. For parallel wingwalls, the minimum distance between wingwalls should should be at least 1.25 times the diameter of the culvert pipe.

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 VIII-9, the actual configuration of the wingwalls varies according to the direction of flow and will also vary according to the topographical requirement placed upon them.

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.

End-Sections and Mitered Entrances

There are a great variety of inlets other than the common ones described. Among these are special end-sections which serve as both outlets and inlets and are available for both corrugated metal pipe 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.

Corrugated Metal Pipe. Special end-sections for corrugated metal pipe 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 corrugated metal pipe with a headwall may be used.

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 (Ke) 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 (Ke) is equal to 0.20.

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 practice usually does not conform with the side slopes, giving essentially a projecting entrance (Ke = 0.7).

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.

Improved Inlets

These improvements represent inlet geometry refinements beyond those normally used in conventional culvert design practice, such as those discussed above. Several degrees of improvements are presented, including bevel-edged, side-tapered, and slope-tapered inlets.

Bevel-Edged Inlets. The first degree of inlet improvement is a beveled edge. The bevel is proportioned based on the culvert barrel or face dimension and operates by decreasing the flow contraction at the inlet. A bevel is similar to a chamfer except that a chamfer is smaller and is generally used to prevent damage to sharp concrete edges during construction. Adding bevels to a conventional culvert design with a square-edged inlet increases culvert capacity by 5 to 20 percent.

As a minimum on major drainage facilities, bevels should be used on all culverts which operate in inlet control, both conventional and improved inlet types. The exception to this is circular concrete pipes where the socket end performs much the same as a beveled edge. Examples of bevels used in conjunction with other improved inlets are shown in Figures VIII-10 and VIII-11. Culverts flowing in outlet control cannot be improved as much as those in inlet control, but the entrance loss coefficient, K_e, is reduced from 0.5 for a square edge to 0.2 for beveled edges.

Side-Tapered Inlets

The second degree of improvement is a side-tapered inlet (Figure VIII-10). It provides an increase in flow capacity of 25 to 40 percent over that of a conventional culvert with a square-edged inlet. This inlet has an enlarged face area with the transition to the culvert barrel accomplished by tapering the sidewalls. The inlet face has the same height as the barrel, and its top and bottom are extensions of the top and bottom of the barrel. The intersection of the sidewall tapers and barrel is defined as the throat section.

Side-tapered inlets of other configurations were tested, some with tops tapered upward but with sidewalls remaining an extension of the barrel walls, and others with various combinations of side and top tapers. Each showed some improvement over conventional culverts, but the geometry shown in Figure VIII-10 produced superior performance.

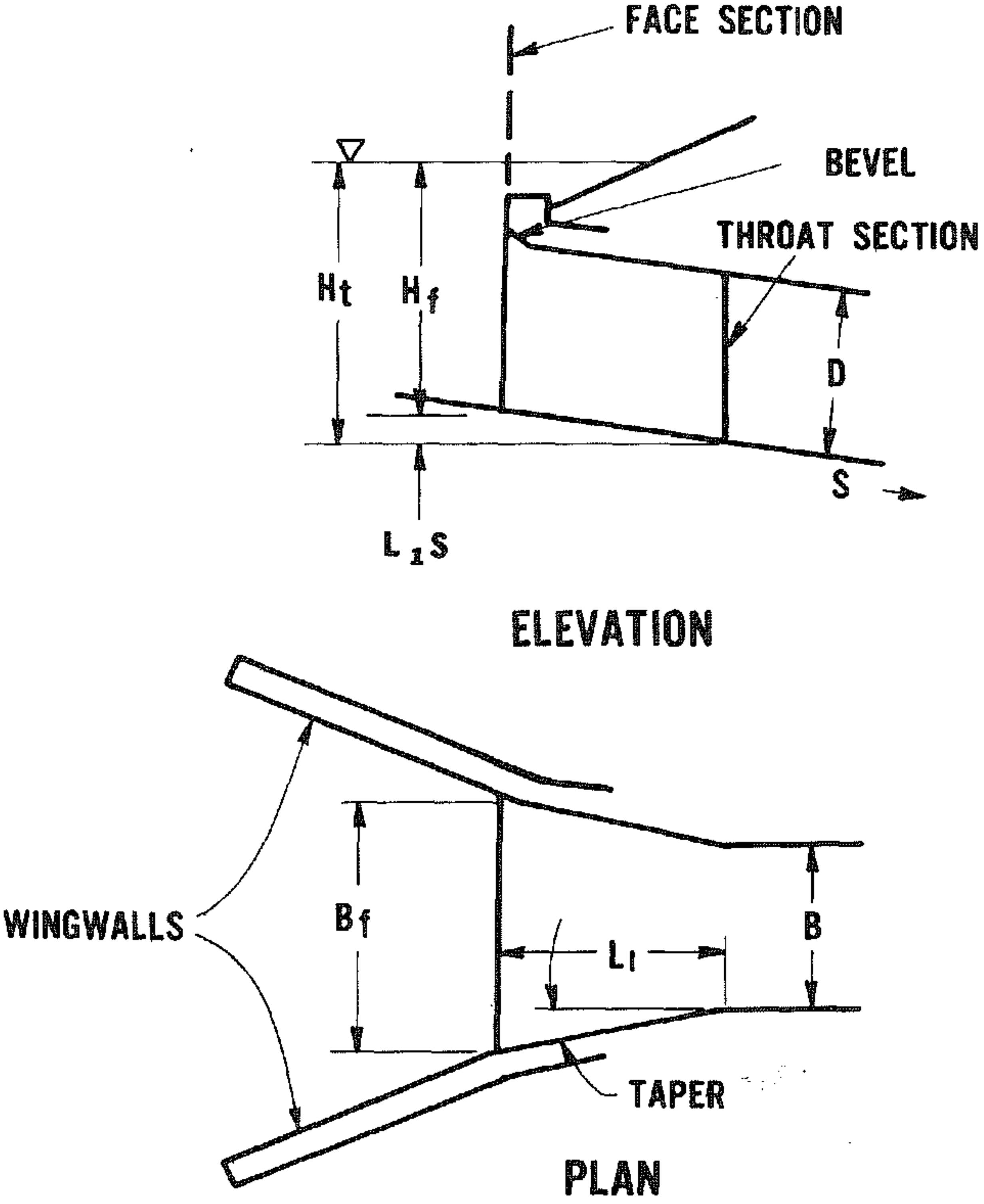


FIGURE VIII - 10
SIDE TAPERED INLET

For the side-tapered inlet, there are two possible control sections: the face and throat. H_f, as shown in Figure VIII-10, is the headwater depth based upon face control. H_t is the headwater depth based upon throat control.

The advantages of a side-tapered inlet operating in throat control are: the flow contraction at the throat is reduced, and for a given pool elevation, more head is applied at the throat control section. The latter advantage is increased by utilizing a slope-tapered inlet or a depression in front of the side-tapered inlet.

Slope-Tapered Inlets

A slope-tapered inlet is the third degree of improvement. Its advantage over the side-tapered inlet without a depression is that more head is available at the control (throat) section. This is accomplished by incorporating a FALL in the enclosed entrance section (Figure VIII-11).

This inlet can have over 100 percent greater capacity than a conventional culvert with square edges. The degree of increased capacity depends largely upon the amount of FALL available between the invert at the face and the invert at the throat section. Since this FALL may vary, a range of increased capacities is possible.

Slope-tapered inlets of alternate designs were considered and tested during the research. The inlet shown in Figure VIII-11 is recommended on the basis of its hydraulic performance and ease of construction. As a result of the FALL concentrated between the face and the throat of this inlet, the barrel slope is flatter than the barrel slope of a conventional or side-tapered structure at the same site.

Both the face and throat are possible control sections in a slope-tapered inlet culvert. However, since the major cost of a culvert is in the barrel portion and not the inlet structure, the inlet face should be designed with a greater capacity at the allowable headwater elevation than the throat.

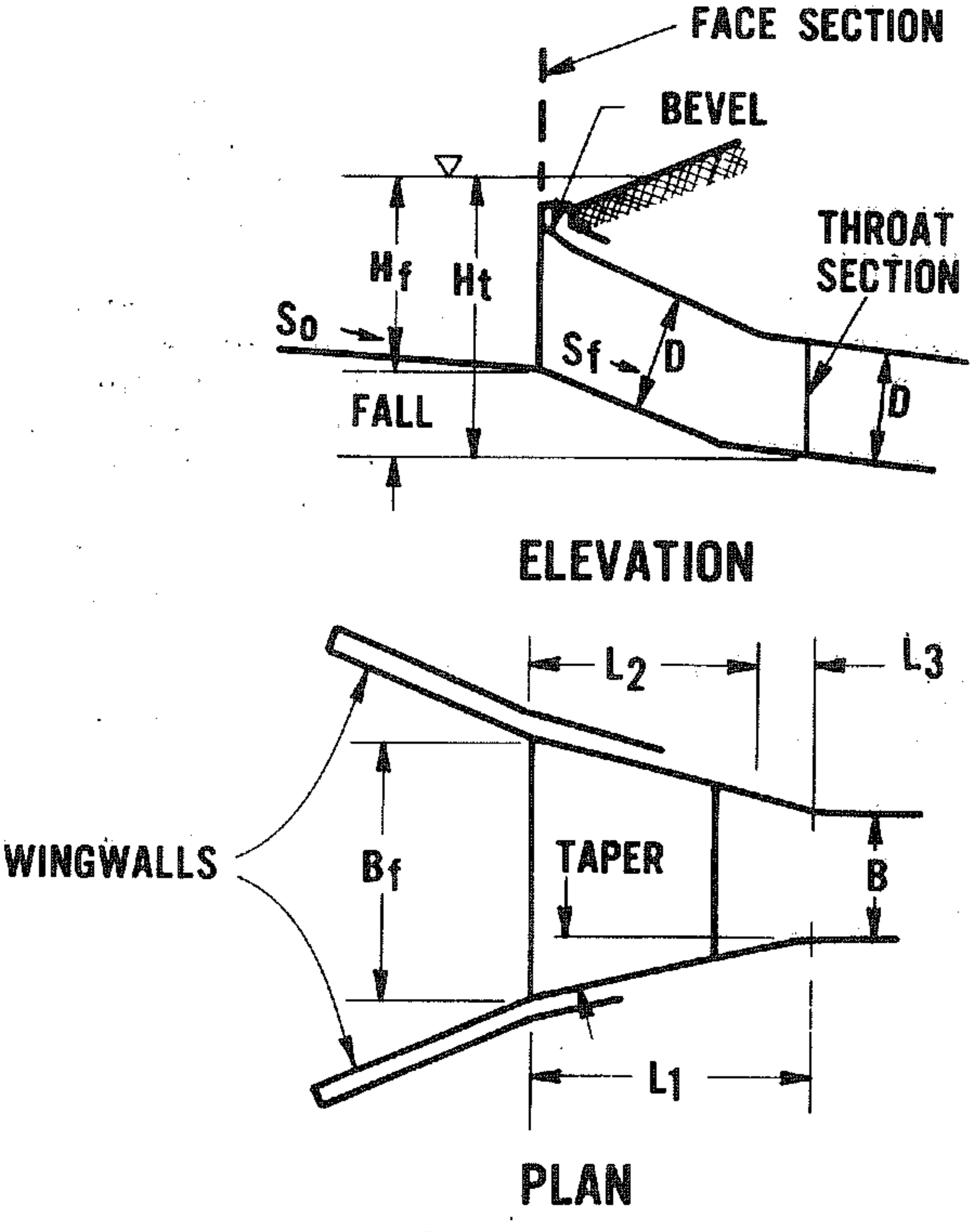


FIGURE VIII-11

SLOPE-TAPERED INLET

This insures that flow control will be at the throat and more of the potential capacity of the barrel will be utilized.

DESIGN CONCEPTS

For small diameter or hydraulically "short" culverts, it is unlikely that improved inlets will be required, however, for culverts of larger size, improved inlets will be particularly useful in reducing cost by reducing barrel size. The design procedure described in this Chapter is identical regardless of the ultimate entrance type, beginning from unimproved entrances and progressing to the highest degree of improvement.

The design charts from HEC-10, Capacity Charts for the Hydraulic Design of Highway Culverts have not been included. Their use is limited to free drop outfalls and inlet control. The former case occurs rarely in Stillwater, and the latter case can be determined from other graphs contained in this Chapter. Table VIII-2 lists the design charts contained in this Chapter, the type of conduit, boundary conditions, and what type of control (inlet, outlet, face, throat, and crest) to which the chart is applicable. Face, throat, and crest control apply to improved entrances. Charts for improved entrances are applicable only to box culvert or round conduits. Standard entrances will be used for other culvert geometries, and inlet/outlet control computations for standard entrances are all that can be accomplished for other than round or box culvert geometry.

Because the normal outlet and inlet control computations for any culvert are an integral part of the design process, a description of the specific information and design parameters for improved inlets follows. For small culverts, the designer can proceed to design procedure.

Performance Curves

To understand how a culvert at a particular site will function over a range of discharges, a performance curve, which is a plot of discharge versus headwater depth or elevation, must be drawn. For side-tapered and slope-tapered inlets, it is necessary to compute the performance of the face section (face control curve), the throat section (throat control curve), and

TABLE VIII-2
SUMMARY OF CULVERT DESIGN CHARTS

CHART	DESCRIPTION	CONTROL
VIII-1	Concrete Box Culverts	Outlet
VIII-2	Concrete Pipe Culverts	Outlet
VIII-3	Oval Concrete Pipe Culverts, Long Axis Vertical or Horizontal	Outlet
VIII-4	C.M. Pipe Culverts	Outlet
VIII-5	Structural Plate, C.M. Pipe Culverts	Outlet
VIII-6	C.M. Pipe-Arch Culverts	Outlet
VIII-7	Structural Plate, C.M. Pipe Arch Culverts, 18-inch Corner Radius	Outlet
VIII-8	Concrete Box Culverts	Inlet
VIII-9	Concrete Box Culverts, 90° Headwall, Beveled Inlet Edges	Inlet
VIII-10	Single Barrel Box Culverts, Skewed Headwalls, Beveled Inlet Edges	Inlet
VIII-11	Concrete Box Culverts, Flared Wingwalls, Top Beveled Edge	Inlet
VIII-12	Concrete Pipe Culverts	Inlet
VIII-13	Oval Concrete Pipe Culverts, Long Axis Horizontal	Inlet
VIII-14	Oval Concrete Pipe Culverts, Long Axis Vertical	Inlet
VIII-15	C.M. Pipe Culverts	Inlet
VIII-16	C.M. Pipe-Arch Culverts	Inlet
VIII-17	Circular Pipe with Beveled Ring	Inlet
VIII-18	Concrete Box Culverts, Side & Slope Tapered Inlet	Throat

TABLE VIII-2, Continued

CHART	DESCRIPTION	CONTROL
VIII-19	Concrete Box Culverts, Side-tapered	Face
VIII-20	Concrete Box Culverts, Slope-tapered	Face
VIII-21	Pipe Culverts, Side Tapered, Circular Sections only	Throat
VIII-22	Pipe Culverts, side-tapered, Circular Sections only	Face
VIII-23	All culverts	Crest

the barrel (outlet control curve), in order to develop the culvert performance curve for a range of discharges. In the lower discharge range, face control governs, in the intermediate range, throat control governs, and in the higher discharge range, outlet control governs.

Performance curves should always be developed for culverts with side-tapered or slope-tapered inlets to insure that the designer is aware of how the culvert will function over a range of discharges, especially those exceeding the design discharge. It should be recognized that there are uncertainties in the various methods of estimating flood peaks, that larger floods than the design event occur, and that there is a chance that the design frequency flood will be exceeded during the life of the project. Culvert designs should be evaluated in terms of the potential for damage to the highway and adjacent property from floods greater than the design discharge.

As alternate culverts are possible using improved inlet design, a performance curve should be plotted for each alternate considered. The performance curve will provide a basis for selection of the most appropriate design.

The advantages of various improved inlet designs are demonstrated by the performance curves shown in Figure VIII-12. These curves represent the performance of a single 6 feet by 6 feet reinforced concrete box culvert 200 feet long, with a 4-foot difference in elevation from the inlet to the outlet. For a given headwater, the culvert can convey a wide range of discharges, depending on the type of inlet used.

Curves 1 through 4 are inlet control curves for a 90° wingwall with a square-edged inlet, a 1.5:1 bevel-edged inlet, a side-tapered inlet, and a slope-tapered inlet with minimum FALL, respectively. Curves 5 and 6 are outlet curves. Curve 5 is for the square-edged inlet and curve 6 is for the other three inlet types. As previously discussed, curves 5 and 6 show that improved entrances can increase the performance of a culvert operating in outlet control, but the improvement is not as great as for culverts operating in inlet control, as demonstrated by curves 1 through 4.

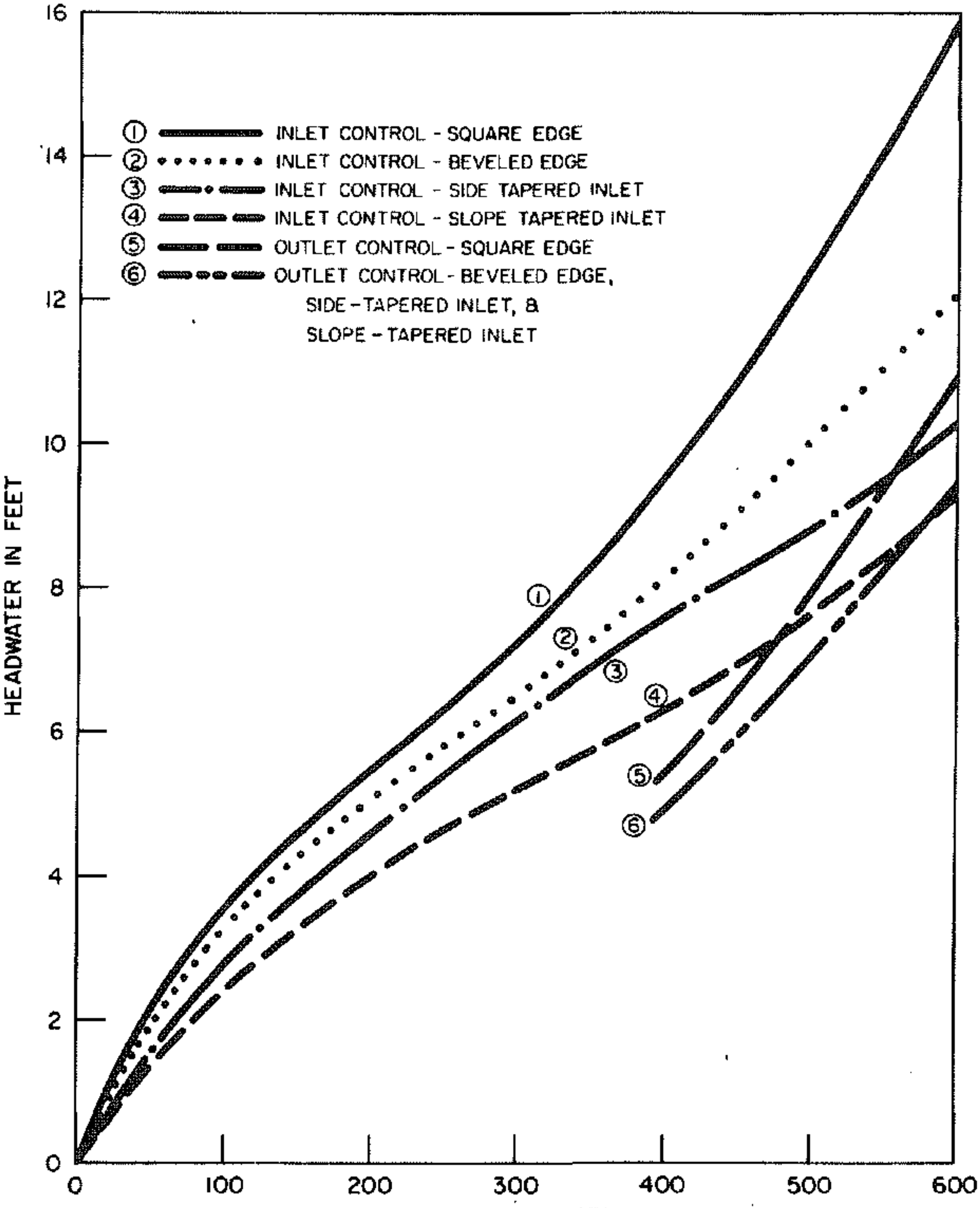


FIGURE VIII-12

PERFORMANCE CURVES FOR
SINGLE 6' X 6' BOX CULVERT
90 DEGREE WINGWALL

Table VIII-3 compares the inlet control performance of the different inlet types. It shows the increase in discharge that is possible for a headwater depth of 8 feet. The bevel-edged inlet, side-tapered inlet and slope-tapered inlet show increases in discharge over the square-edged inlet of 16.7, 30.4 and 55.6 percent, respectively. It should be noted that the slope-tapered inlet incorporates only the minimum FALL of D/4. Greater increases in capacity are often possible if a larger FALL is used.

TABLE VIII-3

COMPARISON OF INLET PERFORMANCE AT
CONSTANT HEADWATER FOR 6 FT. x 6 FT. RCB

Inlet Type	Headwater	Discharge	% Improvement
Square-edge	8.0'	336 cfs	0
Bevel-edge	8.0'	392 cfs	16.7
Side-tapered	8.0'	438 cfs	30.4
*Slope-tapered	8.0'	523 cfs	55.6

* Minimum FALL in inlet = D/4 = 1.5 ft.

The performance curves in Figure VIII-12 illustrate how inlet geometry affects the capacity of a given culvert. The practical use of performance curves to compare the operation of culverts of various sizes and entrance configurations for a given discharge are discussed in detail in the subsequent sections.

In improved inlet design, the inverts of the face sections for the different types of improved inlets fall at various locations, depending on the design chosen. Therefore, it is difficult to define a datum point for use in comparing the performance of a series of improved inlet designs. The use of elevations is suggested, and this concept is used in the design procedure section of this Chapter. The example problem performance curves are plots of discharge versus required headwater elevations. Allowable headwater is also expressed as an elevation.

BOX CULVERTS IMPROVED INLET DESIGN

Beveled-Edged Inlets

Four inlet control charts for culverts with beveled edges are included in this Circular: Chart VIII-9 for 90° headwalls (same as 90° wingwalls), Chart VIII-10 for skewed headwalls, Chart VIII-11 for wingwalls with flare angles of 18 to 45 degrees, and Chart VIII-17 for circular pipe culverts with beveled rings. Instructions for the use of nomographs are given later in this Chapter. Note that Charts VIII-9 through VIII-11 apply only to bevels having either a 33° angle (1.5:1) or a 45° angle (1:1). For example, the minimum bevel dimension for an 8 ft. x 6 ft. box culvert designed using Chart 8 for a 1:1 bevel, or 45° angle, would be d = 6 ft. x 1/2 in/ft = 3 in. and b = 8 ft. x 1/2 in/ft = 4 in. Therefore, the top bevel would have a minimum height of 3 inches and the side bevel would be 4 inches in width. Similar computations would show that for a 1.5:1 or 33.7° angle, d would be 6 in. and b would be 8 in.

The design charts in this Chapter are based on research results from culvert models with barrel width, B, to depth, D, ratios of from 0.5:1 to 2:1.

Multibarrel Installations. For installations with more than one barrel, the nomographs are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size. For example, in a double 8 ft. by 8 ft. box culvert, the top bevel is proportioned based on the height, 8 ft., and the side bevels proportioned based on the clear width, 16 feet. This results in a dimension, for the top bevel of 4 in. for the 1:1 bevel, and 8 in. for the 1.5:1 bevel and a b dimension for the side bevels of 8 inc. for the 1:1 bevel and 16 in. for the 1.5:1 bevel. The ratio of the inlet face area to the barrel area remains the same as for a single barrel culvert.

For multibarrel installations exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width, B, or three times the height, whichever is smaller. The top bevel dimension should always be based on the culvert height. Until further research information becomes available, the design charts in this circular may be used to estimate the hydraulic performance of these installations.

The shape of the upstream edge of the intermediate walls of multibarrel installations is not as important to the hydraulic performance of a culvert as the edge conditions of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to act as debris fins as suggested in HEC no. 9 (13).

It is recommended that Chart VIII-10 for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in underdesign due to a greatly reduced capacity. Skewed inlets should be avoided whenever possible, and should not be used with side- or slope-tapered inlets.

Side-Tapered Inlets

The selected configurations of the side-tapered inlet are shown in Figures VIII-13 and VIII-14. The barrel and face heights are the same except for the addition of a top bevel at the face. Therefore, the enlarged area is obtained by making the face wider than the barrel and providing a tapered sidewall transition from the face to the barrel. Side taper ratios may range from 6:1 to 4:1. The 4:1 taper is recommended as it results in a shorter inlet.

The throat and the face are possible flow control sections in the side-tapered inlet. The weir crest is a third possible control section when a FALL is used. Each of the possible control sections should be sized to pass the design discharge without exceeding the allowable headwater elevation.

Throat Control. In order to utilize more of the available culvert barrel area, the control at design discharge generally should be at the throat rather than at the face or crest. Chart VIII-18 presents the headwater depth, referenced to the throat invert, required to pass a given discharge

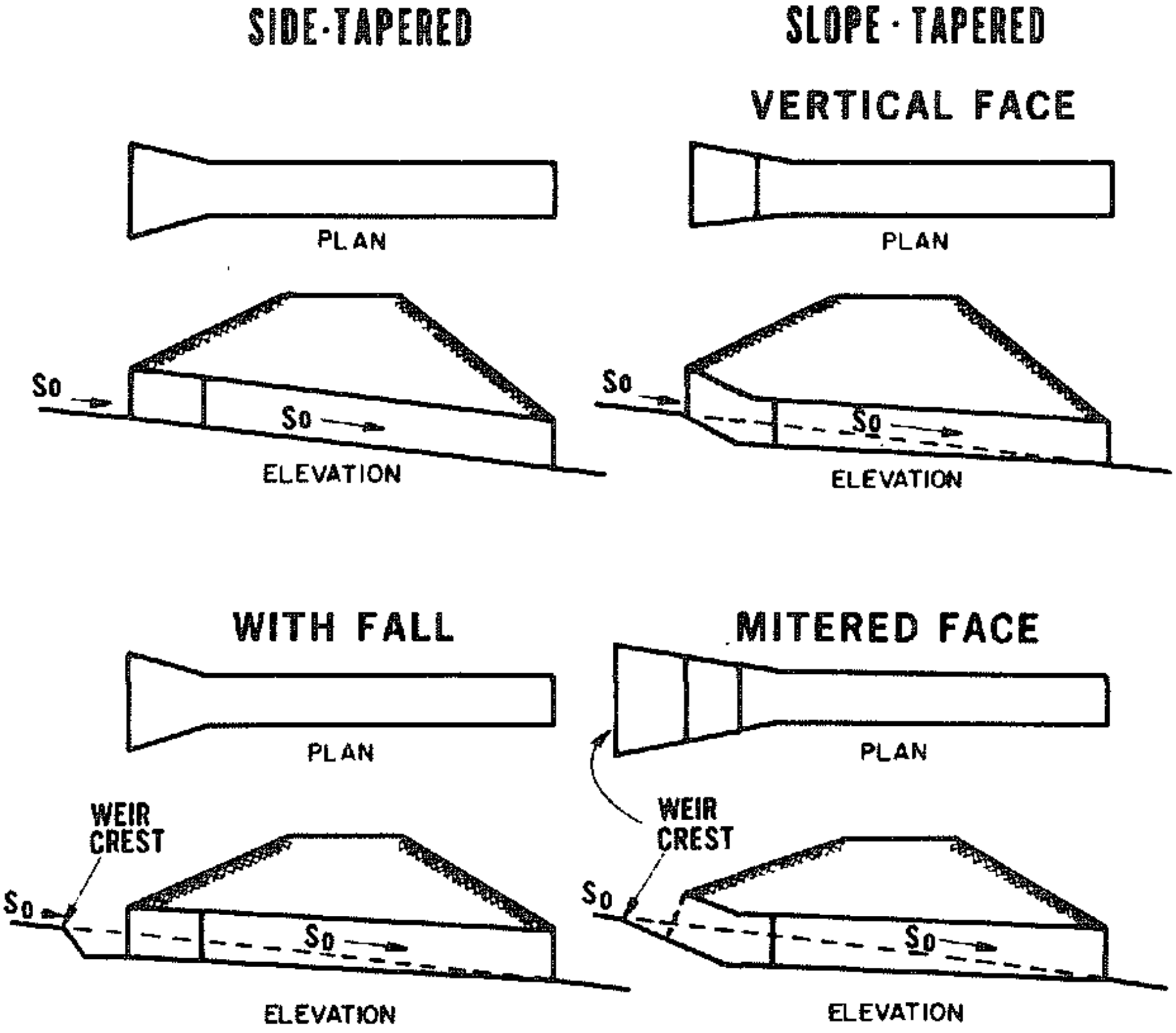


FIGURE VIII-13

TYPES OF IMPROVED INLETS FOR BOX CULVERTS

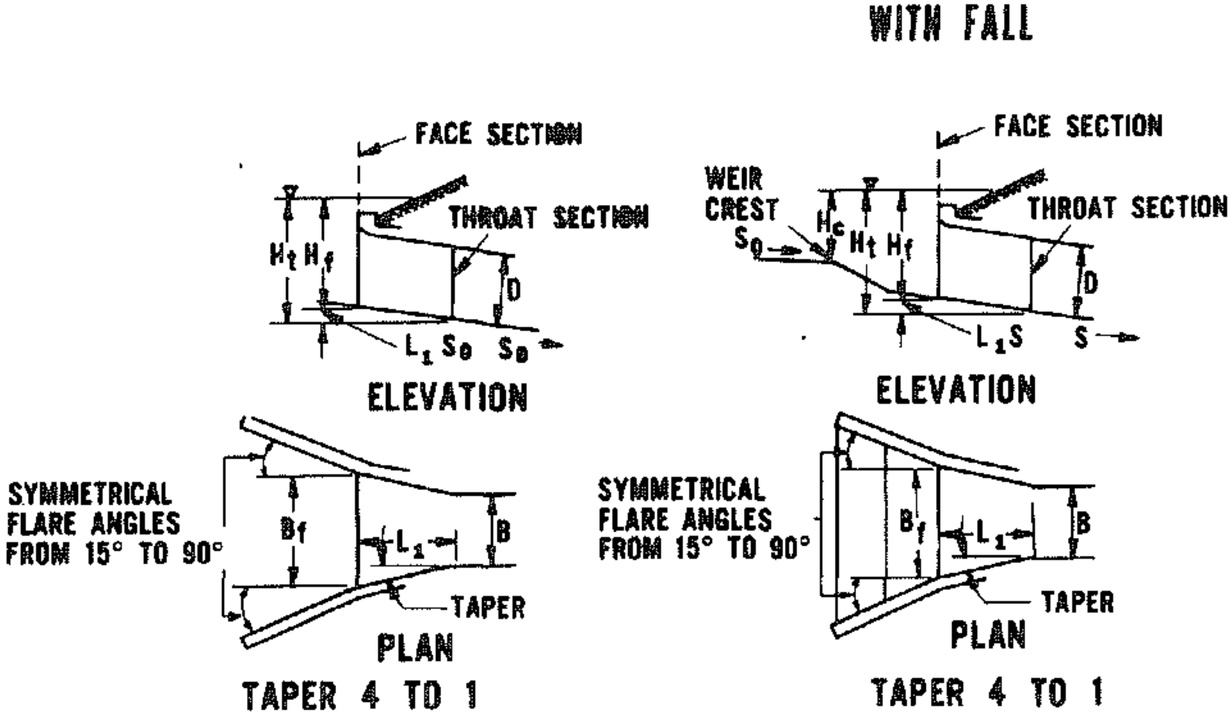
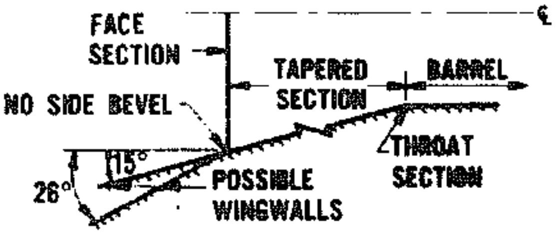
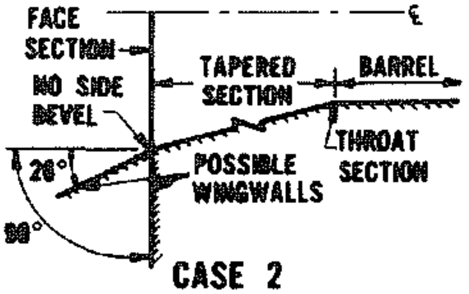


FIGURE VIII-14
IMPROVED INLETS SIDE-TAPERED

DASHED CURVE

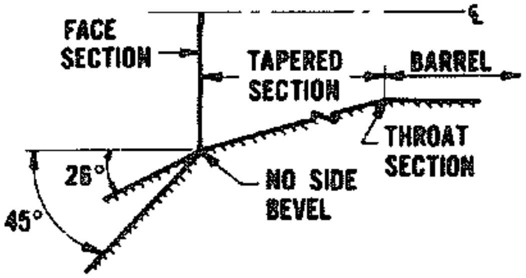


CASE 1
15° TO 26° WINGWALL FLARE ANGLES
TOP BEVEL REQUIRED
NO SIDE BEVEL REQUIRED

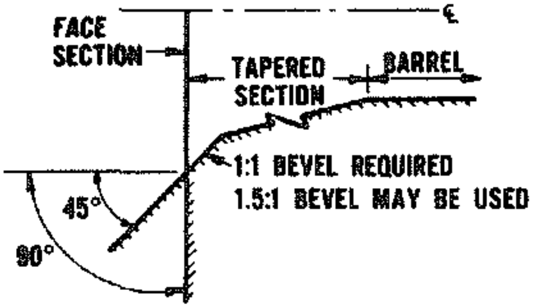


CASE 2
26° TO 90° WINGWALL FLARE ANGLES
NO TOP BEVEL REQUIRED
NO SIDE BEVEL REQUIRED

SOLID CURVE



CASE 1
26° TO 45° WINGWALL FLARE ANGLES
TOP BEVEL REQUIRED
NO SIDE BEVEL REQUIRED



CASE 2
45° TO 90° WINGWALL FLARE ANGLES
TOP BEVEL REQUIRED
SIDE BEVEL REQUIRED

FIGURE VIII-15

DEFINITION OF CURVES ON FACE CONTROL DESIGN CHARTS 19 AND 20 for side- or slope-tapered inlets operating in throat control. This chart is in a semi-dimensionless form, H_t/D plotted against $Q/BD^{3/2}$. The term, $Q/BD^{3/2}$, is not truly dimensionless, but is a convenient parameter and can be made non-dimensional by dividing by the square root of gravitational acceleration, $g^{1/2}$. A table of $BD^{3/2}$ values is contained in this Section.

Face Control. Design curves for determining face width are provided in Chart VIII-19. Both the inlet edge condition and sidewall flare angle affect the performance of the face section. The two curves in Chart VIII-19 pertain to the options in Figure VIII-15. The dashed curve, which is less favorable, applies to the following inlet edge conditions.

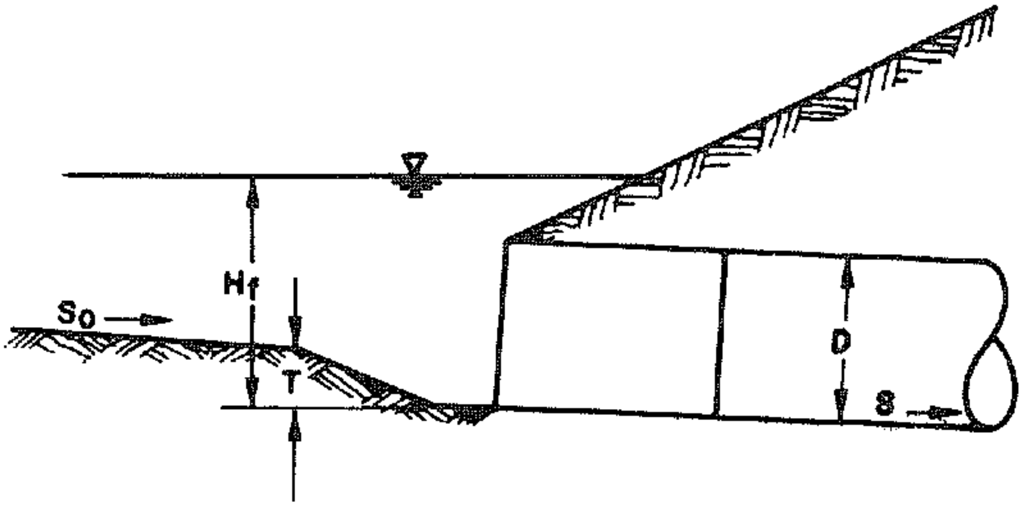
(1) wingwall flares of 15° to 26° and a 1:1 top edge bevel, and
(2) wingwall flares of 26° to 90° and square edges (no bevels). A 90° wingwall flare is commonly termed a headwall.

The more desirable solid curve applies to the following entrance conditions.

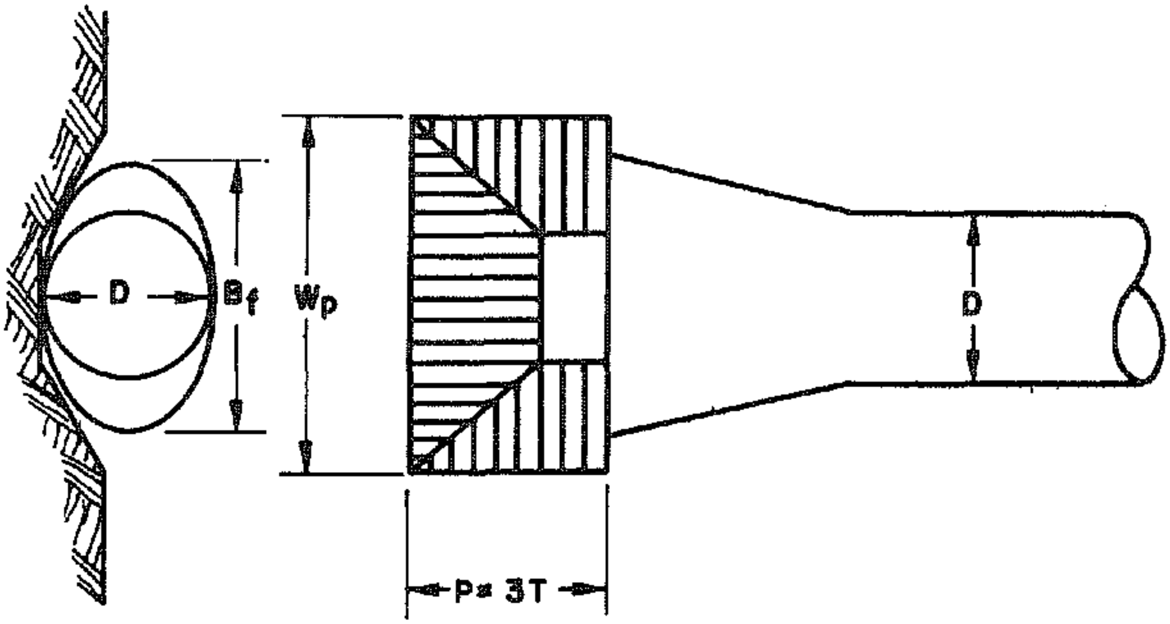
(1) wingwall flares of 26° to 45° with a 1:1 top edge bevel, or
(2) wingwall flares of 45° to 90° with a 1:1 bevel on the side and top edges.

Note that undesirable design features, such as wingwall flare angles less than 15°, or 26° without a top bevel, are not covered by the charts. Although the 1.5:1 bevels can be used, due to structural considerations, the smaller 1:1 bevels are preferred.

Use of FALL Upstream of Side-Tapered Inlet. A depression may be utilized upstream of the face of a side-tapered inlet. As illustrated in Figures VIII-13 and VIII-14, the depression may be constructed in various ways, as an extension of the wingwalls, or by a paved depression similar to that used with side-tapered pipe culvert inlets, shown in Figure VIII-16. The only requirements are: the plant of the invert of the barrel be extended upstream from the inlet face a minimum distance of D/2, to provide a smooth flow transition into the inlet, and the crest be long enough to avoid undesirably high headwater from crest control at design discharges. Chart



ELEVATION



PLAN

$W_p = B_f + T$ or 4T WHICH EVER IS LARGER

FIGURE VIII-16

SIDE-TAPERED INLET WITH CHANNEL
DEPRESSION UPSTREAM OF ENTRANCE

VIII-23 may be used for checking crest control if the fall slope is between 2:1 to 3:1. The length of the crest, W, may be approximated, neglecting flow over the sides of sloping wingwalls. This provides a conservative answer.

Performance Curves. Figure VIII-17 illustrates the design use of performance curves and shows how the side-tapered inlet can reduce the barrel size required for a given discharge.

The hatched performance curve is for a double 6 ft. x 5 ft. box culvert with a side-tapered inlet with no FALL upstream. It is a composite of the throat and face control curves. The outlet control curve was also computed, but falls outside of the limits of the figure. This indicates that further increases in capacity or reduction in headwater are possible. Face control governs to a discharge of 375 cfs, and throat control for larger discharges. Thus, the barrel dimensions (throat size) control the designs at high discharges, which should always be the case. In this example, the size of the culvert was reduced from a double 7 ft. x 6 ft. box to a double 6 ft. x 5 ft. for the same allowable headwater. Use of an upstream FALL would reduce the barrel size still further to a size comparable to that required with a slope-tapered inlet.

Double Barrel Design. Double barrel structures may be designed with improved inlets. The face is proportioned on the basis of the total clear width as described for bevels.

The center wall is extended to the face section with either a square, rounded, chamfered, or beveled edge treatment. A side-wall taper from 4:1 to 6:1 may be used.

The face width, as determined from Chart VIII-19, is the total clear face width needed. The width of the center wall must be added to this value in order to size the face correctly.

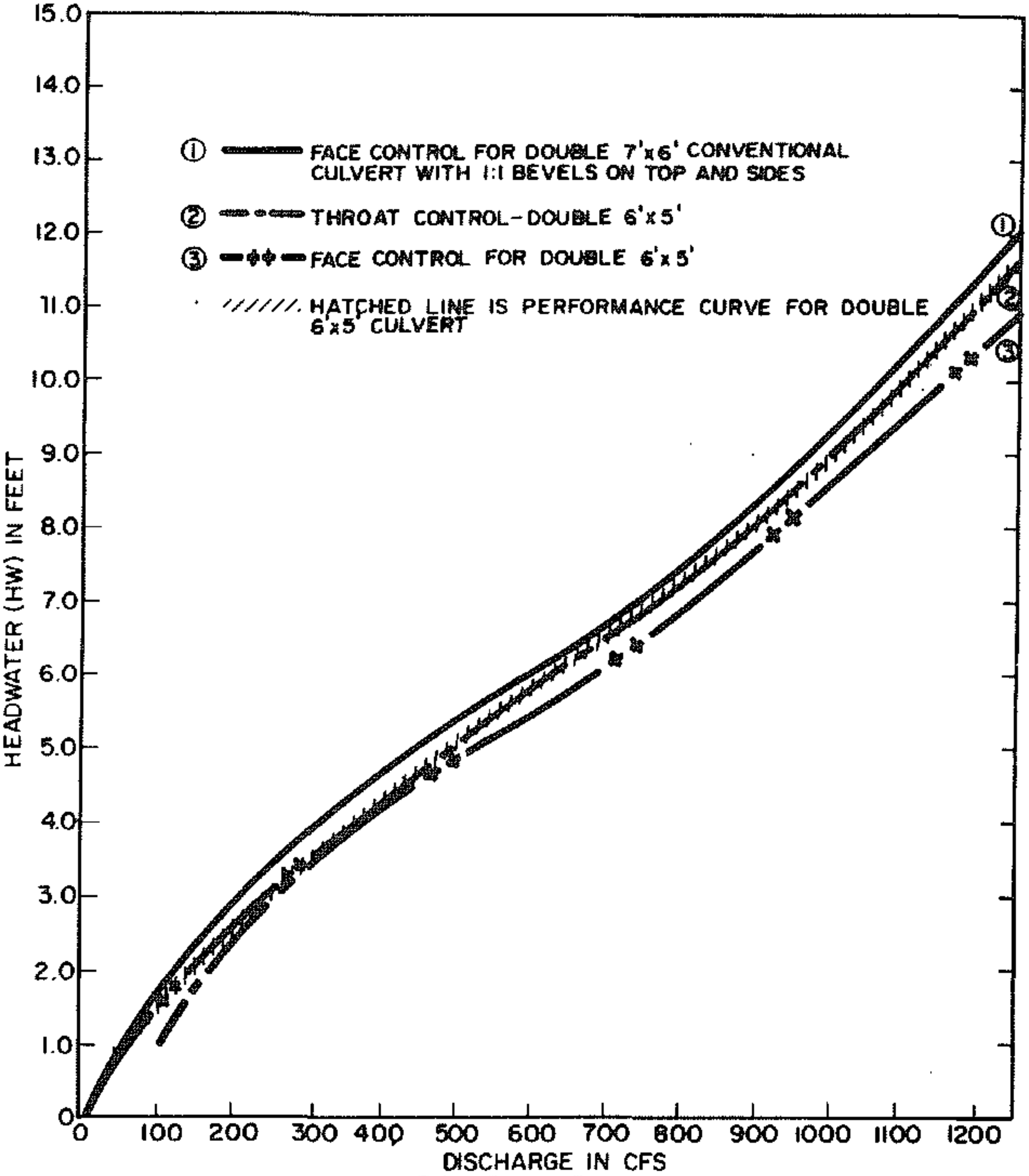


FIGURE VIII-17

PERFORMANCE CURVES FOR DIFFERENT BOX CULVERTS WITH VARYING INLET CONDITIONS (SIDE-TAPERED INLET)

No design procedure is available for side-tapered inlet culverts with more than two barrels.

Slope-Tapered Inlets

The inlets shown in Figure VIII-18 are variations of the slope-tapered inlet and provide additional improvements in hydraulic performance by increasing the head on the control section. The difference between the two types of slope-tapered inlets lies in the face section placement. One type has a vertical face configuration and the other a mitered face. The face capacity of the latter type is not based on its physical face section, but on a section perpendicular to the fall slope intersecting the upper edge of the opening. This is illustrated by the dashed line in Figure VIII-18.

Excluding outlet control operation, the slope-tapered inlet with a vertical face has three potential control sections: the face, the throat, and the bend (Figure VIII-18). The bend is located at the intersection of the fall slope and the barrel slope. The distance, L_3, between the bend and the throat must be at least 0.5B, measured at the soffit or top of the culvert, to assure that the bend section will not control. Therefore, the hydraulic performance needs only be evaluated at the face and throat sections. The slope-tapered inlet with a mitered face has a fourth possible control section, the weir crest.

Throat Control. As with side-tapered inlets, throat control performance should usually govern in design since the major cost is in the construction of the barrel. Chart VIII-18 is the throat control design curve for both slope-tapered inlets. By entering Chart VIII-18 with a computed value for $Q/BD^{3/2}$, H_t can be determined from the value $\frac{H_t}{D}$.

Face Control. Face control design curves for slope-tapered inlets are presented in Chart VIII-20. The two design curves apply to the face edge and wingwall conditions shown in Figure VIII-15.

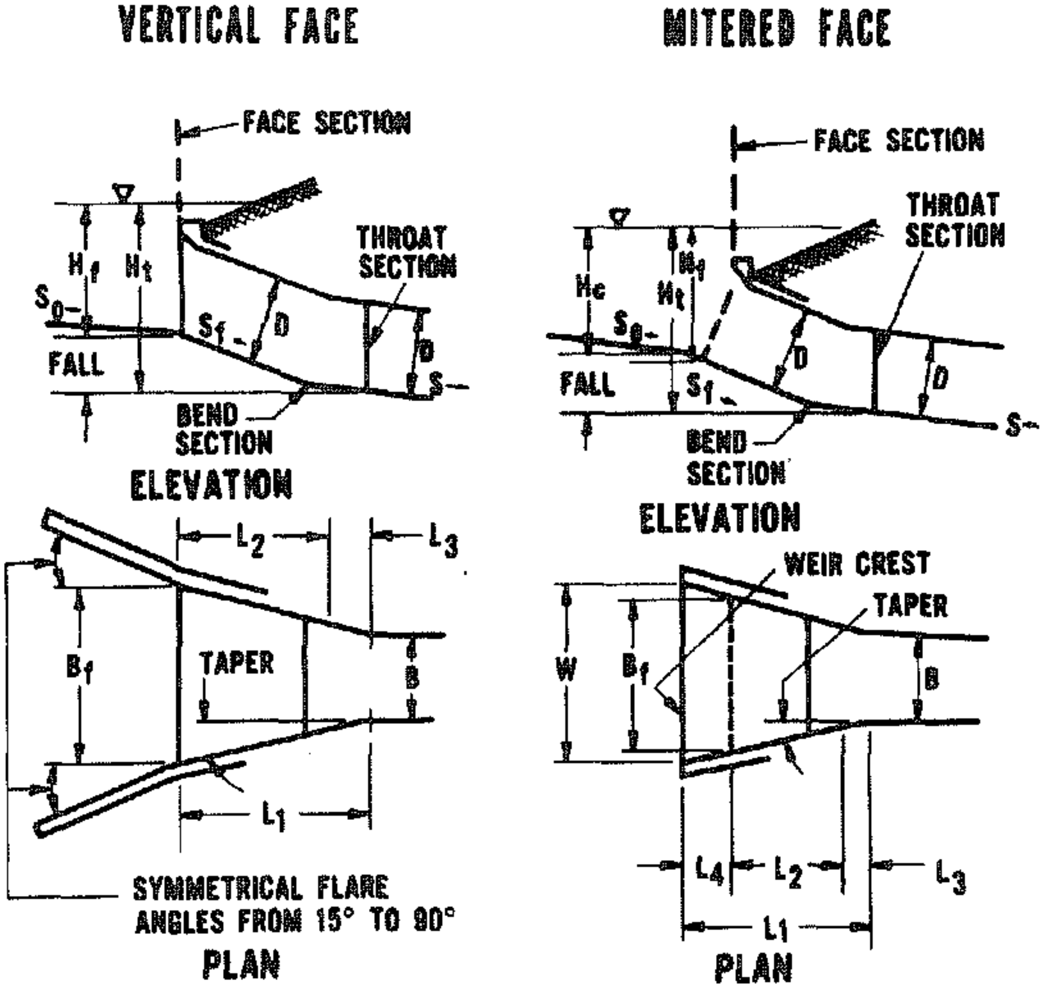


FIGURE VIII-18

IMPROVED INLETS SLOPE-TAPERED

Crest Control. The possibility of crest control should be examined for the slope-tapered inlet with a mitered face using Chart VIII-23. The crest width, W, is shown in Figure VIII-18. Again, there may be flow from the sides over the wingwalls, but generally this can be neglected. As the headwater rises above the wingwalls, there is little chance that the crest will remain the control section.

Design Limitations. In the design of slope-tapered inlets, the following limitations are necessary to insure that the design curves provided will always be applicable. If these limitations are not met, hydraulic performance will not be as predicted by design curves given in this Chapter.

- The fall slope must range from 2:1 to 3:1. Fall slopes steeper than 2:1 have adverse performance characteristics and the design curves do not apply. If a fall slope less than 3:1 is used, revert to design Chart VIII-19 for side-tapered inlets and use the fall slope that is available. Do not interpolate between Charts VIII-19 and VIII-20.
- The FALL should range from D/4 to 1.5D for direct use of the curves. For FALLS greater than 1.5D, frictional losses between the face and the throat must be calculated and added to the headwater. For FALLS less than D/4, use design Chart VIII-19 for side-tapered inlets and the FALL that is available. Do not interpolate between Charts VIII-19 and VIII-20.
- The sidewall taper should be from 4:1 to 6:1. Tapers less than 4:1 are unacceptable. Tapers greater than 6:1 will perform better than the design curves indicate, and the design will be conservative.
- L_3 must be a minimum of 0.5B measured at the soffit or inside top of the culvert. Larger values may be used, but smaller ones will cause the area provided for the bend to be so reduced that the bend section will control rather than the throat section. Do not use an L_3 value less than 0.5B.

Performance Curves. In Figure VIII-19, performance curves for the slope-tapered inlet are shown in addition to the performance curves shown in Figure VIII-17. Detailed calculations may be found in the Examples.

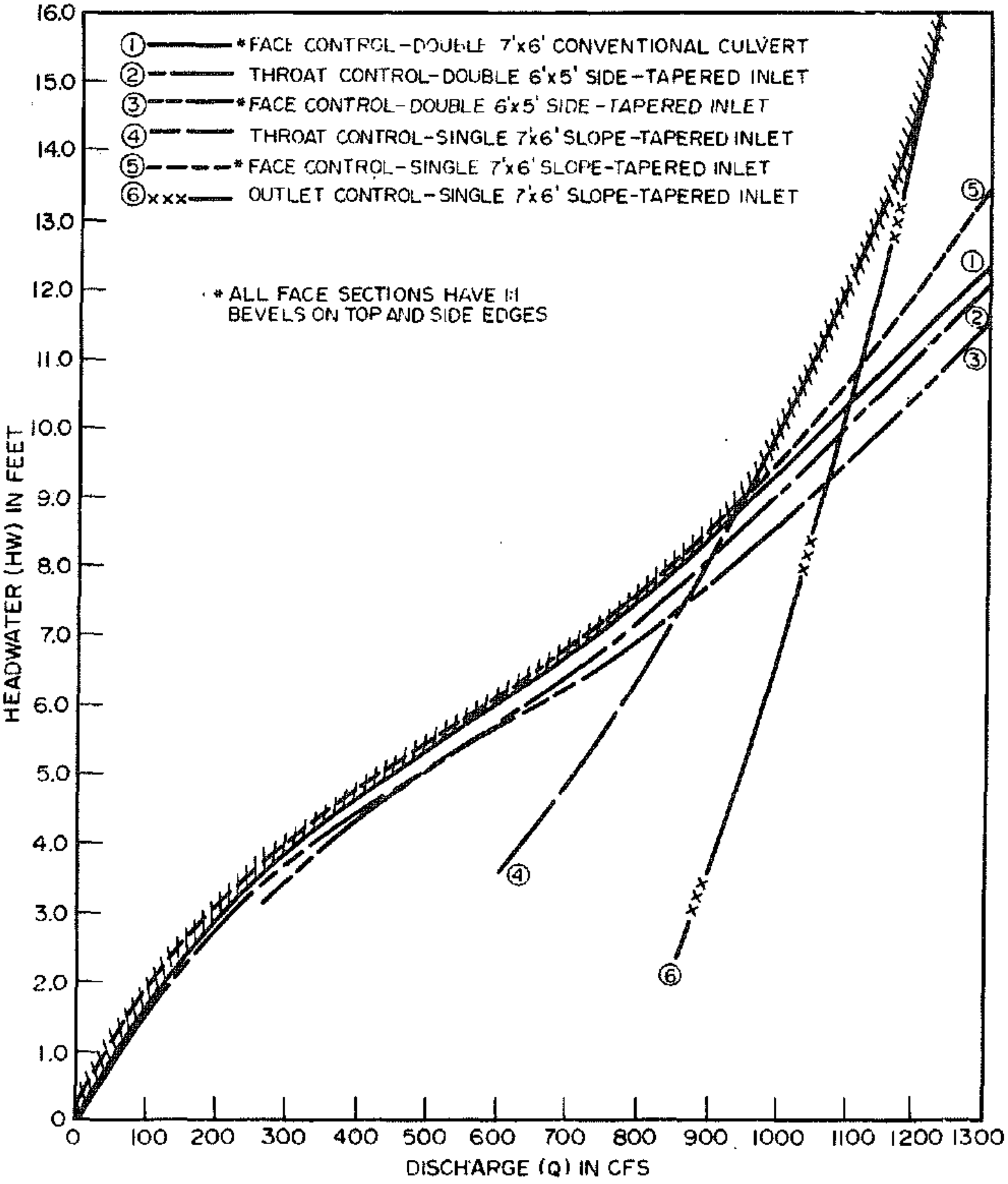


FIGURE VIII-19

PERFORMANCE CURVES FOR DIFFERENT BOX CULVERTS WITH VARYING INLET CONDITIONS

As can be seen from Figure VIII-19, the performance of a single 7 ft. by 6 ft. culvert with a slope-tapered inlet is comparable to a double conventional 7 ft. by 6 ft. culvert with beveled edges. Note that the performance curve for the single 7 ft. x 6 ft. culvert (hatched line) is developed from the face control curve (Curve 5) from 0 to 950 cfs, the throat control curve (Curve 4) from 950 to 1,200 cfs and the outlet control curve (Curve 6) for all discharges above 1,200 cfs. This illustrates the need for computing and plotting the performance of each control section and demonstrates the barrel size reduction possible through use of improved inlets. The performance curves clearly indicate the headwater elevation required to pass any discharge.

Double Barrel Design. Charts VIII-18, VIII-20, and VIII-21 depict single barrel installations, but they are applicable to double barrel installations with the center wall extended to the face section.

In addition to the comments and limitations for single barrel slope-tapered inlets, the face must be proportioned on the basis of the total clear width. The center wall is extended to the face section and may have any desired edge treatment.

The face width, as determined from Chart VIII-20 is the total clear face width. The center wall width must be added to the value found from Chart VIII-20 in order to size the face correctly.

No design procedure is available for slope-tapered inlet culverts with more than two barrels.

PIPE CULVERT IMPROVED INLET DESIGN

As with box culverts, for each degree of pipe culvert inlet improvement there are many possible variations using bevels, tapers, drops and combinations of the three. The tapered inlets are generally classified, as shown in Figure VIII-20, as either side-tapered (flared) or slope-tapered. The side-tapered inlet for pipe culverts is designed in a manner similar to that used

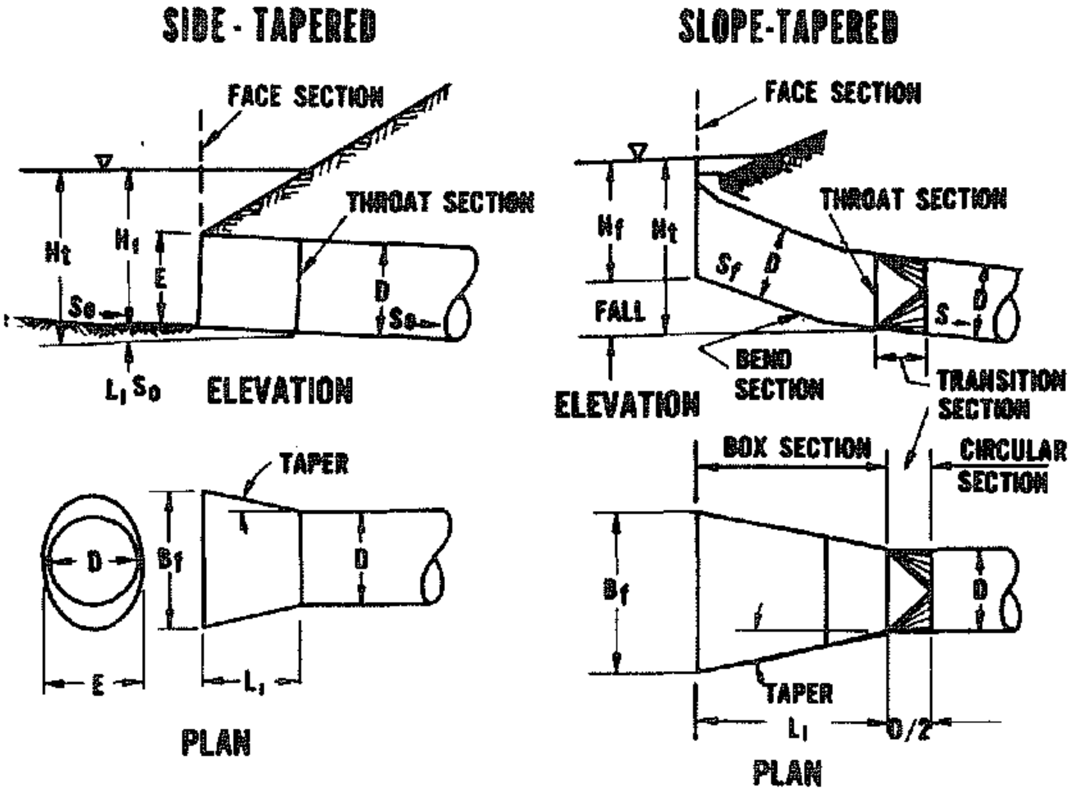


FIGURE VIII - 20
TYPES OF IMPROVED INLETS FOR PIPE CULVERTS

for a side-tapered box culvert inlet. The slope-tapered design for pipes utilizes a rectangular inlet with a transition section between the square and round throat sections.

Bevel-Edged Inlets

Design charts and their instructions for conventional pipe culverts with different entrance edge conditions are contained in this Chapter. The socket end of a concrete pipe results in about the same degree of hydraulic improvement as a beveled edge; therefore the socket should be retained at the upstream end of concrete pipes, even if some warping of the fill slope is required because of the longer pipe or skewed installation.

Multibarrel pipe culverts should be designed as a series of single barrel installations using the appropriate design charts, since each pipe requires a separate bevel.

Side-Tapered Pipe Inlets (Flared Inlets)

Description. The side-tapered or flared inlet shown in Figure VIII-20 is comparable to the side-tapered box culvert inlet. The face area is larger than the barrel area and may be in the shape of an oval as shown in Figure VIII-20, a circle, a circular segment, or a pipe-arch. The only limitations on face shape are that the vertical face dimension, E, be equal to or greater than D and equal to or less than 1.1D and that only the above face shapes be used with inlets designed using Chart VIII-22. Rectangular faces may be used in a manner similar to that described for the side- and slopetapered inlet. The side taper should range from 4:1 to 6:1.

As with the box culvert side-tapered inlet, there are two possible control sections: the face and the throat (Figure VIII-20). In addition, if a depression is placed in front of the face, the crest may control. This variation of the side-tapered inlet is depicted in Figure VIII-16, and will be discussed later.

Throat Control. As stated before, the barrel of a culvert is the item of greatest cost; therefore, throat control should govern in the design of all improved inlets. Throat control design curves for side-tapered inlets are presented in Chart VIII-21. Note that this chart contains two throat control design curves while the box culvert charts have only one. One curve is for entrances termed "smooth", such as those built of concrete or smooth metal, and the other is for "rough" inlets, such as those built of corrugated metal. The need for two curves results from different roughness characteristics and the difference in energy losses due to friction between the face and throat of the inlets.

Chart VIII-21 applies only to circular barrels. It should not be used for rectangular, pipe-arch, or oval sections. No information is available for using improved inlets with pipe arch or oval barrels.

Face Controls. Face control curves for the side-tapered pipe culvert inlet are presented in Chart VIII-22. The three curves on this chart are for: the thin-edged projecting inlet, the square-edged inlet, and the bevel-edged inlet. Note that the headwater is given as a ratio of E rather than D. This permits the use of the curves for face heights from D to 1.1D, as the equations used in developing the curves do not vary within this range of E.

In Chart VIII-22, flexibility is allowed in choosing the face shape by presenting the flow rate, Q, in terms of $Q/A_fE^{1/2}$, rather than $D^{5/2}$. By using the area of the face, A_f, and its height, E, the designer may choose or evaluate any available shape, such as elliptical, circular, a circular segment, or a pipe-arch. However, this chart does not apply to rectangular face shapes.

FALL Upstream of Inlet Face. In order to provide additional head for the throat section of pipe culverts, the slope-tapered inlet may be used, or a depression can be placed upstream of the side-tapered inlet face. There are various methods of constructing such a depression, including a drop similar to that shown for the side-tapered box culvert inlet with flared wingwalls. This configuration consists of a constantly sloping bottom from the crest to a point a minimum distance of D/2 upstream of the face invert, and on line with the barrel invert. Chart VIII-23 should be used to assure that the weir crest is long enough to avoid crest control.

Another means of providing a FALL upstream of the face is depicted in Figure VIII-16. This configuration can be used with 90° wingwalls (headwall). The depression will probably require paving to control upstream erosion. Research results indicated that such a depression could cause a moderate decrease in the performance of the face. To insure that this reduction in performance is not extreme, the following dimensional considerations should be observed (Figure VIII-16).

(1) The minimum length of the depression, P, should be 3T;

(2) the minimum width, W_p, of the depression should be B_f + T or 4T, whichever is larger;

(3) the crest length should be taken as $W_p + 2(P)$ when using Chart VIII-23 to determine the minimum required weir length.

Slope-Tapered Inlets for Pipe Culverts

In order to utilize more of the available total culvert fall in the inlet area, as is possible with the box culvert slope-tapered inlets, a method was devised to adapt rectangular inlets to pipe culverts as shown in Figure VIII-21. As noted in the sketch, the slope-tapered inlet is connected to the pipe culvert by use of a square to circular transition over a minimum length of one-half the pipe diameter. The design of this inlet is the same as presented in the box culvert section. There are two throat sections one square and one circular, and the circular throat section must be checked by use of Chart VIII-21. In all cases, the circular throat will govern the design because its area is much smaller than the square throat section. Thus, the square throat section need not be checked. The culvert performance curve consists of a composite of performance curves for the inlet control sections and the outlet control performance curve.

Square to round transition sections have been widely used in water resource projects. They are commonly built in-place, but also have been preformed. It is recommended that plans permit prefabrication or precasting as an alternate to in-place construction.

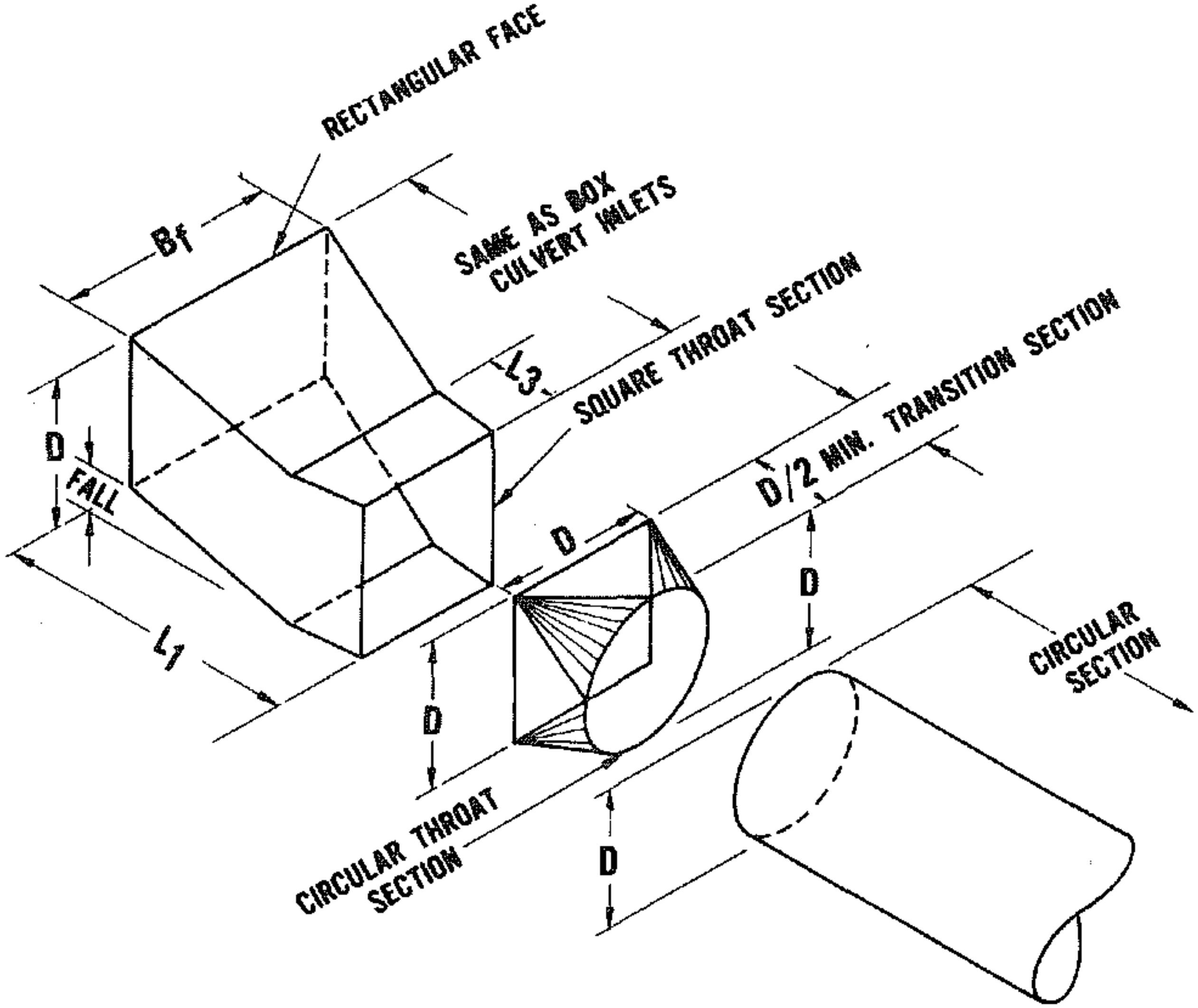


FIGURE VIII-21

SLOPE-TAPERED INLET APPLIED TO CIRCULAR PIPE

Rectangular Side-Tapered Inlets for Pipe Culverts

The expedient suggested for adapting the slope-tapered inlet for use with pipe culverts can also be used on side-tapered inlets where unusally large pipes or sizes not commonly used are encountered. It may not be economical to prefabricate or precast a "one-of-a-kind" side-tapered or flared inlet, in which case, a cast-in-place rectangular side-tapered inlet would be a logical bid alternate. Also, flared inlets for large pipes may be too large to transport or to handle on the job. In this case, the flared or side-tapered pipe inlet could either be prefabricated or precast in two sections or the rectangular side-tapered inlet may be used as a bid or design alternate. Information for determining throat and face control performance is provided in Charts VIII-21 and VIII-19, respectively.

Design Limitations

In addition to the design limitations given previously for box culvert slope-tapered inlets, the following criteria apply to pipe culvert slope-tapered inlets and rectangular side-tapered inlets for pipe culverts:

1. The rectangular throat of the inlet must be a square section with sides equal to the diameter of the pipe culverts.

2. The transition from the square throat section to the circular throat section must be no shorter than one half the culvert diameter, D/2. If excessive lengths are used, the frictional loss within this section of the culvert should be considered in the design.

Multibarrel Designs

The design of multiple barrels for circular culverts using slope-tapered improved inlets can be performed the same as for box culverts, except that the center wall must be flared in order to provide adequate space between the pipes for proper compaction of the backfill. The amount of flare required will depend on the size of the pipes and the construction techniques used. No more than two barrels may feed from the inlet structure using the design methods of this Chapter.

An alternative would be to design a series of individual circular culverts with slope-tapered inlets. This permits the use of an unlimited number of barrels, and the curves and charts of this publication are applicable.

Use of Nomographs for Outlet and Inlet Control

Charts VIII-1 through VIII-7 are for outlet control and Charts VIII-8 through VIII-17 apply to inlet control. There are charts for most types of barrels commonly used for culverts. The following is a brief description of the use of these charts.

Outlet Control - Charts VIII-1 Through VIII-7

Chart VIII-1 will be used as an example.

1. List design data; Q(cfs), L (ft), invert elevations in and out (ft), allowable headwater (AHW, ft), Tailwater (TW, ft), type of culvert, and entrance type for first trial.

2. Compute the Hw for outlet control, Chart VIII-1. Enter the graph with the length, the entrance coefficient for the entrance type, and the trail size. Connect the length scale and the culvert size scale with a straight line, 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). Compute Hw elevation from the equation:

   $$HW_o = Hw\ Elev = H + h_o + \text{Outlet Invert Elevation} \qquad \text{Eq. VIII-7}$$

   For TW greater than or equal to the top of the culvert, ho = TW, and for TW less than the top of the culvert:

   $$ho = dc + D/2 \text{ or } TW \qquad \text{Eq. VIII-8}$$

   whichever is the greater. If TW is less than dc, the nomographs cannot be used. See the critical depth charts contained in this Chapter.

Inlet Control - Charts VIII-8 through VIII-17

Chart VIII-8 will be used as an example. For inlet control, entering Chart VIII-8, connect a straight line through D and Q to scale (1) of H/D scales and project horizontally to the proper scale. After computing H_f, compute HWo by adding H_f to the upstream invert elevation.

Design Procedure

The design procedure hinges on the selection of a culvert barrel based on its outlet control performance curve, which is unique when based on elevation. The culvert inlet is then manipulated using edge improvements and adjustment of its elevation in order to achieve inlet control performance with the outlet control performance. The resultant culvert design will best satisfy the criteria set by the designer and make optimum use of the barrel selected for the site.

The design calculations are to be done on the design forms from HEC-12 (12), Figures VIII-22 (outlet Control), Figures VIII-23 (Inlet Control), Figure VIII-24 (Side Taper), and Figures VIII-25 (Slope-Taper).

Step 1. Determine and Analyze Site Characteristics

Site characteristics include the generalized shope of the highway embankment, bottom elevations and cross sections along the stream bed, the approximate length of the culvert, and the allowable headwater elevation. In determining the allowable headwater elevation (AHW El.), roadway elevations and the elevation of upstream property should be considered. The consequences of exceeding the AHW El. should be evaluated according to the criteria contained in Section II of Part II.

Culvert design is actually a trial-and-error procedure because the length of the barrel cannot be accurately determined until the size is known, and the size cannot be precisely determined until the length is known. In most cases, however, a reasonable estimate of length will be accurate enough to determine the culvert size.

The culvert length is approximately $2S_eD$ shorter than the distance between the points defined by the intersections of the embankment slopes and the

PROJECT: ______ DESIGNER: ______

STATION: ______ DATE: ______

INITIAL DATA:

Q____ = ____cfs

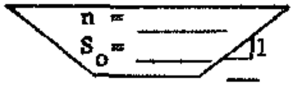
AHW El. = ____ ft.

S_o = ____

L_a = ____ ft.

El. Outlet Invert ____ ft.

Stream Data:

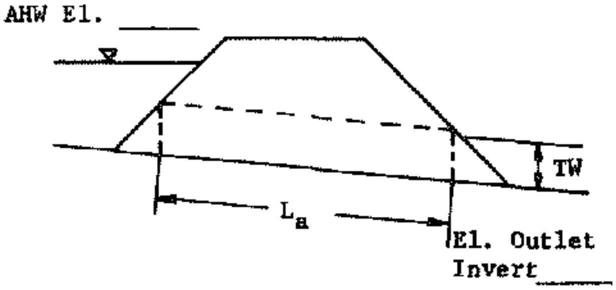


n = ______

S_o = ______

Barrel Shape and Material ______

SKETCH



AHW El. ____

El. Outlet Invert ____

Q	$\frac{Q}{N}$	* H	$\frac{Q}{NB}$	(1) d_c	$\frac{d_c+D}{2}$	Qn	(2) TW	(3) h_o	(4) HW_o	(5) V_o	COMMENTS
Trial No. ____, N = ____, B = ____, D = ____, k_e = ____											
Trial No. ____, N = ____, B = ____, D = ____, k_e = ____											
Trial No. ____, N = ____, B = ____, D = ____, k_e = ____											

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

(3) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.

(4) $HW_o = H + h_o$ + El. Outlet Invert

(5) Outlet Velocity (V_o) = Q/Area defined by d_c or TW. not greater than D.

SELECTED DESIGN

N = ____

B = ____ ft.

D = ____ ft.

k_e = ____

At Design Q:

HW_o = ____ ft.

V_o = ____ f/s

$$* \quad H = \left[1 + k_e + \frac{29n^2 \cdot L}{R^{1.33}}\right] \frac{V^2}{2g}$$

FIGURE VIII-22

OUTLET CONTROL DESIGN CALCULATIONS

PROJECT: ________ DESIGNER: ________

STATION: ________ DATE: ________

<u>INITIAL DATA:</u>

Q _____ = _____ cfs

AHW El. = _____ ft.

S_o = _____

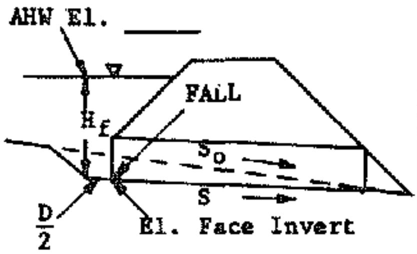
L_a = _____ ft.

El. Stream Bed at Face _____ ft.

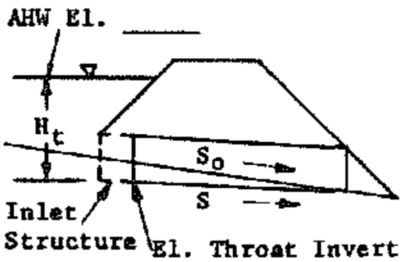
Barrel Shape and Material __________

N = _____, B = _____

D = _____, $NBD^{3/2}$ = _____



CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION
(Upper Headings)



TAPERED INLET
THROAT CONTROL SECTION
(Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

	$\frac{Q}{NB}$	$\frac{H_f}{D}$	H_f	(1) El. Face Invert	El. Stream Bed At Face	(2)	(3) HW_f	(4)	(5)	Note: Use Upper Headings for Conventional or Beveled Face; Lower Headings for Tapered Inlet Throat
Q	$\frac{Q}{NBD^{3/2}}$	$\frac{H_t}{D}$	H_t	El. Throat Invert		FALL	HW_t	S	V_o	COMMENTS
Trial No. _____ Inlet and Edge Description __________										
Trial No. _____ Inlet and Edge Description __________										
Trial No. _____ Inlet and Edge Description __________										

<u>Notes and Equations:</u>

(1) El. Face (or throat) invert = AHW El. − H_f (or H_t)

(2) FALL=El. Stream Bed at Face−El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed

(4) $S \approx S_o - FALL/L_a$

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

<u>Inlet Description:</u>

FALL = _____ ft.

Invert El. = _____ ft.

Bevels:

Angle = _____

b = _____ in., d = ____ in

FIGURE VIII-23
CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS

PROJECT: ____________

STATION: ______ ______

DESIGNER: ____________

DATE: ____________

INITIAL DATA

$Q_{\_}$ = ___cfs S_o = ____

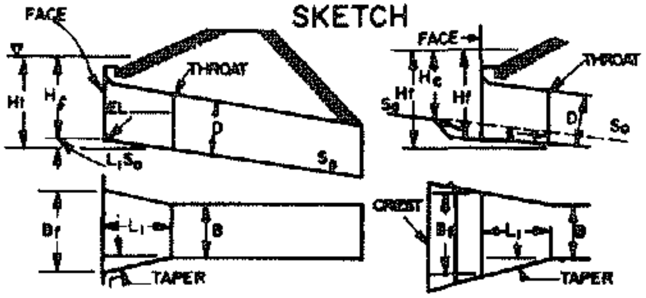
AHW El. = ___ft. L_a = ____ ft.

TAPER = ____:1

Barrel Shape and Material ____________

Face Edge Description ____________

SKETCH



N = ____, B = ____, D = ____

Q	El. Throat Invert	(1) $\frac{H_f}{D}$ $\frac{H_f}{E}$	$\frac{Q}{B_fD^{3/2}}$ $\frac{Q}{A_fE^{1/2}}$	$D^{3/2}$ $E^{1/2}$	(2) Min. B_f Min. A_f	B_f	(3) L_1	(4) S	L_1S	(5) El. Face Invert	COMMENTS
Trial No. ____, Q = ____, HW_f = ____											
											$B_fD^{3/2}$[or $A_fE^{1/2}$] = ____
Trial No. ____, Q = ____, HW_f = ____											
											$B_fD^{3/2}$[or $A_fE^{1/2}$] = ____
Trial No. ____, Q = ____, HW_f = ____											
											$B_fD^{3/2}$[or $A_fE^{1/2}$] = ____

Notes and Equations:

(1) H_f/D[or H_f/E] = (HW_f-El. Throat Invert-1)/D [or E]

$D \leq E \leq 1.1D$

(2) $\text{Min. } B_f = Q/\left[(D^{3/2})\ \frac{Q}{B_fD^{3/2}}\right]$

$\text{Min. } A_f = Q/\left[(E^{1/2})\ \frac{Q}{A_fE^{1/2}}\right]$

(3) $L_1 = \left[\frac{B_f - NB}{2}\right]$ TAPER

(4) From throat design

(5) El. Face Invert-El.Throat Invert>1 ft., recompute. Face and Throat may be lowered to better fit site.

SELECTED DESIGN

B_f = ____ ft.

L_1 = ____ ft.

Bevels: Angle ____°

d = ___in., b = ___ in.

Crest Check:

HW_c = ____ ft.

H_c = ____ ft.

Q/W = ____ (Chart 17)

Min.W = ____ ft.

FIGURE VIII-24
SIDE-TAPERED INLET DESIGN CALCULATIONS

PROJECT: ____________________ DESIGNER: ______________

STATION: ____________________ DATE: ______________

INITIAL DATA

Q ___ = ___ cfs S_o = ______

AHW El. = ______ ft. L_a = _____ ft.

El. Stream bed at crest _______ ft.

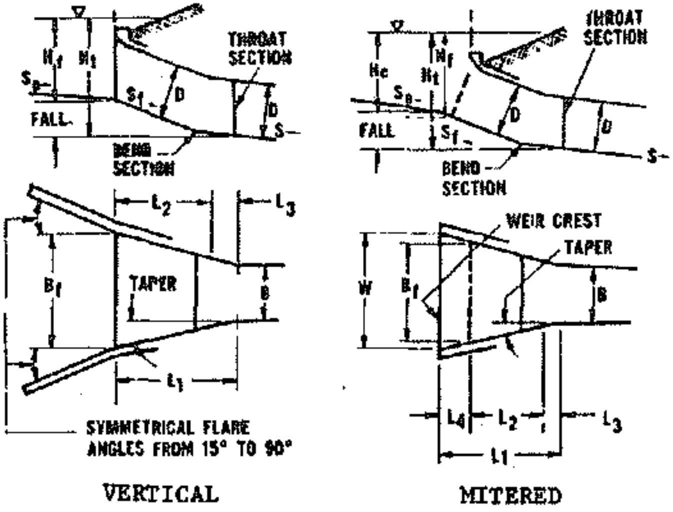
El. stream bed at face ________ ft.

TAPER = ___:1 (4:1 to 6:1)

S_f = ____:1 (2:1 to 3:1)

Barrel Shape and Material____________________

Inlet Edge Description____________________



N = ______, B = ______, D = ______

	Q	HW_f	El. Throat Invert	(1) El. Face Invert	(2) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3) Min. B_f	B_f	S	COMMENTS
Trial 1												$B_f D^{3/2}$ = ______
Trial 2												$B_f D^{3/2}$ = ______

Note: Use only throat designs with FALL > 0.25D

(1) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest - 0.4D (Approx.), but higher than throat invert elevation.

(2) $H_f = HW_f$ - El. face invert

(3) Min $B_f = Q/[(D^{3/2})\ Q/B_f D^{3/2}]$

(4) Min. L_3	(5) L_4	(6) L_2	(7) Check L_2	(8) Adj. L_3	(9) Adj. TAPER	(10) L_1	(11) W	$\frac{Q}{W}$	H_c	(12) Max. Crest El.	GEOMETRY
											B_f = ___ft. L_3 = ___ ft. L_1 = ___ft. L_4 = ___ ft.
											L_2 = ___ft. d = ____ in. b = ____ in. TAPER = ___:1

(4) Min $L_3 = 0.5B$

(5) $L_4 \approx S_{fv} + D/S_f$

(6) L_2 = (FALL) $S_f - L_4$

(7) Check $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER$-L_3$

(8) If (7)>(6), Adj. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER$-L_2$

(9) If (6)>(7) Adj. TAPER$=(L_2+L_3)/\left[\frac{B_f - NB}{2}\right]$

(10) $L_1 = L_2 + L_3 + L_4$

(11) Miêered: $W = NB + 2\left[\frac{L_1}{\text{TAPER}}\right]$

(12) Max. Crest El $= HW_f - H_c$

FIGURE VIII-25
SIDE-TAPERED INLET DESIGN CALCULATIONS

stream bed, where S_e is the embankment slope, and D is the culvert height. The inlet invert elevation will be approximately S_oS_eD lower than the upstream point of intersection and the outlet invert elevation is approximately S_oS_eD higher than the downstream point of intersection, where S_o is the stream bed slope.

All points referenced to the stream bed should be considered approximate since stream beds are irregular and not straight lines as shown in the schematic site representation.

Step 2. Perform Hydrologic Analysis

Define the design flow rate according to the procedures outlined in Chapter I of Part II.

Step 3. Perform Outlet Control Calculations and Select Culvert (Charts VIII-1 through VIII-7)

These calculations are performed before inlet control calculations in order to select the smallest feasible barrel which can be used without the required headwater elevation in outlet control (HW_o) exceeding the allowable headwater elevation (AHW El.). For use in this procedure, the equation for headwater is in terms of elevation.

The full flow outlet control performance curve for a given culvert (size, inlet edge, shape, material) defines its maximum performance. Therefore, inlet improvements beyond the beveled edge or changes in inlet invert elevation will not reduce the required outlet control headwater elevation. This makes the outlet control performance curve an ideal limit for improved inlet design.

When the barrel size is increased, the outlet control curve is shifted to the right, indicating a higher capacity for a given head. Also, it may be generally stated that increased barrel size will flatten the slope of the outlet control curve, although this must be checked.

The outlet control curve passing closest to and below the design Q and AHW El. on the performance curve graph defines the smallest possible barrel which will meet the hydraulic design criteria. However, that curve may be very steep (rapidly increasing headwater requirements for discharges higher than design) or use of such a small barrel may not be practical.

a) Calculate HW_o at design discharge for trial culvert sizes, entrance condition, shapes, and materials.
b) Calculate headwater elevations at two additional discharge values in the vicinity of design Q in order to define outlet control performance.
c) Plot outlet control performance curves for trial culvert sizes.
d) Select culvert barrel size, shape and material.

This selection should not be based solely on calculations which indicate that the required headwater at the design discharge is near the AHW El., but should also be based on outlet velocity as affected by material selection, the designer's evaluation of site characterisitcs, and the possible consequences of a flood occurrence in excess of the estimated design flood. A sharply rising outlet control performance curve may be sufficient reason to select a culvert of different size, shape or material.

In order to zero in on the barrel size required in outlet control, the applicable outlet control nomograph may be used as follows:

(1) Intersect the "Turning Line" with a line drawn between Discharge and Head, H. To estimate H, use the following equation:

$$H = \text{AHW El.} - \text{El. Outlet Invert} - h_o \qquad \text{Eq. VIII-9}$$

where h_o may be selected as a culvert height. Accuracy is not critical at this point.

(2) Using the point on the "Turning Line," K_e, and the barrel length, draw a line defining the barrel size.

This size gives the designer a good first estimate of the barrel size and more precise sizing will follow rapidly.

Step 4. Perform Inlet Control Calculations for Conventional and Beveled Edge Culvert Inlets (Charts VIII-8 through VIII-17.

The calculation procedure is similar to that used in HEC No. 5, except that headwater is defined as an elevation rather than a depth, a FALL may be incorporated upstream of the culvert face, and performance curves are an essential part of the procedure. The depression or FALL should have dimensions as described for side-tapered inlets.

a) Calculate the required headwater depth (H_f) at the culvert face at design discharge for the culvert selected in Step 3.

b) Determine required face invert elevation to pass design discharge by subtracting H_f from AHW El.

c) If this invert elevation is above the stream bed elevation at the face, the invert would generally be placed on the stream bed and the culvert will then have a capacity greater than design Q with headwater at the AHW El.

d) If this invert elevation is below the stream bed elevation at the face, the invert must be depressed, and the amount of depression is termed the FALL.

e) Add H_f to the invert elevation to determine HW_f. If HW_f is lower than HW_o, the barrel operates in outlet control at design Q. Proceed to Step 8.

f) If the FALL is excessive in the designer's judgment from the standpoint of aesthetics, economy and other engineering reasons, a need for inlet geometry refinements is indicated. If square edges were used in Steps 3 and 4 above, repeat with beveled edges. If beveled edges were used, proceed to Step 5.

g) If the FALL is within acceptable limits, determine the inlet control performance by caculating required headwater elevation using the flow rates from Step 3 and the FALL determined above.

$HW_f = H_f +$ El. face invert.

h) Plot the inlet control performance curve with the outlet control performance curve plotted in Step 3.

i) Proceed to Step 6.

Step 5. Perform Throat Control Calculations for Side- and Slope-Tapered Inlets (Charts VIII-18 or VIII-21)

The same concept is involved here as with conventional or beveled edge culvert design.

a) Calculate required headwater depth on the throat (H_t) at design Q for the culvert selected in Step 3.

b) Determine required throat elevation to pass design discharge by subtracting H_t from AHW El.

c) If this throat invert elevation is above the stream bed elevation, the invert would probably be placed on the stream bed and the culvert throat will have a capacity greater than the design Q with headwater at the AHW El.

d) If this throat invert elevation is below the stream bed elevation, the invert must be depressed, and the elevation difference between the stream bed at the face and the throat invert is termed the FALL. If the FALL is determined to be excessive, a larger barrel must be selected. Return to Step 5 (a).

e) Add H_t to the invert elevation to determine HW_t. If HW_t is lower than HW_o, the culvert operates in outlet control at design Q. In this case, adequate performance can probably be achieved by the use of beveled edges with a FALL. Return to Step 4.

f) Define and plot the throat control performance curve.

Step 6. Analyze the Effect of FALLS on Inlet Control Section Performance

It is apparent from performance charts that either additional FALL or inlet improvements would increase the culvert capacity in inlet control by moving the inlet control performance curve to the right toward the outlet control performance curve. If the outlet control performance curve of the selected culvert passes below the point defined by the AHW El. and the design Q, there is an opportunity to optimize the culvert design by selecting the inlet so as to either increase its capacity to the maximum at the AHW El. or to pass the design discharge at the lowest possible headwater elevation.

Some possibilities are illustrated in Figure VIII-26. The minimum inlet control performance which will meet the selected design criteria is illustrated by Curve A. This design has merit in that minimum expense for inlet improvements and/or FALL is incurred and the inlet will pass a flood in excess of design Q before performance is governed by outlet control. This performance is adequate in many locations, including those locations where headwaters is excess of the AHW El. would be tolerable on the rare occasion of floods in excess of design Q.

Curve B illustrates the performance of a design which takes full advantage of the potential capacity of the selected culvert and the site to pass the maximum possible flow at the AHW El. A safety factor in capacity is thereby incorporated in the design. This can be accomplished by the use of a FALL, by geometry improvements at the inlet or by a combination of the two. Additional inlet improvements and/or FALL will not increase the capacity at or above the AHW El.

There may be reason to pass the design flow at the lowest possible headwater elevation even though the reasons are insufficient to cause the AHW El. to be set at a lower elevation. The maximum possible reduction in headwater at design Q is illustrated by Curve C. Additional inlet improvement and/or FALL will not reduce the required headwater elevation at design Q.

The water surface elevation in the natural stream may be a limiting factor in design i.e., it is not productive to design for headwater at a lower elevation than natural stream flow elevations. The reduction in headwater elevation illustrated by Curve C is limited by natural water surface elevations in the stream. If the water surface elevations in the natural stream had fallen below Curve D, this curve would illustrate the maximum reduction in headwater elevation at design Q. Tailwater depths calculated by assuming normal depth in the stream channel may be used to estimate natural water surface elevations in the stream at the culvert inlet. These may have been computed as a part of Step 3.

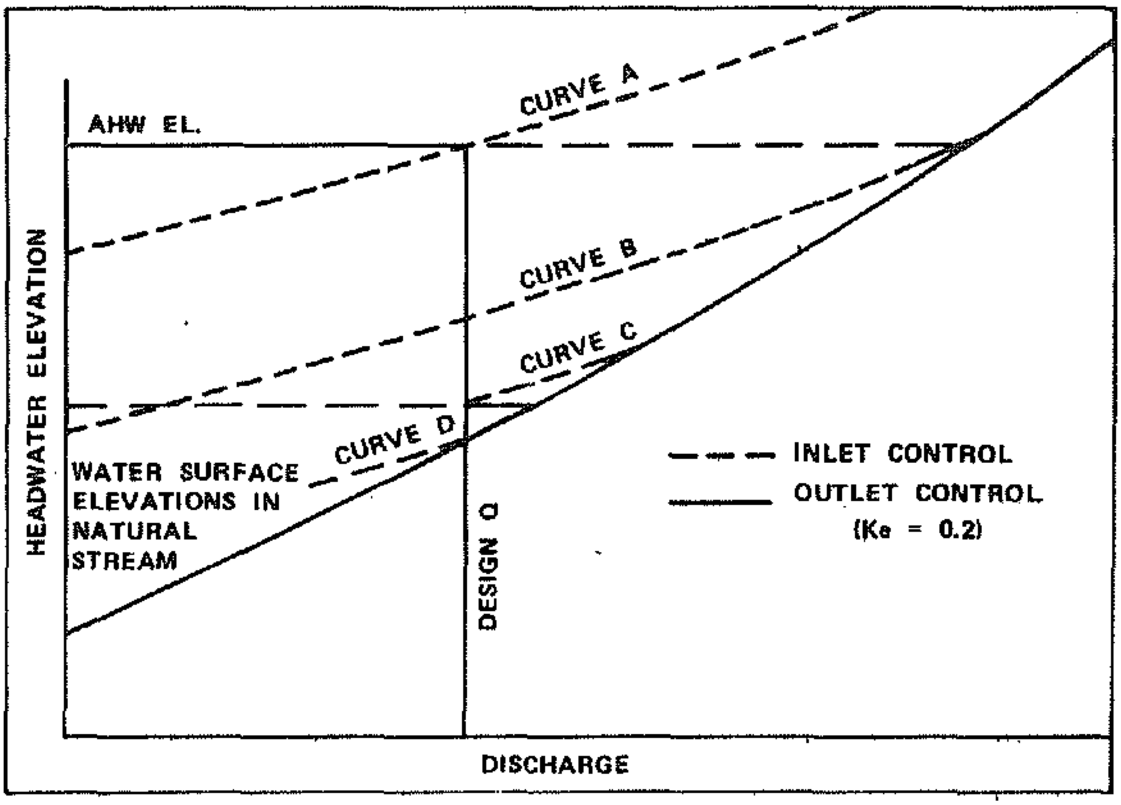


FIGURE VIII-26
OPTIMIZATION OF PERFORMANCE IN THROAT CONTROL

Curve A has been established in either Step 4 for conventional culverts or Step 5 for improved inlets. To define any other inlet control performance curve such as B, C, or D for the same control section:

a) Select a point on the outlet control performance curve.

b) Measure the vertical distance from this point to Curve A. This is the difference in FALL between Curve A and the curve to be established, e.g., the FALL on the control section for Curve A plus the distance between Curves A and B is the FALL on the control section for Curve B.

For conventional culverts only:

c) Estimate and compare the costs incurred for FALLS (structural excavation and additional culvert length) to achieve various levels of inlet performance.

d) Select design with increment in cost warranted by increased capacity and improved performance.

e) If FALL required to achieve desired performance is excessive, proceed to Step 5.

f) If FALL is acceptable and performance achieves the design objective, proceed to Step 8.

Step 7. Design Side- and/or Slope-Tapered Inlet (Charts VIII-19, VIII-20, VIII-22, and VIII-23

Either a side-or slope-tapered inlet design may be used if a FALL is required on the throat by use of a depression (FALL) upstream of the face of a side-tapered inlet or a FALL in the inlet of a slope-tapered inlet.

The face of the side- or the slope-tapered inlet should be designed to be compatible with the throat performance defined in Step 6. The basic principles of selecting the face design are illustrated in Figure VIII-27.

The minimum face design is one whose performance curve does not exceed the AHW El. at design Q. However, a "balanced" design requires that full advantage be taken of the increased capacity and/or lower headwater requirement gained through use of various FALLS. This suggests a face performance curve which intersects the throat control curve : (1) at the AHW El., (2) at design Q, (3) at its intersection with the outlet control curve, or (4) other. These options are illustrated in Figure VIII-27 by points a through e representing the interesections of face control performance curves with the throat control performance curves. The options are explained as follows: (1) Intersection of face and throat control performance curves at the AHW El. (Point a or b): For the minimum acceptable throat control performance (Curve A), this is the minimum face size that can be used without the required headwater elevation (HW_f) exceeding the AHW El. at design Q (Point a). For throat control performance greater than minimum but equal to or less than Curve B, this is the minimum face design which makes full use of the FALL placed on the throat to increase culvert capacity at the AHW El. (Point b). (2) Intersection of face and throat control performance curves at design Q (Points a, c or d): This face design option results in throat control performance at discharges equal to or greater than design Q. It makes full use of the FALL to increase capacity and reduce headwater requirements at flows equal to or greater than the design Q. (3) Intersection of the face control performance curve with throat control performance curve at its intersection with the outlet control performance curve (Points b or e): This option is the minimum face design which can be used to make full use of the increased capacity available from the FALL placed on the throat. It cannot be used where HW_f would exceed AHW El. at design Q; e.g., with the minimum acceptable throat control performance curve. (4) Other: Variations in the above options are available to the designer. The culvert face can be designed so that culvert performance will change from face control to throat control at any discharge at which inlet control governs. Options (1) through (3), however, appear to fulfill design objectives of minimum face size to achieve the maximum increase in capacity possible for a given FALL, or the maximum possible decrease in the required headwater for a given FALL for any discharge equal to or greater than design Q.

Figure VIII-28 illustrates the optional tapered inlet designs possible. Note that the inlet dimensions for the side-tapered inlet are the same for all options. This is because performance of the throat and an increase in headwater on the throat by virtue of an increased FALL results in an almost

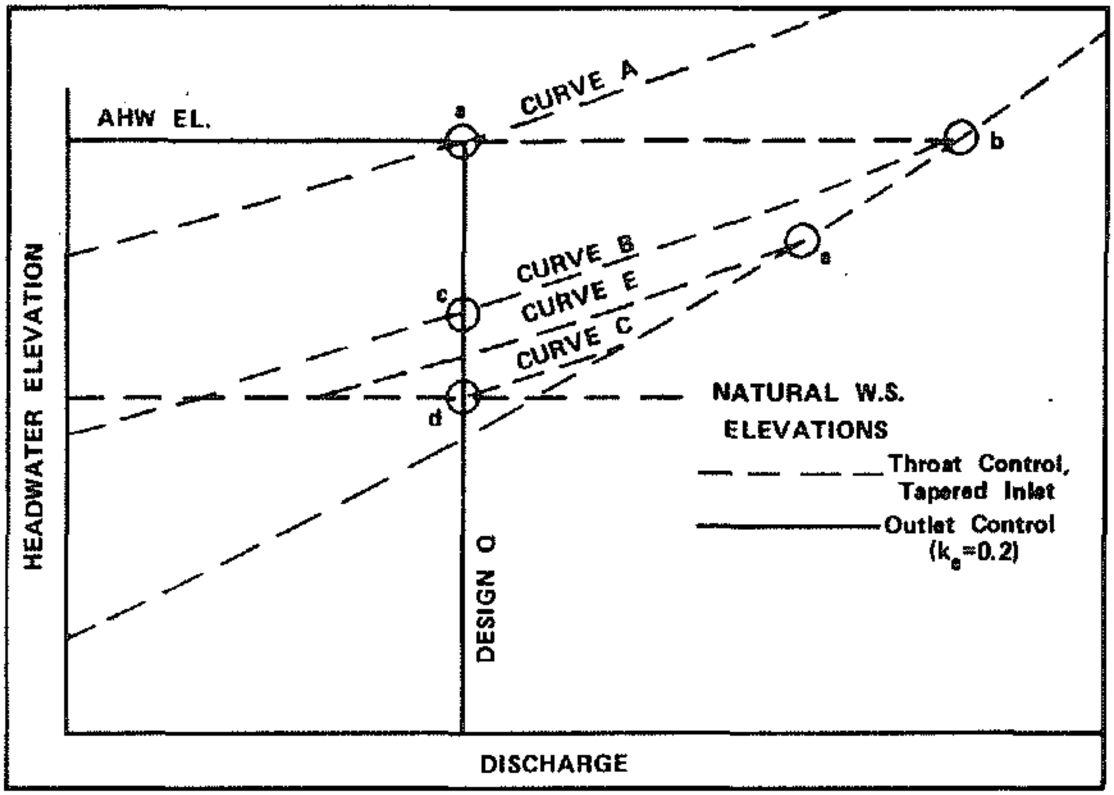
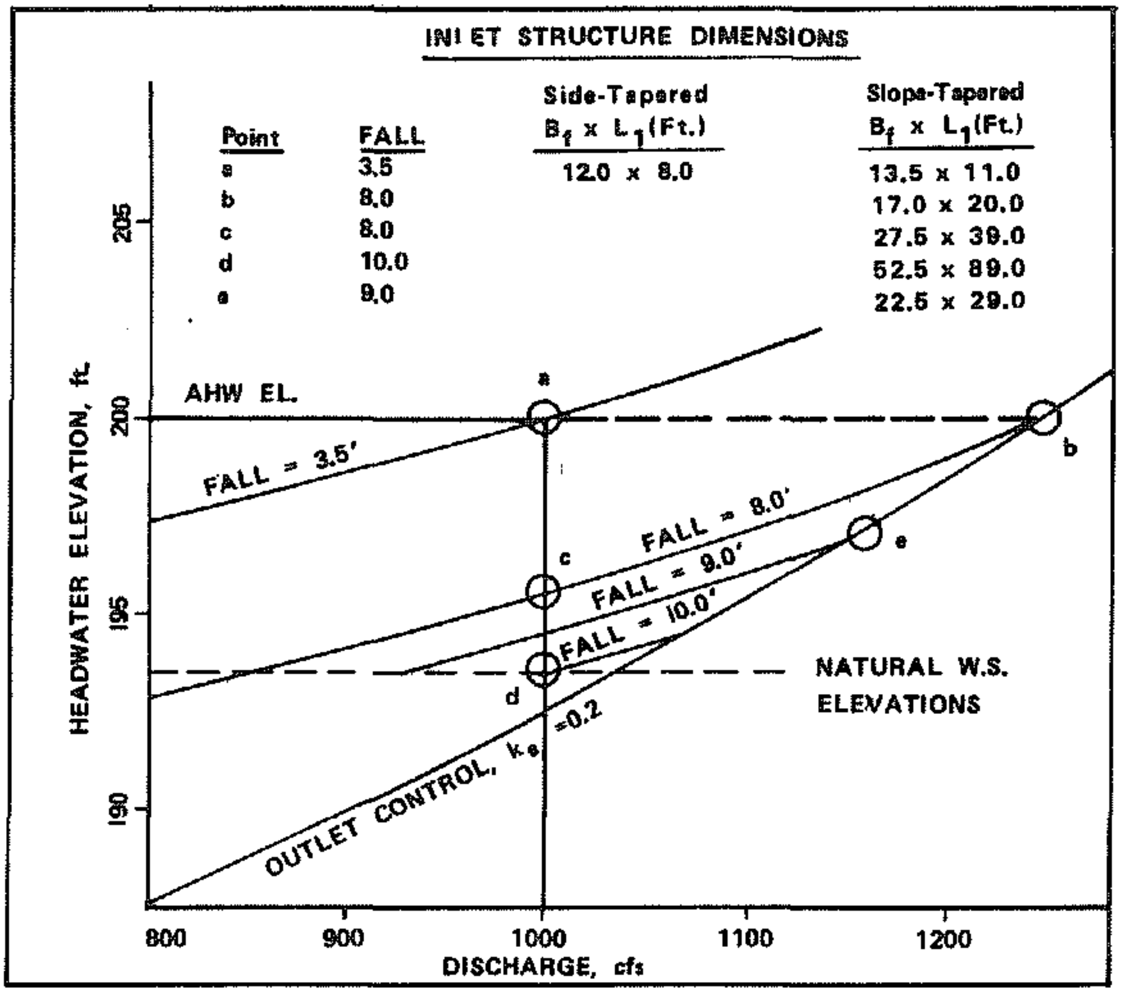


FIGURE VIII-27
POSSIBLE FACE DESIGN SELECTIONS



INLET STRUCTURE DIMENSIONS

Point	FALL	Side-Tapered $B_f \times L_1$(Ft.)	Slope-Tapered $B_f \times L_1$(Ft.)
a	3.5	12.0 x 8.0	13.5 x 11.0
b	8.0		17.0 x 20.0
c	8.0		27.5 x 39.0
d	10.0		52.5 x 89.0
e	9.0		22.5 x 29.0

FIGURE VIII-28

INLET DESIGN OPTIONS 8' x 6' REINFORCED CONCRETE BOX CULVERT equal increase in headwater on the face. Each foot of FALL on the throat of a culvert with a side-tapered inlet requires additional barrel length equal to the fill slope; e.g., if the fill slope is 3:1, use of 4 ft. of FALL rather than 3 ft. results in a culvert barrel 3 ft. longer as well as increased culvert capacity and/or reduced headwater requirements.

Face dimensions and inlet length increase for the slope-tapered inlet as the capacity of the culvert is increased by additional FALL on the throat. No additional head is created for the face by placing additional FALL on the the throat. On the other hand, use of a greater FALL at the throat of a culvert with a slope-tapered inlet does not increase culvert length.

The steps followed in the tapered inlet designs are:

a) Compute H_f for side- and slope-tapered inlets for various FALLS at design Q and other discharges. Side-Tapered Inlet: $H_f = H_t - 1.0'$ (Approximate) Slope-Tapered Inlet: H_f = HWE El. - Stream bed El. at Face.

b) Determine dimensions of side- and slope-tapered inlets for trial options.

c) For slope-tapered inlets with mitered face, check for crest control.

d) Compare construction costs for various options, including the cost of FALL on the throat.

e) Select design with incremental cost warranted by increased capacity and improved performance.

From the above, it is apparent that in order to optimize culvert design, performances are an integral part of the design procedure. At many culvert sites, designers have valid reasons for providing a safety factor in designs. These reasons include uncertainty in the design discharge estimate, potentially disastrous results in property damage or damaage to the highway from headwater elevations which exceed the allowable, the potential for development upstream of the culvert, and the chance that the design frequency flood will be exceeded during the life of the installation. Quantitative analysis of these variable would amount to a risk analysis, but at present, many of these factors must be evaluated intuitively. Procedures described here enable the designer to maximize the performance of the selected culvert or to optimize the design in accordance with his evaluation of site constraints, design parameters, and costs for construction and maintenance.

Step 8. Complete File Documentation

Documentation of the culvert hydraulic design consists of the compilation and preservation of all hydrologic and hydraulic information and the design decisions made on the basis of this information. This should include site information such as highway profile, upstream development and land use, estimates of the costs that would be incurred if the allowable headwater were exceeded, and other data used in determining the allowable headwater elevation. Several decisions in this procedure are based on the designer's knowledge and evaluation of site conditions. These decisions should be well founded on field information and documented for future reference. The design documentation is to be turned into the City Engineer.

DIMENSIONAL LIMITATIONS

Side Tapered Inlets

1. $6:1 \geq \text{Taper} \geq 4:1$

   Tapers greater than 6:1 may be used but performance will be underestimated.
2. Wingwall flare angle from 15° to 26° with top edge beveled or from 26° to 90° with or without bevels.
3. If FALL is used upstream of face, extend barrel invert slope upstream from face a distance of D/2 before sloping upward more steeply.
4. For pipe culverts, these additional requirements apply:
   - a. $D \leq E \leq 1.1D$
   - b. Length of square to round transition $\geq 0.5D$
   - c. FALL (Figure VIII-16)

     $P \geq 3T$

     $W_p = B_f + T$ or $4T$, whichever is larger.

Slope-Tapered Inlets

1. $6:1 \geq \text{Taper} \geq 4:1$

Tapers > 6:1 may be used, but performance will be underestimated.

2. 3:1 $\geq S_f \geq$ 2.1

   If S_f > 3:1, use side-tapered design

3. Minimum $L_3 = 0.5B$

4. 1.5D $\geq$ FALL $\geq$ D/4

   For FALL < D/4, use side-tapered design

   For FALL > 1.5D, estimate friction losses between face and throat.

5. Wingwall flare angle from 15° to 26° with top edge beveled or from 26° to 90° with or without bevels.

6. For pipe culvert, these additional requirements apply:
   a. Square to circular transition length > 0.5D.
   b. Square throat dimension equal to barrel diameter. Not necessary to check square throat performance.

Supporting Technical Information

Figures VIII-29 through VIII-34 are used to determine critical depth in open channels for various conduit cross-sections.

Tables VIII-4 through VIII-9 are tables of geometric properties to be used in the culvert design procedure.

SPECIAL CULVERT CONSIDERATIONS

Scour and Sedimentation

Scour and sedimentation are difficult to analyze and are not adapted to tables or formulas; therefore, the following guides are presented.

Where doubt exists concerning silt or scour, sufficient protection commensurate with the value of the structure and surrounding property should be afforded the structure to insure that damage to the structure or failure will not occur.

Sedimentation. While artificial channels are less prone to sedimentation problems than are natural channels, all culvert entrances are likely to experience some sedimentation problems. The basic design approach for culverts allows for higher headwater elevation than for bridges, reducing the

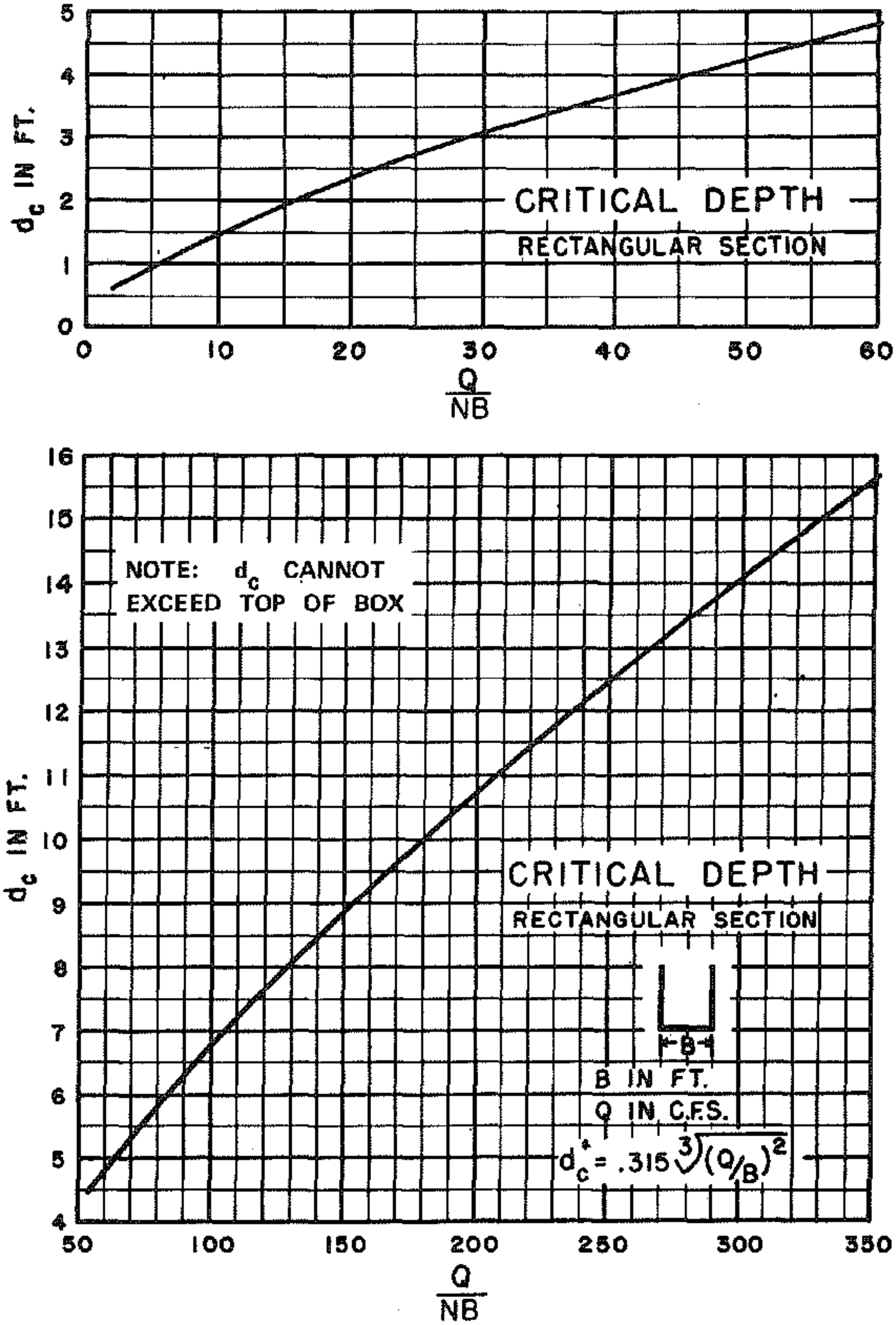


FIGURE VIII-29
CRITICAL DEPTH RECTANGULAR SECTION

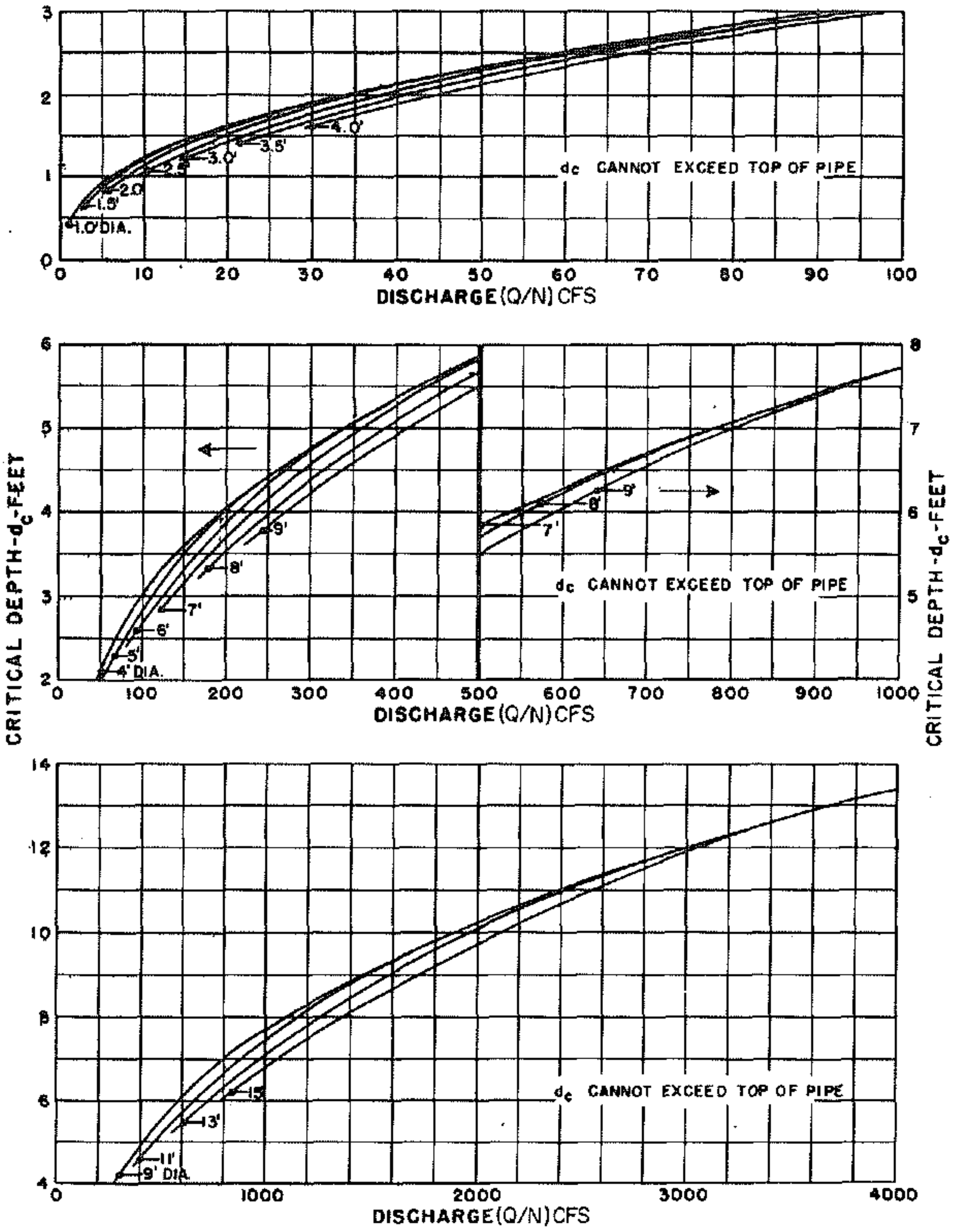


FIGURE VIII-30

CRITICAL DEPTH CIRCULAR PIPE

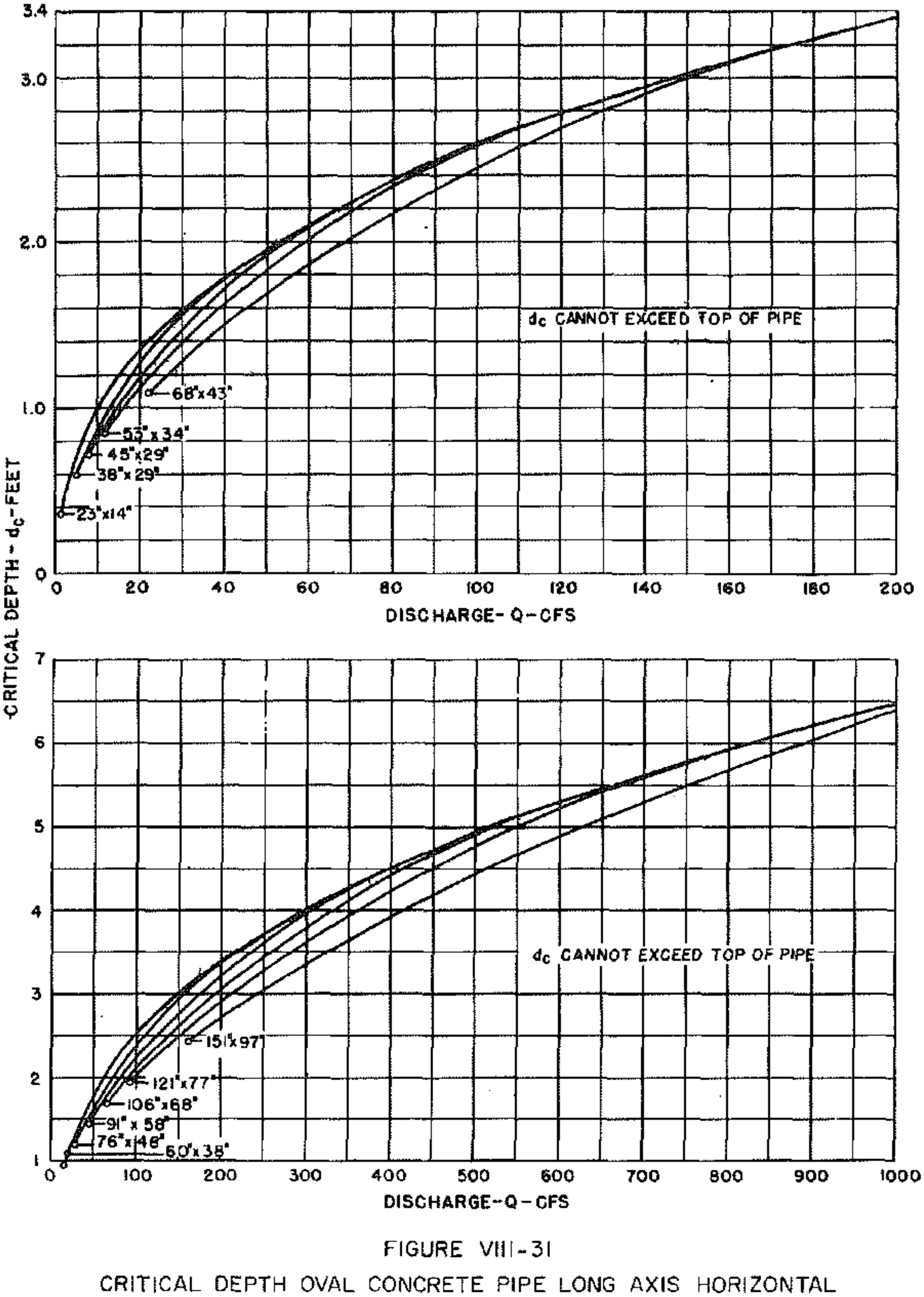


FIGURE VIII-31

CRITICAL DEPTH OVAL CONCRETE PIPE LONG AXIS HORIZONTAL

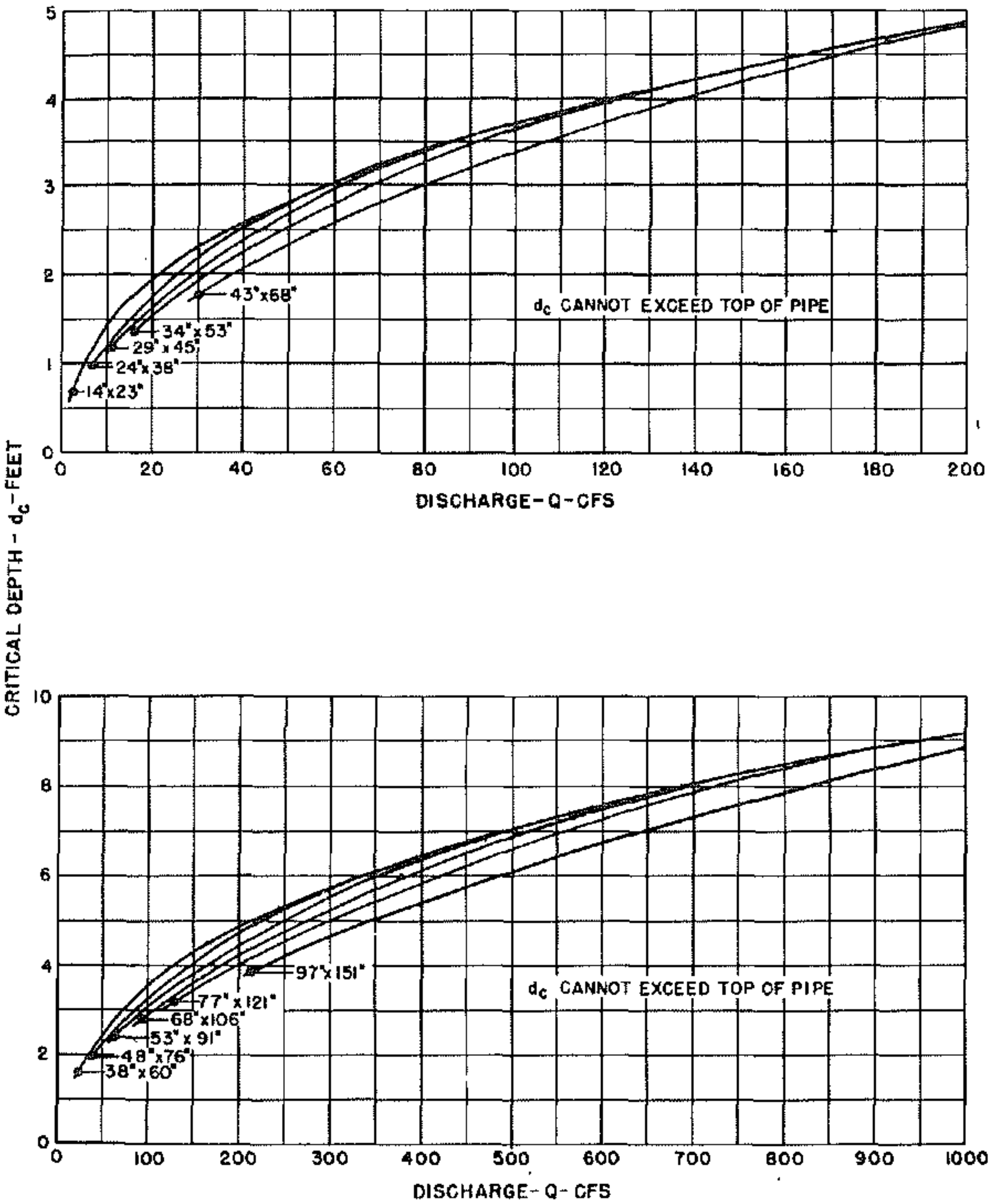


FIGURE VIII-32

CRITICAL DEPTH OVAL CONCRETE PIPE LONG AXIS VERTICAL

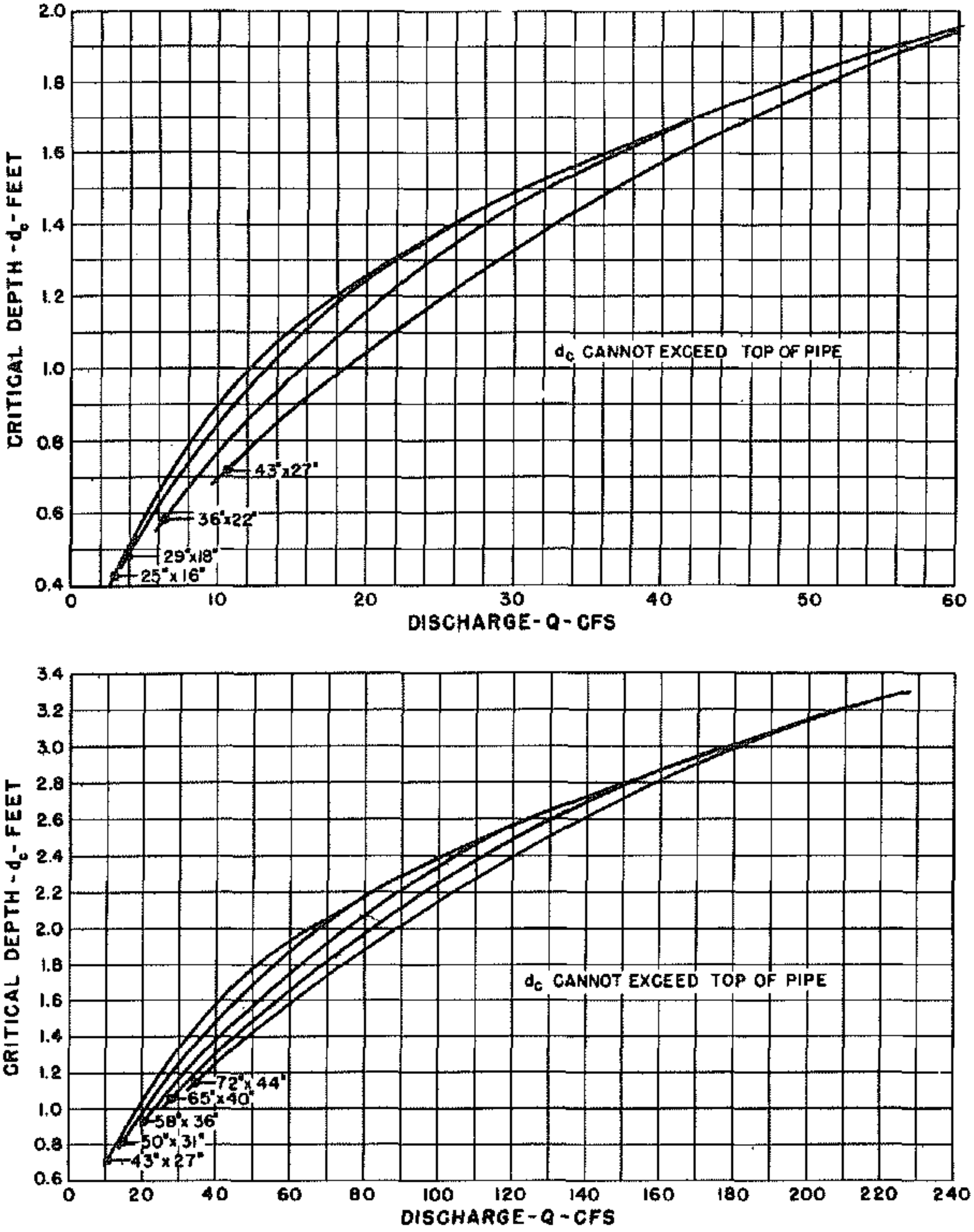


FIGURE VIII-33

CRITICAL DEPTH STANDARD C.M. PIPE ARCH

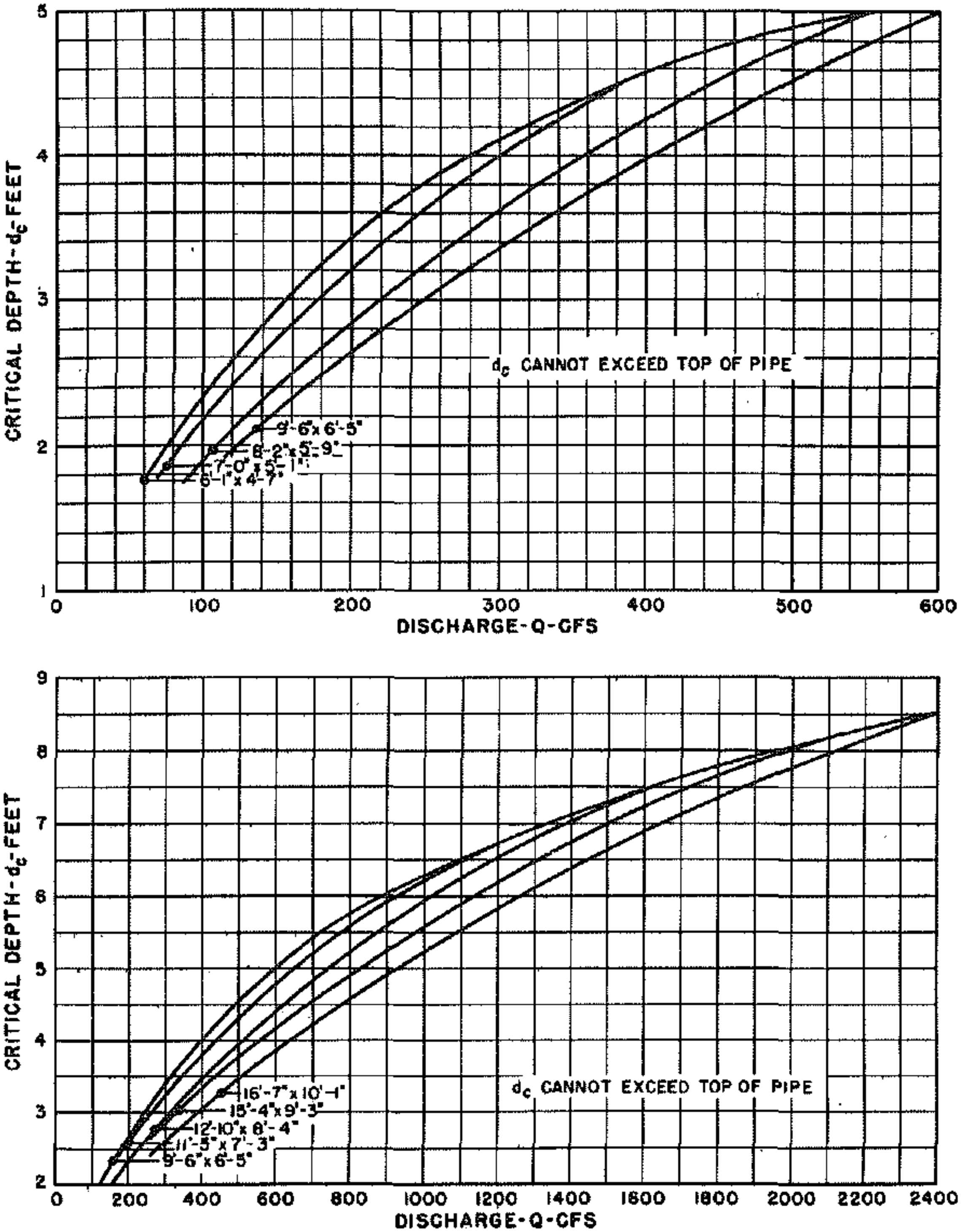


FIGURE VIII-34

CRITICAL DEPTH STRUCTURAL PLATE C.M. PIPE-ARCH
18 INCH CORNER RADIUS

TABLE VIII-4

VALUES OF $BD^{3/2}$

B x D	$BD^{3/2}$	B x D	$BD^{3/2}$	B x D	$BD^{3/2}$
4 x 4	32.0	7 x 7	129.6	10 x 10	316.2
5 x 4	40.0	8 x 7	148.2	12 x 10	379.4
6 x 4	48.0	9 x 7	166.7	14 x 10	442.7
7 x 4	56.0	10 x 7	185.2	16 x 10	505.9
8 x 4	64.0	12 x 7	222.2		
		14 x 7	259.3	12 x 12	498.8
5 x 5	55.9			14 x 12	582.0
6 x 5	67.1	8 x 8	181.0	16 x 12	665.1
7 x 5	78.3	9 x 8	203.7	18 x 12	748.3
8 x 5	89.4	10 x 8	226.3		
9 x 5	100.6	12 x 8	271.6	14 x 14	733.3
10 x 5	111.8	14 x 8	316.8	16 x 14	838.1
				18 x 14	942.8
6 x 6	88.2	9 x 9	243.0		
7 x 6	102.9	10 x 9	270.0		
8 x 6	117.6	12 x 9	324.0		
9 x 6	132.3	14 x 9	378.0		
10 x 6	147.0				
12 x 6	176.4				

TABLE VIII-5

VALUES OF $D^{3/2}$

D	$D^{3/2}$	D	$D^{3/2}$	D	$D^{3/2}$
4	8.0	8	22.6	12	41.6
5	11.2	9	27.0	13	46.9
6	14.7	10	31.6	14	52.4
7	18.5	11	36.5	15	58.1

TABLE VIII-6

VALUES OF $D^{5/2}$

D	$D^{5/2}$	D	$D^{5/2}$	D	$D^{5/2}$
1.0	1.0	5.0	55.9	9.0	243.0
1.5	2.8	5.5	70.9	9.5	278.2
2.0	5.7	6.0	88.2	10.0	316.2
2.5	9.9	6.5	107.7	10.5	357.3
3.0	15.6	7.0	129.6	11.0	401.3
3.5	22.9	7.5	154.0	11.5	448.5
4.0	32.0	8.0	181.0	12.0	498.8
4.5	43.0	8.5	210.6	12.5	552.4

TABLE VIII-7

VALUES OF $E^{1/2}$

E	$E^{1/2}$	E	$E^{1/2}$	E	$E^{1/2}$
1.0	1.00	5.0	2.24	9.0	3.00
1.5	1.22	5.5	2.35	9.5	3.08
2.0	1.41	6.0	2.45	10.0	3.16
2.5	1.58	6.5	2.55	10.5	3.24
3.0	1.73	7.0	2.65	11.0	3.32
3.5	1.87	7.5	2.74	11.5	3.39
4.0	2.00	8.0	2.83	12.0	3.46
4.5	2.12	8.5	2.92	12.5	3.54

TABLE VIII-8

Area in Square Feet of Elliptical Sections

$$\left(A_f = \pi/4\ B_f E \text{ or } A_f = \pi/4\ E^2\ \frac{B_f}{E}\right)$$

B_f \ E	24"	30"	36"	42"	48"	54"	60"	66"	72"	78"	84"	90"	96"	102"	108"
24"	3.14	------	------	------	------	------	------	------	------	------	------	------	------	------	------
30"	3.93	4.91	------	------	------	------	------	------	------	------	------	------	------	------	------
36"	4.71	5.89	7.07	------	------	------	------	------	------	------	------	------	------	------	------
42"	5.50	6.87	8.25	9.62	------	------	------	------	------	------	------	------	------	------	------
48"	6.28	7.85	9.42	11.00	12.56	------	------	------	------	------	------	------	------	------	------
54"	7.07	8.84	10.60	12.37	14.14	15.90	------	------	------	------	------	------	------	------	------
60"	7.85	9.82	11.78	13.74	15.71	17.67	19.63	------	------	------	------	------	------	------	------
66"	8.64	10.8	12.96	15.12	17.28	19.44	21.60	23.76	------	------	------	------	------	------	------
72"	9.42	11.78	14.13	16.49	18.85	21.21	23.56	25.92	28.27	------	------	------	------	------	------
78"		12.76	15.32	17.87	20.42	22.97	25.52	28.08	30.63	33.18	------	------	------	------	------
84"		13.74	16.49	19.24	21.99	24.74	27.48	30.24	32.98	35.74	38.48	------	------	------	------
90"			17.67	20.62	23.56	26.51	29.45	32.40	35.34	38.29	41.23	44.18	------	------	------
96"			18.85	21.99	25.13	28.27	31.41	34.56	37.69	40.84	43.97	47.12	50.26	------	------
102"			20.03	23.37	26.70	30.04	33.38	36.72	40.05	43.39	46.73	50.07	53.41	56.75	------
108"			21.2	24.74	28.27	31.81	35.34	38.88	43.40	45.95	49.47	53.01	56.54	60.08	63.61
120"				27.49	31.41	35.34	39.26	43.20	47.12	51.05	54.97	58.91	62.82	66.76	70.67
132"					34.55	38.88	43.19	47.52	51.83	56.16	60.46	64.80	69.10	73.43	77.74
144"					37.69	42.41	47.12	51.84	56.54	61.26	65.96	70.69	75.38	80.11	84.81
156"						45.95	51.04	56.16	61.25	66.37	71.46	76.58	81.67	86.79	91.87
168"							54.97	60.48	65.96	71.47	76.95	82.47	87.95	93.46	98.94
180"							58.89	64.80	70.67	76.58	82.45	88.36	94.23	100.14	106.00
192"								69.12	75.38	81.68	87.95	94.25	100.51	106.81	113.08

TABLE VIII-9

AREA OF FLOW PRISM IN PARTLY FULL CIRCULAR CONDUIT

Let $\frac{\text{Depth of Water}}{\text{Diameter of Conduit}} = \frac{y'}{D}$ and tabulated Value = C_a. Then Area = $C_a D^2$

$\frac{y'}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.0000	.00013	.00037	.0069	.0105	.0147	.0192	.0242	.0294	.0350
.1	.0409	.0470	.0534	.0600	.0668	.0739	.0811	.0885	.0961	.1039
.2	.1118	.1199	.1281	.1365	.1449	.1535	.1623	.1711	.1800	.1890
.3	.1982	.2074	.2167	.2260	.2355	.2450	.2546	.2642	.2739	.2836
.4	.2934	.3032	.3130	.3229	.3328	.3428	.3527	.3627	.3727	.3827
.5	.393	.403	.413	.423	.433	.443	.453	.462	.472	.482
.6	.492	.502	.512	.521	.531	.540	.550	.559	.569	.578
.7	.587	.596	.605	.614	.623	.632	.640	.649	.657	.666
.8	.674	.681	.689	.697	.704	.712	.719	.725	.732	.738
.9	.745	.750	.756	.761	.766	.771	.775	.779	.782	.784

culvert size and total cost. The reduction in sediment capacity will almost always be of some significance after the flood peak has passed in the upstream channel but it is normally maintenance problem. However, an already aggrading stream may be subject to avulsion during high flows and this condition needs special investigations.

Erosion. Because the basic design approach to culvert design results in high exit velocities, erosional problems can be expected at the outlet. Where the water surface profile is steep (unsubmerged) erosion protection is required.

Skewed Channels. Skewed culverts (culverts not parallel to the direction of flow) will not be acceptable, unless in the opinion of the City Engineer that no other alignment is reasonable.

Uplift and Bending at Inlet

Because of a combination of high head on the outside of the inlet and the large region of low pressure on the inside of the inlet due to separation, a large bending movement is exerted on the end of the culvert which may result in failure. This problem has been noted in the case of culverts under high fills, on steep slopes, and with projecting inlets. Where upstream detention storage requires headwater depth in excess of 20 feet, reducing the culvert size is recommended rather than using the inefficient projecting inlet to reduce discharge.

TRASH RACKS

The use of typical gratings at inlets to culverts is not compatible with the use of the culvert for carrying storm runoff. While there is a sound argument for the use of gratings for safety reasons, field experience has clearly shown that when the culvert is needed to most, that is during the heavy runoff, normal gratings often become clogged and the culvert is rendered ineffective.

It is not, however, necessary to rule out use of all gratings for effective storm drainage work. Rather, it is a matter of working out the safety hazard aspects of the problem with care, defining them clearly and then taking reasonable steps to minimize safety hazards, yet protecting the integrity of the water carrying capability of the culvert.

Generally, the most common aspect involved in considering the safety hazard of a culvert opening is in regard to the possibility of children being carried into the culvert by the upstream flowing water. In reviewing hazards, it is necessary to consider depth and velocity of upstream flow, the fact that storm runoff occurs during unusually heavy rain storms when most people, particularly children, are indoors, the general character of the neighborhood upstream, the length and size of culvert, and other similar factors. Furthermore, in the event that someone is carried to the culvert with the storm runoff, the exposure hazard may in some cases be even greater if the person is pinned to the grating by the hydrosatic pressure of the water rather than being carried through the culvert. The designer is referenced to Debris-Control Structural (13) for more detailed discussions for selecting and designing trash racks.

A frequent objection to the use of improved inlets on highway culverts is that use of the side- and slope-tapered inlet configurations will increase problems with drift and debris. As with conventional culvert design, if the drainage basin will contribute a large amount of drift and debris, the debris control design procedures presented in HEC No. 9 (15) should be utilized.

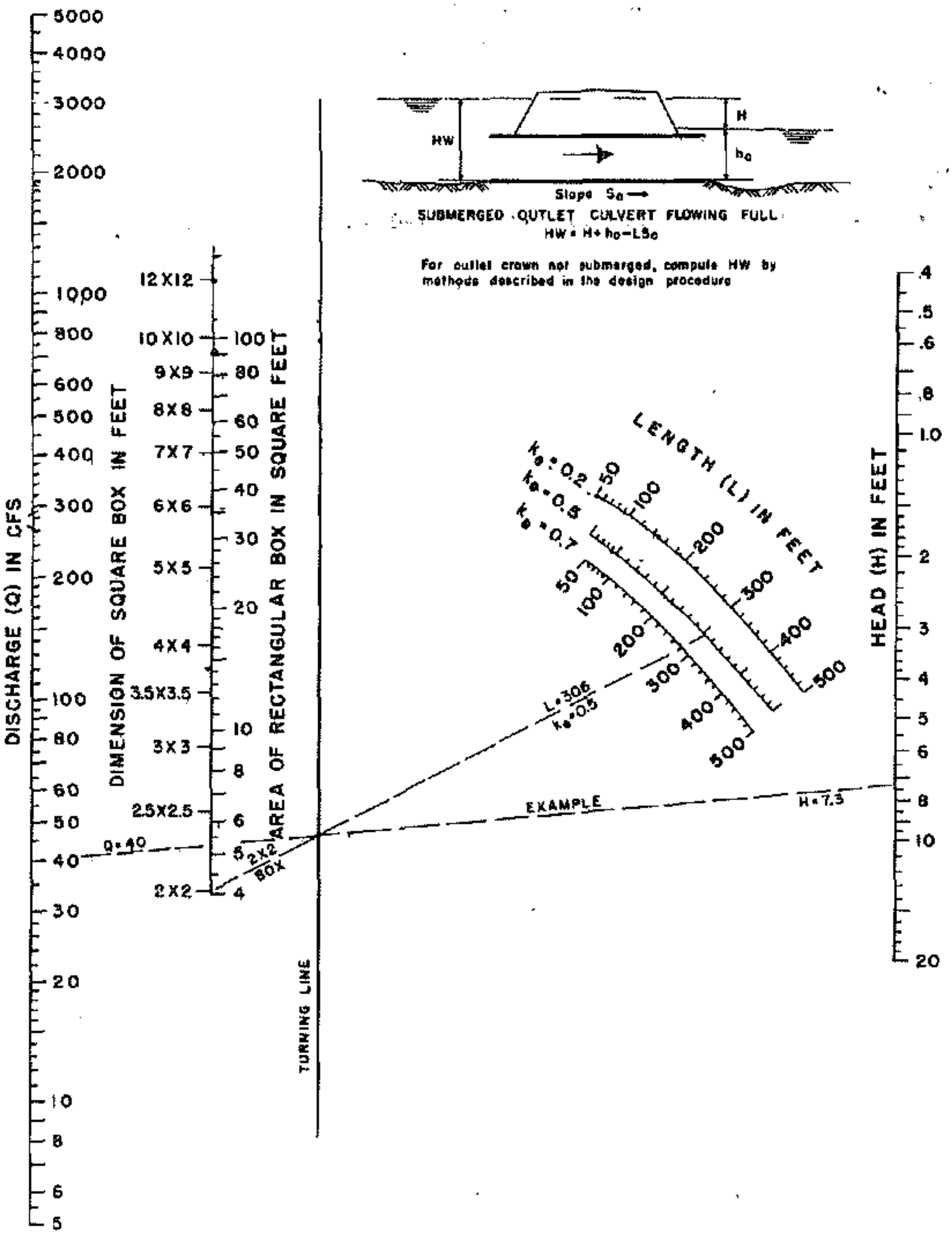
To prevent large drift material from lodging in the the throat section of inlets with side tapers, a vertical column may be placed in the center of the inlet face. Any material passing the face section should then easily clear the culvert throat.

ALLOWABLE HEADWATER ELEVATION

The maximum permissible elevation of the headwater pool of the culvert at the design discharge is termed the Allowable Headwater Elevation. This elevation must be selected by the designer based on his evaluation of many factors, all of which should be well documented. These include highway elevations, upstream development and land use, feature elevations, historical high water marks, importance of the highway, and damage risks. Possible loss of life and property, and traffic delay and interruption should be considered in the damage risk analysis.

DESIGN CHARTS

The following design charts are to be used in culvert design.



SUBMERGED OUTLET CULVERT FLOWING FULL

$HW = H + h_o - LS_o$

For outlet crown not submerged, compute HW by methods described in the design procedure

CHART VIII-1

HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL n=0.012

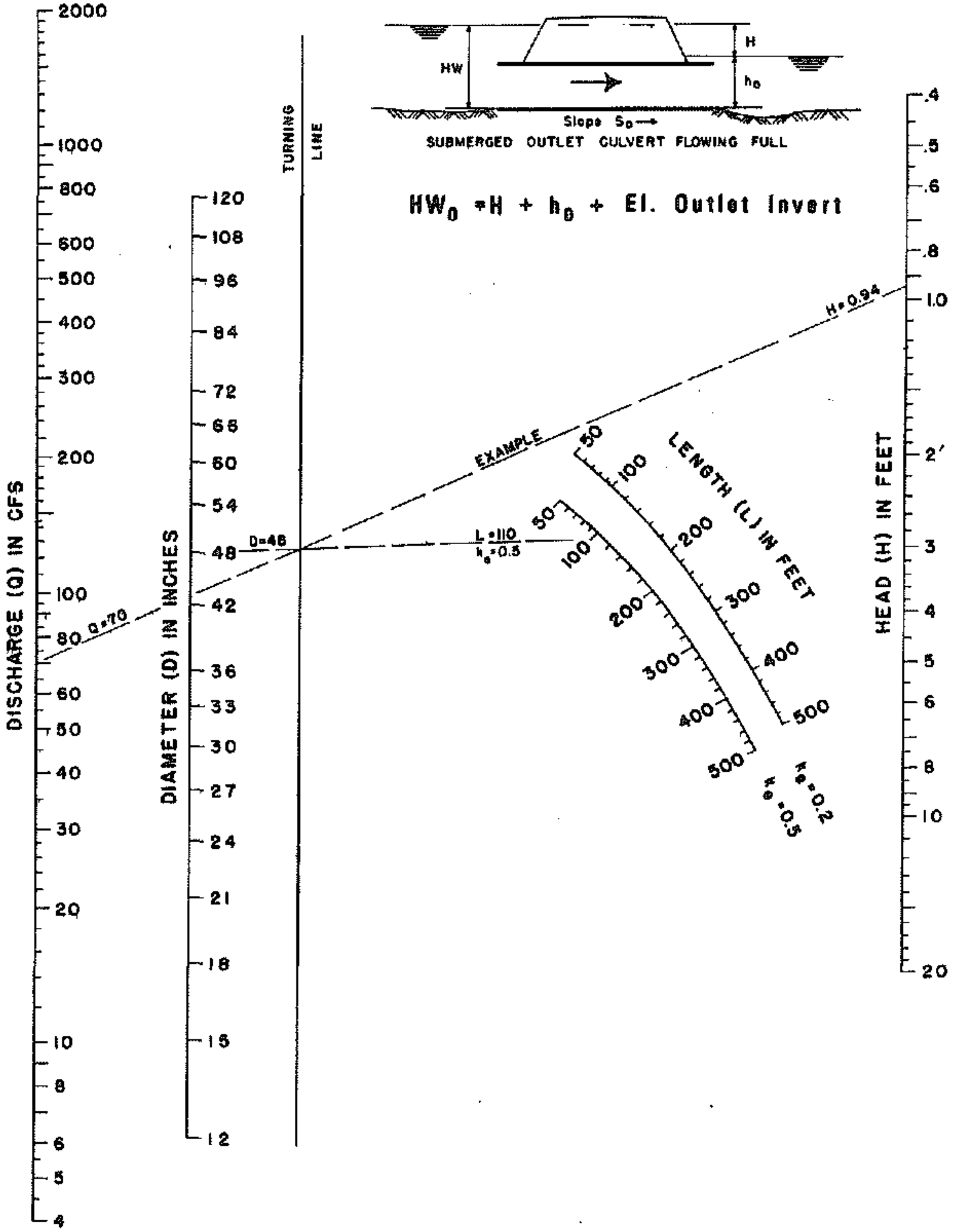
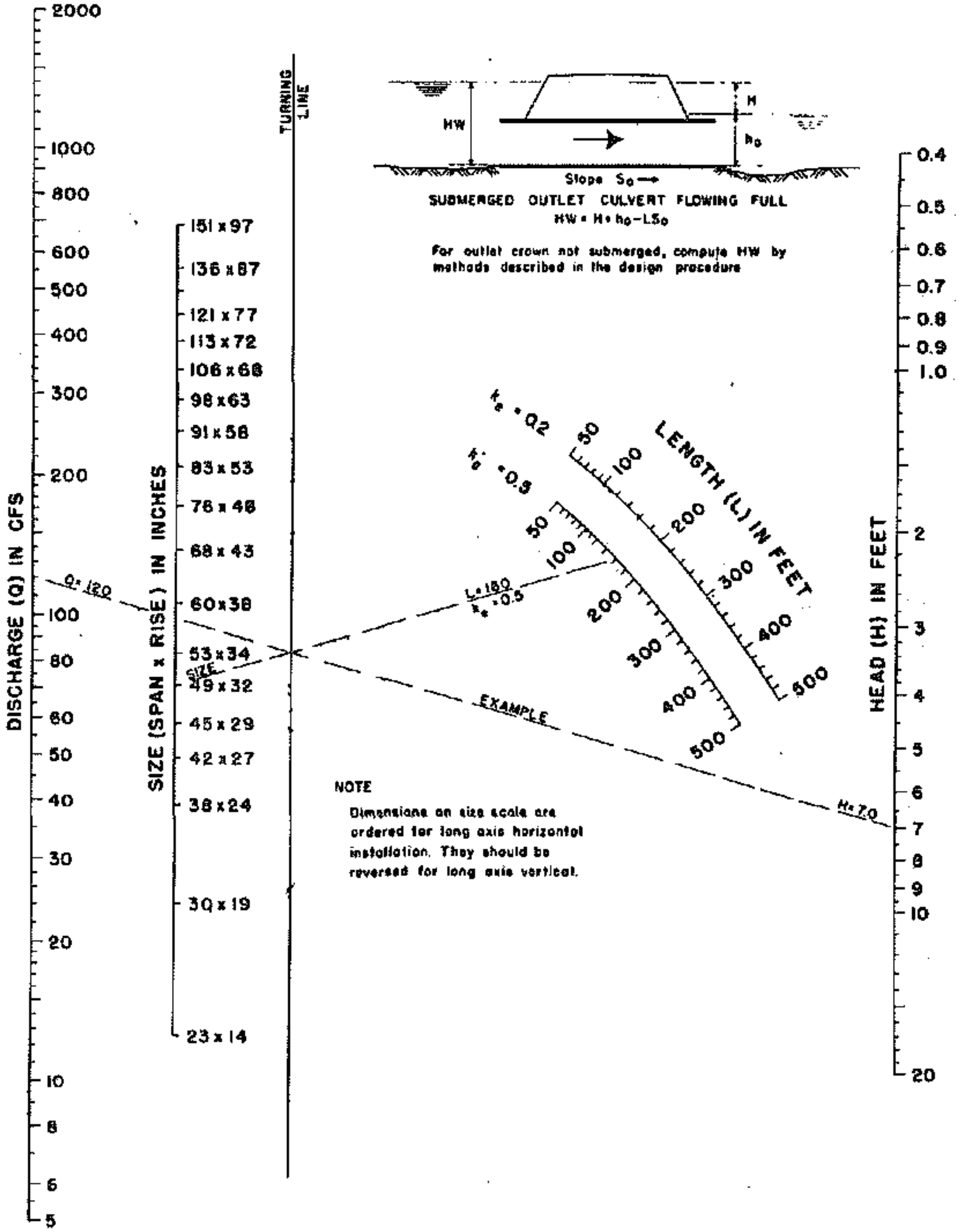


CHART VIII-2
HEAD FOR CONCRETE PIPE CULVERT
FLOWING FULL
n=0.012



SUBMERGED OUTLET CULVERT FLOWING FULL

$HW = H + h_o - LS_o$

For outlet crown not submerged, compute HW by methods described in the design procedure

NOTE

Dimensions on size scale are ordered for long axis horizontal installation. They should be reversed for long axis vertical.

CHART VIII-3

HEAD FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZ. VERT. FLOWING FULL n=0.012

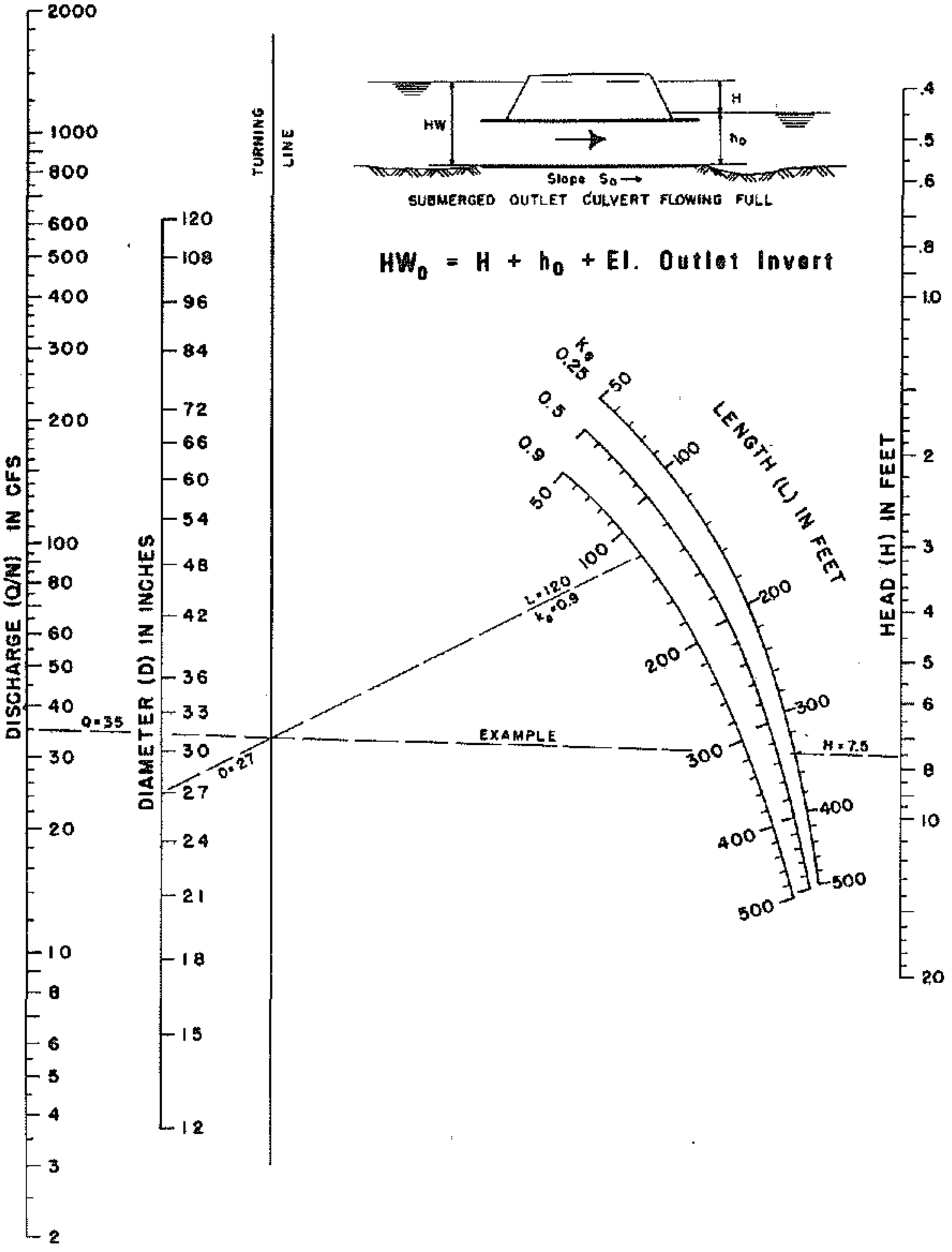


CHART VIII-4
HEAD FOR STANDARD C.M. PIPE CULVERTS FLOWING FULL n = 0.024

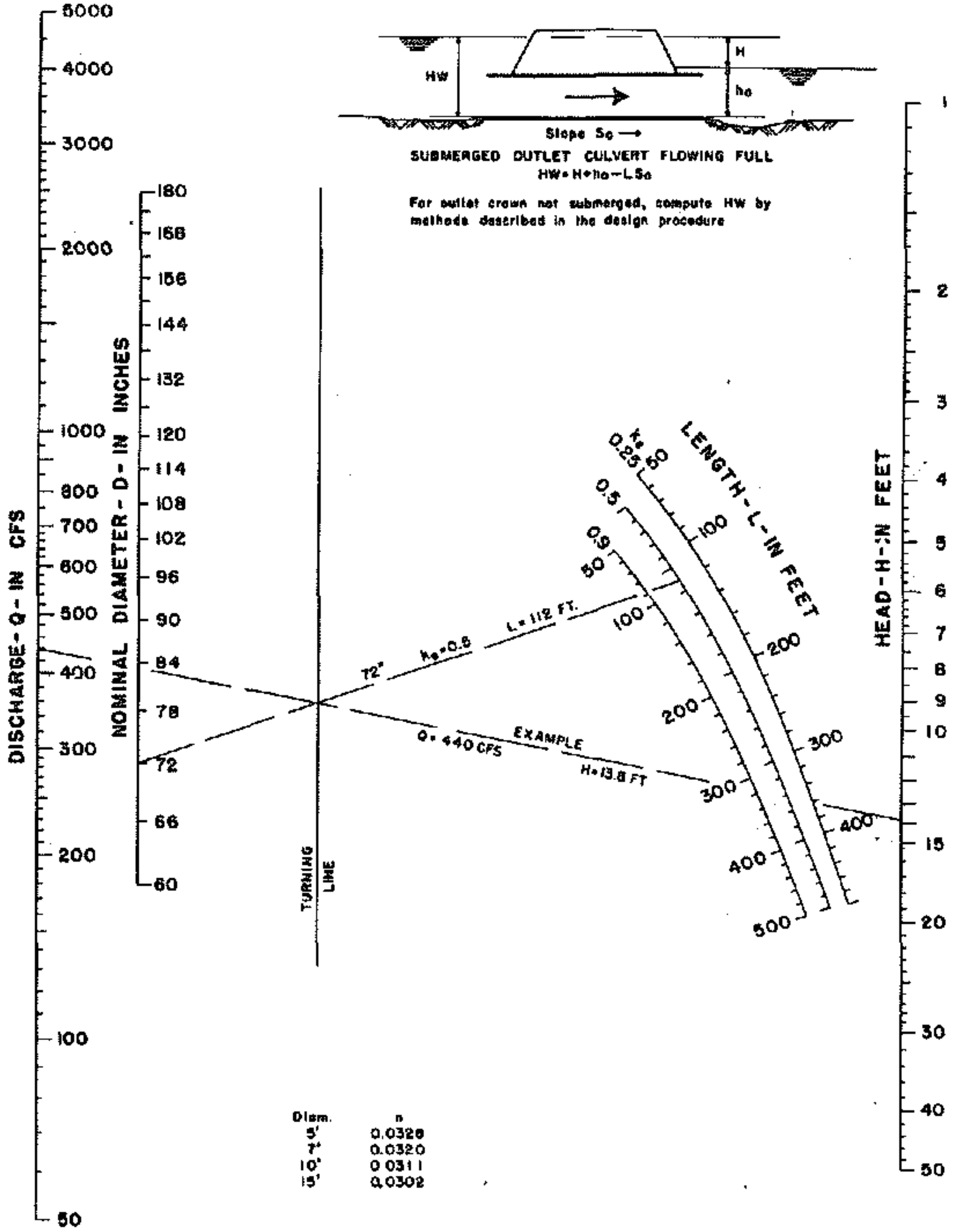


CHART VIII-5

HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL n = 0.0328 TO 0.0302

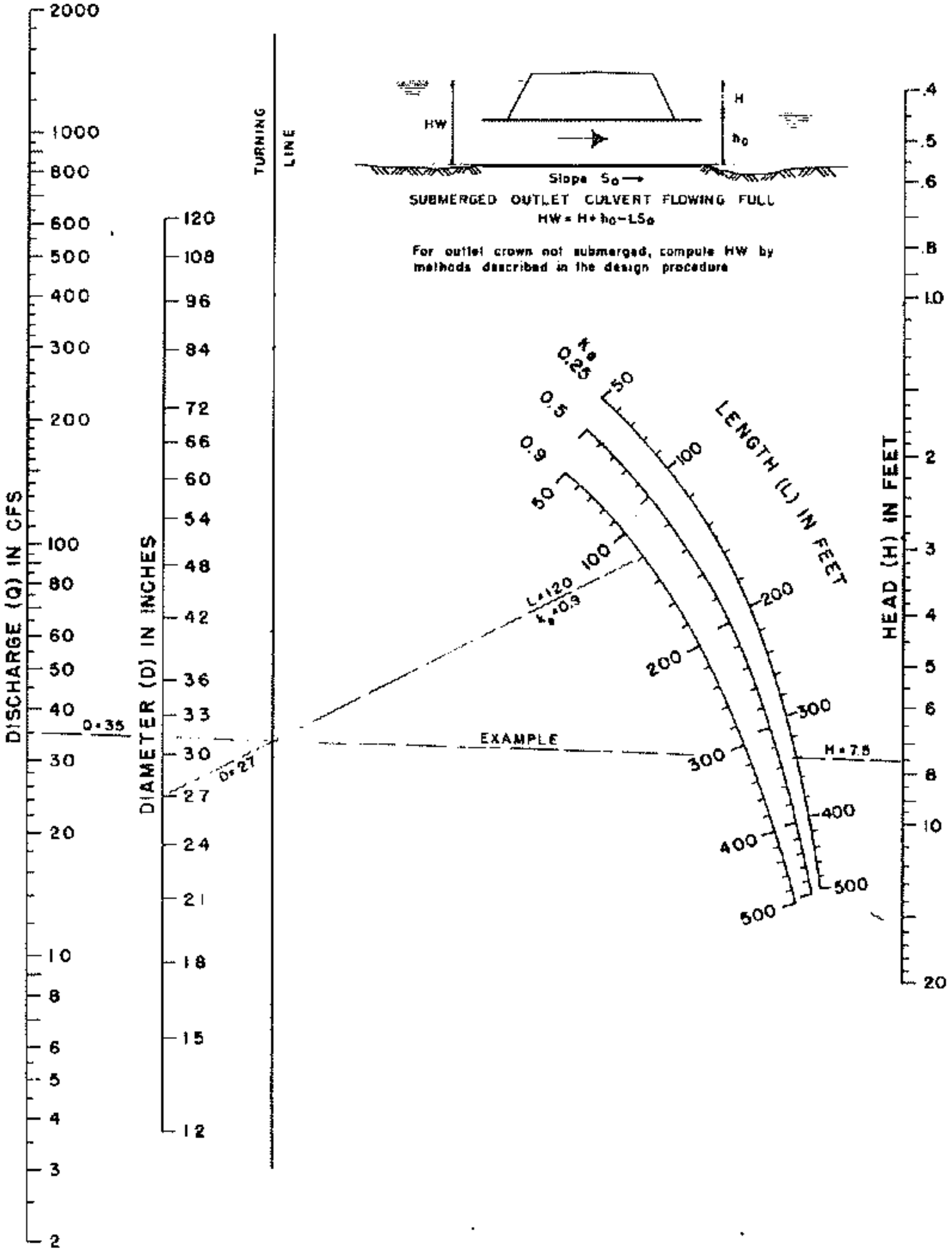
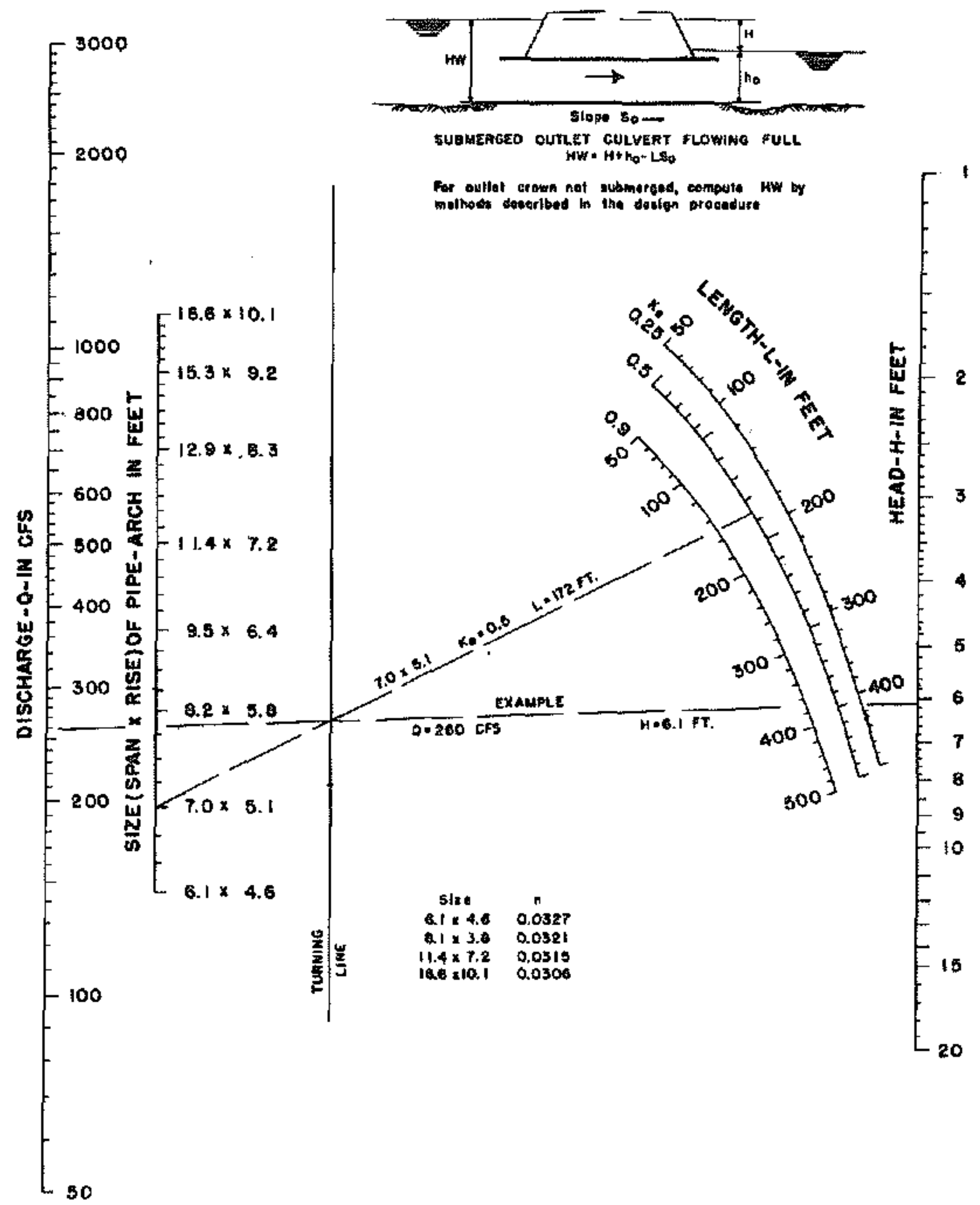


CHART VIII-6

HEAD FOR STANDARD C.M. PIPE-ARCH CULVERTS FLOWING FULL n=0.024



SUBMERGED OUTLET CULVERT FLOWING FULL

$HW = H + h_o - LS_o$

For outlet crown not submerged, compute HW by methods described in the design procedure

Size	n
6.1 x 4.6	0.0327
8.1 x 5.8	0.0321
11.4 x 7.2	0.0315
16.6 x10.1	0.0306

CHART VIII-7

HEAD FOR
STRUCTURAL PLATE CORRUGATED METAL
PIPE ARCH CULVERTS 18 IN. CORNER RADIUS
FLOWING FULL $n=0.0327$ TO 0.0306

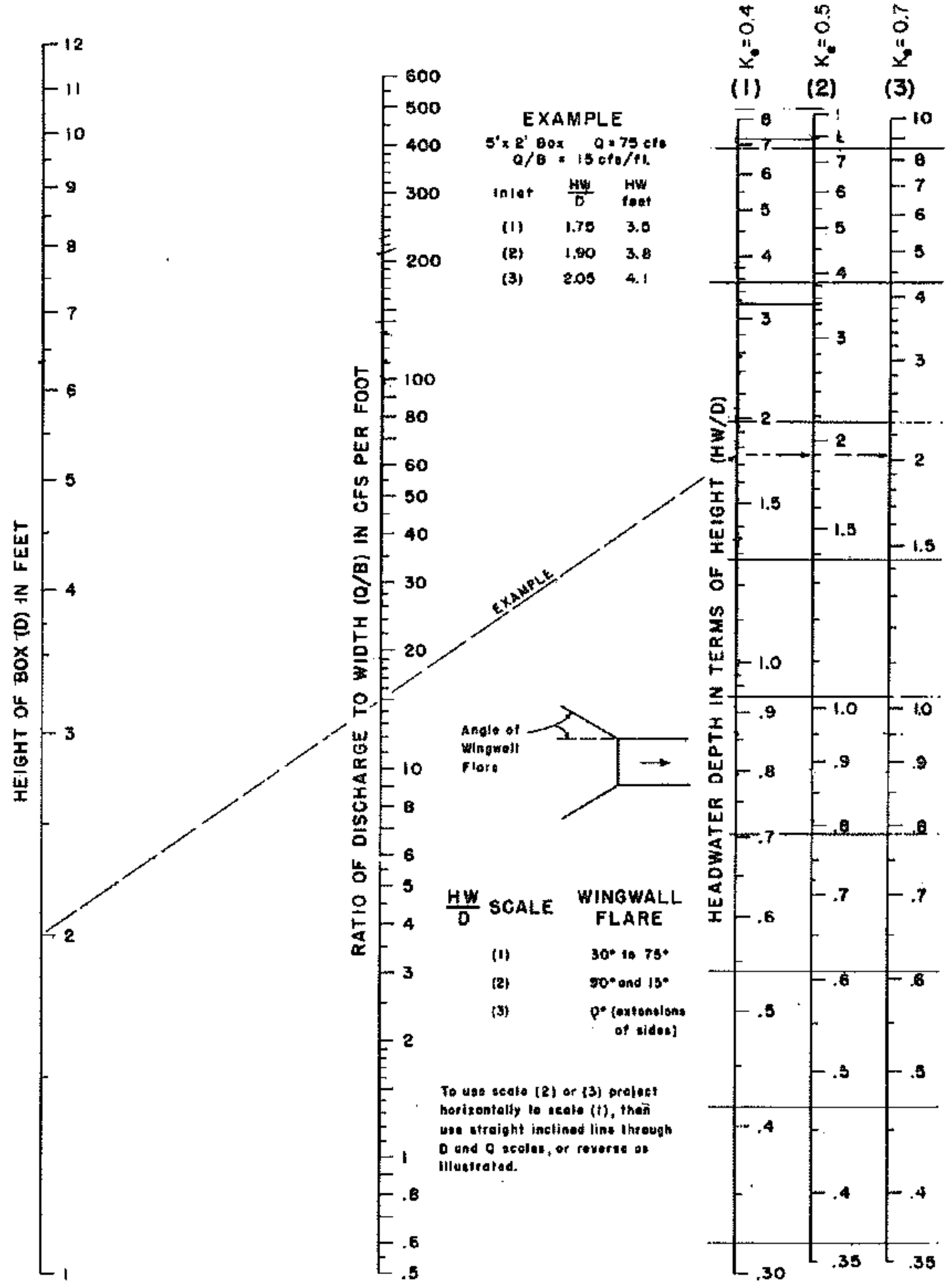


CHART VIII-8
HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

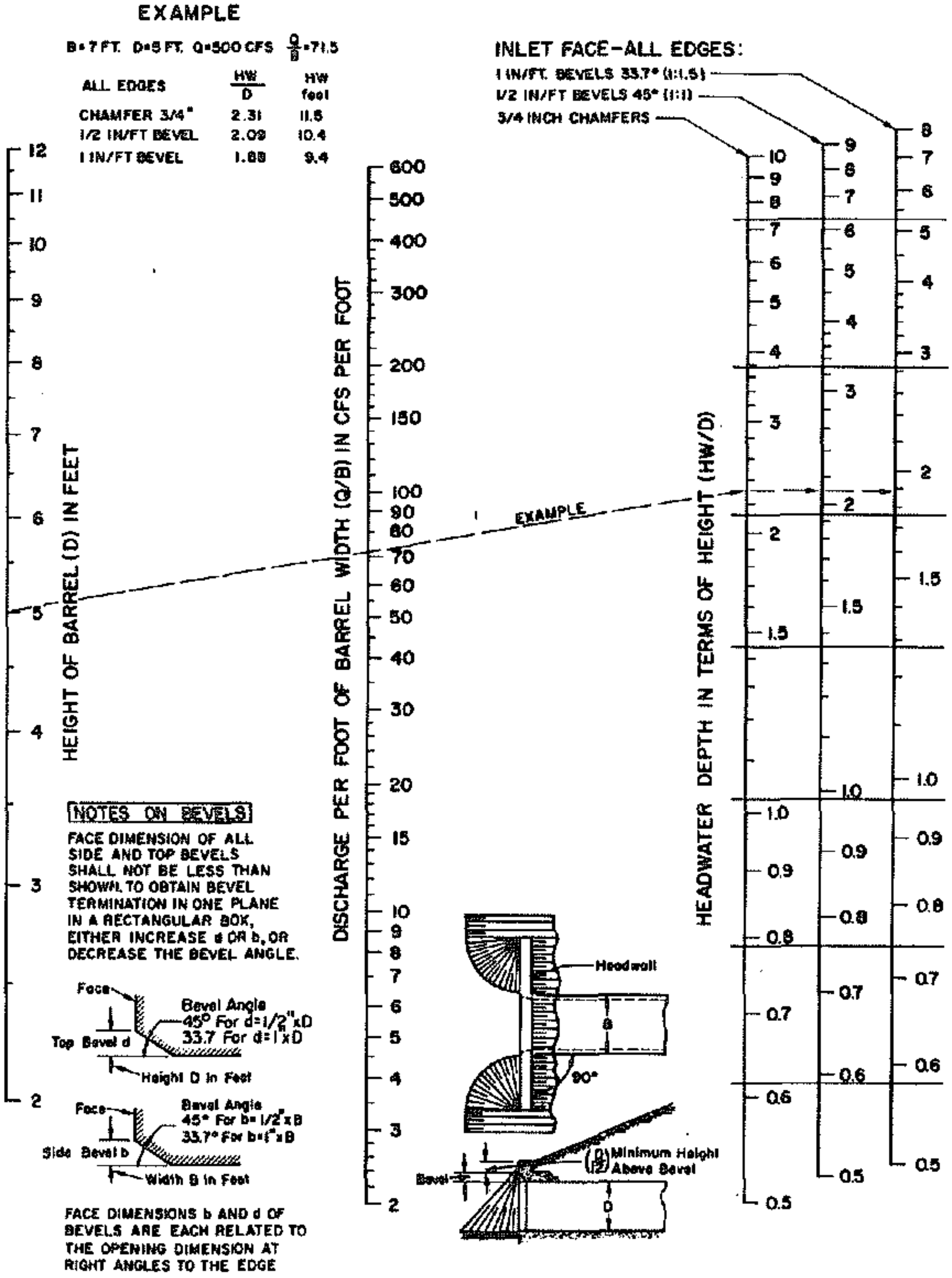
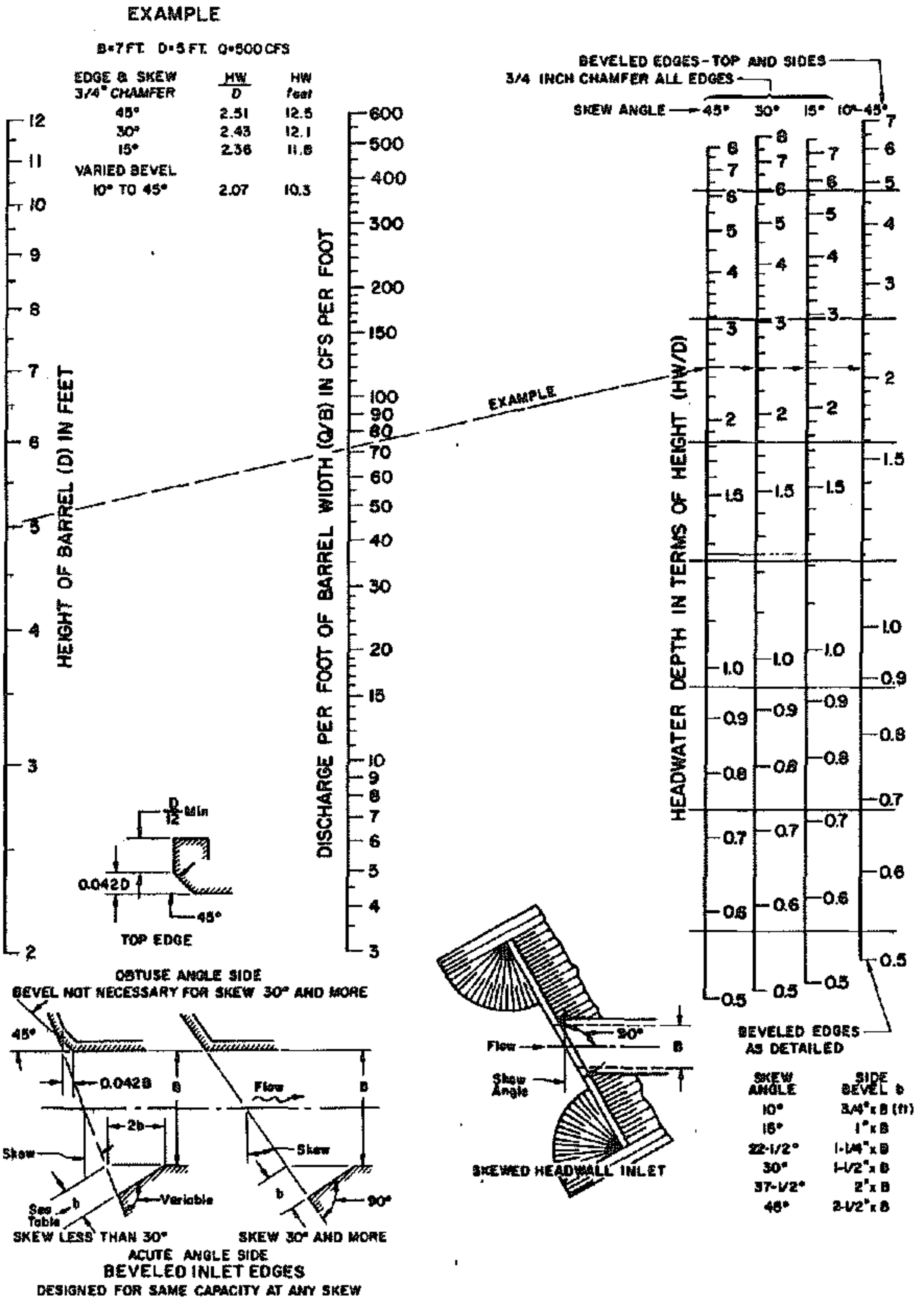


CHART VIII-9

HEADWATER DEPTH FOR INLET CONTROL RECTANGULAR BOX CULVERTS
90° HEADWALL CHAMFERED OR BEVELED INLET EDGES



EDGE & SKEW 3/4" CHAMFER	HW/D	HW feet
45°	2.51	12.5
30°	2.43	12.1
15°	2.36	11.8
VARIED BEVEL 10° TO 45°	2.07	10.3

SKEW ANGLE	SIDE BEVEL b
10°	3/4"x B (ft)
15°	1"x B
22-1/2°	1-1/4"x B
30°	1-1/2"x B
37-1/2°	2"x B
45°	2-1/2"x B

CHART VIII-10

HEADWATER DEPTH FOR INLET CONTROL SINGLE BARREL BOX CULVERTS SKEWED HEADWALLS CHAMFERED OR BEVELED INLET EDGES

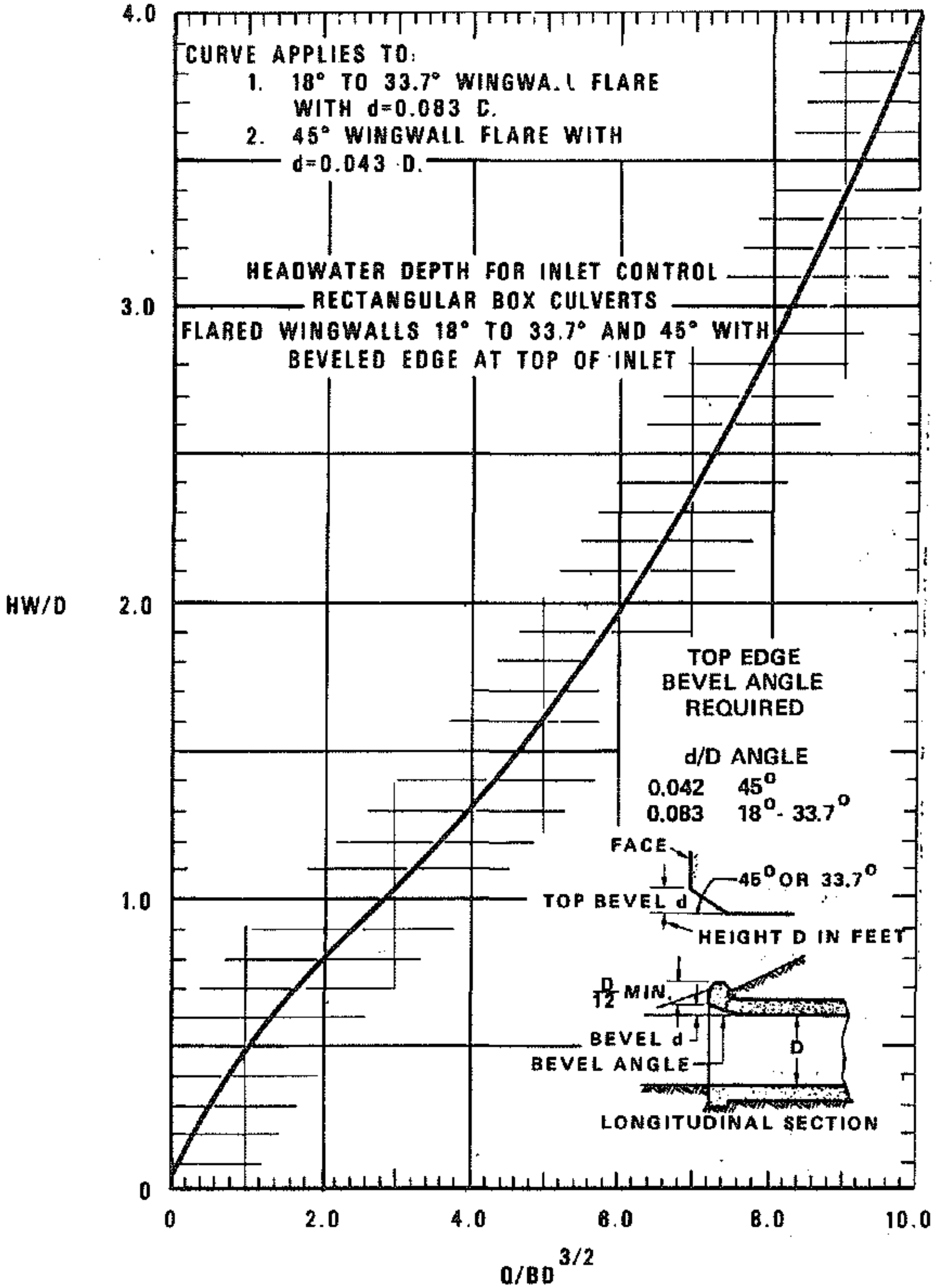
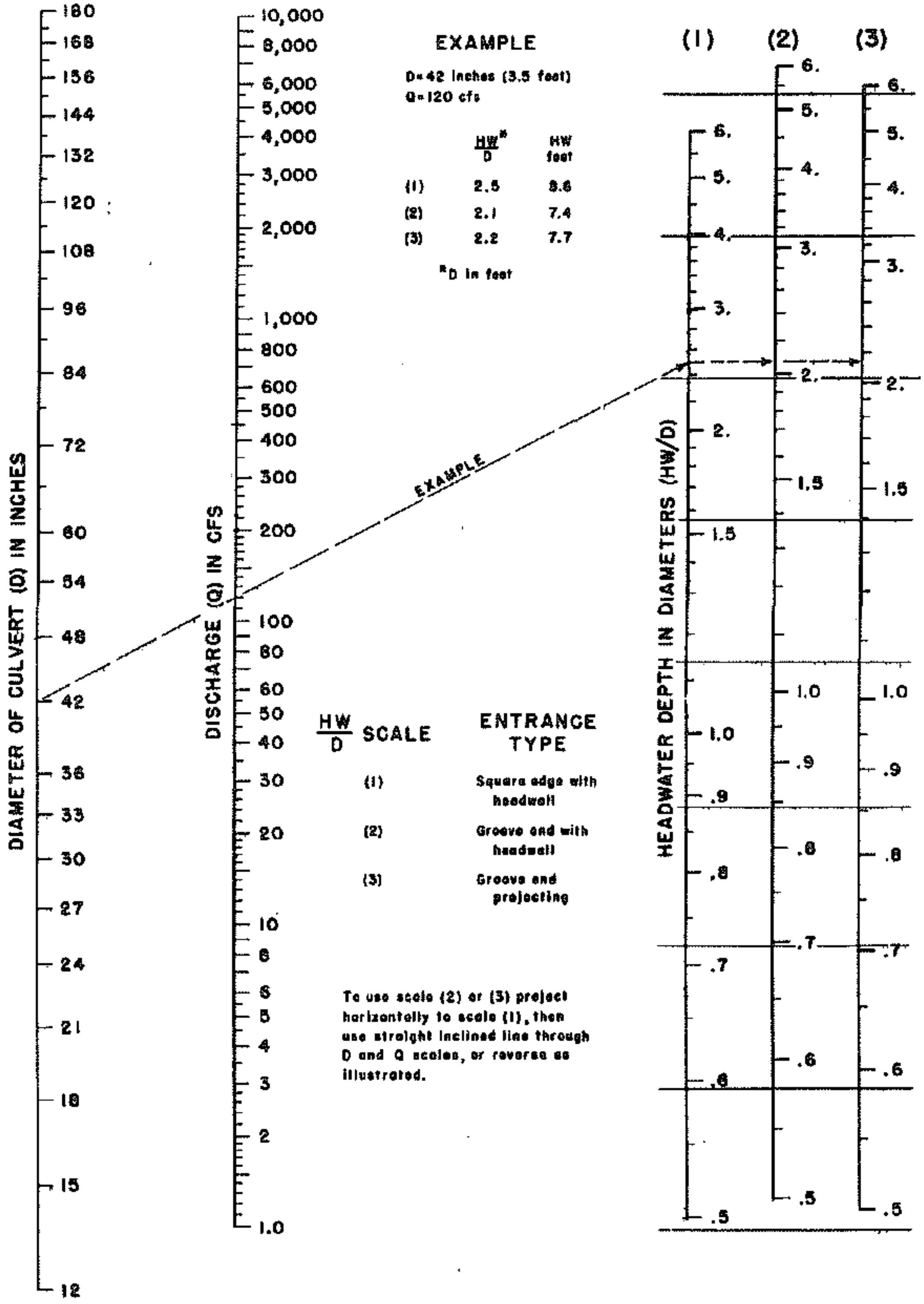


CHART VIII-11

HEADWATER DEPTH FOR INLET CONTROL RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° AND 45° WITH BEVELED EDGE AT TOP OF INLET



EXAMPLE

D=42 inches (3.5 feet)
Q=120 cfs

	$\frac{HW}{D}$*	HW feet
(1)	2.5	8.8
(2)	2.1	7.4
(3)	2.2	7.7

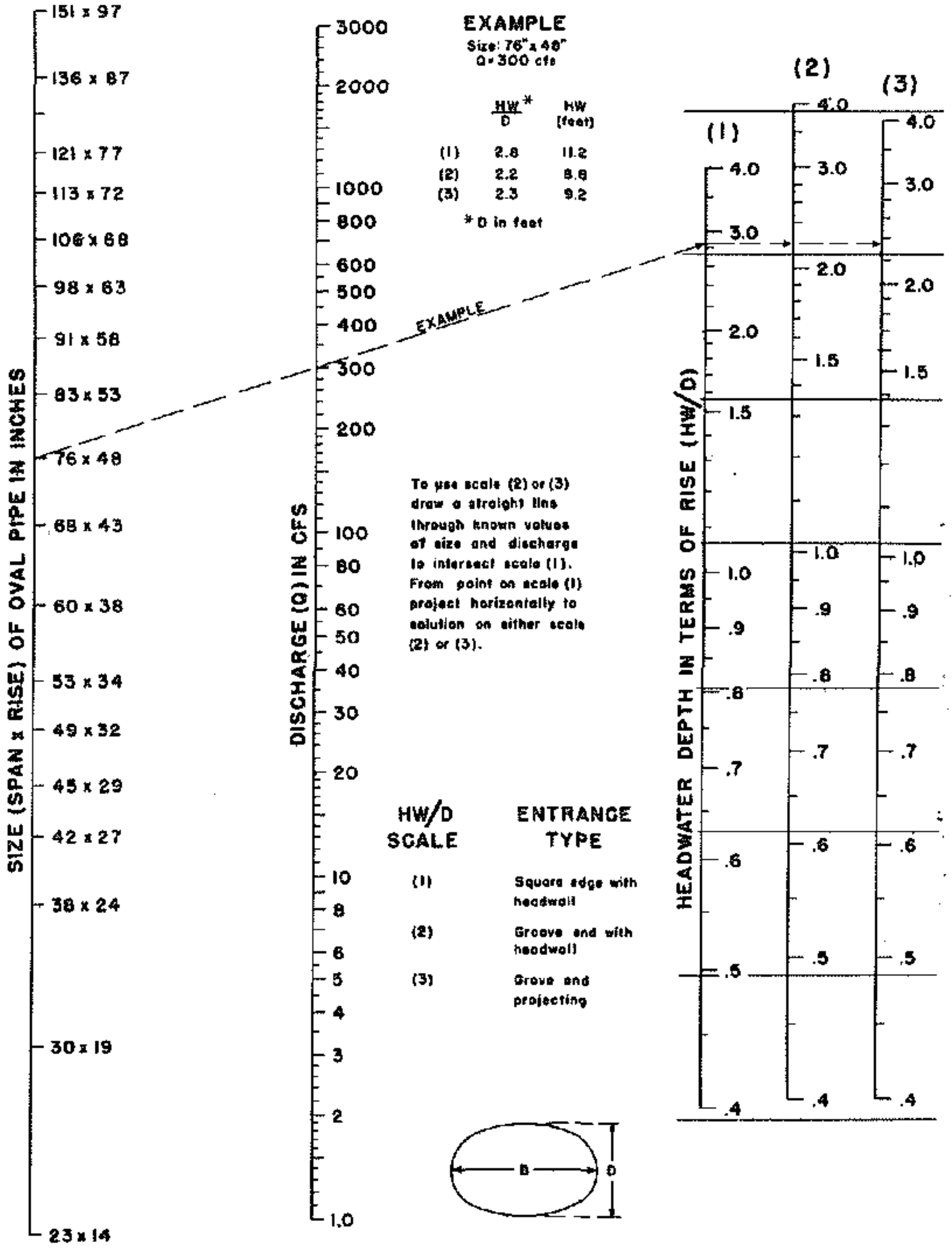
*D in feet

$\frac{HW}{D}$ SCALE	ENTRANCE TYPE
(1)	Square edge with headwall
(2)	Groove end with headwall
(3)	Groove end projecting

To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.

CHART VIII-12

HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL



EXAMPLE

Size: 76"x 48"
Q=300 cfs

	$\frac{HW}{D}$*	HW (feet)
(1)	2.8	11.2
(2)	2.2	8.8
(3)	2.3	9.2

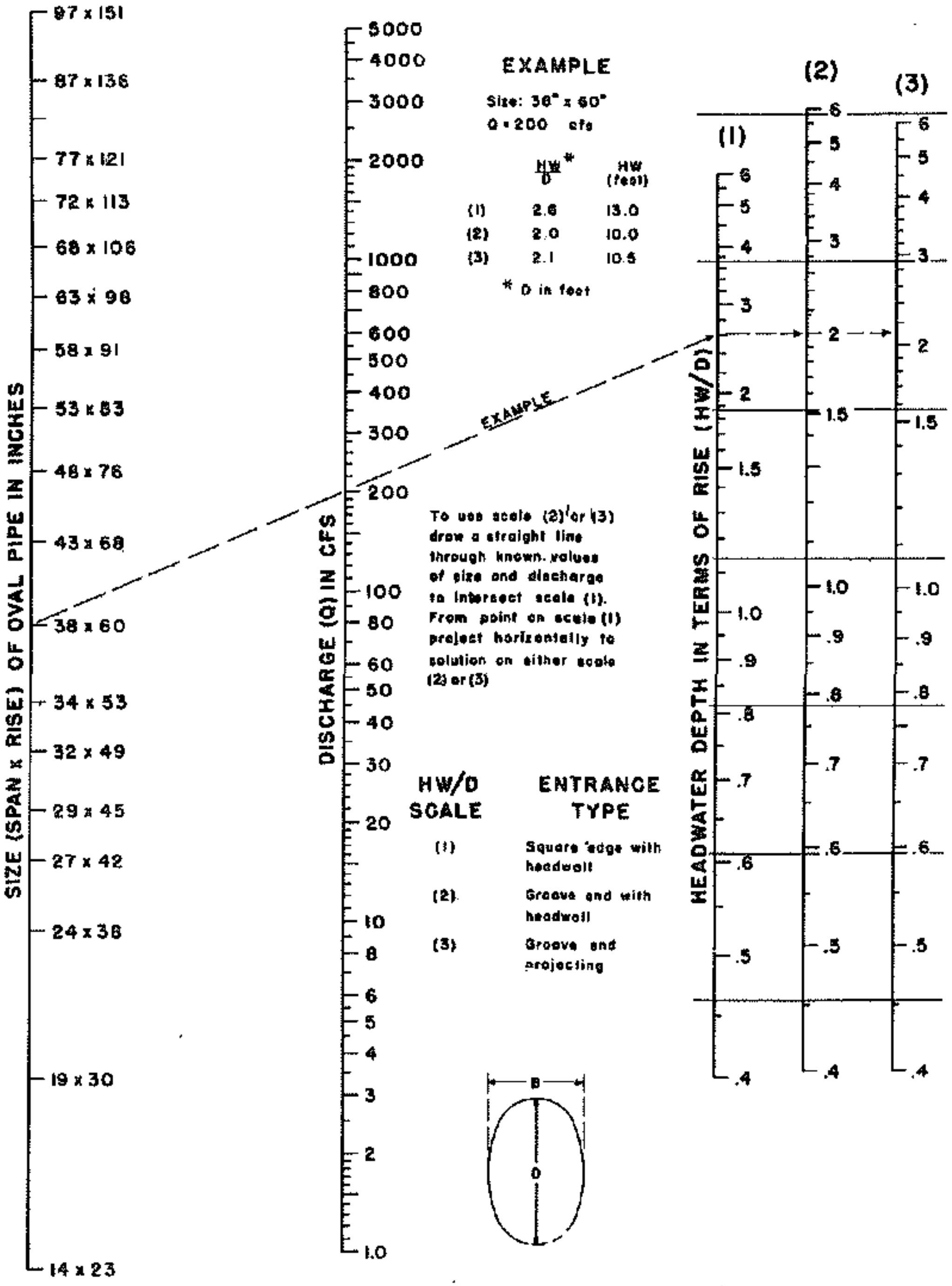
*D in feet

To use scale (2) or (3) draw a straight line through known values of size and discharge to intersect scale (1). From point on scale (1) project horizontally to solution on either scale (2) or (3).

HW/D SCALE	ENTRANCE TYPE
(1)	Square edge with headwall
(2)	Groove end with headwall
(3)	Groove end projecting

CHART VIII-13

HEADWATER DEPTH FOR OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL WITH INLET CONTROL



EXAMPLE

Size: 38" x 60"
Q = 200 cfs

	$\frac{HW}{D}$*	HW (feet)
(1)	2.6	13.0
(2)	2.0	10.0
(3)	2.1	10.5

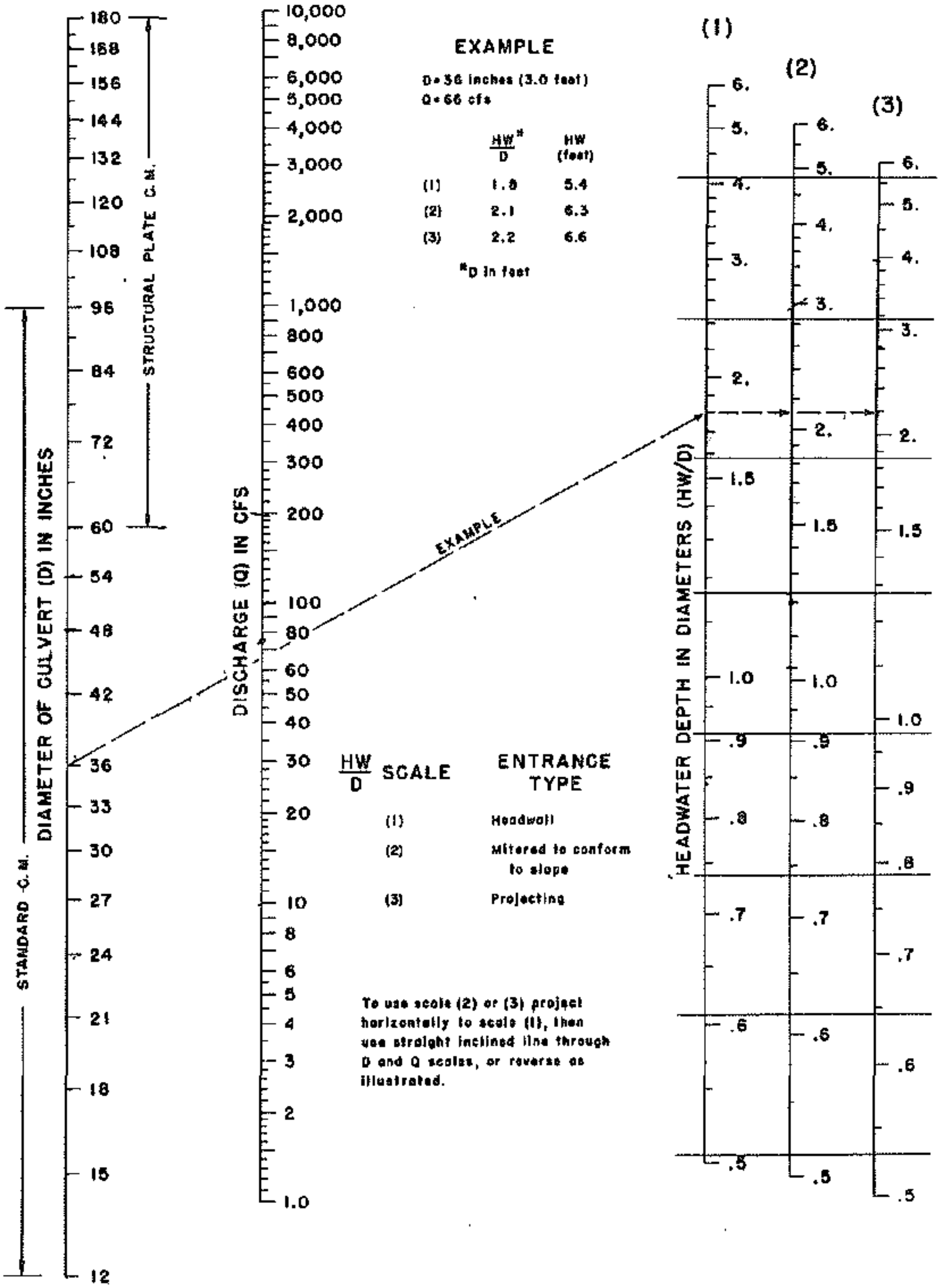
* D in feet

To use scale (2) or (3) draw a straight line through known values of size and discharge to intersect scale (1). From point on scale (1) project horizontally to solution on either scale (2) or (3)

HW/D SCALE	ENTRANCE TYPE
(1)	Square edge with headwall
(2)	Groove end with headwall
(3)	Groove end projecting

CHART VIII-14

HEADWATER DEPTH FOR OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL WITH INLET CONTROL



EXAMPLE

D=36 inches (3.0 feet)
Q=66 cfs

	$\frac{HW}{D}$*	HW (feet)
(1)	1.8	5.4
(2)	2.1	6.3
(3)	2.2	6.6

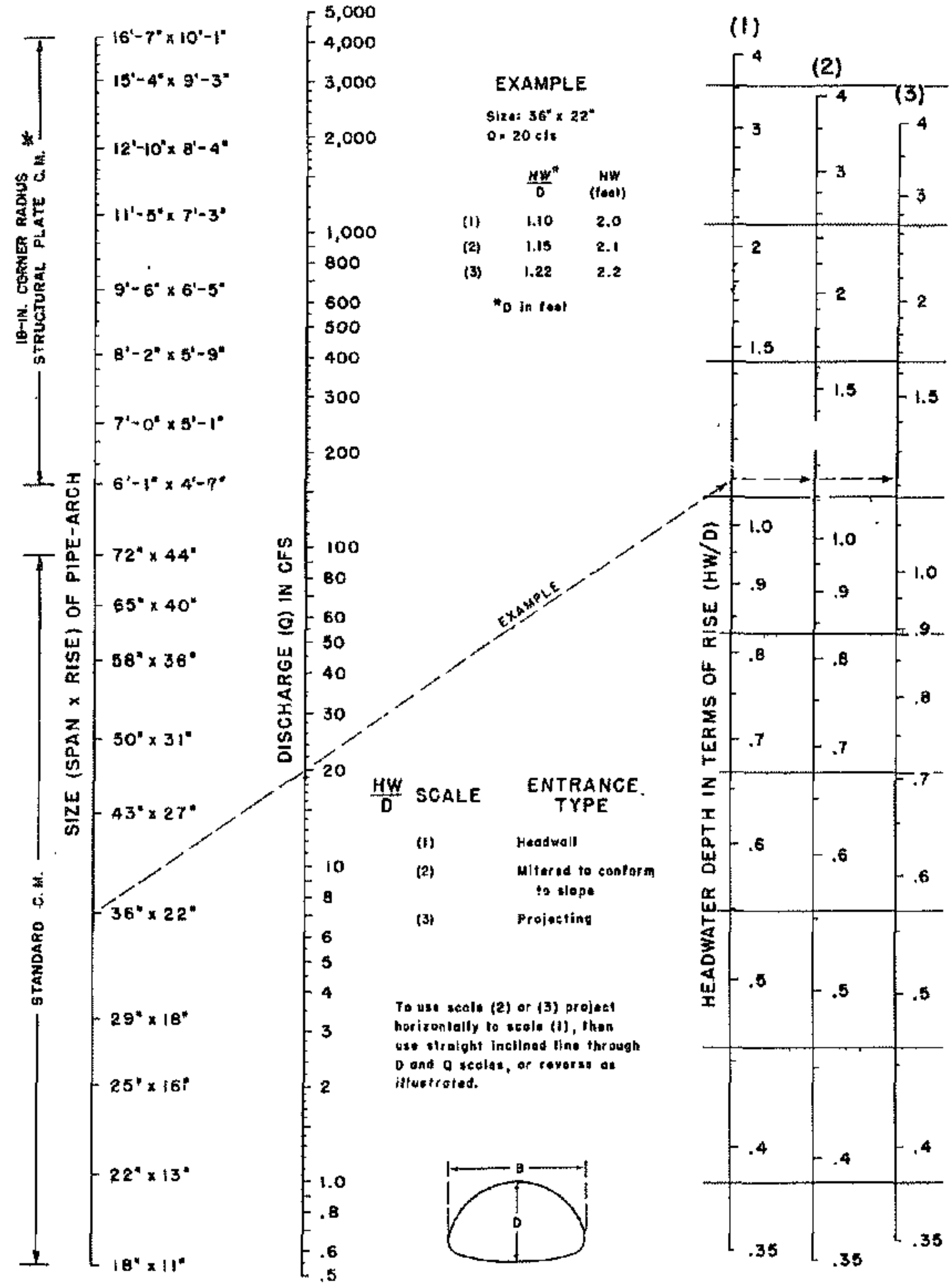
*D in feet

$\frac{HW}{D}$ SCALE	ENTRANCE TYPE
(1)	Headwall
(2)	Mitered to conform to slope
(3)	Projecting

To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.

CHART VIII-15

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



EXAMPLE

Size: 36" x 22"
Q = 20 cfs

	$\frac{HW}{D}$*	HW (feet)
(1)	1.10	2.0
(2)	1.15	2.1
(3)	1.22	2.2

*D in feet

$\frac{HW}{D}$ SCALE	ENTRANCE TYPE
(1)	Headwall
(2)	Mitered to conform to slope
(3)	Projecting

To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.

* Additional sizes not dimensioned are listed in fabricator's catalog

CHART VIII-16

HEADWATER DEPTH FOR C.M. PIPE-ARCH CULVERTS WITH INLETS CONTROL

$\frac{a}{D}$	$\frac{b}{D}$	$\frac{c}{D}$	$\frac{d}{D}$	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B

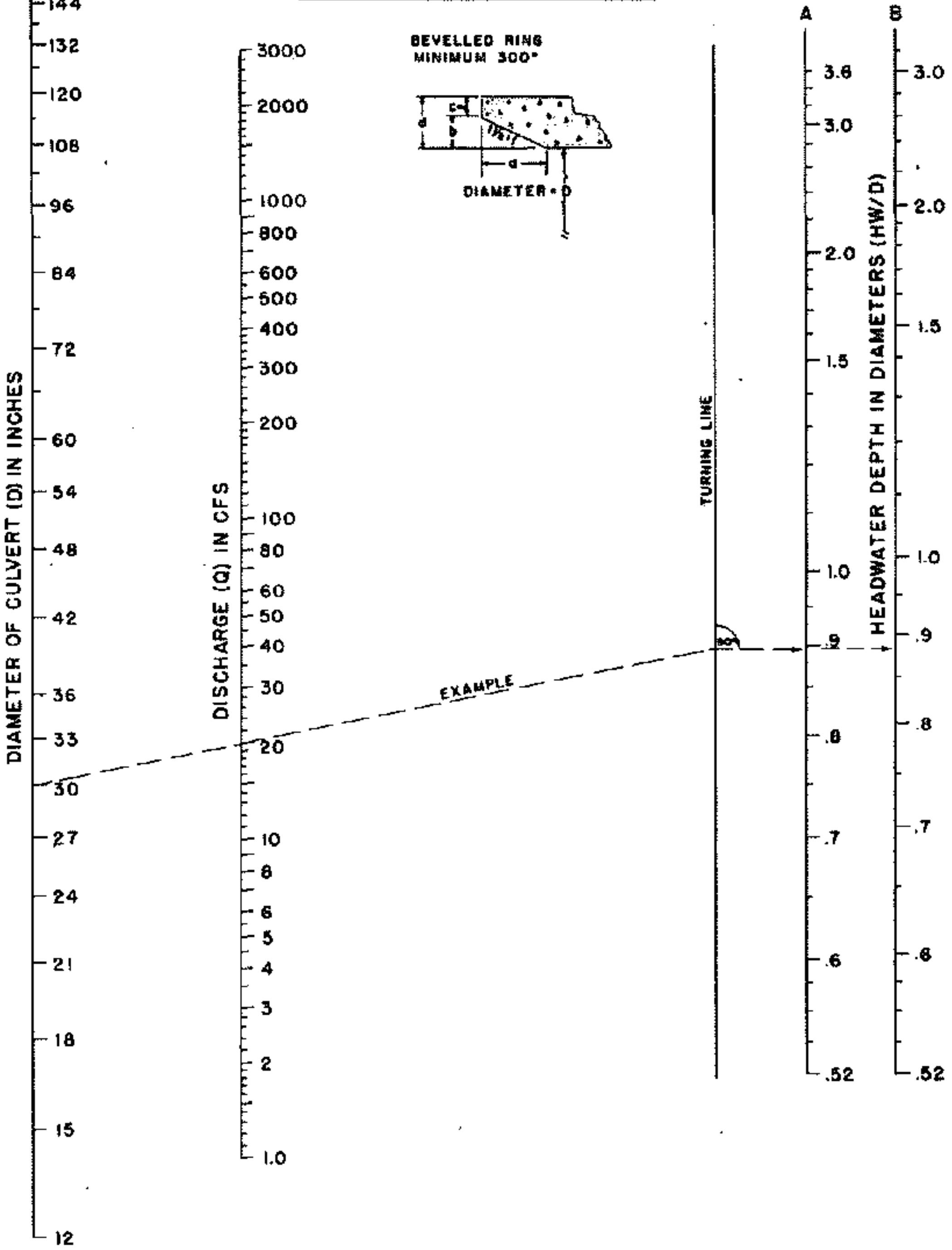
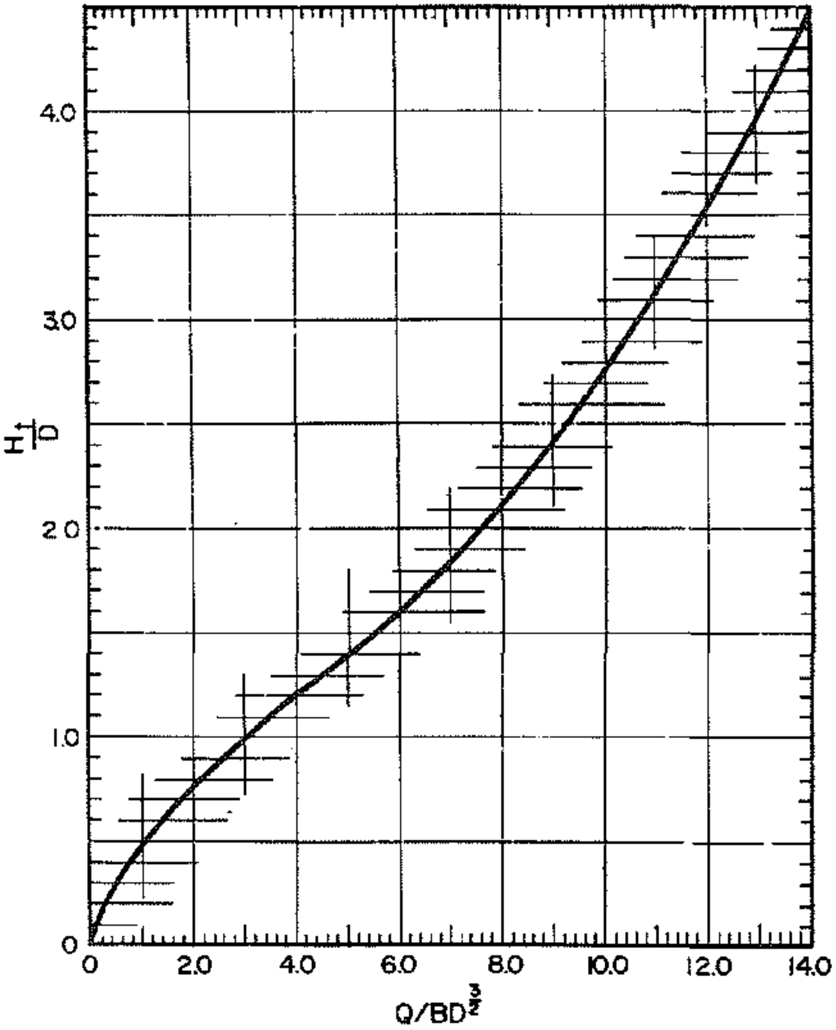


CHART VIII - 17

HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL



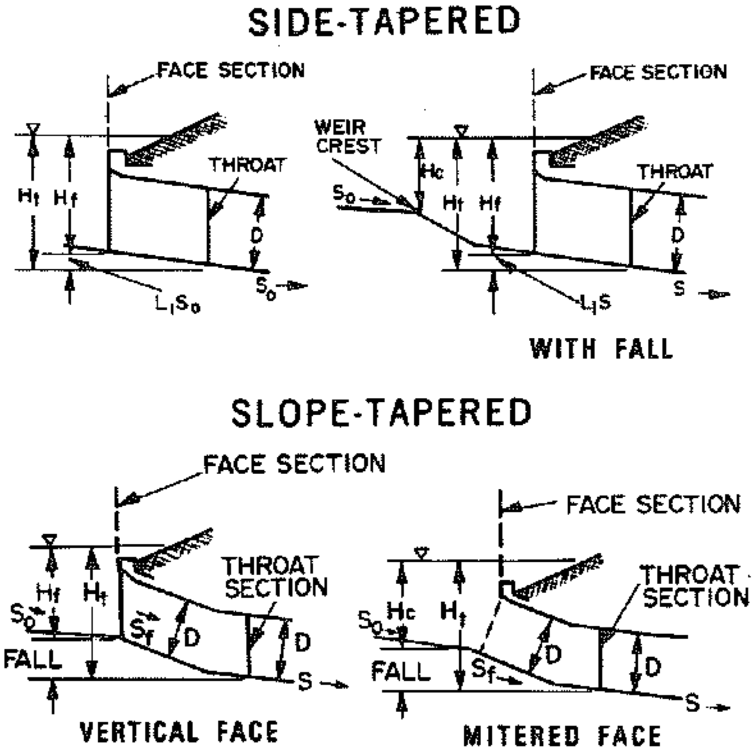


CHART VIII-18

THROAT CONTROL CURVE FOR BOX CULVERTS TAPERED INLETS

WME, June, 1979, 11

FACE CONTROL CURVES FOR BOX CULVERTS SIDE-TAPERED INLETS

CHART VIII-19

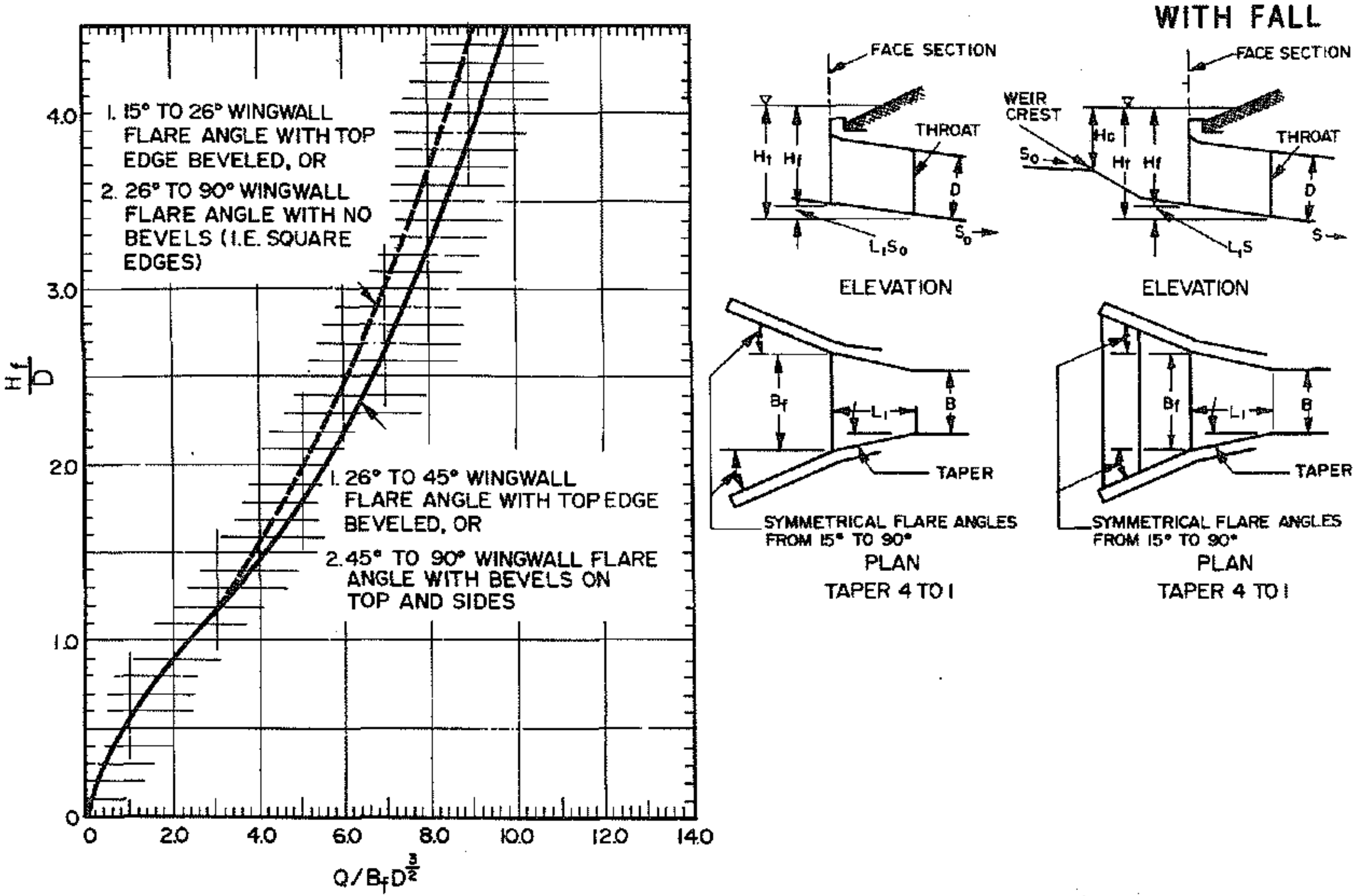
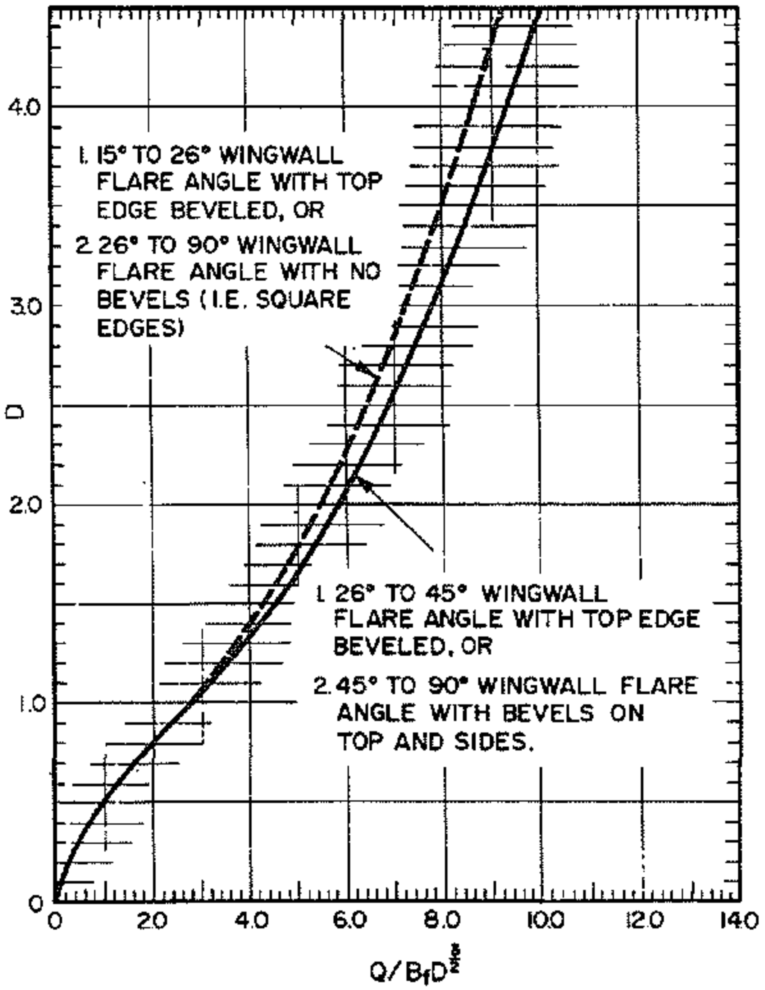
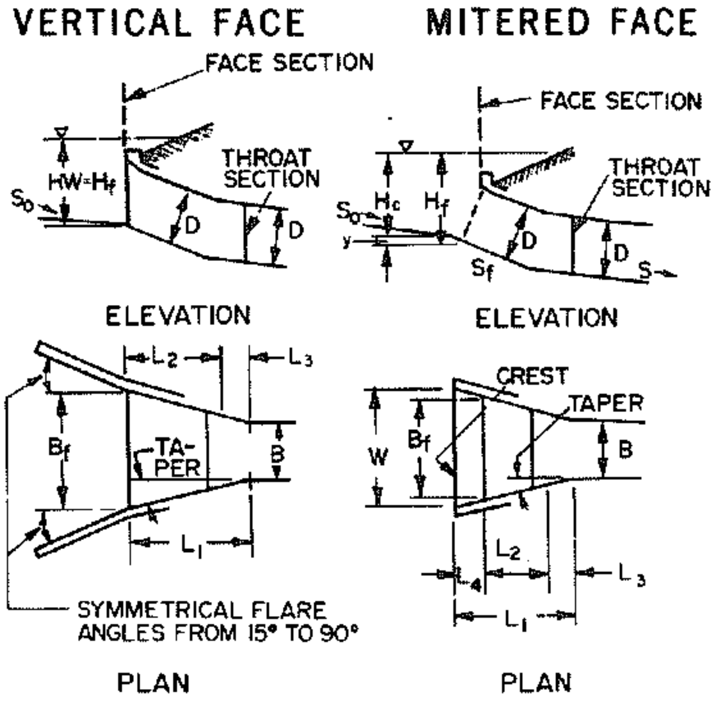


CHART VIII-20
FACE CONTROL CURVES FOR
BOX CULVERTS SLOPE-TAPERED INLETS





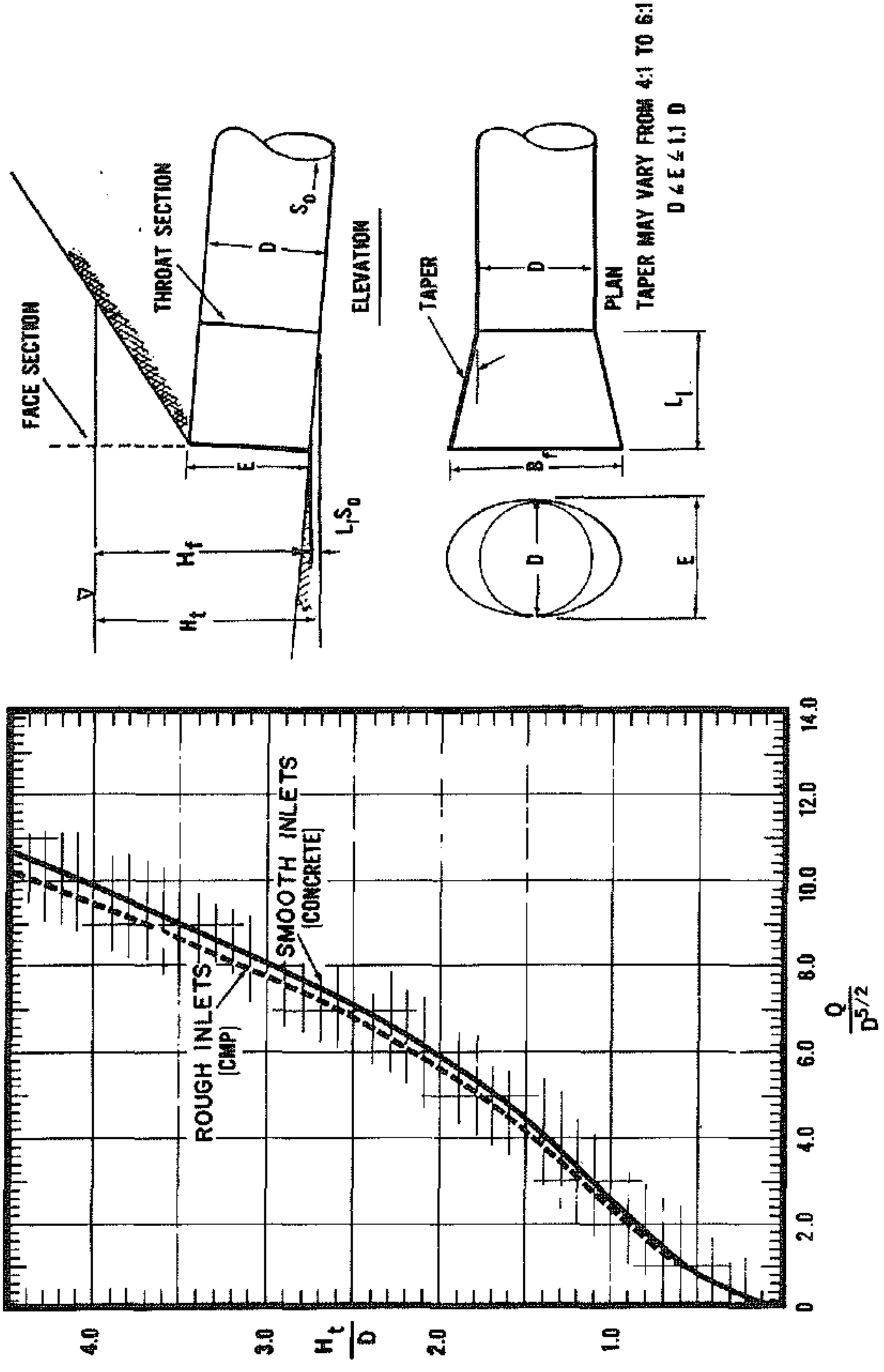
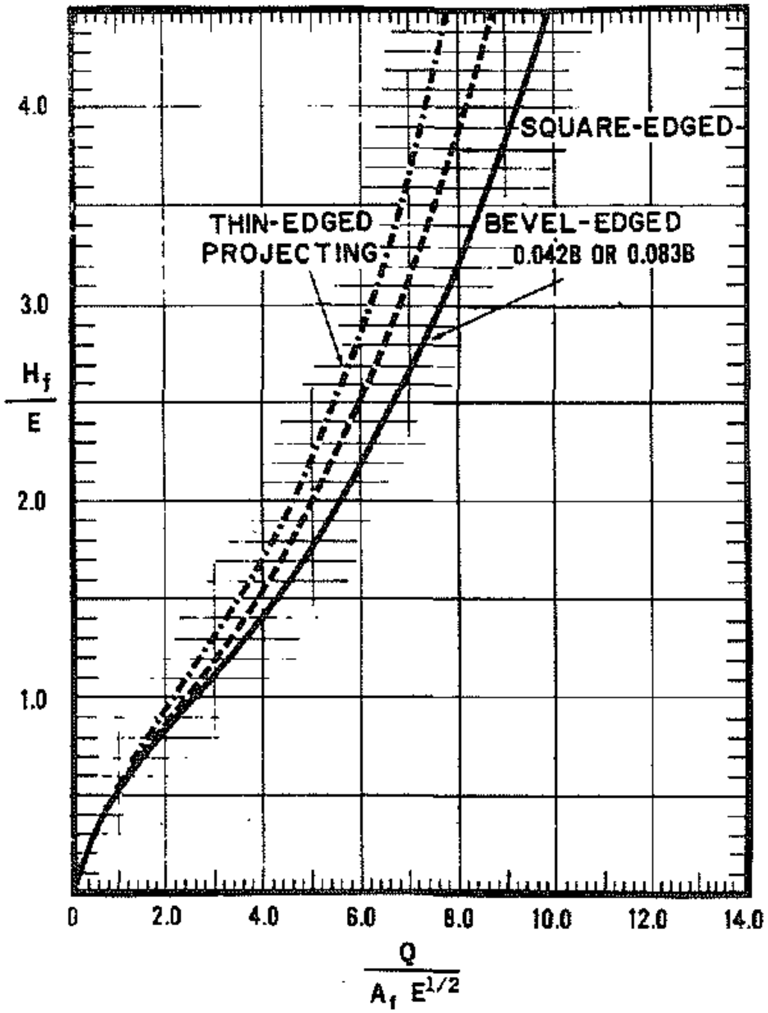
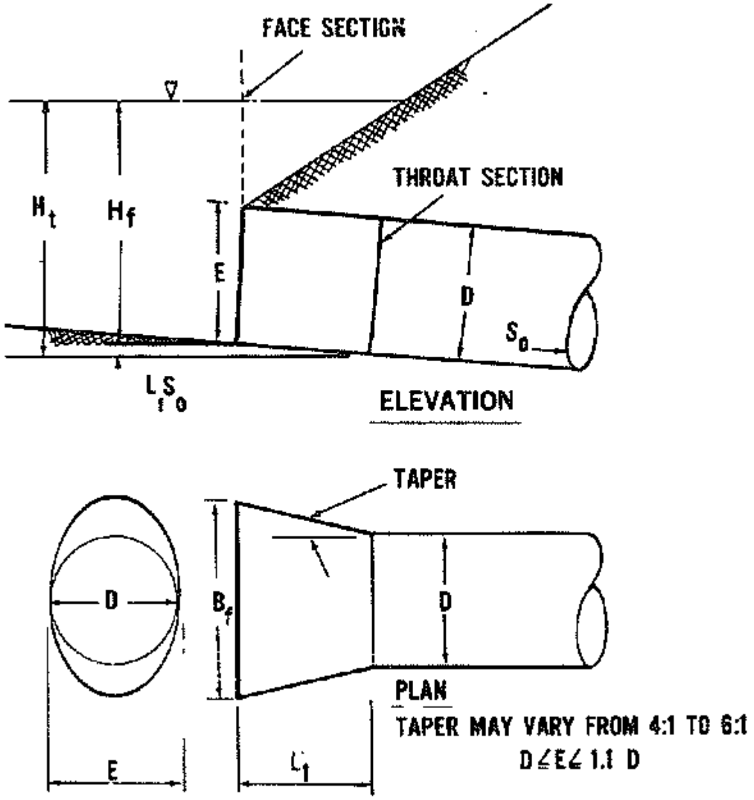


CHART VIII-21
THROAT CONTROL CURVES FOR SIDE-TAPERED INLETS TO PIPE CULVERT
(CIRCULAR SECTIONS ONLY)

CHART VIII-22

FACE CONTROL CURVES FOR SIDE-TAPERED INLETS TO PIPE CULVERTS (NON-RECTANGULAR SECTIONS ONLY)





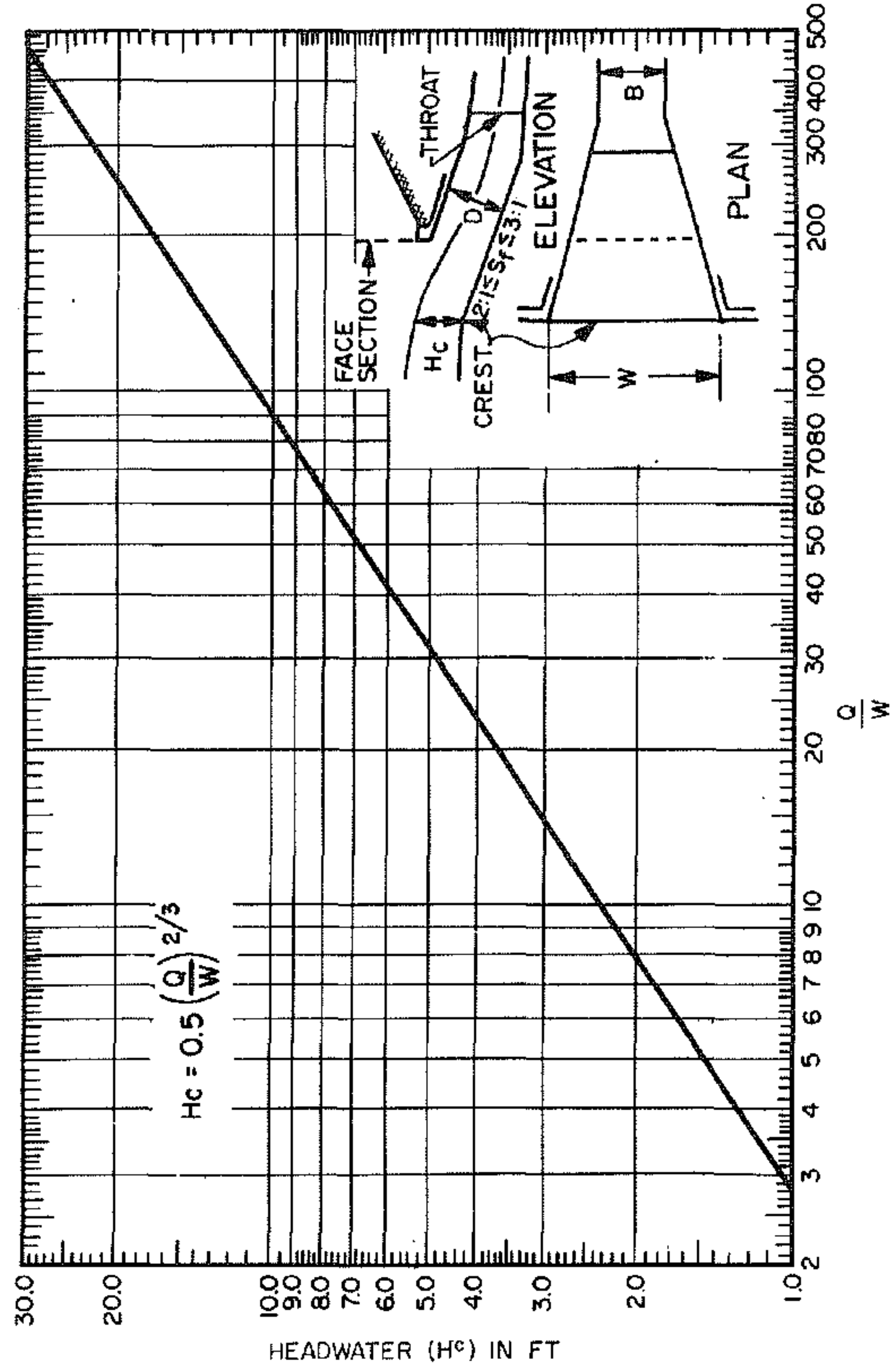


CHART VIII-23

HEADWATER REQUIRED FOR CREST CONTROL

DESIGN EXAMPLE

The following design example from Hydraulic Design of Improved Inlets for Culverts (12) illustrates the procedure for culvert design.

Given: Design Discharge (Q_{50}) = 150 cfs
Allowable Headwater Elevation = 96./0 ft.
Elevation Outlet Invert = 75.0 ft.
Culvert Length (L_a) = 350 ft.

Downstream channel approximates of 5 ft. wide trapezoidal channel with 2:1 side slopes and a Manning n of 0.03. S_o = 0.05.

Requirements: Hydrological estimates are accurate and exceeding the AHW El. at higher discharges is not important at this site. Therefore, use the smallest barrel possible.

The outlet control curves of Problem 4 are applicable in this situation. The 48" C.M.P. is the smallest barrel which will meet 4HW El. = 96.0 and Q = 150 cfs.

From the inlet control curves, it is clear that a FALL must be used on the tapered inlet to meet the AHW El. Try a side-tapered inlet, with FALL, and a slope-tapered inlet.

CONCLUSION

Selection of side-tapered or slope-tapered inlet must be based on economics, as either will perform the required function. Additional FALL is not warranted at this site. Face design was selected to pass 150 cfs at AHW El. = 96.0.

The culvert performance curves for the example illustrate that when a prefabricated side-tapered inlet (rough) or a cast-in-place slope-tapered inlet (smooth) may be chosen for an installation, but the smooth and rough inlet throat control curves should be plotted. The difference between the throat control curves represents the difference in friction losses between the face and throat sections of the inlet.

PROJECT: *Example No. 4*

DESIGNER: *JMN*

SAMPLE

STATION: ________

DATE: *12-10-75*

INITIAL DATA:

Q *50* = *150* cfs

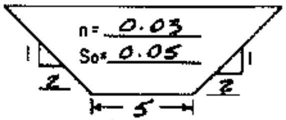
AHW El. = *100* ft.

So = *0.05*

Lo = *350* ft.

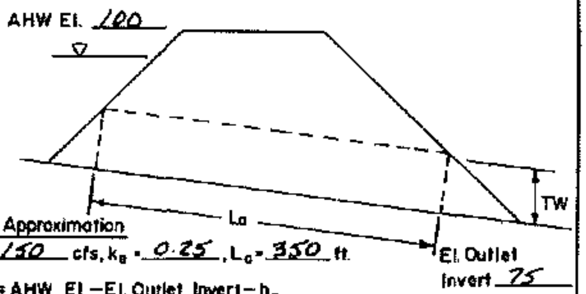
El. Outlet Invert *75* ft

Stream Data:



Barrel Shape and Material *Circular CMP* Barrel n = *0.024*

SKETCH



First Approximation

Q = *150* cfs, k_e = *0.25*, L_a = *350* ft.

H = AHW El. − El. Outlet Invert − h_o

= *100'* − *75'* − *5'* = *20'*

∴ A = ____ ft.^{2} or D = *46"* ft.; Try *42"*

Q	$\frac{Q}{N}$	* H	$\frac{Q}{NB}$	(1) d_c	$\frac{d_c+D}{2}$	Qn	(2) TW	(3) h_o	(4) HW_o	(5) V_o	COMMENTS
Trial No *1*, N = *1*, B = *—*, D = *3.5'*, k_e = *0.25*											
150	*150*	chart 3 *31*	*150*	chart 6 *>3.5*	*3.5*	*—*	NOS 8 *1.6*	*3.5*	*109.5*	table 8	*75 + 31 + 3.5 = 109.5*
											HW_o > AHW El. Try 48"
Trial No. *2*, N = *1*, B = *—*, D = *4'*, k_e = *0.25*											
150	*150*	*15.6*	*150*	*3.6*	*3.8*	*—*	*1.6*	*3.8*	*94.4*		*Ok – Check square edge*
100	*100*	*7.0*	*100*	*3.1*	*3.5*	*—*	*1.4*	*3.5*	*85.5*		
200	*200*	*27.8*	*200*	*>4*	*4.0*	*—*	*1.9*	*4.0*	*106.8*		
Trial No *3*, N = *1*, B = *—*, D = *4'*, k_e = *0.5*											
150	*150*	*16.2*					*1.6*	*3.8*	*95.0*		*From inlet Control section*
100	*100*	*7.2*	*Same as trial # 2*				*1.4*	*3.5*	*85.7*		*Calculations, FALL req'd*
200	*200*	*28.8*					*1.9*	*4.0*	*107.8*		*∴ Use improved inlet*

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

(3) $h_o = \frac{d_c+D}{2}$ or TW, whichever is larger

(4) $HW_o = H + h_o +$ El. Outlet Invert.

(5) Outlet Velocity (V_o=Q/Area defined by d_c or TW, not greater than D. Do not compute until control section is known.

SELECTED DESIGN

N = *1* At Design Q:

B = *—* ft.

D = *4* ft. HW_o = *94.4* ft.

k_e = *0.25 or 0.5* V_o = ____ f/s

$$* \quad H = \left[1 + k_e + \frac{29n^2 \cdot L}{R^{1.33}}\right] \frac{V^2}{2g}$$

FIGURE VIII-35
OUTLET CONTROL DESIGN CALCULATIONS

PROJECT: Example No. 5 DESIGNER: JMN

SAMPLE

STATION: ______ DATE: 12-10-73

INITIAL DATA:

Q 60 = 150 cfs

AHW El. = 96.0 ft.

S_o = 0.05

L_a = 350 ft.

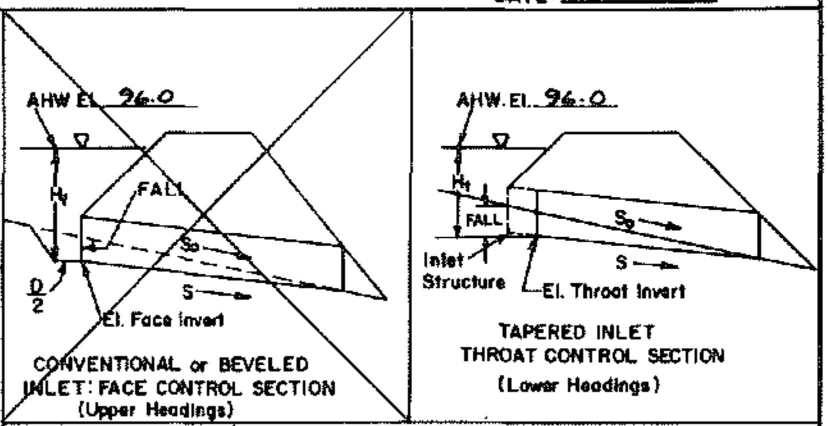
El. Stream Bed at Face 92.5 ft.

Barrel Shape and Material Circ. CMP Barrel n = 0.024

N = 1, B = –

D = 4, $NBD^{3/2}$ = ______

(Pipe) $ND^{5/2}$ = 32.0 (Table 6)



DEFINITIONS OF INLET CONTROL SECTION

	~~$\frac{Q}{NB}$~~	~~$\frac{H_f}{D}$~~	~~H_f~~	(1) ~~El. Face Invert~~	El. Stream Bed At Face	(2)	(3) ~~HW_f~~	(4)	(5)	Note: Use Upper Headings for Coventional or Beveled Face; Lower Headings for Tapered Inlet Throat.
Q	$Q/NBD^{3/2}$	$\frac{H_t}{D}$	H_t	El. Throat Invert		FALL	HW_t	S	V_o	COMMENTS
Trial No. 1 Inlet and Edge Description Tapered inlet throat, smooth, FALL = 2.8'										
150	4.7	chart 15 1.57	6.3	89.7	92.5	2.8	96.0	0.042	HDS 3 14	Decided that at this site
100	3.1	1.13	4.5				94.2			no additional FALL is justi-
200	6.2	2.12	8.5				98.2			fied. Design inlet for HW_t = 96.0
Trial No. 2 Inlet and Edge Description Tapered inlet throat, rough, FALL = 3.1'										Q = 150 cf
150	4.7	1.65	6.6	89.4	92.5	3.1	96.0	0.041	.14	Use smooth inlet for slope-
100	3.1	1.21	4.8				94.2			tapered, rough for side-
200	6.2	2.22	8.9				98.3			tapered.
Trial No. ____ Inlet and Edge Description ______										

Notes and Equations:

(1) El Face (or throat) invert = AHW El. − H_f (or H_t)

(2) FALL = El. Stream Bed at Face − El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed.

(4) $S \approx S_o - FALL/L_a$

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description:

FALL = 2.8 ft. or 3.1 ft.

Invert El. = 89.7 ft. 89.4

Bevels:

Angle = N/A

b = ______ in., d = ______ in.

FIGURE VIII-36
CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS

PROJECT: *Example No. 5* DESIGNER: JMN

SAMPLE

STATION: ________ DATE: 12-10-73

INITIAL DATA

Q_{50} = 150 cfs S_o = 0.05

AHW El. = 96.0 ft. L_a = 350 ft.

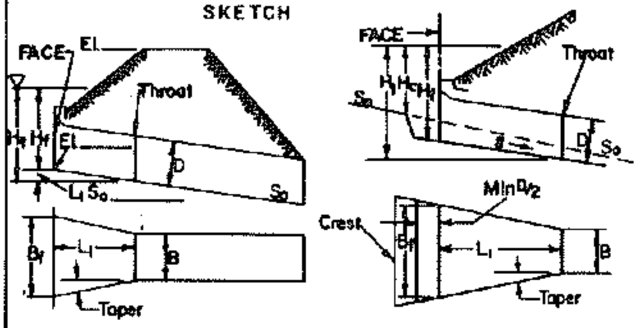
TAPER = 4 : 1

Barrel Shape and Material Circular C.M.P.

Face Edge Description 45° Bevels

N = 1, B = — ft, D = 4 ft

SKETCH



Q	El. Throat Invert	(1) ~~H_f/D~~ H_f/E	~~$\frac{Q}{B_f D^{3/2}}$~~ $\frac{Q}{A_f E^{1/2}}$	~~$D^{1/2}$~~ $E^{1/2}$	(2) ~~Min. B_f~~ Min. A_f	B_f	(3) L_1	(4) S	$L_1 S$	(5) El. Face Invert	Upper Headings for Box Culverts, Lower Headings for Pipes COMMENTS
Trial No. 1, Q = 150, HW_f = 96.0 (Use lower column headings)											
150	89.4	1.4	4.0	Table 6 2.0	18.8	Table 7 6.0	4.0	0.041	0.2	89.6	~~$B_f D^{3/2}$~~[or $A_f E^{1/2}$] = 18.85
											CMP (rough) side-
											tapered inlet
Trial No ____, Q = ____, HW_f = ____											
											$B_f D^{3/2}$[or $A_f E^{1/2}$] = ____
Trial No. ____, Q = ____, HW_f = ____											
											$B_f D^{3/2}$[or $A_f E^{1/2}$] = ____

Notes and Equations: $(96.0 - 89.4 - 1)/4 = 1.4$

(1) H_f/D[or H_f/E] = (HW_f − El. Throat Invert − 1)/D[or E]

$D \le E \le 1.1 D$

(2) Min. $B_f = Q/(D^{3/2}) \left[\frac{Q}{B_f D^{3/2}}\right]$

Min. $A_f = Q/(E^{1/2}) \left[\frac{Q}{A_f E^{1/2}}\right]$

(3) $L_1 = \left[\frac{B_f - NB}{2}\right]$ TAPER $\left[\frac{6.0 - 4.0}{2}\right] 4 = 4.0$

(4) From throat design

(5) El. Face Invert − El. Throat Invert > 1 ft., recompute.
Face and Throat may be lowered to better fit site, but do not raise.

SELECTED DESIGN

B_f = 6.0 ft.

L_1 = 4.0 ft.

Bevels: Angle 45 °

d = — in., b = 3 in.

Crest Check:

HW_c = 96.0 ft.

H_c = 3.0 ft.

$$\begin{array}{r} 96.0 \\ -93.0 \\ \hline 3.0 \end{array}$$

Q/W = 15 (Chart 17)

Min. W = 10.0 ft.

FIGURE VIII-37
SIDE-TAPERED INLET DESIGN CALCULATIONS

PROJECT: Example No. 5

STATION: ______

SAMPLE

DESIGNER: JMN

DATE: 12-10-73

INITIAL DATA:

Q60 = 150 cfs So = 0.05

AHW EL. 96 ft. L_a = 350 ft.

El. Stream bed at crest — ft.

El. stream bed at face 92.5 ft

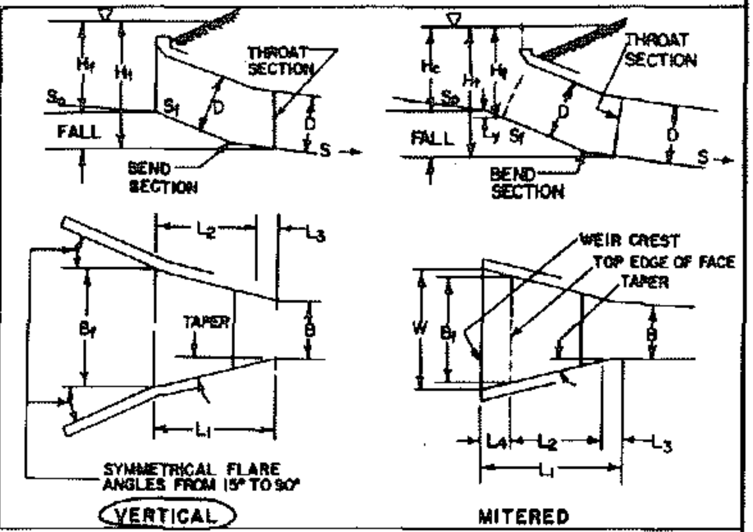
TAPER = 4 :1 (4:1 to 6:1)

S_f = 2 :1 (2:1 to 3:1)

Barrel Shape and Material Circular C.M.P.

Inlet Edge Description Beveled

N= 1 , B= — ft, D= 4 ft.



Note: Use square to circular transition section, D = B = 4′ (smooth conc. inlet)

	Q	HW_f	El. Throat Invert	(1.) El. Face Invert	(2.) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3.) Min. B_f	B_f	S	Comments
Trial 1	150	96.0	89.7	92.5	3.5	0.88	chart 16 2.4	Table 4 8.0	7.8	8.0	0.042	$B_f D^{3/2}$ = ______
												Vertical face,
												Min. FALL required
Trial 2												$B_f D^{3/2}$ = ______

Note: Use only throat designs with FALL > 0.25D

(1.) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest − y, where y = 0.4D (Approx.), but higher than throat invert elevation.

(2.) $H_f = HW_f$ − El. face invert

(3.) Min. $B_f = Q/((D^{3/2})\ Q/B_f D^{3/2})$

(4.) Min. L_3	(5.) L_4	(6.) L_2	(7.) Check L_2	(8.) Adj L_3	(9.) Adj TAPER	(10.) L_1	(11.) W	$\frac{Q}{W}$	H_c	(12.) Max. Crest El.
2.0	—	5.6	< 6.0	2.4	—	8.0	—	chart 17 —	—	—

GEOMETRY

B_f = 8.0 ft. L_3 = 2.4 ft.

L_1 = 8.0 ft. L_4 = — ft.

L_2 = 5.6 ft d = 2 in.

b = 4 in.

TAPER = 4 :1

(4.) Min. L_3 = 0.5NB = 0.5 (4) = 2.0

(5.) $L_4 = S_f y + D/S_f$ N/A

(6.) L_2 = (El. Face (Crest) Invert − El. Throat Invert) $S_f - L_4$ = (92.5 − 89.7)2 = 5.6

(7.) Check $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER − L_3.

(8.) If (7) > (6), Adj. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER − L_2

(9.) If (6) > (7) Adj. TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(10.) $L_1 = L_2 + L_3 + L_4$

(11.) Mitered: $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

(12.) Max. Crest El. = $HW_f - H_c$

FIGURE VIII-38

SIDE-TAPERED INLET DESIGN CALCULATIONS

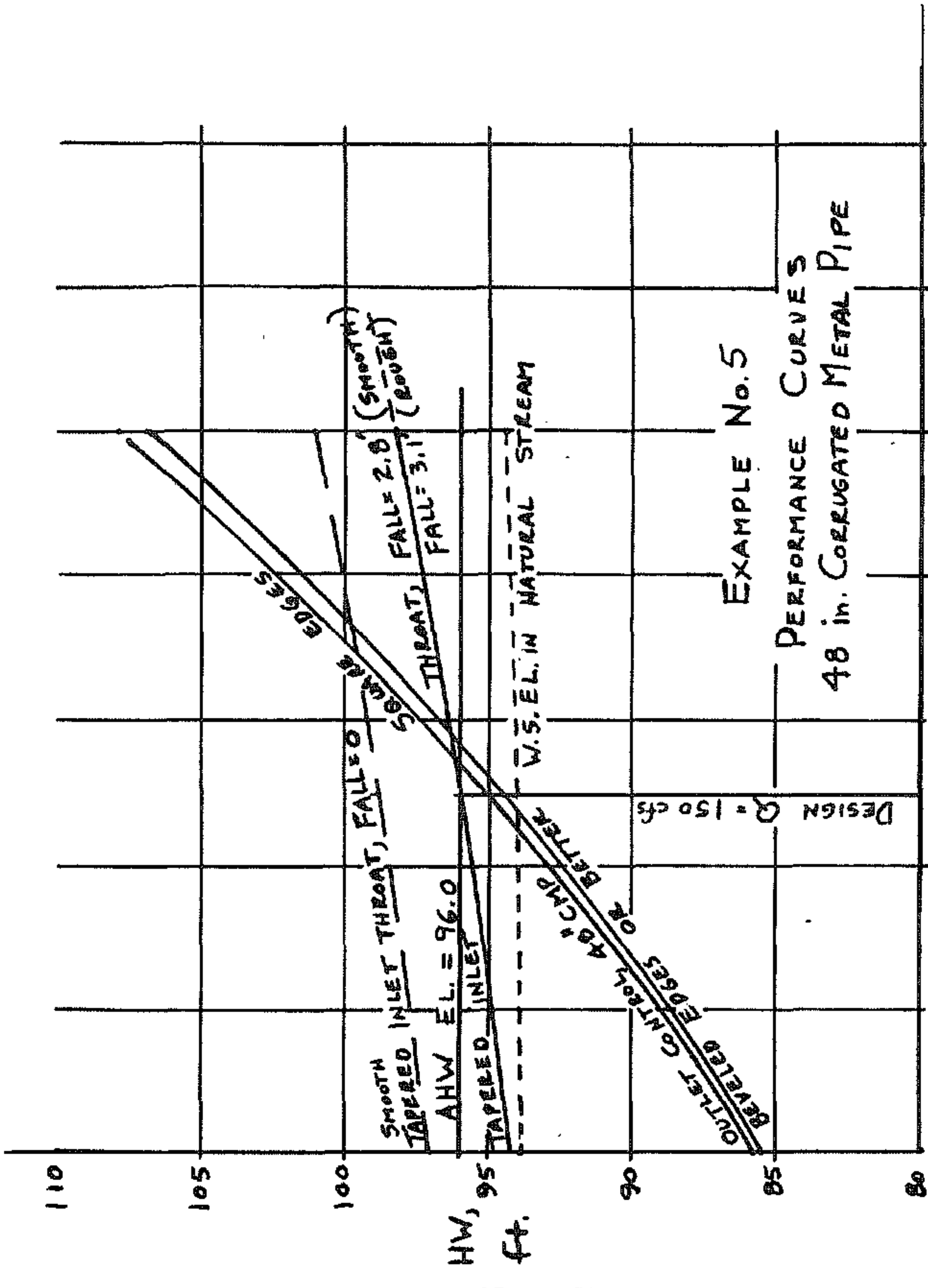


FIGURE VIII-39

SAMPLE RATING CURVE FOR DESIGN EXAMPLE

LIST OF SYMBOLS

Symbol	Units	Description
A_b	sq.ft.	Area of bend section of slope-tapered inlets
A_f	sq.ft.	Area of inlet face section
A_t	sq.ft.	Area of inlet throat section
AHW El.	ft.	Allowable headwater elevation at culvert entrance
B	ft.	Width of culvert barrel or diameter of pipe culvert
b	in.	Dimension of side bevel
B_b	ft.	Width of bend section of slope-tapered inlets
B_f	ft.	Width of face section of improved inlets
C_b		Discharge coefficient based on bend section control
C_f		Discharge coefficient based on face section control
C_t		Discharge coefficient based on throat section control
cfs	cu.ft./sec.	Cubic feet per second
CMP		Corrugated metal pipe
D	ft.	Height of box culvert or diameter of pipe culvert
d	in.	Dimension of top bevel
d_c	ft.	Critical depth of flow
E	ft.	Height of side-tapered pipe culvert face section, excluding bevel dimension
f		Darcy resistance factor
FALL	ft.	Approximate depression of control section below the stream bed

LIST OF SYMBOLS (Cont'd)

Symbol	Units	Description
g	ft./sec./sec.	Acceleration of gravity = 32.2
H	ft.	Head or energy required to pass a given quantity of water through a culvert flowing in outlet control
H_b	ft.	Depth of pool, or head, above the bed section invert
H_c	ft.	Depth of pool, or head, above the crest
H_f	ft.	Depth of pool, or head, above the face section invert
H_t	ft.	Depth of pool, or head, above the throat section invert
H*	ft.	Specific head at minimum energy
HG Line	ft.	Hydraulic grade line
HW	ft.	Headwater elevation; subscript indicates control section (HW, as used in HEC #5, is a depth and is equivalent to H_f in this Circular)
HW_c	ft.	Headwater elevation required for flow to pass crest in crest control
HW_f	ft.	Headwater elevation required for flow to pass face section in face control
HW_o	ft.	Headwater elevation required for culvert to pass flow in outlet control
HW_t	ft.	Headwater elevation required for flow to pass throat section in throat control
h_o	ft.	Elevation of equivalent hydraulic grade line referenced to the outlet invert

LIST OF SYMBOLS (Cont'd)

Symbol	Units	Description
K		A constant relating to free surface nonsubmerged entrance flow
k_e		Entrance energy loss coefficient
k_b		A dimensionless effective pressure term for bend section control
k_f		A dimensionless effective pressure term for inlet face section control
k_t		A dimensionless effective pressure term for inlet throat control
L_a	ft.	Approximate total length of culvert, including inlet
L_1, L_2, L_3, L_4	ft.	Dimensions relating to the improved inlet as shown in sketches of the different types of inlets
N		Number of barrels
n		Manning roughness coefficient
P	ft.	Length of depression
Q	cu.ft./sec.	Volume rate of flow
R	ft.	Hydraulic radius = $\frac{\text{Area}}{\text{Wetted Perimeter}}$
S	ft./ft.	Slope of culvert barrel
S_e	ft./ft.	Slope of embankment
S_f	ft./ft.	Slope of FALL for slope tapered inlets (a ratio of horizontal to vertical
S_o	ft./ft.	Slope of natural channel
T	ft.	Depth of the depression
Taper	ft./ft.	Sidewall flare angle (also expressed as the contangent of the flare angle.

LIST OF SYMBOLS (Cont'd)

Symbol	Units	Description
TW	ft./ft.	Tailwater depth at outlet of culvert referenced to outlet invert elevation
V	ft./sec.	Mean velocity of flow
W	ft.	Width of weir crest for slope tapered inlet with mitered face
W_p	ft.	Top width of depression
WW		Wingwall of culvert entrance
y	ft.	Difference in elevation between crest and face section of a slope-tapered inlet with mitered face
θ_s	degrees	Flare angles of side walls of tapered inlet with respect to extension of culvert side wall
θ_t	degrees	Angle of departure of the top slab from a plane parallel to the bottom slab

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CHAPTER IX
URBAN RUNOFF POLLUTION

The nonpoint pollution sources in the Water Quality Management Plan for the Cimarron River Basin have been described by the Oklahoma Department of Pollution Control. This description is presented in the Areawide Waste Treatment Management Planning (208) report of February, 1979, for Segment 620900.

The City of Stillwater has been identified as being the source of two-thirds of the Segment 620900 pollutants originating from urban storm runoff. While the Department of Pollution Control was unable to provide specific data in regard to the quantity, constituents, or severity of the urban runoff pollution, it has strongly indicated the need for voluntary stormwater pollution controls by the main municipalities. The state has concluded that implementation of water controls may be the most effective means of enhancing the stream water quality in the Cimarron River.

Specifically, the Department of Pollution Control has recommended city ordinances requiring detention or retention of stream runoff and erosion control in developing areas. These recommendations are consistant with the objectives of this Manual.

TYPES OF POLLUTANTS

The Oklahoma Water Quality Standards cover a wide range of pollutants which are to be managed to assure satisfactory stream water quality. Most of the pollution indicators or parameters listed are related in varying degrees to urban runoff. A partial list follows:

- Toxic Pollutants
  - Ammonia
  - Arsenic
  - Barium
  - Cadmium
  - Chromium
  - Copper
  - Iron
  - Herbicides
  - Lead
  - Manganese
  - Mercury
  - Sulfides
  - Zinc
  - Phenolic Compounds
  - Pesticides
  - Nitrate
- Color

TABLE IX-1

ESTIMATED CONTRIBUTION OF URBAN RUNOFF TO TOTAL STREAM POLLUTION FROM THE CITY OF STILLWATER

CONSTITUENT	STORM RUNOFF POLLUTION OR PERCENT OF TOTAL ANNUAL POLLUTION LOAD
BOD	20%
COD	30%
Suspended Solids	85%
Organic Kjeldahl Nitrogen	30%
Soluble Orthophosphate	5%

Stillwater contributes a source of metals which accumulates on the streets. These are susceptible to being washed into the streams during urban runoff events.

By comparing Stillwater to tyical urban runoff pollution leads from similar cities estimates can be made of runoff pollution metal loading from Stillwater to this area stream system. These estimates are given in Table IX-2 for a total population of 50,000 people, including students.

TABLE IX-2

ESTIMATED METAL POLLUTION IN URBAN RUNOFF RESULTING FROM SHORT INTENSE PRECIPITATION OF 0.1 INCHES

METAL	POLLUTANT LOAD IN POUNDS/HOUR
Lead	600
Cadmium	1
Nickel	10
Copper	35
Zinc	150
Iron	8000
Manganese	150
Chromium	80

SOURCES OF POLLUTANTS

To understand and control urban runoff pollution, it is necessary to be familiar with the usual sources of pollutants. This information is presented in Table IX-3.

TABLE IX-3

ORIGIN OF STORM RUNOFF POLLUTANTS

CATEGORY	PARAMETER	SOURCE
Bacterial	Fecal Coliform and Fecal Strep	Humans, other mammals, and birds
Organic	Biochemical Oxygen Demand Chemical Oxygen Demand	Leaves, grass clippings, garbage, Mammals, oil, and grease
Nutrients	Nitrogen, Phosphate	Fertilizers, leaching from minerals, decomposition of organic matter, animal waste
Solids	Suspended Solids Clay, Silica, organic matter	Erosion of cleared land, dust, dirt from streets unimproved drainage channels, sand for ice control
	Dissolved Solids Carbonates Chlorides Sulfates Phosphates Nitrites of calcium organic matter	Erosion of cleared land, leaching from minerals soluble dust and dirt from streets, salt for ice control

Urban stormwater pollution is closely related to suspended solids because many of the consituents attach themselves to particles in the water. Thus, by controlling the suspended solids carried by runoff, a city can take a major step towards water quality enhancement. It is for this reason, among others, that the Oklahoma Department of Pollution Control has recommended use of detention or retention storage.

Urban erosion contributes large quanitities of suspended sediment to storm runoff. Much of the erosion is easily eliminated by following simple rules of land husbandry. The Soil Conservation Service ethic of controlling erosion before it starts should be incorporated into the policies and ordinances of the City of Stillwater.

Any soil can erode. The most erosive soils in the Stillwater metropolitan area are discussed. New development on these soils should be subject to specific erosion and sedimentation regulations.

Erodibility of Stillwater Soils

Erosion and subsequent sedimentation are natural earth processes which are often accelerated during periods of construction in urbanizing areas. This accelerated soil erosion will, in turn, contribute to increased sediment loads in receiving streams and waterways during periods of runoff, thereby worsening downstream water quality.

Within the Stillwater Metropolitan Area, a number of soils groups have been identified as being naturally erosive. Table IX-4 (at the back of this Chapter) lists these soils and describes their degree of susceptibility to erosion.

The areal distribution of the soils identified in Table IX-4 can be found on the Soil Map, City of Stillwater, Oklahoma prepared by the Soil Conservation Service. This 1" = 800' scale uncontrolled mosaic was compiled from 1949 and 1963 soil survey field sheets and completed in August, 1969.

URBAN RUNOFF CONTROL MEASURES

This Manual has previously articulated the goals, objectives, and policies of the City of Stillwater to enhance the quality of streams by reducing the pollution generated by urban storm runoff. This Section outlines various pollution control measures which can be incorporated into a water quality enhancement strategy by the City of Stillwater.

Both structural and nonstructural control measures can reduce water pollution from urban areas. A nonstructural water quality measure is an action, either physical or legislative, which attempts to prevent pollution before it becomes a problem. Structural control measures generally do not strive to eliminate sources of pollution or actions that cause it, but collect and concentrate pollutants for treatment.

Nonstructural controls have been given special emphasis because prevention of a pollution problem is preferable to its correction through collection and treatment.

Nonstructural Controls for Urban Runoff

The nonstructural controls may be subdivided into two categories: those that are aimed toward reducing the volume of runoff and those that improve urban cleanliness.

Land Use Controls. Local land use planning efforts can be oriented to quality of groundwater recharge areas, wetlands, floodplains, stream/lake border areas, steep slopes and areas with highly erodible soils. Alternatives for controlling land use include zoning, public acquisition of green belts, critical areas' delineation, phased development, and urban service area delineation.

Land use control for preservation of water quality can be implemented into existing land use programs.

Retention/Detention. This involves utilizing temporary storage followed by infiltration to reduce runoff of storm water. Water which falls on newly developed or redevelopment areas should be absorbed or retained on site to the extent that the quality and the rate of water leaving the site is not significantly different than that under the pre-urbanized condition. This involves a movement away from the traditional method of development that required efficient storm drainage systems to compensate for large expanses of impervious area. Rather than the use of curbs, gutters, and storm sewerage systems, this approach utilizes measures such as: (1) maximum use of natural drainage, (2) contour landscaping, (3) swale storage, (4) porous pavement, (5) turf grids, (6) parking lot storage, (7) rooftop storage, (8) multi-use areas, (9) storm water detention ponds, and (10) parking lot drainage structures.

Improved Street Sweeping. Street sweeping has been primarily for urban beautification, not water quality. However, water quality benefits are achieved with regular sweeping.

Litter Ordinances. Littering is partially responsible for the build-up of pollutants experienced in urban areas.

Recycle Programs for Waste Oil. Improper discharge of waste oil results in a water quality problem especially if oil is dumped into storm sewers.

It may be possible to reduce illegal discharges of waste oil through recycling programs encouraged by public education, and by strict enforcement of sanitary and litter ordinances.

Enforcement of Sanitary Codes. The purpose of this control is to assure that sanitary wastes are not discharged to the storm sewer system. In many areas, gas stations, vehicle washing operations and laundromats are connected to the storm sewerage system rather than to the sanitary system. Consequently, waste waters that should receive treatment are discharged without it. Enforcement of sanitary codes involves an aggressive inspection program which identifies illegal dischargers and requires them to connect to the sanitary system.

Street Maintenance. Roads that are poorly maintained experience a much higher pollutant build-up than those that are in good condition. This is principally due to the additional sediments common to deteriorating road surfaces.

Maintenance of Private Parking Lots. Next to street surfaces, parking lots are one of the largest contributors of nonpoint source pollution. As a result, frequent cleaning of parking lots would benefit water quality.

Animal Control. Much of the fecal coliform found in urban runoff is from the wastes of urban animals. Most domestic animals have a daily waste production that ranges from five to eight percent of their total body weight. Animal control ordinances are an aid to storm runoff water quality.

Ordinances Requiring Protection of Stockpiles. Stockpiles that are unprotected can erode and present a water quality problem. This is especially true of stockpiled salt.

Because of increasing concern regarding water quality, it is anticipated that this control would be implementable. The control would involve some public costs for inspection, but this is anticipated to be minimal. Stockpile protection would not necessarily have to involve the actual development of a structure, but could involve a simple measure that trapped eroded materials and prevented them from entering a water course.

Collection of Vegetative Debris. Leaves and grass clippings are a significant contributor of BOD and, to a lesser extent, phosphates and nitrogen. The major problem with these materials is that they are often deposited in gutters and storm sewers, where they decompose. During heavy rainfall, these materials are then washed to a nearby water course. Vegetative debris also increases the cost of catch basin maintenance.

Limiting of Fertilizers and Irrigation. Many of the landscapes in urban areas could not be maintained without fertilization and irrigation. Over application of both nutrients and water can cause a water quality problem. Over irrigation results in runoff, which can carry fertilizers to the street area, where they are either immediately discharged to the storm sewerage system or remain on the street surface until the next storm event. Fertilizers can also be washed off of urban landscapes during heavy rainfall.

Structural Control Measures

Structural control measures for managing urban runoff are physical actions that either strive to imitate the natural hydrologic (predevelopment) system, or involve the collection, storage and treatment of runoff water.

Natural Drainage. This involves designing developments so they maximize the use of the pre-development drainage system. Natural drainageways can be lined with vegetation or slightly modified in other ways to increase infiltration and retention. Natural drainage can be most effective if supplemented by onsite detention, so that peak runoff can be reduced for subsequent release to the natural drainageway.

Contour Landscaping. Contour landscaping involves grading the surface so that infiltration is increased and runoff is reduced. This concept is the reverse of most traditional means of development where subdivisions were graded to promote the discharge of stormwater. In addition to careful grading, contour landscaping also involves the use of vegetation, so that runoff is discharged to vegetated areas for infiltration and storage, rather than to the streets and storm sewerage systems. This control is best applied in combination with one or more of the controls that are mentioned in this Section.

Swale Storage. Swales are small grass-lined depressions that can either be natural or manmade, which collect storm runoff. To be most efficient, they should be graded wide and shallow and slightly sloped. Infiltration and storage can be increased by maintenance of vegetation in the swale.

Grass-Lined Ditches. As the name implies, these are small grassed drainageways that can be used to replace storm sewers. The principle advantage of this method of drainage is that infiltration of runoff can be increased through ditch losses, and the roughness in the channel provided by the vegetation reduces water velocities and peak discharge. In addition, the grass in the ditch aids in filtering out many of the pollutants carried by the runoff.

One of the major advantages of grass-lined ditches is that they are typically cheaper than a traditional storm sewerage system.

Porous Pavement. Use of traditional asphalt or concrete for parking lots and roads prevents the infiltration of precipitation and increases runoff.

By contrast, the use of porous pavement, which is permeable, allows infiltration and groundwater recharge. On the average, porous pavement is more expensive than conventional pavement for parking lot construction.

Turf Grids. A turf grid is a new product that allows parking lots to be planted in grass, yet still supports vehicles without damaging the turf.

Turf grids are similar to large concrete waffles with interstices that allow the grass to grow through to the surface. A parking lot built with turf grids gives an appearance similar to cobblestones, and it effectively reduces runoff.

The principle disadvantage of turf grids is that the cost of parking lot construction is several times greater than required for asphalt paving. In addition, it is possible that the oil, gas and other liquids that leak from automobiles may cause the grass to die.

Multiple Use Areas. This concept involves the use of an area that is normally used for field-type recreational events for the storage and infiltration of runoff. Typically, runoff is discharged to the field which through grading, is such that the water ponds there for subsequent evaporation and infiltration. The principle advantage of this measure is that it is relatively inexpensive if incorporated during the initial phase of development. It also provides useful open space.

Road Embankments. This concept utilizes the roadway embankment as a dam where roads cross drainageways. That is, the road is designed to function as a dam during runoff events, so that water is detained upstream of the road. It is necessary that the roadway embankment be adequately protected against erosion.

Detention/Retention Facilities. Detaining water before it runs off is the primary means of reducing peak discharge. Although detention and retention are very similar, there is a subtle difference. Detention involves the short-term storage of water for a subsequent release to the drainage system, where the amount of runoff is not reduced, only the timing is changed. By contrast, retention of storm water involves the capture and long-term storage, so that both peak runoff and total flow are reduced.

Historically, detention facilities have principally been developed for flood protection, not water quality improvement. As discussed earlier in this Chapter, reduction of peak runoff can improve water quality by reducing erosion and stream sedimentation.

Sedimentation Basins. These facilities are used to impound urban runoff for the purpose of settling out wastewater contaminants, after which the wastewater can be discharged, or subjected to additional treatment.

Screening. There are three general categories of screens: coarse, fine, and microscreens. The first category provides only rudimentary treatment, whereas the latter two achieve significant removals of suspended material.

Biological Treatment. Biological treatment is accomplished by converting a portion of the organic matter present in wastewater into cell tissue, which subsequently can be removed by gravity settling. Potential biological treatment for urban runoff includes oxidation, aerated, or facultative lagoons.

Land Treatment. This procedure involves detention followed by application to the land. In order of increasing rate of application, the alternatives include: (1) irrigation; (2) high-rate irrigation; and (3) infiltration-percolation. The higher the application rate the lower the land requirement.

RESIDENTIAL EROSION AND SEDIMENTATION

The magnitude of sediment contribution from residential development operations is higher than it should be in Stillwater. This problem on a national scale has caused the National Association of Home Builders (NAHB) to undertake the development and publishing of a manual on the subject for use by builders, citizens, and government officials. The publication, entitled *Residential Erosion and Sediment Control* was a joint effort with the American Society of Civil Engineers and the Urban Land Institute.

The importance of erosion control in the City of Stillwater and environs cannot be overemphasized. Local planners and engineers may consult with the Soil Conservation Service in Stillwater for assistance in erosion control matters and to obtain valuable design criteria and methodology.

The reader is also referred to a publication of the U.S. Environmental Protection Agency entitled Guidelines for Erosion and Sediment Control Planning and Implementation numbered EPA-R2-72-015 dated August, 1972.

The NAHB publication entitled Residential Erosion and Sediment Control is available from the headquarter offices of either ASCE, NAHB, or the ULI at a cost of $10 per copy. Due to the fact that this publication is a product of the homebuilding industry itself and that it represents a practical and reasonable approach to erosion control, the manual should be considered a mandatory reading for the drainage planner and engineer. Excerpts are presented in the Appendix to this Chapter.

Earth Change Ordinance

An ordinance for urban erosion and sediment control paralleled with a county resolution having similar regulation will be adopted by the Stillwater Commission. At that time, the ordinance and resolution should be inserted in this location of the manual. The Residential Erosion and Sediment Control publication can be used on a basis for the regulation.

TABLE IX-4
ERODIBLE SOILS - STILLWATER, OKLAHOMA[2]
HIGHLY SUSCEPTIBLE

MAPPING SYMBOL	NAME	DESCRIPTION
5sCD4, 5rCD4	Renfrow Soils; 2-6% slope	Severely eroded, clayey land; not suited for cultivation; drought.
6rCD4	Zaneis and Norge	Severely eroded loamy upland soils; frequent gullies, suited only for permanent vegetation; erosion has removed or much of original topsoil.
70CD4	Stephenville-Darnell complex; 3-8% slope	Deep and shallow, severely eroded upland soils; formerly cultivated land; water and wind erosion are serious problems.
17DE	Vernon Clay Loam 5-12% slope	Shallow to very shallow, steep clayey soil; highly susceptible to water erosion; suited for limited amounts of grazing.
COCD4	Lucien-Zaneis Soils 3-8% slope	Shallow, severely eroded, sandy to clayey soil; gullies frequent; high susceptibility to water erosion.
OW	Oil Waste Land	Land that has been used as a disposal for oil and salt-water waste from oil well and drilling operations; agricultural value low.

TABLE IX-4 (cont'd)

ERODIBLE SOILS - STILLWATER, OKLAHOMA

MODERATELY SUSCEPTIBLE

MAPPING SYMBOL	NAME	DESCRIPTION
5pB3	Kirkland Silt loam; 0-3% slope	Deep gently sloping, moderately eroded upland soil with a clay subsoil; con- of water erosion is a problem.
5rBC3	Renfrow-Kirkland soils; 2-5% slope	Deep, gently to moderately sloping, moderately susceptible to water erosion.
6rC3	Zaneis loam 3-5% slope	Deep, moderately sloping, eroded, reddish brown colored prairie soil; moderately susceptible to water erosion.
6rD3	Zaneis loam 3-5% slope	Moderately deep, strongly sloping eroded upland soil; highly susceptible to water erosion.
60C3	Zaneis-Slickspot complex 3-5% slope	Moderately deep, moderately sloping, eroded upland soil containing numerous solonetz areas of alkali or white slickspots; droughty.
6+C3	Chickasha loam 3-5% slope	Deep, moderately sloping, eroded, loamy upland soil; very susceptible to water erosion.

TABLE IX-4 (cont'd)

ERODIBLE SOILS - STILLWATER, OKLAHOMA

MODERATELY SUSCEPTIBLE

MAPPING SYMBOL	NAME	DESCRIPTION
6nC3	Norge loam; 3-5%	Deep, moderately sloping, eroded, loamy prairie soil; quite susceptible to water erosion.
6nD3	Norge loam; 5-8%	Deep, strongly sloping, eroded, loamy upland soil subject to severe erosion when cultivated.
7+BC3 7+C3	Teller fine sandy loam; 1-5 % slope	Deep, moderately sloping, eroded, permeable prairie soil; high susceptibility to water and wind erosion.
7BC3 7C3	Teller loam; 1-5% slope	Deep, gently to moderately sloping; eroded, loamy prairie soil; high susceptibility to water.
7D3	Teller loam; 5-8% slope	Deep, strongly sloping, eroded, permeable prairie soil; occasional small crossable gullies high susceptibility to water erosion.
70BC3	Stephenville-Darnell complex, 4-5% slope	Deep to shallow, gently to moderately sloping, eroded upland soil, high susceptibility to water and wind erosion.

TABLE IX-4 (cont'd)

ERODIBLE SOILS - STILLWATER, OKLAHOMA

MODERATELY SUSCEPTIBLE

MAPPING SYMBOL	NAME	DESCRIPTION
17C	Vernon clay loam; 3-5% slope	Shallow, moderatey sloping, clayey upland soil, draughty; highly susceptible to water erosion.
20DE	Lucien-Vernon complex; 5-12% slope	Shallow, strongly sloping to steep, sandy to clayey prairie soils; sandstone and clay outcrops; careful range management required for maximum production.

REFERENCES

1. Guidelines for Erosion and Sediment Control Planning and Implementation, Environmental Protection Agency, EPA-R2-72-015, August 1972.

2. Residential Erosion and Sediment Control, NAHB.

3. *Soil Survey of Payne County, Oklahoma*, *U.S. Department of Agriculture*, August 1969.

4. Storm Water Pollution from Urban Land Activity, 11034, FKL, July 1970.

APPENDIX IX-A

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APPENDIX IX-A
RESIDENTIAL EROSION & SEDIMENT CONTROL

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RESIDENTIAL EROSION & SEDIMENT CONTROL

Increased erosion and sediment movement rates caused by man are superimposed on the natural rates of the cycle. Often the impact of these superimposed erosion and sediment rates causes nature's ability to readjust the cycle to be overtaxed. The overtaxing of nature due to urban erosion and sediment movements is the problem.

High, localized erosion rates can result in the loss of soil and its many elements required to support vegetation from areas where it is needed for embankments, construction areas, or for development. Hand in hand with high erosion rates are high rates of sediment transport and deposition in areas where it is not wanted--on roads, in ponds, reservoirs, streams, rivers, and harbors. Excessive sedimentation in lakes and streams can reduce or destroy their aesthetic and practical values for recreation, flood control, and water supply, and it can cause the loss of recreational fishing activities by covering and destroying food sources.

When man's activities increase the rate of erosion and sedimentation, the effect of this change must be evaluated. If the changes have adverse impacts, steps must be taken to limit erosion, usually at the source.

Concepts

Several basic concepts are the foundation for dealing with urban erosion and sedimentation. These are:

- Sediment movement should not be permitted at rates or in quantities which will cause significant residual damage. Under ideal conditions any change in the nature or amount of sediment leaving a site as a result of construction should maintain or improve environmental quality when compared to pre-construction conditions.

- Strong emphasis needs to be placed on "natural" engineering and land planning techniques, which will not only preserve and enchance natural features of the land, both on and off the site, but protect them. There are techniques which use and improve the natural processes taking place at a construction site, during and after the actual construction period, rather than ignoring or replacing them with artificial systems.
- There must be increasing recognition that each site has its own set of natural resources, land use limitations, environmental conditions, and occupancy requirements. These factors and the inter-relationships vary from site to site within a community, and variations in design standards will be required for achievement of optimum off-site protection.
- There must be continuing recognition of a balance of responsibilities and obligations between individual land owners and the public for the protection of the environment from adverse impacts of excessive erosion and sedimentation. It must be understood that significant immediate and long-term expenditures for the construction and maintenance of this protection will be incurred by individual home owners and the community. A balance must be struck in determining the acceptable ranges of damage.

Objectives of Erosion Control

The objectives of a program of managing urban sedimentation and erosion are:

- To provide a clear understanding of erosion and sediment control processes and philosophies.
- To demonstrate the impact of these philosophies on an environment for human habitation that maintains a level of quality which stimulates the reaction that life is a rewarding experience.
- To promote realistic achievements consistent with alternative design solutions, environmental quality, and sound judgment.
- To provide a method of determining the type and degree of investigation required for finding acceptable design solutions which maximize environmental quality throughout the anticipated life of the capital investment.
- To encourage design methods and approaches which will result in effective facilities requiring minimum maintenance.

- To encourage the development of new and better understanding of the long-term results of erosion and sediment control practices.
- To encourage rules, regulations, and laws at all levels of government that will be sensitive to the particular environmental conditions and values and human needs associated with each specific location.
- To summarize recent developments for the benefit of professionals not involved in the actual design of erosion and sediment control measures.

Principles

Principles for effective soil and conservation in urban development areas include the following:

- There shall be a minimization of changes in the rate of existing erosion and the amount of sediment movement throughout the life of a project as part of the residential development design process, leading to the preservation of environmental quality.
- There should be a balance between the use of on-site and off-site techniques, legislative and regulatory modifications are needed to achieve this balance.
- Flexibility and creativity are needed in the emerging field of erosion and sediment control.
- Control measures selected should be based on evaluations of costs, benefits, and other needs.
- A well-conceived residential development can result in a reduction or an elimination of erosion and sediment problems that exist prior to construction.
- Overall catchment area plans and objectives are desirable and often help provide a uniform basis for evolving site-specific measures.
- Specific requirements to prevent erosion and sedimentation should recognize the issue of risk, most particularly, the probable frequency of events for which protective measures are provided. (This is especially true for temporary measures.)

- Measures used will vary in their effectiveness at different scales. The suitability of measures for specific applications should be evaluated.
- Long-term maintenance is an integral aspect of erosion and sediment control design.
- The timing and location of construction affects the degree of risk and the effectiveness of measures required to control erosion and sediment deposition.
- The fundamental consideration in erosion and sediment control is the protection, maintenance, or establishment of ground cover.
- There should be a balance between the measures required of private developers, public and private utilities, public works, and agricultural and extractive activities, in relationship to their proportionate share in causing erosion and sediment problems.

Erosion Processes

There are five types of erosion by water: raindrop or splash erosion, sheet erosion, rill erosion, gully erosion, and streambank erosion. Each type may be aggravated when the natural landscape is disturbed.

Raindrop or splash erosion is the initial phase of water erosion. The impact of raindrops supplies the initial kinetic energy that starts soil erosion. Raindrop impact has a high capacity for detaching soil particles, but only a low capacity for transporting them. The amount of soil detached increases the intensity, velocity, and drop size. Raindrop impact splashes small amounts of soil but tends to compact the soil mass, reducing its ability to absorb water and, in some cases, increasing its resistance to erosion by tractive forces.

The second type of water erosion is sheet erosion, which is characterized by the general removal of a fairly uniform thin layer of soil from the land surface. This type of erosion is associated with runoff that often is referred to as sheet flow, due to its characteristic of flowing like a sheet over the ground. In contrast to raindrop impact, sheet flow usually has a low detachment capacity and a higher transport capacity. Much damaging erosion and sediment movement occur this way.

Streambank or channel erosion is the removal of soil from streambanks and stream bottoms. Clearing protective vegetative cover from banks, straightening and realigning channels, and construction projects which substantially increase the rates and volumes of runoff within the watershed can result in degradation and channel enlargement, and an increase in sediments transported far downstream from the disturbed area. For this reason, channel alterations should be avoided whenever possible; they should only be made when it becomes necessary to safely transport expected flood flows.

Rill erosion occurs when sheet flow moves down fairly steep slopes, forming small channels with depths up to 1 foot, fairly evenly spaced across a slope. When slopes are of loessial soils (unstratified loams chiefly deposited by wind) and are improperly finished, rill erosion can be unusually serious.

Gullies are an advanced form of soil erosion resulting from concentrated stormwater flow. Uncontrolled runoff in rill channels can continue to remove soil, often rapidly, and may turn into gullies up to 100 feet or more in depth. Gully erosion often moves more soil than sheet erosion. Gully erosion rates are highest for silty soils.

Erosion Factors

Climatic factors, soil erodibility, slope length, slope gradient, and vegetation are the primary factors involved in the water erosion process.

Rainfall. The climatic factors important in determining soil loss include the amount, intensity, and frequency of rainfall, and especially its seasonal distribution. The amount and intensity of rainfall relate to the rate of runoff, which detaches and transports soil particles downhill and downstream. The frequency with which some given amount of rainfall occurs determines how often the effects associated with it will be felt.

Soils. The main soil properties which determine erodibility include particle size distribution, clay and organic content, pore water chemistry, soil structure, permeability, specific gravity, and root structure; some of these characteristics will be discussed briefly below. The erodibility of bare disturbed soil or subsoil can be estimated if these properties are known.

Particle size distribution refers to the relative proportion by weight of the various sizes of soil particles found in a sample of the soil. Soil texture has a direct effect upon the water infiltration rate and permeability of a given soil.

Soil erodibility decreases as the organic matter content increases. The organic matter decomposes, and the resulting soil humus is important in producing organic clods which result in a less erodible soil structure.

Porosity, capillarity, and water content also affect soil erodibility. Granular non-cohesive soils which contain large amounts of fine sands and silts with little clay and organic matter are usually more erodible than soils with blocky or massive structures.

Soil permeability is the ability of soil to transmit water, horizontally or vertically. Soild permeabilities vary from a high of 60 inches per hour to virtually zero. Soils with higher clay contents are generally less erodible than those with lower clay contents, even though the former are less permeable. Other properties not discussed here are responsible for these differences in erodibility.

Slope Gradient and Length. The rate of erosion occurring on a slope during a rainstorm increases roughly in proportion to the square of increasing slope steepness because of the proportionality to the velocity of runoff flowing down slope. The tractive force of flowing water determines the size and number of soil particles detached and transported.

Total soil erosion also increases with the length of down slope distance, due to the relative cumulative increase in runoff volume and hence in runoff flow velocity. As an increase in velocity causes faster erosion, the increased volume also results in a higher sediment transport capacity, so that more soil particles are carried away.

Vegetation. Well-established and well-maintained vegetation is a major deterrent to soil erosion because it shields the soil from the raindrop impact and decreases flow velocity by increasing flow friction (resistance). Root systems may increase soil porosity, permitting greater water infiltration and reinforcing the soil mass. Stems, stalks, leaves, and roots break up flow patterns, increase flow friction and cause deposition of some soil particles. Plants also remove water from the soil by transpiration, so the soil can absorb more water, potentially decreasing the amount of runoff. It is important to recognize that planting and maintenance of vegetation is practical only on slopes flatter than three horizontal to one vertical, and then only where there will be regularly distributed rainfall.

Universal Soil Loss Equation

The Universal Soil Loss Equation (USLE) applies only to sheet, rill, and inter-rill erosion (it cannot be used to predict gully or streambed erosion), and it applies to large areas of loose soil, bare and exposed for 2 or more years. Frequently, applications of the USLE assume that all soil lost due to erosion will appear as downstream sediments. This assumption ignores the fact that substantial amounts of eroded soils will be deposited where slope gradients decrease or where runoff flow velocities are reduced for any other reason.

Quantification of the factors in the USLE is mostly a matter of very coarse judgment. The results obtained should be used with great caution, with recognition of the uncertainties involved. If they are used to estimate erosion at construction sites, USLE results should only be viewed as relative erosion rates, not absolute or reliable quantities.

The basic form of the equation is:

$$E = RKLSCP$$

Where:

E = Soil loss, in tons per acre per year
R = rainfall factor
K = soil erodibility factor
LS = slope length gradient factor
C = vegetative cover factor
P = conservation practice factor

From its form, it is apparent that the USLE applies only to a single, large, homogeneous area, so it is necessary to evaluate separately the soil loss for areas having differing conditions.

Erosion can be aggravated by frost action which expands and loosens soil particles, increasing their susceptibility to loss during runoff after thaws. Large quantities of soil may be lost during the spring if runoff from snow melt acts upon frost-loosening soil, a factor which should be considered in areas subject to heavy snowfalls. For short-term projects constructed during winter or spring months, the USLE estimates can be significantly wrong.

From the foregoing, it is evident that caution is necessary in applying the USLE to construction sites to determine potential sediment movements to streams. Precise results are unobtainable, but the USLE can be used for a rough evaluation of erosion control alternatives. It can provide some insight into various design alternatives at a given site, but it is questionable whether such insights will be an improvement upon basic considerations of comparative soil erodibility.

RESIDENTIAL DEVELOPMENT

The most important steps in controlling urban erosion and sedimentation on residential construction sites are briefly summarized below.

- Study the site and surrounding area and assess soil limitations and suitability of the site in view of the topography, geology, natural drainage, hydrology, prevailing winds, and other factors.
- Identify potential problem soils, if any.
- Select a method of development that will be compatible with site conditions.
- Examine existing and proposed drainage patterns.
- Examine the lengths and gradients of existing slopes.
- Evaluate watershed problems, upstream erosion conditions, and sediment conditions downstream from the construction site.
- Minimize through proper site planning the amount of site grading needed for development and utility construction.
- Avoid removal of existing vegetation insofar as possible.

The following general principles should be recognized through the site design and construction processes.

- Integrate clearing and grading with layout design.
- Keep clearing to a minimum and preserve as much of the existing vegetation as possible.
- Limit grading to those areas involved in current construction activities.
- Limit the time during which unprotected graded areas are exposed to rain and wind.
- Protect disturbed areas by using stabilization measures as soon as possible.
- Plan structural and vegetative measures to control the velocity and volume of runoff, and to provide windbreaks where needed.
- Divert and convey surface runoff safely through the area with structural measures such as diversion, storm drains, channels, or waterways.
- Ensure runoff velocities high enough to prevent unwanted deposition and low enough to prevent erosion.

- Construct sediment traps and basins to trap sediment on site when necessary.
- Stabilize exposed soils by adhering to time limits set out in the schedule for site grading, seeding, and mulching.
- Assure adequate maintenance of structural measures and of all plantings.

RESIDENTIAL DEVELOPMENT EROSION CONTROL

If control is necessary or desirable, water erosion control methods will include soil stabilization methods (vegetative and other), runoff control, and structural controls. To achieve the best results, these types of measures should complement each other. Erosion control measures reduce the duration of soil exposure and perform one or both of the following two functions: protect the soil by shielding it, and hold the soil in place. These functions may improve soil capacity to absorb stormwater runoff and thereby reduce the amount of overland runoff and its power to erode soil materials. The staging of grading operations and immediate re-vegetation will help minimize the exposed soil area at any one time. The control of surface runoff may be accomplished by interception, diversion, and safe disposal of runoff, in coordination with staged construction activities, designed grading methods, and the preservation of natural vegetation.

Protection of exposed soil from raindrop impact and subsequent erosion is obtained by applications of organic mulches, rock, chemical additives, sheets of jute netting, planting, and paving. The choice of which material should be based on economy, the future use of the area to be protected, and the degree of protection required. Details of some measures used for erosion control may be found in U.S. Environmental Protection Agency guidelines and various standards and specifications prepared by the U.S. Soil Conservation Service, Soil and Water Conservation Districts (in cooperation with local government agencies).

In some instances it may be possible to achieve an acceptable areawide average erosion rate by stabilizing existing off-site areas to reduce their erosion potential during a short on-site construction period of high sediment yield.

Stabilization

Temporary measure typically should be used if the soil is to remain exposed for more than 30 days. Permanent structural measures must be installed prior to or during active construction, not after.

Vegetative. Fast-growing annual and perennial grasses may be used on partially completed construction projects to protect them from erosion for short periods of time. Completion of final grading during seasons unfavorable for permanent vegetative stabilization may necessitate temporary structural surface stabilization. Certain areas such as drainageways, cut and fill slopes, borrow pit areas, excavations, and soil stockpiles often require immediate structural surface stabilization, although this need may be temporary.

The need for temporary stabilization generally should be avoided, as it is costly and rarely can be salvaged or incorporated into final protective measures.

Permanent vegetative stabilization should be long-lived and require minimal care or maintenance. Grasses and legumes are generally superior to shrubs and ground covers because of their more complex root systems which encourage formation of a water-stable soil structure. In addition, their leaves and stems protect the ground against erosion from wind and water. The selection of plant material should be based upon specific site growth expectancies, the purpose of the planting, and foreseeable assured level of maintenance activities. Any representation that a particular plant material is proper for a given slope, soil condition, and maintenance expectancy should be viewed skeptically unless the performance of comparable installations in the general area provides certainty.

The more fertile surface layer of the soil, if present, is usually removed and stockpiled during grading activities. Typically, exposed subsurface layers are less fertile, have lower organic matter content, and are more susceptible to erosion than surface soil horizons. For this reason, the physical and chemical properties of newly exposed soils should be considered. The principal chemical factors are nutritive elements such as nitrogen, phosphorus, magnesium, potassium, and occasionally certain trace elements. Systematic soil analyses of various horizons performed during the site investigation can be helpful in estimating the plant requirements and the proper application of fertilizers and other conditioning materials. For plant growth, factors such as soil texture, soil drainage, porosity, degree of aeration, structure, degree of compaction, soil temperature, slope gradient, pH, available nutrients, and exposure to the sun and wind must be carefully considered.

The steeper the slope, the more drought-resistant plantings should be. South-facing slopes will usually be drier than others. Two or three fertilizer applications may be necessary to ensure establishment of good stands of grass and legumes. Deep fertilization before the addition of topsoil on a slope may increase the potential for long-term growth.

Manmade cut and fill slopes in construction projects steeper than three to one are often impractical to stabilize in order to prevent excessive long-term erosion. Maintenance equipment can be safely operated on slopes with a maximum gradient no steeper than five to one, and it is hard to perform difficult maintenance even on a slope no steeper than three to one. If steeper slopes are incorporated in the grading plan, there should be positive assurance that plantings will flourish over the long term without maintenance. Plants should be selected accordingly. Except under unusual circumstances, vegetative and slope stability factors, as well as maintenance and other requirements, should preclude slopes steeper than three to one. Exceptions include rock slopes which may be safe and stable on faces as steep as one to nine, and slopes of loess or similar soils which should be finished as nearly vertical as possible but no more than 10 feet in height. Rock faces more than about 8 to 10 feet high should be benched, and drained soil pockets should be created to permit landscaping of the rock face. Slopes of loess soils should not have drainage passing over them from above. If vertical faces are impractical in loess soils, controllable slopes generally cannot be assured with slopes steeper than five to one.

Seeding can be used for both temporary and permanent soil stabilization. A common method is hydroseeding in which the seed is applied in a spray which also includes various soil surface stabilizers. As required, fertilizers and sometimes a fiber mulch or chemical soil stabilizer may be mixed with the spray.

After seed and fertilizer are applied to slopes, a mulch is usually needed for temporary protection. This may be applied as an asphalt emulsion which is also sprayed on, or straw mulch distributed by a blower can be used. Often, fertilizer, seed, and mulch are applied in a single operation. Straw mulches must often be held in place. This is accomplished by machine "cleating" (a tracked vehicle such as a bulldozer is run over the mulch), by spraying asphalt emulsion, or by staking plastic netting down over the straw.

Seed may be applied by machine drilling in furrows, a method most applicable to large areas having gentle slopes. The more expensive hydro-seeding method is best adapted to long, relatively narrow areas having steeper slopes. Drilling is unsuitable for areas with moderate or steeper slopes.

The cost of seeding, fertilizing, and mulching varies greatly, depending primarily on the size and shape of the area and the season and level of treatment required, but the range is typically between $800 and $1,000 per acre (1978 prices), which makes this process the least expensive approach to stabilization. Dependent on the area and season, the cost of regular watering until a strong cover is established may also be necessary. On erosion-resistant soils, seeding may be adequate for intermittent waterways when the design flow velocity is less than 2 feet per second. Sodding is recommended for waterways when design flow velocity is less than 2 feet per second. Sodding is recommended for waterways with design flow velocities between 2 and 4 feet per second. For velocities above 4 feet per second, structural stabilization of some kind (concrete, treated timber, or riprap) is usually required if excessive erosion and swale or channel maintenance is to be avoided.

Protection against waterway erosion can be achieved if proper consideration is given to the erodibility of designed slopes, flow velocity, flow resistance of the selected vegetation, and method of establishing the vegetation, provided foreseeable extreme flows will not be appreciably faster than erosion prevention design velocities. It is important to recognize that hydraulically efficient channel sections, which require minimum widths, are inconsistent with the low flow velocities needed to avoid damaging channel erosion.

Sodding is used for the immediate establishment of a permanent ground cover, but it will not adhere well without several weeks of growth after it is placed. It should be used on critical areas such as steep slopes, channels, and areas adjacent to paved land and buildings where splash from walls may cause erosion during storms. Where sod is laid on slopes steeper than five to one, it should be pegged to prevent it from washing away. Caution should be exercised in selecting pegs; long-lasting ones can remain as a hazard to foot traffic and machinery, whereas pegs made of such materials as soft wood will eventually decay. While sodding is fairly expensive (usually in the range of $1.00 to $2.00 per square yard in 1978), it may be economical if it can obviate the need for structural measures. Sodding also requires fertilization, watering, and initial maintenance.

In the proper seasons seed-bearing hay is occasionally used to establish a temporary cover, especially when further grading will be deferred. In this method, the hay forms the mulch. Costs for this type of cover and mulch vary considerably, and materials of suitable quality are often unavailable or inappropriate during non-growing seasons. Where it can be used, its effect is similar to temporary seeding and mulching.

Sprigging is sometimes used to establish Bermuda grass and other plants which are easily propagated. Sprigging is propagating by cuttings which may be spaded or cleated into the ground. Costs vary but are usually between transplanting and seeding where labor is inexpensive. If springs can be cleated in place, the resulting roughened ground may be more resistant to wind erosion than if they are set with a sprigging tool.

Transplanting, which also includes plugging, is another method of establishing vegetative cover from live plants. It is used for propagation of grasses such as Zoysia, and to establish shrubs and trees. Relatively mature plants that are transplanted can significantly reduce wind erosion and enhance the aesthetic qualities of a site. This method of establishing cover is more costly than seeding or sprigging, but it is usually less expensive than sodding. The cost of transplanting trees and shrubs varies greatly, depending on the species of plant, labor rates, feeding, watering and maintenance expenses, and site conditions. When transplanting is used to establish cover, special stabilization of intervening exposed soil generally is necessary to prevent its washing away, even on fairly flat areas.

Non-Vegetative. Non-vegetative soil stabilization also includes temporary and permanent measures. As well as giving temporary protection until permanent vegetative covers are established, temporary non-vegetative stabilization can protect during grading delays. Mulches, nettings, and chemical binders are typical temporary practices.

During periods of extreme drought, cold, or other conditions unfavorable for plant growth, a protective layer of mulch should be applied over exposed soil. Mulching is an important erosion control measure even when no vegetation is used, because it protects the soil against erosion. It is also important when establishing vegetation, particularly grasses and legumes, because it prevents seeds, fertilizer, and other soil additives from washing away, improves capacity for rainfall infiltration into the soil, prevents wide variations in soil temperature, encourages retention of moisture by reducing surface evaporation, and shields delicate young plants.

The most common mulch materials are hay, small grain straw, wood chips, jute matting, glass fiber netting, plastic and asphalt emulsions, and various paper products. Most fiber mulches require immediate anchoring to prevent dispersal. Using plastic sheeting as a mulch is unwise because direct sunlight may cause it to kill seeds and plants.

Permanent non-vegetative stabilization is used where conditions preclude the use of vegetation. Structural treatment may be required for excessively steep slopes, areas of ground water seepage, droughty soils (soils which for one reason or another do not absorb or retain moisture well), or waterways subject to high flow velocities. Coarse crushed rock and gravel are commonly used materials where slopes are more gentle. Since non-vegetative measures do not regenerate as live plants do, their use is limited to relatively level areas with maximum gradients no steeper than about five to one. Costs vary widely, depending on availability of materials and ease of application. This type of protection can be integrated with a permanent landscaping plan, permitting some degree of cost recovery. Except in unusual circumstances, rock and gravel should not be used for temporary stabilization, as they will interfere with establishment of permanent vegetative covers.

Structural Measures. Structural measures are designed and built to fulfill a specific function. The most common structures are those which intercept surface runoff and convey it to a safe disposal area to keep runoff away from erodible soil or to prevent gully erosion. Sometimes runoff is intercepted to trap moving sediment.

There are numerous structural measures which can be employed. For a full discussion of these measures, the reader is referred to Item (2) in the references.