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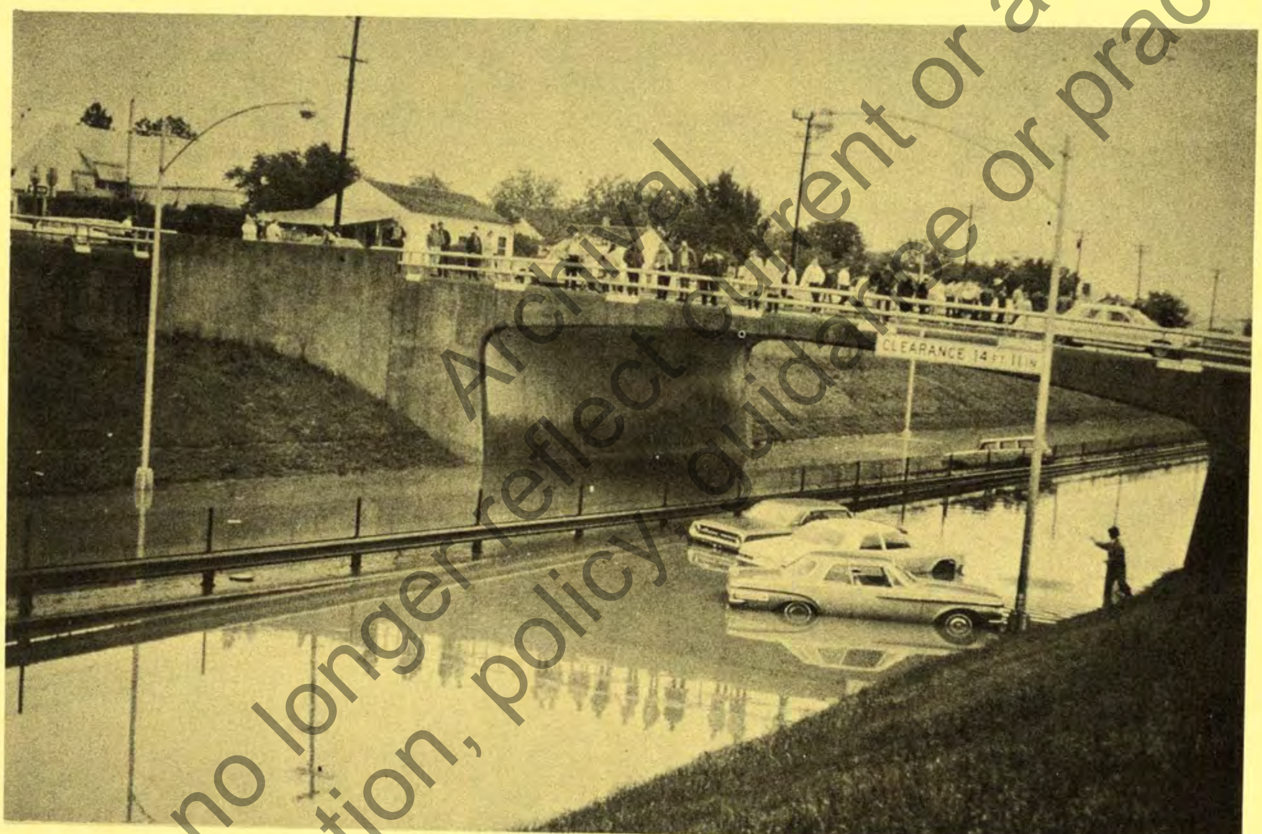
DESIGN OF URBAN HIGHWAY DRAINAGE

THE STATE OF THE ART



U.S. DEPARTMENT OF TRANSPORTATION
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U.S. Department of Transportation
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PREFACE

The contract for the preparation of this publication states in part that it "....is to be limited to urban storm drainage considerations and deal primarily with surface water collection and disposal and the incorporation of storage where appropriate. The hydraulics of bridges and culverts (cross-drains) is to be excluded. The work will not require the development of new concepts, design techniques or computer programs, rather, it will involve the review and evaluation of available design information and the assembly of the most useful information into a manual. The manual is to present design philosophy and concepts along with the best available methods of analysis which can be carried out by hand or on a programmable calculator. Design techniques are to be illustrated through the use of examples. Useful design aids are to be included".

The Contractor acknowledges the constructive criticisms of the several FHWA reviewers of early drafts and is especially grateful to Messrs. Leonard Greer and Daniel S. O'Connor for their counsel and understanding guidance throughout the prosecution of this work.

Discussion of curb-opening inlets and the pertinent design aids was prepared by Carl F. Izzard. Similar material concerning the design of grate inlets including the appropriate design charts was prepared by Daniel S. O'Connor.

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LIST OF SYMBOLS

A	Tributary area in acres or hectares.
A_o	Area of one orifice in square feet or square metres.
A_p	Cross-sectional area of pipe in square feet or square metres.
a	A constant.
b	A constant.
b	Width of emergency spillway in feet or metres.
c	A constant.
a_1, b_1, c_1	Standard storm parameters describing the ratios of various duration intensities to the 1-hour intensity for the same duration.
C	Coefficient in the Rational Formula.
C_{es}	Coefficient in emergency spillway formula.
C_w	Discharge coefficient in weir equation.
C_o	Discharge coefficient in circular orifice formula.
C_t	Coefficient in synthetic unit hydrograph formula for lag time.
C_p	Coefficient in synthetic unit hydrograph formula for peak flow.
cfs	Cubic feet per second.
CN	Curve number in Soil Conservation Service method of runoff determination.
D_p	Pipe diameter in feet or metres; inches or millimetres.
D_o	Diameter of circular orifice in inches or millimetres.
d	Depth of curb flow in feet or metres.
d'	Depth of triangular flow (spread) at any distance from face of curb in feet or metres.

d_c	Critical depth of flow in feet or metres.
E	The hydraulic efficiency of an inlet grate, in percentage.
E_o	Inlet grate efficiency without splash-over, in percentage.
F	Infiltration or actual retention in inches or millimetres occurring after runoff begins.
F_w	Froude number of gutter flow related to the depth of approach flow to an inlet at a distance w from the curb.
g	Gravitational constant; 32.16 feet per second per second or 9.8024 metres per second per second.
H_p	Effective head on outfall pipe or emergency spillway in feet or metres.
H_w	Effective head above top rim of weir in feet or metres.
H_o	Effective head at an orifice, in feet or metres.
h	Height of curb-opening of an inlet, in inches or millimetres.
h_f	Friction loss in feet or metres.
h_j	Head loss at a junction in feet or metres.
h_m	Minimum height of curb opening required for weir-type operation, in inches or millimetres.
h_t	Head loss at a transition, in feet or metres.
h_v	Velocity head in feet or metres.
I_1	Inflow rate in cfs or m^3/s at beginning of time Δt.
I_2	Inflow rate in cfs or m^3/s at end of time Δt.
I_a	Initial abstraction from rainfall in inches or millimetres.
i	Average rainfall rate in inches or millimetres per hour.
I-D-F	Intensity-Duration-Frequency.
j	Counter for data points.

K	Coefficient
K_c	Pipe head loss coefficient.
K_e	Entrance head loss coefficient.
L'	Effective or unclogged grate length in feet or metres.
L_i	Length of curb-opening inlet in feet or metres.
L_o	Overland flow length in feet or metres.
L_p	Length of pipe in feet or metres.
L_w	Length of weir in feet or metres.
L	Length in miles or kilometres of the mainstream from the point of interest to the watershed divide.
L_{ca}	Distance in miles or kilometres along the mainstream from the point of interest to a point opposite the centroid of the basin.
L_1, L_2, L_3	See Figure 5-5.
m^3/s	Cubic metres per second.
n	Roughness coefficient in the Manning Formula.
N_i	Number of circular orifices under the same effective head.
O_1	Rate of outflow in cfs or m^3/s at beginning of time Δt.
O_2	Rate of outflow in cfs or m^3/s at end of time Δt.
P	Storm rainfall in inches or millimetres.
P	Percent of impervious area.
P_a	Percentage of watershed that is impervious.
P_e	Potential runoff or effective storm rainfall (storm rainfall, P, minus the initial abstraction) in inches or millimetres.
Q	Rate of flow in cubic feet per second or cubic metres per second.
Q_{es}	Emergency spillway flow in cfs or m^3/s.

Q_f — Frontal flow approaching a grate inlet; i.e. the flow in that part of the gutter width equal to the grate width.

Q_i — Intercepted flow at inlet, in cfs or m^3/s; or discharge in cfs or m^3/s per foot or metre of channel width.

Q_o — Orifice flow rate in cfs or m^3/s.

Q_p — Flow rate in pipe in cfs or m^3/s.

Q_r — Flow through the riser in cfs or m^3/s.

Q_t — Total flow in cfs or m^3/s.

Q_w — Flow rate over weir in cfs or m^3/s.

q_p — Runoff rate in cfs per square mile or m^3/s per square kilometre.

R — Hydraulic radius or area of flow cross-section divided by wetted perimeter.

r — Rainfall rate in inches per hour or millimetres per hour.

r_{av} — Average rainfall intensity in inches per hour or millimetres per hour.

r_{av}^{j} — j-th data point for average rainfall intensity in inches per hour (iph) or millimetres per hour (mm/h).

r_{av}^{T,t_d} — T-year, t_d-hour (or minute) average rainfall intensity in inches per hour or millimetres per hour.

$r_{av}^{T,1}$ — T-year, 1-hour average rainfall intensity in iph or mm/h.

r_{av}^{10,t_d} — 10-year, t_d-hour (or minute) average rainfall intensity in iph or mm/h.

r_{av}^{100,t_d} — 100-year, t_d-hour (or minute) average rainfall intensity in iph or mm/h.

$r_{av}^{10,1}$ — 10-year, 1-hour average rainfall intensity in iph or mm/h.

$r_{av}^{10,24}$ — 10-year, 24-hour average rainfall intensity in iph or mm/h.

$r_{av}^{100,1}$ — 100-year, 1-hour average rainfall intensity in iph or mm/h.

S	Potential retention in inches or millimetres.
S_c	Critical slope in feet per foot or metres per metre.
S_o	Longitudinal slope in feet per foot or metres per metre.
S_x	Cross-slope in feet per foot or metres per metre.
S_1	Storage in cubic feet or cubic metres at beginning of time Δt.
S_2	Storage in cubic feet or cubic metres at end of time Δt.
T	Return period in years.
T	Top width of water surface (spread) from curb face toward crown of pavement, in feet or metres.
T_c	Time of concentration in minutes.
t	Time in minutes.
t_d	Duration of rainfall in minutes.
t_p	Lag time in hours; the time from the centroid of effective rainfall to the runoff peak.
t_r	Standard unit duration of excess rainfall in hours.
t_R	Other than standard unit duration of excess rainfall in hours.
t_1	Original lag time in hours.
t_{pR}	Adjusted lag time in hours.
Δt	A finite interval of time in minutes (usually short).
V_c	Critical velocity in feet per second or metres per second.
V_F	Frontal flow velocity in feet per second or metres per second.
W	Width of inlet grate in feet or metres.
ΔW	The extra grate width which would be necessary to reflect the interception of both frontal and side flows.
W_E	$W+ \Delta W$, the effective grate width to reflect the interception of both frontal and side flows.

W_{75}, W_{50}	Time widths in hours for the unit hydrograph at flow rates 75% and 50% of the peak runoff rate.
x	Exponent; ratio of 100-year, 1-hour rainfall to the 10-year, 1-hour rainfall.
Δy	Change in hydraulic grade line or water surface elevation through a junction, in feet or metres.
Z	Ratio of water surface width to its depth at the curb.
γ	Gamma, a factor between 0 and 1 indicating the position of the center of the mass of the most intense burst of rainfall in a rainfall event. 0 is the beginning of the storm and 1 is at the end of the rainfall event.
τ	Tau, the integration variable for time.

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CHAPTER 1

GENERAL PRINCIPLES OF STORMWATER DRAINAGE

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GENERAL PRINCIPLES OF STORMWATER DRAINAGE

1.1 Introduction

Stormwater drainage attitudes and consequent policies have been undergoing a significant redirection in the past decade (1968-1978). Historical practice has involved a philosophy of intercepting, collecting and disposing of stormwater runoff as rapidly as possible. The cumulative effects of such past concepts of urban storm drainage have been a principal cause of increased frequency of downstream flooding, often accompanied by diminishing groundwater supplies as direct results of urbanization; or they have necessitated development of large-scale downstream engineering works to prevent flood damage. There is increased attention in urban area master planning of storm drainage to the desirability of detaining or storing rainfall close to where it falls on-site, which sometimes requires trade-offs with short-term, localized inconvenience.

Water quality has become one of the most prominent issues in the increasing public awareness of the environmental impact of man's activities. There is accumulating evidence (Ref. 1-6) that storm runoff includes significant amounts of contaminants. A significant portion of the contaminants originate in the surface runoff from pavements. Consequently, planning stormwater facilities may have to consider the possibility of treatment of stormwater prior to its ultimate disposal. Any treatment of contaminated runoff is most cost-effective if the treatment facilities are handling as unvarying a flow rate as is possible. This makes storage virtually mandatory.

Today's urban drainage master plan should include collection, storage, treatment and disposal. Logically, each should be an integral and interrelated part of any stormwater management system. Furthermore, for any specific project, there is an optimum mix of these interrelated components of a system. This optimum mix changes from project to project.

1.2 Basic Concepts

The principles, objectives and design considerations in the current approaches to stormwater drainage involve a variety of basic concepts of which the following are the more important (Ref. 1-1).

For ordinary design rainfall frequencies (about 1 to 10 years) the peak runoff after the provision of drainage facilities, should not be significantly different after development of an area than it would be if such development had not taken place.

The increasing focus on water quality in urban water resources has as a corollary the identification and application of engineering techniques that will preserve and enhance the natural features of a locale and maximize economic-environmental benefit. Improvement of the effectiveness of natural systems rather than replacing, downgrading or ignoring them is an objective of current (1978) engineering design.

In the middle 1960's (Ref. 1-2) there was initiated a heightened consciousness of the fact that in all instances of stormwater drainage there actually exist two principal systems for handling surface water runoff. The one on which engineering planning, design and operations have been almost wholly concentrated in the past has been termed the "Minor System" and might better be called the "Convenience System". This, in turn, is part of the larger major storm drainage system which includes all the natural and man-made drainage facilities in an entire watershed. The "Convenience System" is that scheme of curbs, gutters, inlets, pipes or other conduits, swales, channels and appurtenant facilities all designed to minimize nuisance, inconvenience and hazard to persons and property from storm runoffs which occur at relatively frequent intervals (usually all runoffs associated with a 10-year or less recurrence interval rainfall). Current progressive engineering recognizes the need to devote more detailed attention to the planning and design of the supplementary aspects of the overall "major system" which carry the excess flow over and above the hydraulic capacity of the various components of the convenience system. The initial portion of the collection process in the convenience system, i.e. gutters and inlets, should have as much design attention as the conveyance system after the water has been removed from streets, sidewalks, parking and landscaped areas, etc. For example, when the inlets, pipes or conduits become overtaxed, the excess runoffs use the hydraulic capacity of the roads and streets and flow overland. Past practice has not consciously recognized in design detail the functioning of the supplementary facilities in the major storm system which come into operation when the less frequent higher-intensity storms occur. Lack of conscious attention to the supplementary functioning of the major storm drainage system is no longer acceptable.

There is a continuing and growing recognition that there are interrelated responsibilities and obligations for collection, storage and possible treatment of stormwater. These responsibilities and obligations should be shared by all involved, both private developers and property owners, as well as public agencies including that which in an urban area bears primary responsibility for stormwater drainage.

In addition to specific recognition of the convenience and major drainage systems, there should be recognition of the use of on-site detention storage and "blue-green" (Ref. 1-2) development. The increased use of storage to balance out handling or treatment of peak flows; use of land treatment systems for handling and disposal of stormwater; and perhaps most important, a recognition of temporary ponding at various points in a system, are potential design solutions rather than problems in many situations.

Another basic reality is the fact that every site or situation presents an unique array of physical resources, land use conditions and environmental values. Variations of such factors generally will require variation in design standards for optimal achievement of runoff management objectives.

An overall consideration of optimum design of stormwater collection, storage and treatment facilities indicates that a balance should be struck among the capital costs, operation and maintenance costs, public convenience, environmental enhancement and other design objectives. Such an optimum balance is dynamic, changing over time with changing physical conditions and value perceptions.

Stormwater is a component of the total water resources of an area and should not be casually discarded but rather, where feasible, should be used to replenish that resource. In many instances, stormwater problems signal either misuse of a resource or unwise land occupancy.

Finally, there is increasing awareness of the need to reevaluate approaches to basin-wide management which is the responsibility of and should be an objective of the public sector.

The irrefutable desirability of basin-wide plans, to which individual developments should conform is strongly borne out by the following. Current practices, based on traditional drainage concepts of the past, allow upstream development to increase runoff. As a consequence, downstream development relying on new concepts might be unable to accomodate, without significant additional cost, the upstream excess runoff thereby generated. However, if the approaches suggested herein for individual projects use the strategy of retention and attenuation of peak runoff and total runoff (to values not significantly different from pre-development levels) such development would normally be compatible with any future plan that might evolve for a watershed. It seems clear that the public sector should develop basin-wide plans incorporating the best current philosophies and knowledge.

1.3 Highway Drainage Needs and Requirements

A highway, traversing an urban area in various stages of development, rarely involves in its storm drainage considerations all of the subwatersheds in the principal traversed watershed(s). Furthermore, in any urban areas public agencies such as towns, cities, counties and special storm drainage districts have jurisdiction over the planning and provision of storm drainage. The local agency, particularly if it is regional in character, should have the responsibility for developing master plans for stormwater drainage. Because of this responsibility, such regional agency generally develops criteria and design standards.

Usually, the detailed provision of stormwater drainage for the highway will utilize the available outfall facilities. If the existing local outfall facilities are inadequate, the highway agency and local drainage authority will have to negotiate the most acceptable solution to both parties.

Traffic safety is intimately related to surface drainage. Rapid removal of stormwater from the pavement minimizes the conditions which can result in the hazardous phenomenon of hydroplaning. Adequate cross-slope and longitudinal grade ensure such rapid removal. Where curb and gutter are necessary, the provision of sufficient inlets and satisfactory cross-slope and longitudinal slope can limit the spread of water on the pavement. Extra inlets at profile sags will minimize ponding due to clogging. Inlets at strategic points on ramp intersections and approaches to superelevated curves will reduce the liklihood of gutter flows spilling across roadways. Satisfactory cross-drainage facilities will limit the buildup of pondage against the upstream side of roadway embankments. Where there is a probability of the overflow of a roadway by flash floods in remote areas, an automatic warning system should be installed.

Bridge foundations should be designed to be safe from scour. An automatic warning system should alert traffic to the formation of ice on a bridge deck.

All grate inlets should be bicycle-safe and hydraulically adequate.

Where safety considerations make it desirable, open channels and storage basins should avoid where possible the delivery of slope runoff directly onto pavements to prevent the presence of silt or ice on the pavement (the latter could occur in winter when daytime thawing of a slope can result in night-time freezing on the slab).

Since many communities and urban areas use less than a 10-year frequency design for their storm drainage facilities, coordination of the highway drainage with that of the local urban area is a primary factor requiring very careful consideration. Location studies of a highway through a builtup area require close attention to how the proposed highway's drainage requirements can be satisfactorily coordinated with those of the community. Necessarily, both horizontal and vertical location of the proposed highway improvements are of great significance since most major city streets are likely to have existing storm sewers and buried utilities.

The cross-drainage needs of a highway will usually require a culvert or bridge. Design of highway culverts often results in the placing of the invert of the culvert at approximately the elevation of the flowline of the natural watercourse. Under some circumstances, the local drainage authority has a practice of placing trunk storm sewers below the bed of a natural watercourse. Whether the highway culvert or the local trunk storm drain is constructed first, cooperative consideration of the needs of both agencies should be involved in the planning and design of each. The highway designer should carefully examine the capability of existing closed drains to handle the runoff rates for which the highway facilities would ordinarily be designed.

Whether the highway is at grade, in cut or elevated, significantly affects the handling of the surface water drainage. At grade, the surface drainage of the roadway is a part of the surface drainage system as it serves the local streets and developed areas. The provision of adequate, suitably located inlets to provide rapid removal of surface water from the trafficways is the primary need with probable delivery of the collected roadway surface water into the existing urban drainage system facilities. If the highway is in cut, there is a likelihood there will be low points or sumps at which excess surface runoff will collect and pumping may be needed. In cut, there may be encountered difficult problems of potential interference of the profile grade with sewers and other underground utilities in intersecting local streets. The designer of drainage for an elevated roadway may have more freedom of choice of pickup of collected surface water.

Directional or other interchanges pose particular surface runoff collection problems in that it is more difficult to achieve efficient pickup of gutter flows where the longitudinal slope of the gutter is high. Ramp quadrants may offer opportunities for development of detention storage.

The provision of retardation or detention storage as a part of the facilities to handle runoff from the urban freeway very probably involves a cooperative provision of such storage with the local drainage authority. The acquisition of rights-of-way for freeways in urban or urbanizing area does not often afford economic opportunities to acquire locations with site characteristics suitable for the development of pondage or other economic detention storage. The variety of location and character of storage to be incorporated as part of a stormwater management program is discussed in detail in Chapter 4 of this manual.

The distinctive criterion for surface drainage of highways is the great need to remove surface water from high traffic pavements as rapidly as possible. This criterion arises from the fact that at the speeds of traffic on freeways, the presence of a film of water which does not drain off pavement rapidly enough can, under certain circumstances, involve moving traffic in the very hazardous phenomenon known as "hydroplaning". The texture and character of the pavement surface and the condition of the tire treads on the individual vehicles are vital parts of this problem.

1.4 Economics of Storm Drainage

The economics of storm drainage is concerned principally with the costs associated with proper handling of runoffs of various frequencies versus the associated inconveniences or damages. The rarer the design frequency the larger are the design capacities of the storm drainage facilities, particularly the collecting system. This leads to larger first cost which translates into larger annual investment charges.

As the capacity of a storm system increases, storm sewers will be overtaxed less frequently and consequently, less inconvenience and damage related to such overflows can occur. The design cost objectives are to minimize the total annual cost of the stormwater drainage facilities (capital costs, maintenance and operating costs, etc.). An associated objective is the reduction in average annual costs of damages by overflow or other aspects associated with lack of capacity in the system. Where overflows are evidences of incapacity of the storm system, investment to reduce the frequency of such overflows is more likely to be justified. It has generally been impractical to develop a realistic evaluation of damages associated with each of several alternative stormwater systems and its cost. This is because urban stormwater damages related to rainfall events of known frequency of recurrence are difficult to measure and evaluate; and the collection of suitable data is very costly. Usually such studies are not made for an urban storm drainage project. Judgement based upon performance experience in similar developed areas is generally the basis for selecting a design frequency. It should be further noted that for storm drainage the frequency used as a guideline for the criteria is that of the rainfall since there are available sufficient rainfall data to develop reasonably reliable frequency relationships particularly for recurrence intervals of about 50 years or less. Unfortunately, in urban areas there are very few runoff records.

Methods for determination of runoff all require the choice of a design rainfall. Since there exist no suitable urban runoff records upon which to predicate the choice of a runoff frequency related to the desired quality of storm drainage, it becomes most practical to base the drainage design on the frequency of rainfall which can be readily determined for any United States location. Sometimes, the rainfall and runoff frequencies are thought of as identical but that this is erroneous is illustrated by the fact that identical rainfall on the same area can result in different runoffs if the area has been dry for an appreciable period in one case and the same area has been thoroughly wetted in the other. The practical base to which to tie the relative quality of storm drainage to be provided therefor is the causative rainfall and its probable frequency of recurrence.

The relative hazards to persons, property and traffic associated with each of the runoffs related to rainfalls of several selected frequencies should be used in storm drainage design. Mitigation of drainage-related damages or losses is theoretically balanced as a benefit against the associated drainage costs. In practice, judgement has largely been relied upon to choose the design rainfall frequency.

The majority of large American cities use a 5- or 10-year rainfall recurrence interval for their storm drainage with several adopting a 15- to 20-year frequency. The shorter recurrence intervals generally are standards for urban areas of flatter terrain. Costs limit the design frequency in some instances more than in others, but in all cases, there is a relationship between the quality of storm drainage and what the benefitted area is willing to pay.

Urban highways such as the interstate system should use high drainage standards. At locations where water can pond on the roadway and create a hazard to life, traffic and property, as in sag vertical curves, underpasses and depressed sections, roadway drainage systems should be designed for a relatively infrequent rainfall event (perhaps five times the recurrence interval of locations where water cannot pond). At such locations the flow should include bypass amounts from upstream inlets and tributary areas with facilities designed to a lesser standard. At locations where water cannot pond, inlets for roadway and bridge drainage should be designed so that spread on the pavement from a 10-year rainfall event will be limited to the highway shoulder. Roadside and median ditches should be designed to convey at least the runoff from a 10-year rainfall event without encroachment on the shoulders.

Urban highways other than interstate should preferably be provided with drainage systems based upon a 10-year rainfall. If local drainage facilities and practices have provided drains of a lesser standard to which the highway system must connect, especial consideration should be given to whether it is realistic to design the highway drainage system to a higher standard than the available outlet(s). If the local facilities and policies of the local drainage authority require a higher standard than normally used for the highway, the drainage system for the latter should give consideration to a basis of design compatible with that locally followed.

Detention storage should be considered where economies can be achieved or downstream flooding problems would otherwise be worsened by drainage from the highway development.

Cooperative projects with other agencies should be considered where a savings of public funds can be realized from the joint effort.

Where practicable, existing outfalls should be utilized to dispose of flow from the highway drainage system. Improvements to the existing natural or man-made outfall should be made only to the extent necessary for assurance that the roadway drainage system will operate as designed and will satisfy legal responsibility.

1.5 Cost Considerations

The basic factors making up the total costs of a highway drainage system are:

- Capital investment costs (debt service)
- Right-of-way or land acquisition costs
- Damage to other properties
- Environmental studies: permits
- Construction costs
- Traffic delays
- Maintenance
- Operation
- Administration

Existing serviceable facilities including natural drainage swales, ditches, creeks, ponding areas, etc. should be used wherever possible to reduce initial costs. For highways in urban areas, incremental land costs can and are usually held to a minimum by acquiring sufficient right-of-way width to include most of the drainage facilities within that right-of-way. Elsewhere, existing or future streets, water courses, ravines or other property unlikely to become developed should be used for the location of drainage facilities.

A recent storm drainage cost study (Ref. 1-4) shows that irrespective of the degree of development (percent of imperviousness) there was a rapid increase in the cost per acre of storm drainage facilities for the 1- to 10-year frequency recurrence interval as compared to a slow increase in unit cost between the 10- and 100-year. This study reflects the very important initial (principally construction) costs and does not include such other costs as maintenance and operation. The significant fact is that the dollar of incremental cost invested over and above the cost of storm drainage facilities for a 10-year design frequency achieves more desirable quality of drainage than does that same dollar incremental investment in improving facilities designed for any recurrence interval of less than 10 years.

Detention storage costs have been given some study (Ref. 1-5) but the wide variety of circumstances governing each installation precludes any general unit costs. Earthen basins have the lowest costs and covered concrete tanks the highest. The meager operating and maintenance cost information is unsatisfactory as a guide to probable costs. For any specific project, the conditions influencing design of a drainage system are unique and the designer must select a system on the basis of total costs applicable to the specific circumstances. There always should be considered the overall cost during the life of the project rather than initial installation costs only.

Of significant importance to the location of urban highways and their drainage are the Flood Insurance Rate Maps of the National Flood Insurance Program of the Federal Insurance Administration. These have been prepared for very many urban areas throughout the country and show for the principal watercourses traversing a community, the flood boundaries for the 100- and 500-year events. Various zones within such boundaries are designated to indicate flooding depths and overflow velocity characteristics of importance to insurance rate determination. The information given by these maps could be of value in the location and drainage of highways. The maps often bear the name of an incorporated community within which boundaries, the flood determinations have been made from the best available hydrologic information. The development of improved hydrologic information sometimes results in revisions.

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PRECIPITATION

2.1 Introduction

Hydrologists use the general term "precipitation" to describe all types of moisture that can fall from the clouds to the ground. In storm-generated runoff, rainfall is the primary form of precipitation. Under certain circumstances, the melting of snow can contribute significantly to runoff but such instances are so special that in general, this manual will consider precipitation to mean rainfall.

Where water vapor is present in the atmosphere, anything that can bring about a cooling of the air may cause the moisture to condense to form water droplets. For significant amounts of precipitation to occur, large regions of air must be cooled and this is usually achieved by a lifting of the air. The factor which causes the air-lifting phenomenon leads to a classification of the resulting precipitation.

The movement of air masses from high pressure to low pressure areas results in what is termed "cyclonic" precipitation. Unequal heating of the earth's surface causes the pressure differences. Cyclonic precipitation can be categorized as frontal or non-frontal. The frontal cyclonic storms can be the warm front type in which cold air is replaced by warm air or the cold front where cold air replaces the warm air. A stationary front indicates no movement of the front.

Heating of moisture-laden air near the earth's surface can result in "convective" precipitation. Water vapor is taken up when the heated air expands and the warm, moist air rises and is surrounded by cold, dense air which occasions precipitation. The variable spottiness with sometimes light showers and occasional high intensity rains are frequently termed "thunderstorms". Because of its spatial variability, convective precipitation is often the most difficult to accurately record.

Where topography causes air to rise with resulting precipitation, it is termed "orographic precipitation" and can vary significantly in intensity and quantity. Obviously, mountainous and hilly regions cause particularly pronounced variations. Warm air rising on the windward side of a slope moves upward and as the warm, moist air comes into contact with the cooler air at higher altitudes, precipitation forms. Consequently, the windward side of major slopes or mountains is the rainy side.

The form and intensity of rainfall also leads to National Weather Service (NWS) classifications. Drops larger than 0.02 inches (0.508mm) with intensities greater than 0.04 inches per hour (1.016mm per hour) are classified as "rain". Water drops less than this size and intensity are termed a "drizzle". Recorded total precipitation of less than 0.005 inches (0.127mm) is termed a "trace". The usually localized "thunderstorms" are

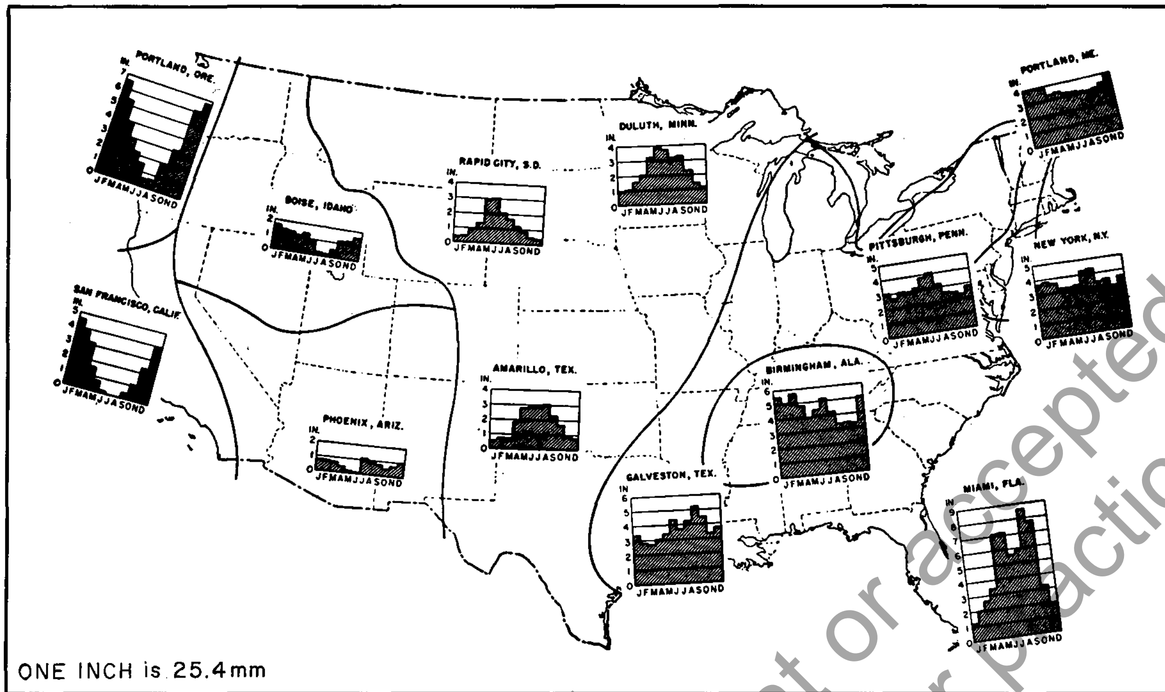

FIG. 2-1 Typical monthly distribution of precipitation in various climatic regions.
From: Hydrology For Engineers by Ray K. Linsley, Jr., Max A. Kohler and Joseph L. H. Paulhus, Copyright 1958 by the McGraw-Hill Book Co. Used with permission of McGraw-Hill Book Company.

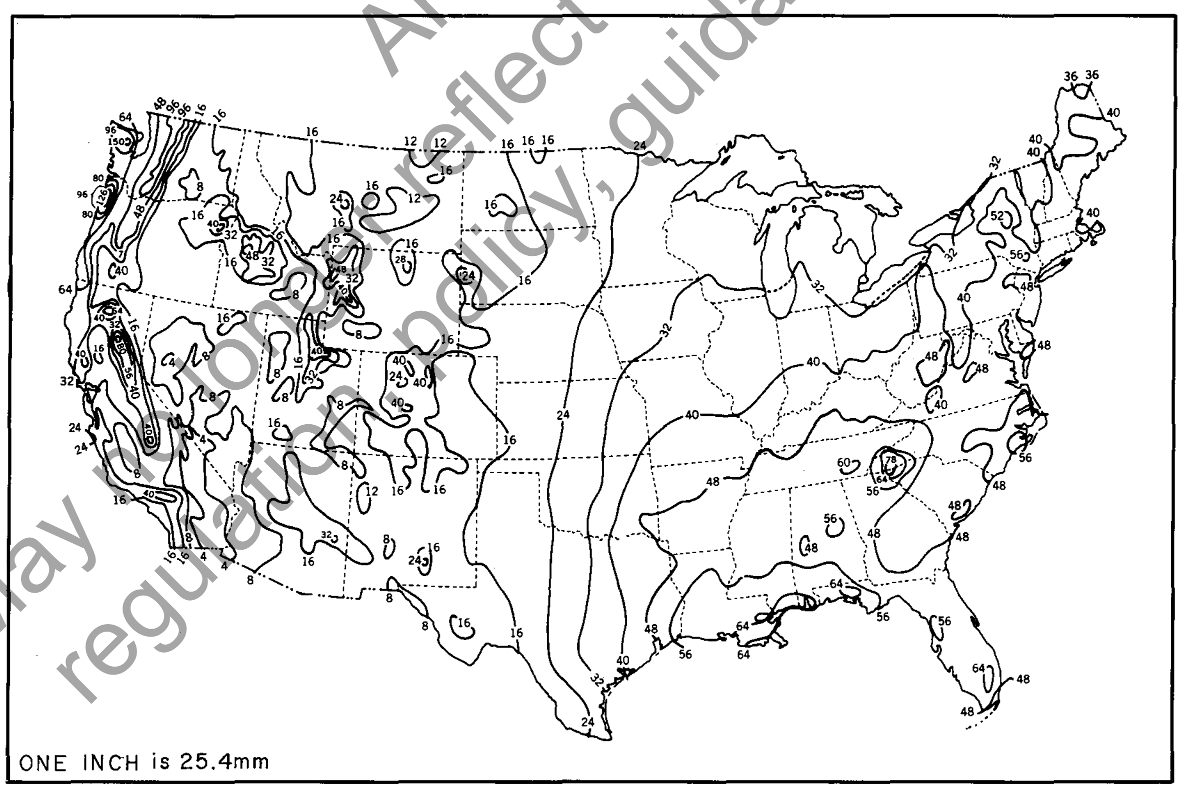

FIG. 2-2 Average annual precipitation in the United States.
(After National Weather Service).

high intensity, short-duration (15-30 minutes) forms of precipitation.

Intensity and variable monthly distribution vary for specific geographic and climatic areas. This is evidenced by Fig. 2-1 which indicates that most of the eastern third of the country has reasonably uniform rainfall throughout the year. The plains states in the central third of the country have wet summer seasons as compared to the winter months. Mountainous areas have light rainfall, the majority of it occurring in the fall, winter and spring, with very little in the summertime. The West Coast states secure the majority of their rainfall with the highest intensities in the winter months. The average annual precipitation varies across the United States as shown in Fig. 2-2. The effects of the topographic or orographic influences of the western part of the country are evident in this figure.

2.2 Available Precipitation Data

Precipitation information is collected by vertical cylindrical rain gauges of about 8 inches (203mm) diameter and is usually designated as "point rainfall". The National Weather Service collects precipitation data and publishes the results in the documents listed in Table 2-1. The majority of the information is presented as isohyetal lines on geographic maps of the conterminous United States with separate studies having been made for Hawaii, Alaska, Puerto Rico and the Virgin Islands. The technical publications, under subheadings A and B, give the precipitation to be expected within certain durations and return periods. A total rainfall amount in inches for a specific duration and for a specific recurrence interval, is given on each of the published maps. This presents the rainfall data required in peak discharge methods such as the Rational Formula.

Intensity-duration relationships can be presented as either a rainfall hyetograph or as an accumulated rainfall mass curve. Fig. 2-3 sketches such hypothetical precipitation curves. Neither of these can be obtained from the usually available precipitation data but require that the original gaugings (with sequential measurements at relatively short time intervals) be available to develop the constantly changing hyetograph. For no significantly long period of time does the duration of a certain intensity of rainfall persist before it becomes either greater or lesser. For practical purposes it often is useful to represent the temporal pattern of a rainfall event as a bar graph with each short interval assuming an average rainfall consistent with the continuous hyetograph (see Fig. 2-3). A pattern of distribution of intensities during a storm is of practical importance where design in the storm management process must consider storage or pumping.

Of most practical interest for urban highway drainage are the data in the publications listed under "A" in Table 2-1. Technical Publication No. 40 gives the inches of rainfall for durations of 30 minutes, 1, 2, 3, 6, 12 and 24 hours for frequencies of recurrence of 2, 5, 10, 25, 50 and 100

National Weather Service Publications* - Precipitation Data

A. <u>Durations to 1 day and return periods to 100 years</u>

NOAA Technical Memorandum NWS HYDRO-35 "5 to 60-Minute Precipitation Frequency for Eastern and Central United States", 1977

Technical Paper 40. 48 contiguous states (1961)
(Use for 37 contiguous states east of the 105th meridian for durations of 2 to 24 hours. Use NOAA NWS HYDRO-35 for durations of 1 hour or less.)

Technical Paper 42. Puerto Rico and Virgin Islands (1961)
Technical Paper 43. Hawaii (1962)
Technical Paper 47. Alaska (1963)

NOAA Atlas 2. Precipitation Atlas of the Western United States (1973)

Vol. I, Montana	Vol. II, Wyoming	Vol. III, Colorado
Vol. IV, New Mexico	Vol. V, Idaho	Vol. VI, Utah
Vol. VII, Nevada	Vol. VIII, Arizona	Vol. IX, Washington
Vol. X, Oregon	Vol. XI, California	

B. <u>Durations from 2 to 10 days and return periods to 100 years</u>

Technical Paper 49. 48 contiguous states (1964)
(Use SCS West Technical Service Center Technical Note - Hydrology - PO-6 Rev. 1973, for states covered by NOAA Atlas 2.)

Technical Paper 51. Hawaii (1965)
Technical Paper 52. Alaska (1965)
Technical Paper 53. Puerto Rico and Virgin Islands (1965)

C. <u>Probable maximum precipitation (see Fig. 2-13)</u>

Hydrometeorological Report 33. States east of the 105th meridian (1956)
(Use Fig. 4-12, NWS map for 6-hour PMP (1975). This map replaces ES-1020 and PMP maps in TP-40** which are based on HM Report 33 and TP-38.)

Hydrometeorological Report 36. California (1961)

Hydrometeorological Report 39. Hawaii (1963)
(PMP maps in TP-43** are based on HM Report 39)

Hydrometeorological Report 43. Northwest States (1966)

Technical Paper 38. States west of the 105th meridian (1960)
Technical Paper 42** Puerto Rico and Virgin Islands (1961)
Technical Paper 47** Alaska (1963)

Unpublished Reports:
*** Thunderstorms, Southwest States (1972)
Upper Rio Grande Basin, New Mexico, Colorado (1967)

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* National Weather Service (NWS), National Oceanic and Atmospheric Administration (NOAA), U.S. Department of Commerce, formerly U.S. Weather Bureau.

** Technical papers listed in both A and C

Being replaced by Hydrometeorological Report No. 51 "Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 20,000 Square Miles and Durations from 6 to 72 Hours", available end of 1977.

*** Being replaced by Hydrometeorological Report No. 49 "Probable Maximum Precipitation, Colorado and Great Basin Drainages".

<u>TABLE 2-1</u>

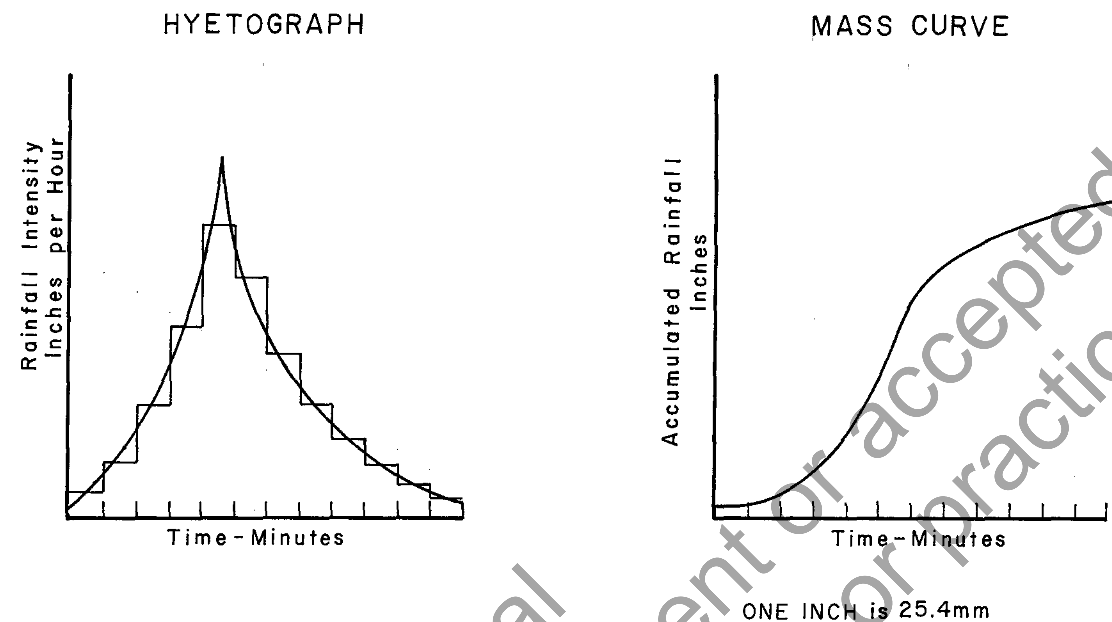


FIG. 2-3 Hypothetical precipitation curves.

years. For example, Figs. 2-4, 2-5 and 2-6 show the 10-year 1-hour, 10-year 24-hour and 100-year 1-hour rainfall as given by lines of equal depth drawn on maps of the continental United States. Note that each of the states in the part of the United States east of about the 105° meridian has county lines and the principal parallels of latitude and longitude. It is relatively easy to locate geographically any specific urban area of design interest. Then, it is possible to read off of the maps, the values for any specific frequency and sequence of durations and plot a duration-intensity-frequency graph. The entire family of such curves for the various frequencies is readily and quickly obtained for any location. This procedure is so simple that it is preferable to utilizing the closest first order station record and then assuming the particular location under consideration has identical intensity-duration-frequency values.

A first order station of the National Weather Service collects continuous records of precipitation, temperature, humidity, wind direction and velocity, and other meteorological data. These data are published by the National Weather Service. Since 1973 the NOAA Atlas No. 2, with its 11 volumes each covering one of the western states, replaces for those states any information given with respect to them in NWS TP No. 40. Technical Memorandum NWS HYDRO-35 was published in June 1977: "Five to Sixty-Minute Precipitation and Frequency for Eastern and Central United States". The

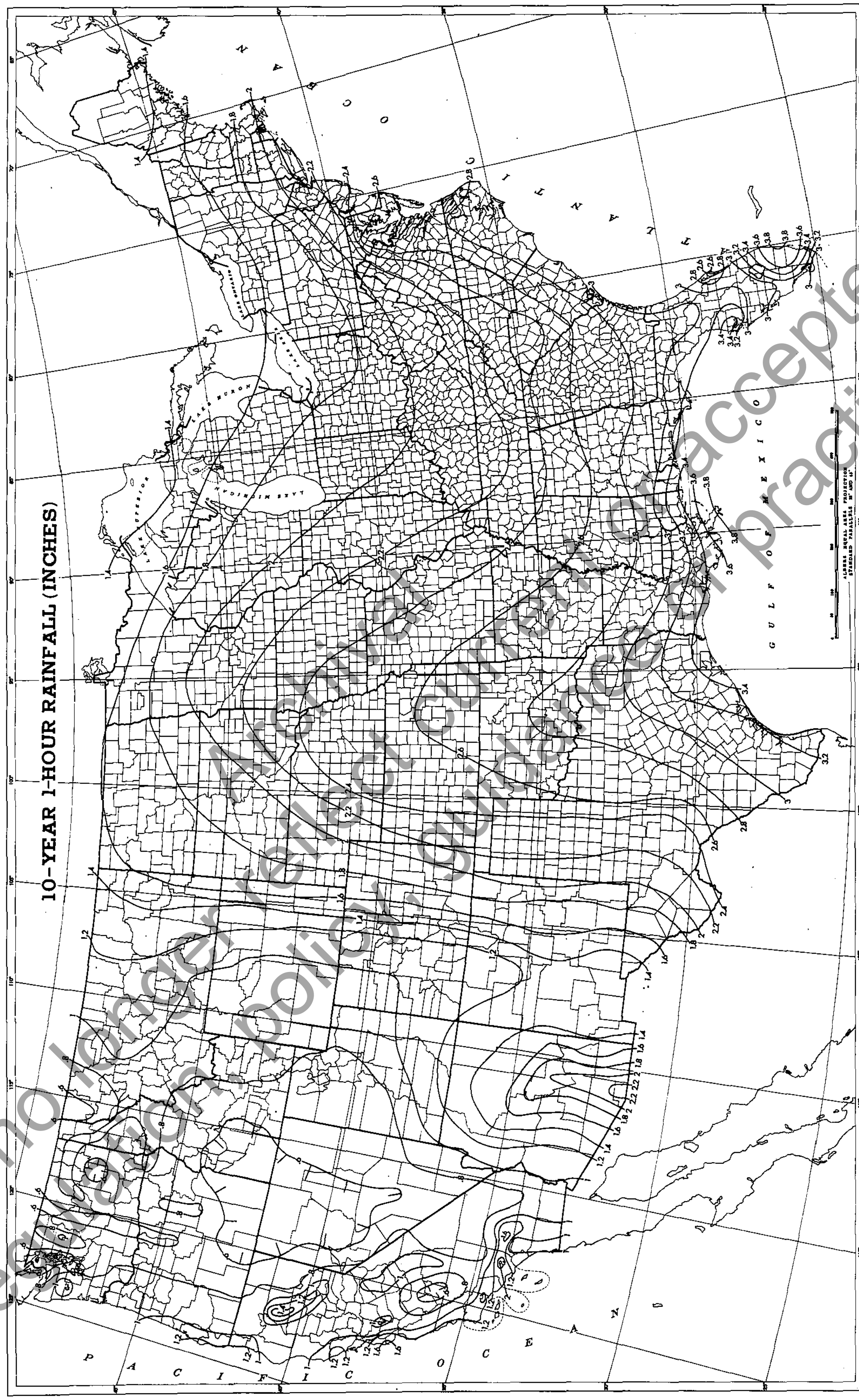


FIG. 2-4 *(After National Weather Service).*

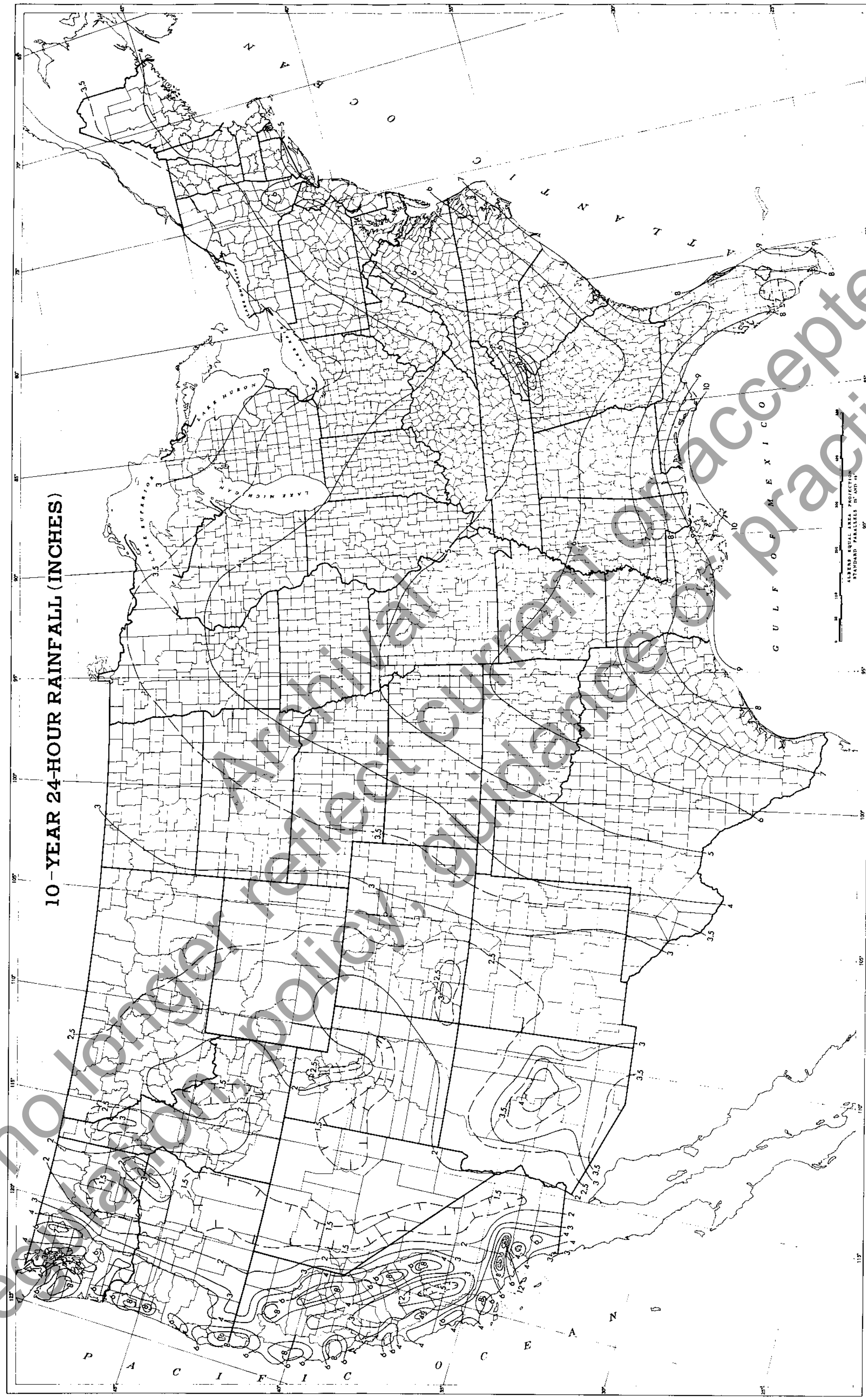


FIG. 2-5 *(After National Weather Service).*

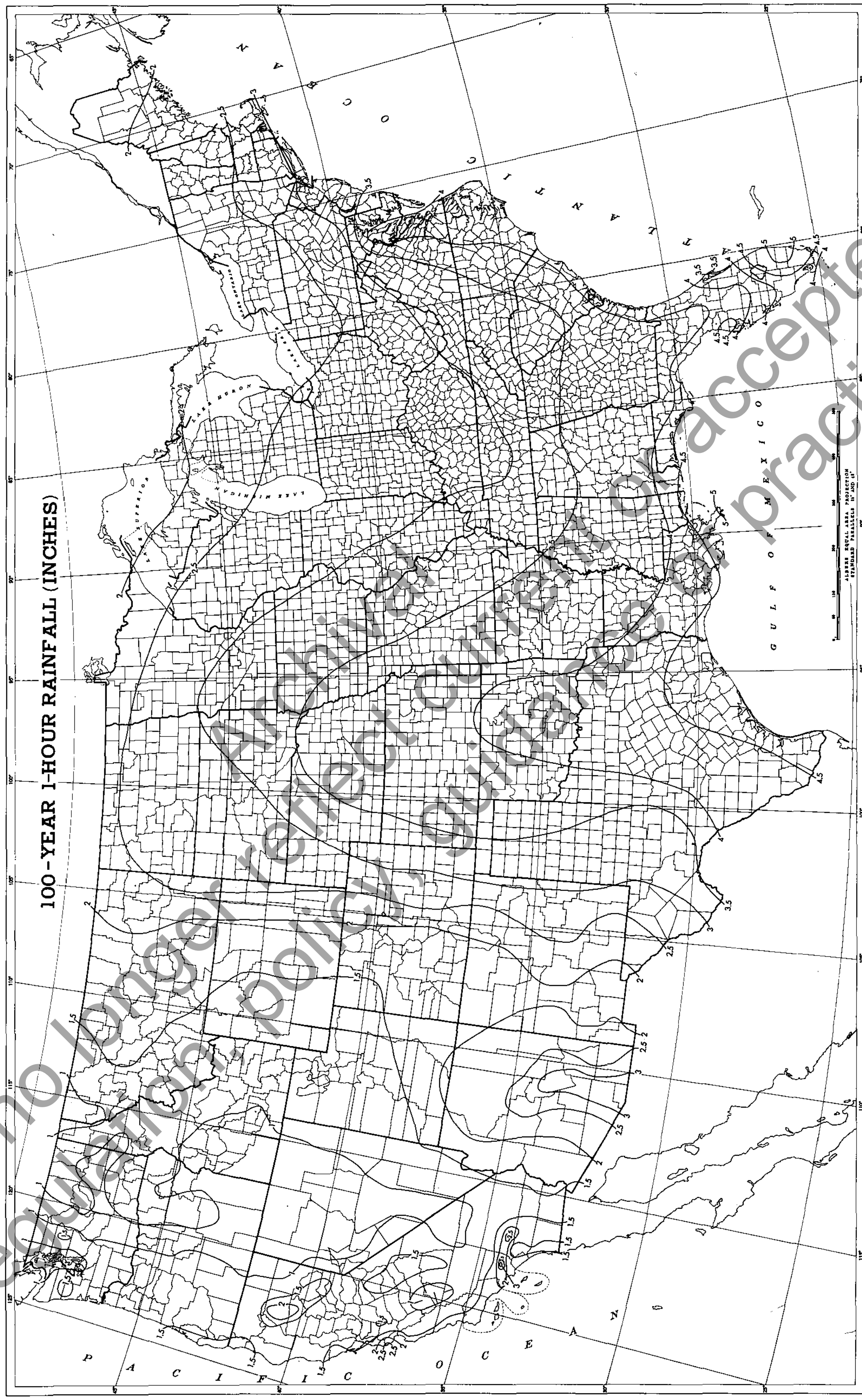


FIG. 2-6 *(After National Weather Service).*

information in TP 40 for use for the eastern and central portions of the United States consequently should be used only for durations greater than one hour.

For the 11 western states, each with some mountainous terrain, individual volumes have been developed for each as part of NOAA Atlas No. 2. This was necessary because of the many areas in those states where orographic influences of the mountains severely affect the precipitation regimes. Each of these volumes has plats of isopluvials (lines of equal rainfall depth) for 6- and 24-hour durations for 2, 5, 10, 25, 50 and 100 years recurrence intervals. Each of the volumes in Atlas No. 2 also has procedures for estimating amounts for durations other than 6- and 24-hour. Such procedures estimate the 1-, 2-, 3- and 12-hour precipitation frequency values and give factors for developing the 5-, 10-, 15- and 30-minute depth values as related to the 1-hour values.

The detailed maps of NOAA Atlas No. 2, showing the variations in rainfall frequency values in the 11 western United States were developed to depict rainfall frequency values for average conditions along orographic barriers and in mountain valleys. At some locations, where the topography departs significantly from average conditions, amounts determined from the generalized chart may possibly be either an under- or over-estimate. For these locations, locally available data could be considered to modify values obtained from the generalized charts. Possible additional data sources are the local National Weather Service office, State Highway Office, State Hydrographer's office, United States Geological Survey, Corps of Engineers, City Engineer's office and local drainage district or utility companies. Unless there is ample evidence that the local data are more applicable than the generalized charts, such locally derived data should not be used.

Similar procedures to that outlined above for the western states can be followed in the eastern part of the United States, Puerto Rico and the Virgin Islands, Hawaii and Alaska, using charts from the appropriate NOAA National Weather Service Technical Papers to obtain values for various durations and frequencies (see subsection 2.4).

2.3 Development of Rainfall Intensity-Duration-Frequency Curves

For engineering purposes in implementing stormwater management, it is essential that it be known for a specific locality, how much rainfall may be anticipated for a specific time period with an anticipated recurrence interval of x years. For example, what total rainfall may be expected at St. Louis over a duration of 15 minutes with an expected recurrence of once in two years (or a 50% chance of occurring in any particular year)? The most recent National Weather Service Technical Memorandum NWS HYDRO-35, reflected on Fig. 2-7, shows that 3.6 inches (91.4mm/hr) per hour or 0.90 inches (22.86mm) of rainfall will fall within 15 minutes at St. Louis once every two years on an average. Such information or comparable data for other durations and frequencies is essential to the current methods of design of storm drainage facilities (see Chapter 3, "Runoff").

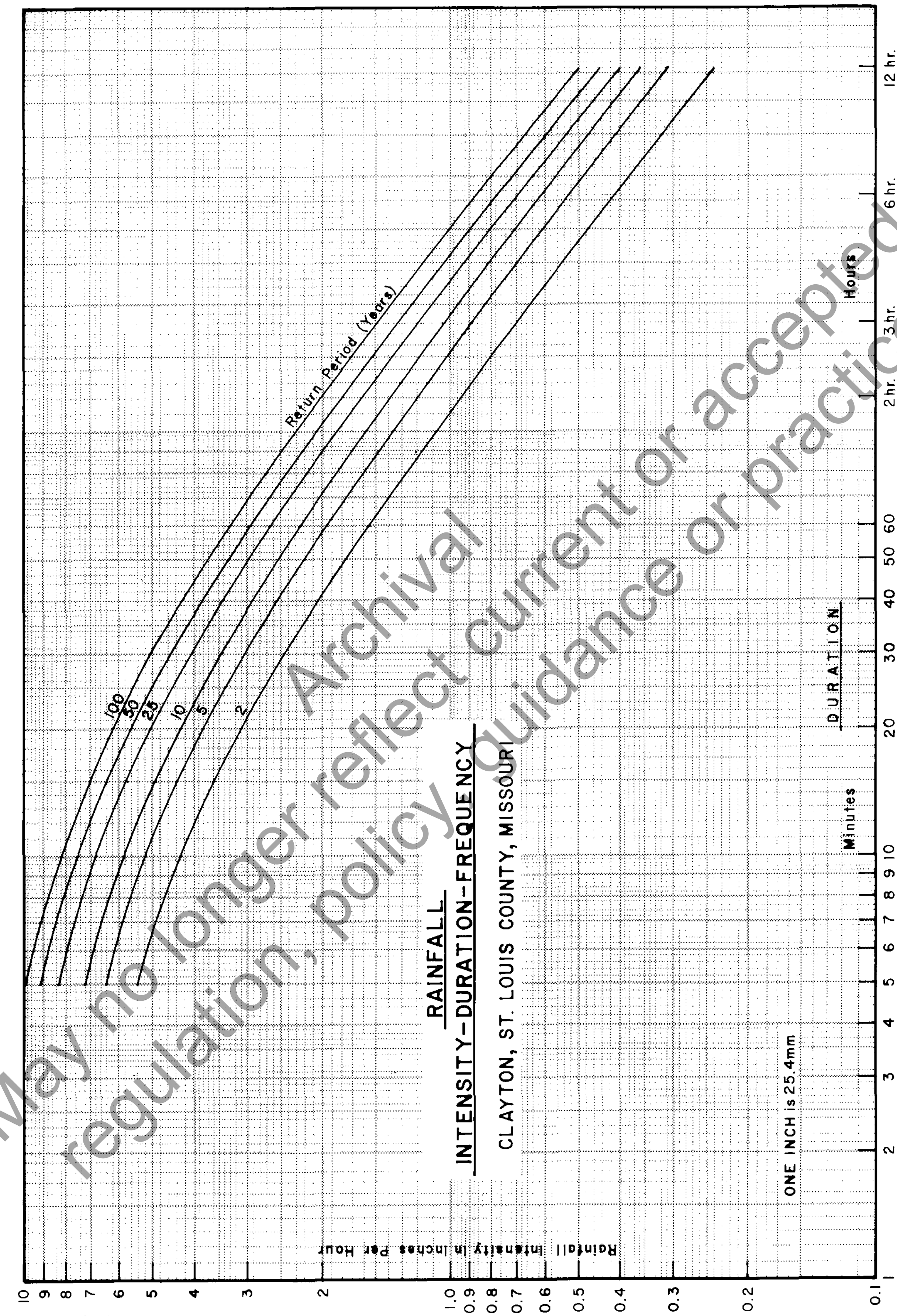

RAINFALL
INTENSITY-DURATION-FREQUENCY
CLAYTON, ST. LOUIS COUNTY, MISSOURI
ONE INCH is 25.4mm
Return Period (Years)
100
50
25
10
5
2
Rainfall Intensity in Inches Per Hour
DURATION
Minutes
Hours
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
2 hr.
3 hr.
6 hr.
12 hr.
10
9
8
7
6
5
4
3
2
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1

FIG. 2-7

As a consequence it is desirable for design purposes to develop rainfall intensity-duration-frequency curves. The manner in which such curves can be developed from available National Weather Service data is illustrated in the two examples which follow:

2.4 Example 2-1: Intensity-Duration-Frequency Relationships - Humid Area Ease of 105th Meridian

The utilization of available National Weather Service NOAA Technical Memorandum NWS HYDRO-35 (Ref. 2-1) and NWS Technical Paper No. 40 (Ref. 2-2) to develop an intensity-duration-frequency curve for a specific location in eastern and central United States is illustrated by the following example:

Given: Location - Clayton, Missouri $38^{\circ}39'N$; $90^{\circ}20'20''W$

Develop I-D-F Curve for 5 minutes to 24 hours, 2 to 100 years frequency

Step 1: From Ref. 2-1, Figs. 4 through 9, obtain the following information: (the figures with an asterisk)

	5 Min.	10 Min.	15 Min.	30 Min.	60 Min.
2-Yr.	0.45*	0.72	0.90*	1.22	1.55*
5-Yr.	0.53	0.86	1.09	1.52	1.96
10-Yr.	0.60	0.97	1.23	1.73	2.25
25-Yr.	0.69	1.13	1.43	2.03	2.66
50-Yr.	0.76	1.25	1.59	2.27	2.98
100-Yr.	0.83*	1.37	1.75*	2.51	3.30*

Figures are in inches. Multiply by 25.4 to obtain mm.

Step 2: Intermediate return period values are calculated using equations 9 through 12 of Ref. 2-1 and are entered encircled in the tabulation under Step 1. The calculation for the 25-year 15-minute value (using equation 11) is as follows: 25-year = 0.669(1.75) + 0.293(.90) = 1.43.

Step 3: For the 10-minute values use the Ref. 2-1 equation: 0.59 (15-min. value) + 0.41 (5-min. value). For the 30-minute values use the Ref. 2-1 equation: 0.49 (60-min. value) + 0.51 (15-min. value). Enter results in rectangles in tabulation under Step 1.

Step 4: From Ref. 2-2 charts 16 through 49 (e.g. Figs. 2-4, 2-5 and 2-6) inclusive, obtain the following information:

	2-Hr.	3-Hr.	6-Hr.	12-Hr.	24-Hr.
2-Yr.	1.92	2.13	2.59	3.06	3.50
5-Yr.	2.40	2.68	3.18	3.72	4.38
10-Yr.	2.75	3.12	3.62	4.33	4.94
25-Yr.	3.15	3.50	4.23	4.82	5.61
50-Yr.	3.50	3.87	4.61	5.42	6.32
100-Yr.	3.92	4.25	5.10	6.00	6.90

Figures are in inches. Multiply by 25.4 to obtain millimetres.

Step 5: Combine tabulations in Steps 1 and 4, converting them to inches per hour in Table 2-2.

Step 6: Plot the rates versus durations for each frequency, resulting in Fig. 2-7.

2.5 Example 2-2: Intensity-Density-Frequency Relationships for the 11 Western States

The use of available NOAA Atlas No. 2 with its 11 volumes, one for each of the western states, to develop a rainfall intensity-duration-frequency curve for a specific location in one of those states will be exemplified by the following:

Given: Location - Santa Fe, New Mexico 35.5°N; 105.9°W

Develop: I-D-F curve for 5 minutes to 24 hours, 2 years to 100 years frequency.

Step 1: From Figs. 19 through 30 of Ref. 2-3, obtain the following (figures with asterisks):

	1-Hr.	2-Hr.	3-Hr.	6-Hr.	12-Hr.	24-Hr.
2-Yr.	0.86⬯	0.98◇	1.07◇	1.22**	1.42*	1.62*
5-Yr.	1.12⬯	1.27◇	1.37◇	1.55	1.77*	2.00
10-Yr.	1.33⬯	1.48◇	1.59◇	1.77	2.01*	2.25
25-Yr.	1.65⬯	1.78◇	1.88◇	2.04	2.33*	2.62
50-Yr.	1.90⬯	2.06◇	2.18◇	2.37	2.64*	2.90
100-Yr.	2.18⬯	2.34◇	2.46◇	2.65**	2.95*	3.25**

(⬯ See Steps 4 and 5)
(◇ See Step 6)
(* See Step 7)

Figures are inches. Multiply by 25.4 to obtain mm.

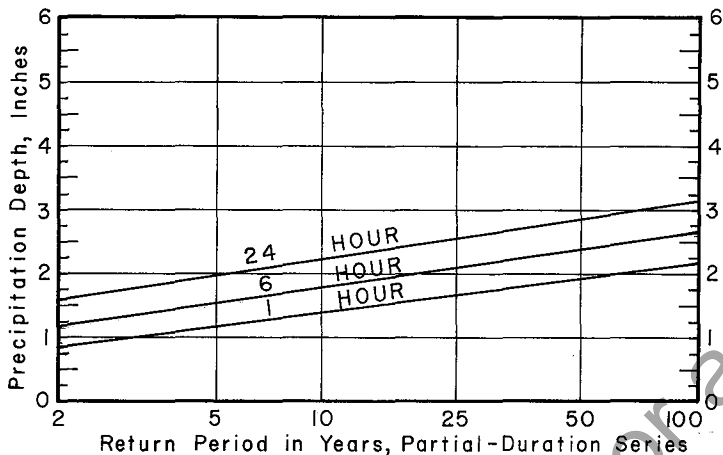
Step 2: Plot the 6-hour and 24-hour values of Step 1 on the nomograph (Fig. 6 of Ref. 2-3) of Fig. 2-8.

PRECIPITATION INTENSITY DURATION FREQUENCY
CLAYTON (ST. LOUIS COUNTY), MISSOURI
90°20'20"W., 38°39'N.

(1) Recurrence Interval	(2) 5-Min iph	(3) 10-Min iph	(4) 15-Min iph	(5) 30-Min iph	(6) 60-Min iph	(7) 2-Hr iph	(8) 3-Hr iph	(9) 6-Hr iph	(10) 12-Hr iph	(11) 24-Hr iph
2-Yr.	5.40	4.32	3.60	2.44	1.55	0.96	0.71	0.43	0.24	0.15
5-Yr.	6.36	5.16	4.36	3.04	1.96	1.20	0.89	0.53	0.31	0.18
10-Yr.	7.20	5.82	4.92	3.46	2.25	1.38	1.04	0.60	0.36	0.21-
25-Yr.	8.28	6.78	5.72	4.06	2.66	1.57	1.17	0.71	0.40	0.23+
50-Yr.	9.12	7.50	6.36	4.54	2.98	1.75	1.29	0.77	0.45	0.26+
100-Yr.	9.96	8.22	7.00	5.02	3.30	1.96	1.42	0.85	0.50	0.29-

One inch is 25.4mm

TABLE 2-2



ONE INCH is 25.4mm

FIG. 2-8 Precipitation depth versus return period for Santa Fe, N. M.
(Grid From National Weather Service).

Step 3: Read the intercept "precipitation depth" for the 5-, 10-, 25- and 50-year return periods. Where these differ noticeably from those tabulated in Step 1, strike the original Step 1 value and insert the intercept figure in its place. These noticeable departures occur because there may be slight registration differences in printing the isopluvial lines on the background printed charts; and precise interpolation between values is difficult.

Step 4: The isopluvial lines in all volumes of NOAA Atlas No. 2 are for 6-hour and 24-hour durations. Values for other durations can be estimated using the 6- and 24-hour values from the maps and the empirical methods outlined in each volume. The 11 western states were separated into several geographic regions each chosen on the basis of meteorological and climatological homogeneity. They are generally combinations of river basins separated by prominent divides. Two of these regions are partially in New Mexico.

Empirical equations for each of the regions are given in Table 2-3 which is taken from Ref. 2-3. That reference suggests that where a point of interest is within a few miles of a regional boundary "computations be made using equations applicable to each region and that the average of such computations be adopted".

From the tabulation in Step 1: $X_1 = 1.22$; $X_2 = 1.62$; $X_3 = 2.65$; $X_4 = 3.25$; from the isopluvial charts in Ref. 2-3, $Z = 7.6$.

Equations for Estimating 1-hr Values in New Mexico With Statistical Parameters for Each Equation

Region of Applicability*	Equation	Corr. Coeff.	No. of Stations	Mean of Computed Stn. Values (Inches)	Standard Error Of Estimate (Inches)
New Mexico east of generalized crest of Sangre de Cristo Range and Sacramento Mountains (1)	$Y_2 = 0.218+0.709\ [(X_1)(X_1/X_2)]$	0.94	75	1.01	0.074
	$Y_{100} = 1.897+0.439\ [(X_3)(X_3/X_4)] - 0.008Z$	.84	75	2.68	.317
New Mexico west of generalized crest of Sangre de Cristo Range and Sacramento Mountains (2)	$Y_2 = -0.011+0.942\ [(X_1)(X_1/X_2)]$	.96	86	0.72	.085
	$Y_{100} = 0.494+0.755\ [(X_3)(X_3/X_4)]$	.90	85	1.96	.290

*Numbers in parentheses refer to geographic regions shown in figure 18 of Ref. 2-3. See text for more complete description.

List of variables

Y_2 = 2-yr 1-hr estimated value

Y_{100} = 100-yr 1-hr estimated value

X_1 = 2-yr 6-hr value from precipitation-frequency maps

X_2 = 2-yr 24-hr value from precipitation-frequency maps

X_3 = 100-yr 6-hr value from precipitation-frequency maps

X_4 = 100-yr 24-hr value from precipitation-frequency maps

Z = point elevation in hundreds of feet

One inch is 25.4mm
One foot is 0.3048m

TABLE 2-3

(After National Weather Service)

With these data, values for the 2-year 1-hour and 100-year 1-hour rainfalls can be estimated from the equations in Table 2-3. Since Santa Fe is only about 11 miles west of the divide between geographic regions 1 and 2 as defined in Ref. 2-3, these 1-hour rainfalls are computed using each set of formulas for each region and the results are averaged. The computations yield the following:

Region	2-Yr. 1-Hr.	100-Yr. 1-Hr.
1	0.870	2.237
2	0.854	2.125
Avg.	0.86	2.18

Figures are inches. Multiply by 25.4 to obtain mm.

Step 5: Plot the Step 4, 1-hour averages on Fig. 2-8, connect the points with a straight line and read off the intercepts for 1-hour values for the 5-, 10-, 25- and 50-year recurrence intervals. Enter all 1-hour values in rectangles in the tabulation under Step 1.

Step 6: Ref. 2-3 gives the following equations for the computation of 2- and 3-hour precipitation-frequency estimates:

For region 1 (east): 2-Hr. = 0.342 (6-Hr.) + 0.658 (1-Hr.)
For region 1 (east): 3-Hr. = 0.597 (6-Hr.) + 0.403 (1-Hr.)
For region 2 (west): 2-Hr. = 0.341 (6-Hr.) + 0.659 (1-Hr.)
For region 2 (west): 3-Hr. = 0.569 (6-Hr.) + 0.439 (1-Hr.)

For each frequency, using the 6- and 1-hour figures in the Step 1 tabulation, calculate the 2- and 3-hour estimates and insert the results under Step 1 as encircled figures.

Step 7: The 12-hour precipitation frequency estimates can be made by averaging the 6- and 24-hour figures given in the Step 1 tabulation. Enter the 12-hour estimates with an asterisk.

Step 8: Compute the 5-, 10-, 15- and 30-minute precipitation estimates using the following information from Ref. 2-3:

Duration (min.)	5	10	15	30
Ratio to 1-Hr.	0.29	0.45	0.57	0.79

These ratios are independent of frequency and were adopted in NOAA NWS Atlas No. 2 from Weather Bureau Technical Paper No. 40 (U.S. Weather Bureau 1961) only after investigation demonstrated their applicability to data from the area covered by Atlas No. 2.

Step 9: Convert the information tabulated under Step 1 and that developed in Step 8 to rates in inches per hour and prepare Table 2-4.

PRECIPITATION INTENSITY-DURATION-FREQUENCY

SANTA FE, NEW MEXICO
35°30'N., 105°55'W.

(1) Recurrence Interval	(2) 5-Min iph	(3) 10-Min iph	(4) 15-Min iph	(5) 30-Min iph	(6) 60-Min iph	(7) 2-Hr iph	(8) 3-Hr iph	(9) 6-Hr iph	(10) 12-Hr iph	(11) 24-Hr iph
2-Yr	3.00	2.34	1.96	1.36	0.86	0.50	0.35	0.20	0.12	0.07
5-Yr	3.84	3.00	2.56	1.76	1.12	0.64	0.45	0.26	0.15	0.08
10-Yr	4.68	3.60	3.04	2.10	1.33	0.74	0.53	0.30	0.17	0.09
25-Yr	5.76	4.44	3.76	2.60	1.65	0.90	0.62	0.34	0.19	0.11
50-Yr	6.60	5.16	4.32	3.00	1.90	1.03	0.72	0.40	0.22	0.12
100-Yr	7.56	5.88	4.96	3.44	2.18	1.17	0.82	0.44	0.25	0.13

One inch is 25.4mm

TABLE 2-4

Step 10: Plot, as Fig. 2-9 on log-log paper, the values in Table 2-4. This gives complete rainfall intensity-duration-frequency data for Santa Fe, New Mexico.

2.6 Areal Variation in Precipitation

Precipitation data and analytical studies such as are reflected by the isopluvial charts of the publications of the National Weather Service (Refs. 2-1, 2-2,2-3) are based upon the fact that the value read for any specific point on a chart is the amount of rainfall for that particular duration which will be equalled or exceeded, on the average, once during the period indicated on the chart. For many engineering problems such as storm drainage, the concern is with the average depth of precipitation over an area and not with the depth at a particular point. Depth-area curves such as Fig. 2-10 were developed to meet this need (Refs. 2-2 and 2-3). These curves represent the geographically fixed-area depth-area relationship where the area of interest is fixed and the storm is displaced so only a portion of the storm affects the area. The other type of depth-area relationship is that in which the storm is centered directly over the area of interest. The average depth-area curves of Fig. 2-10 are for fixed areas and were developed from dense networks used in preparing the National Weather Service Charts.

Note that the curves of Fig. 2-10 bear out the general opinion in the literature (Refs. 2-4, 2-5 and 2-6) that for engineering purposes, point rainfall can be "....a satisfactory index of the frequency distribution of areal rainfall" in a 10 square mile (25.9 Mm^2) area. The Soil Conservation Service (Ref. 2-7) states "No areal adjustments are to be made for areas of less than 10 square miles".

Thus, in addition to climatic and temporal variations in precipitation, for areas of 10 square miles (25.9 Mm^2) or more, there is an areal reduction of statistically processed point rainfall in accordance with the curves of Fig. 2-10. "The data used to develop the area-depth curves exhibited no systematic regional pattern. Duration turned out to be the major parameter for areas up to 400 square miles (1036 Mm^2). It is tentatively accepted that storm magnitude (or return period) is not a parameter in the area-depth relationship. The reliability of this relationship appears to be best for the longer durations" (Ref. 2-2). For highway drainage in urban areas, it rarely will be necessary to consider areas of 10 square miles (25.9 Mm^2) or greater as design parameters.

2.7 Temporal Variation in Precipitation

The growing emphasis on storage in stormwater drainage makes it increasingly important to develop hydrographs of inflow to the detention facility. This, in turn, requires the use of actual or synthetic time distribution of rainfall.

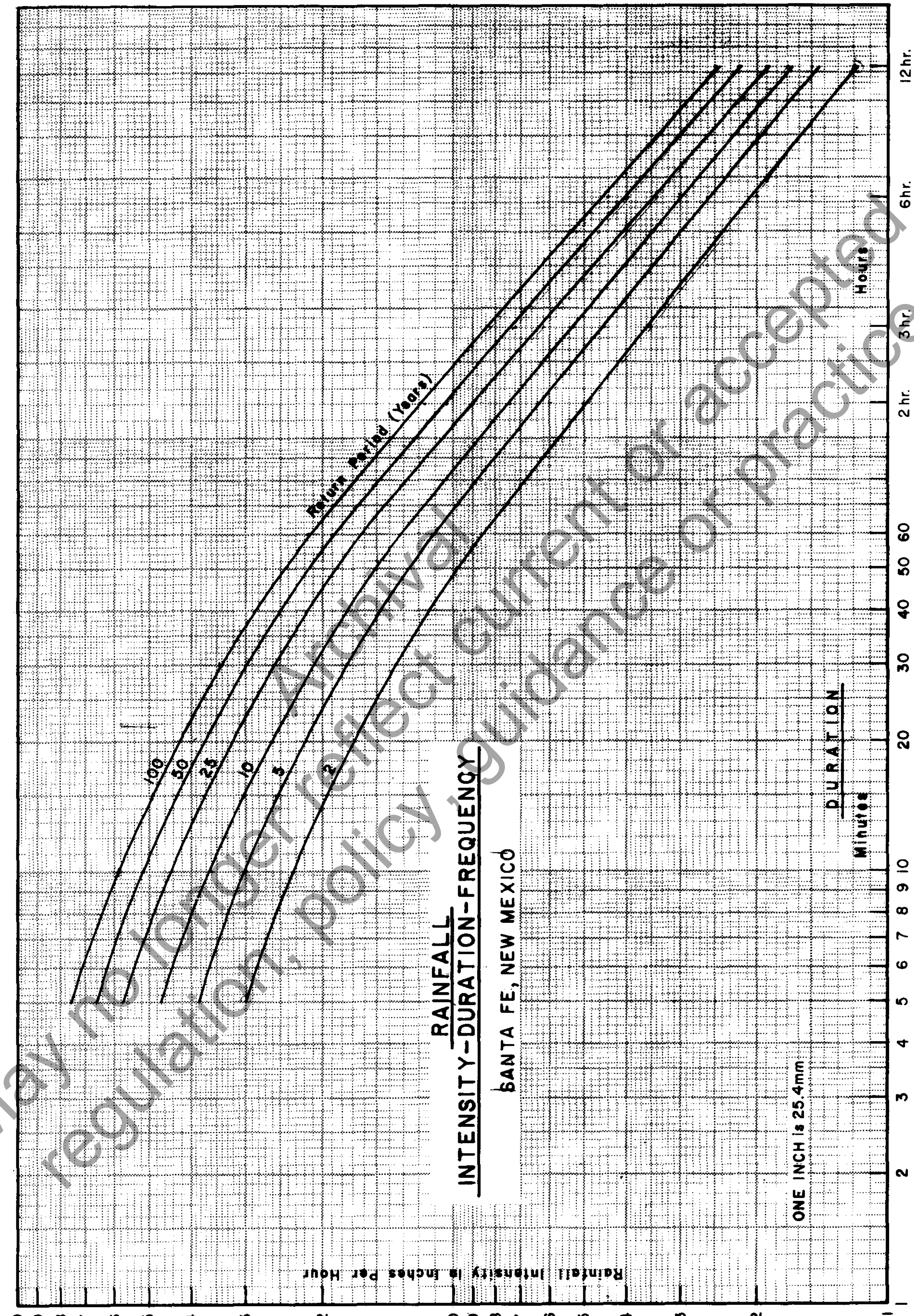

RAINFALL
INTENSITY-DURATION-FREQUENCY
SANTA FE, NEW MEXICO
Return Period (Years)
100
50
25
10
5
2
Rainfall Intensity in Inches Per Hour
10 9 8 7 6 5 4 3 2 1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3 0.2 0.1
DURATION
Minutes
Hours
1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 2 hr. 3 hr. 6 hr. 12 hr.
ONE INCH is 25.4mm

FIG. 2-9

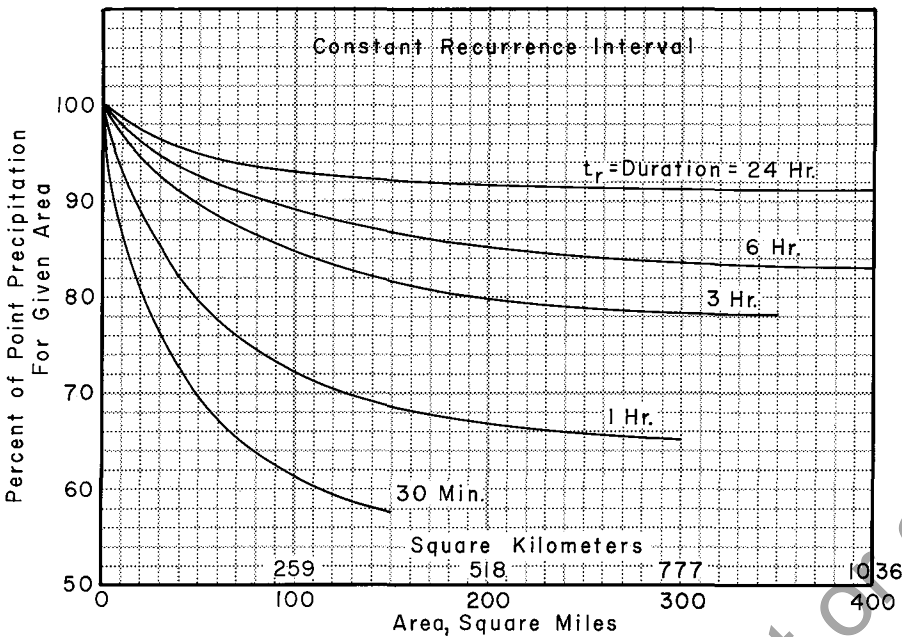


FIG. 2–10 Area-depth curves for use with duration-frequency values. *(After National Weather Service).*

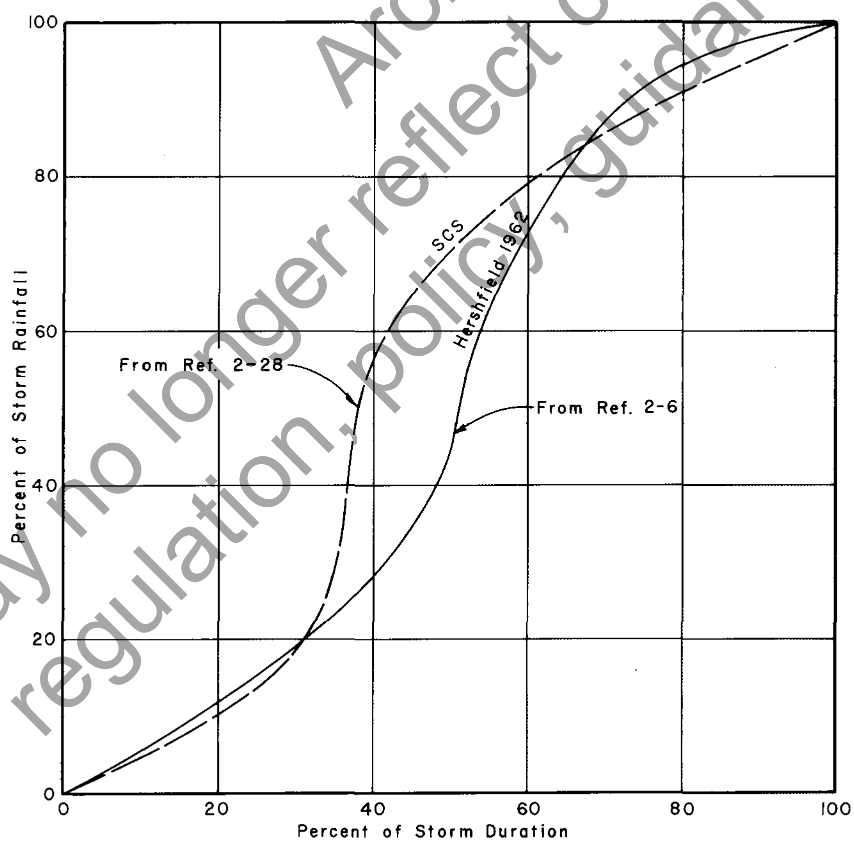


FIG. 2–11 Time Distribution of Rainfall.

Ideally, a continuous rainfall record of about 20 years (representative of the area under study) should be processed and the significant storm events studied to develop, insofar as is practicable, the general time-distribution of the precipitation in a single event of a known frequency.

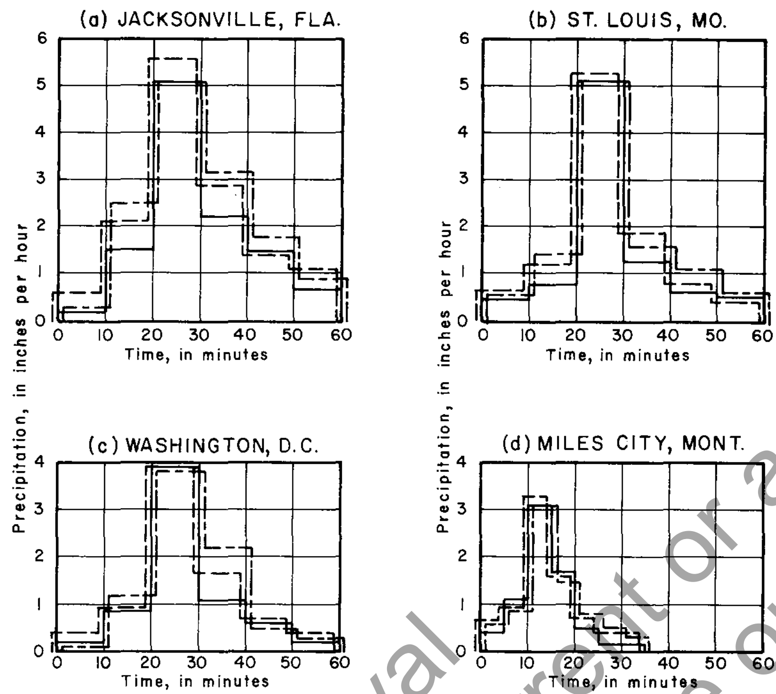
Hershfield (Ref. 2-6) examined a total of 400 storms from 50 widely separated stations with different rainfall regimes. The storm data were extracted from hourly rainfall tabulations. The observed storm quantity increments which were kept in chronological order and expressed as a percentage of storm total, were plotted against percentage of storm duration. The empirical results emphasized the extremely wide range of variability resulting from the random elements associated with the many storms. This erratic incidence of rainfall is the important factor that complicates the relationship of rainfall quantity with time. Hershfield prepared an average curve for each of four durations, 6-hour, 12-hour, 18-hour and 24-hour. Since each curve showed approximately the same average relationship, they were combined into one as shown in the average curve of Fig. 2-11, marked "Hershfield 1962". Also given on this same figure is the "Six-Hour Design Storm Distribution" curve used by the Soil Conservation Service (Ref. 2-7).

The design storm concept postulates a rainfall pattern presumed to reflect a single storm event with an assumed frequency of recurrence interval. Several studies (e.g. Refs. 2-6, 2-8 and 2-9) indicate clearly the theoretical unreality of this. The great variability in individual storms is indicated by historical mass rainfall curves. The scatter of the mass curves was so wide that no typical chronological patterns were evident. No doubt the random variation in the time patterns results from the fact that very heavy rainfalls are generally associated with highly turbulent unstable air movements.

The preferable alternative to assuming a synthetic time-intensity rainfall pattern is to analyze 20 years or more of continuous rainfall records on a complex model. As a practical modification of such an approach, such a long historical "record should be applied to a calibrated catchment near the reference weather station to segregate those storms of design interest. Because only the unusual occurrences are of design interest, there may be perhaps only two dozen or so actual storms of concern" (Ref. 2-9). The severe limitation placed upon this suggestion for a practical modification of the ideal approach is the almost complete lack of time-related rainfall-runoff data in urban areas needed to calibrate a representative catchment.

Some engineers have formulated storm patterns on the basis of more or less arbitrary temporal distributions of intensities assumed symmetrical in time or in some fashion that seems reasonable (Refs. 2-10, 2-11, 2-12, 2-13)(See Fig. 2-12).

A second approach derives storm patterns from the rainfall intensity-duration-frequency relationships on the premise that thereby, there are



ONE INCH is 25.4mm

———————— Pattern developed by Jens method.

— - — - — - Synthetic pattern developed from local intensity curve.

— — — — — Synthetic pattern developed from standard intensity curve.

FIG. 2-12 Comparison of rainfall patterns developed by various methods for selected localities. *(After H. M. Williams).*

represented a series of average values from a variety of storms rather than a sequence of intensities in a particular burst of intense rainfall (Refs. 2-14 thru 2-20, inclusive).

A third approach develops average storm patterns from complete storms rather than from intense bursts of individual rainfall and is based upon observed historical rain gage records (Refs. 2-6, 2-8, 2-21 thru 2-28, inclusive).

For long duration storms a combination of the second and third approaches has been developed using the rainfall intensity-duration-frequency relationships (Refs. 2-29, 2-30).

The fourth approach to the problem formulates a statistical model to generate a sequence of short period rainfall (Ref. 2-31).

Usually, the design of urban highway drainage facilities involves small drainage areas (less than 500 acres=202.3 hectares) for which there most often are no reliable records of single bursts of intense rain for a specific location. Without resorting to historical intense rainfall records, the second approach appears to be the most practical way to formulate temporal storm patterns.

While it is true that storm patterns developed from the second approach in no manner represent the characteristics of complete storms of long duration, it is justified for the small drainage areas involved in urban highway drainage to have a design hyetograph which represents an intense burst of short duration as part of a longer duration storm. For lack of a better method presently available in the formulation of design rainfall patterns for such small watersheds, the second approach will be generalized by using a unified time-coordinate system to describe a temporal pattern before and after the peak of a storm. It is equivalent to assuming that a synthetic storm pattern for a small subwatershed of an urban highway drainage system is a relatively short single-burst pattern in a longer rainfall with a duration of rainfall equal to the time of concentration for an entire storm drain system which serves a larger collection system.

2.8 Basic Equation of Rainfall Intensity-Duration-Frequency

The basic equation to be used represents the rainfall intensity-duration-frequency relationship formulated by using the data in the National Weather Service publications. That basic equation is expressed as

$$r_{av} = \frac{K\,a}{(t_d \pm b)^c} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2\text{-}1)$$

r_{av} = average rainfall intensity in inches per hour or metres per hour

t_d = duration of rainfall in minutes

a,b,c = constants based on precipitation data in English units

K = one (1) for English units; K = .0254 for metric units.

These constants can be obtained for any specific location from the precipitation data in Refs. 2-1 and 2-2 and associated National Weather Service Technical Papers (see Table 2-1). The method for obtaining the constants from precipitation data is extended and laborious but Chen (Ref. 2-20) has developed a relatively simple method of obtaining these parameters which so determined are judged to be unique and adequate for each location studied.

Dr. Chen found that the parameter b in the rainfall intensity-duration-frequency formula may be either positive or negative. He states "A preliminary analysis of rainfall data obtained from Weather Bureau Technical Papers Nos. 25 and 40, has indicated that a positive b mainly applies to a large section of the country--perhaps to the portion east of the Rocky Mountains--while a negative b generally applies to west of the Rocky Mountains. However, in some special meteorological areas such as Hawaii (Table 2-1 and Ref. 2-32), the value of b was found to be, or almost zero". In light of the variety of the b value to be found in nature, the intensity-duration-frequency formula should use the ± sign ahead of

$$b \left[r_{av} = a/(t_d \pm b)^c \right]$$

Appendix A2 explains Chen's method to develop the coefficients a, b and c in detail and illustrates it in a step-by-step example.

2.9 The Skewness or Gamma Value Determination for a Synthetic Storm Pattern

The time position of the most intense bursts of precipitation in a storm event is of importance. Do these intense bursts occur in the initial quarter or second, third or fourth quarters of a storm? Clearly, it is of importance since the largest part of the abstractions (depression storage and infiltration) occurs in the earlier portions of storms. The location of the intense part of a storm has been termed its skewness - if the peak is exactly at the midpoint, the pattern can be virtually symmetrical; if the intense parts of the storm are in the initial part, it can be termed an "advanced" storm pattern; if in the latter part, it would be a "delayed" pattern, etc. The symbol γ (gamma) is used to indicate the skewness as reflected in the formulas for a synthetic storm. A completely advanced storm pattern (the intense part of the storm at its beginning) is indicated by a gamma value of 0; a completely delayed pattern by gamma equal to 1; and intermediate positions of the most intensive part of the storm by gamma values between 0 and 1.

The position of the most intense burst is assumed based upon study of the actual storms of a specific locality. A method for determining the skewness or gamma value of a storm pattern was proposed initially by Keifer and Chu (Ref. 2-16) and has been adopted by several investigators since. This was entirely based on antecedent rainfall records of arbitrarily specified durations of 15, 30 and 60 minutes, etc. up to t_c, the time of concentration. The gamma value obtained for each specified duration is weighted in proportion to the amount of antecedent rainfall preceding that duration so that a weighted average value of gamma is computed. The gamma value so obtained should vary with the a, b and c values used in the rainfall intensity-duration-frequency formula as well as the t_c value found in the drainage area under study. This method appears to be an acceptably usable technique.

Analytical studies (Ref. 2-20) show that the gamma value is no longer of importance when considering very small drainage areas with very short times of concentration. Referring to equation 2-1, the gamma value is of decreasing importance as t_d approaches zero and as c approaches 0. The inverse is likewise true: when the drainage area and hence t_c become larger and longer, the position of the peak in the hyetograph (i.e. the gamma value) becomes more important.

2.10 Hyetograph Equations for Synthetic Storms

Chen's presentation of the hyetograph equations for a positive b and his equations for a negative b follow (Ref. 2-20):

"Hyetograph equations for positive b

Three types of storm patterns are specified by using the different values of γ. A completely advanced (initial burst) type storm pattern has $\gamma = 0$ and a completely delayed (final burst) type storm pattern, $\gamma = 1$. Both types which seldom occur in nature may be regarded as extreme cases of the third type, namely an intermediate type storm pattern, which has $0 < \gamma < 1$.

For $0 < \gamma < 1$

$$r = \frac{a[(1-c)(t_d - t/\gamma) + b]}{[(t_d - t/\gamma) + b]^{1+c}} \qquad 0 \leq t \leq \gamma t_d \quad \ldots\ldots\ldots\ldots\ldots(2\text{-}2)$$

$$r = \frac{a[(1-c)(t - \gamma t_d)/(1-\gamma) + b]}{[(t - \gamma t_d)/(1-\gamma) + b]^{1+c}} \qquad \gamma t_d \leq t \leq t_d \quad \ldots\ldots\ldots(2\text{-}3)$$

"Hyetograph equations for negative b

In this case, the value of c cannot exceed unity. Moreover, because of the nature of Eq2-1, a small portion of hyetograph for all three types must be given a constant intensity, $(a/b^c)\,[(1-c)/(1+c)]^c$ in order to avoid the breakdown when $t \leq b$. Hyetograph equations for the three types are derived and listed as follows:

(1) For $\gamma = 0$

$$r = \frac{a}{b^c}\left(\frac{1-c}{1+c}\right)^c \; ; \quad t \leq \frac{2b}{1-c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2\text{-}4)$$

$$r = \frac{a\,[(1-c)\,t - b]}{(t-b)^{1+c}} \; ; \quad t \leq \frac{2b}{1-c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(2\text{-}5)$$

(2) For $\gamma = 1$

$$r = \frac{a\,[(1-c)(t_d - t) - b]}{[(t_d - t) - b]^{1+c}} \; ; \quad 0 \leq t \leq t_d - \frac{2b}{1-c} \quad \ldots\ldots\ldots(2\text{-}6)$$

$$r = \frac{a}{b^c}\left(\frac{1-c}{1+c}\right)^c \; ; \quad t_d - \frac{2b}{1-c} \leq t \leq t_d \quad \ldots\ldots\ldots\ldots\ldots\ldots(2\text{-}7)$$

(3) For $0 < \gamma < 1$

$$r = \frac{a\,[(1-c)(t_d - t/\gamma) - b]}{[(t_d - t/\gamma) - b]^{1+c}} \; ; \quad 0 \leq t \leq \gamma\, t_d - \frac{2b\gamma}{1-c} \quad \ldots(2\text{-}8)$$

$$r = \frac{a}{b^c}\left(\frac{1 - c}{1 + c}\right)^c \quad ; \quad \gamma t_d - \frac{2b\gamma}{1 - c} \leq t \leq \gamma t_d + \frac{2b(1 - \gamma)}{1 - c} \quad \ldots\ldots (2\text{-}9)$$

$$r = \frac{a[(1 - c)(t - \gamma t_d)/(1 - \gamma) - b]}{[(t - \gamma t_d)/(1 - \gamma) - b]^{1 + c}} \quad ; \quad \gamma t_d + \frac{2b(1 - \gamma)}{1 - c} \leq t \leq t_d (2\text{-}10)$$

For examining the validity of Eqs. 2-2 through 2-10, substituting the equation or equations for each case into Eq. 2-11 and performing the integration over the respective integration limits as specified gives exactly $r_{av} t_d$. However, for negative b, if Eq. 2-11 is satisfied, there is an apparent discontinuity in r, for example, at t = 2b/(1 - c) in the case of $\gamma = 0$ with $r = (a/b^c)\,[(1 - c)/(1 + c)]^{1 + c}$ obtained by substituting t = 2b/(1 - c) into Eq. 2-5. For application, the values of the parameters characterizing the hyetograph equations such as a,b,c,t_d and γ need to be evaluated."

$$\int_0^t r\,d\tau = r_{av} t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2\text{-}11)$$

in which r is the rainfall intensity in inches per hour (mm per hour) at any time in the synthetic storm; τ is the integration variable for time; and r_{av} is the average rainfall intensity in inches per hour (mm per hour) and is assumed to be expressible in the form of Eq. 2-1.

The choice of a value for gamma (γ) to be used in the equations 2-2 through 2-10 can be guided by the experience of past investigators such as those listed in Table 2-5. Wherever possible it is recommended that a study be made of the closest hydrologically applicable first order station's precipitation records choosing the major storms over a significant period of record and analyzing them for a possible average gamma value. Where such studies are not feasible, it is suggested that a gamma value of 0.37 to 0.50 be adopted with the lesser value used for the shorter times of concentration (e.g. the smaller watersheds). The advanced type of storm pattern is most likely to occur as short thunderstorms and where conditions of design suggest such will dominate, the gamma value can be reduced somewhat. Only with the strongest supportive local information should gamma be below 0.25. Most practical design methods utilize rainfall based on frequency-duration data which are derived from intense bursts of recorded rainfall rather than from complete storms. Since the temporal patterns discussed herein are also based on the recorded intense bursts, the use of such temporal patterns is both consistent and logical. A typical such synthetic hyetograph is given in Fig. 2-13.

Synthetic rainfall patterns developed as above described, have the following unique characteristic. If any one of the average rainfall intensities

Gamma (γ) Values Determined by Various Investigators

Reference	Investigator	Location	Recommended γ
2-16	Keifer & Chu	Chicago - 83 station rainfalls	0.37
2-19	MacLaren	Winnipeg, Canada - 60 rainfall events	0.31
2-33	Mitci	Montreal, Canada - 22 major storms	0.50
2-6	Hershfield*	400 storms - 50 U.S. Precip. Stas.	0.53
2-28	S.C.S.*	Avg. of many nationwide records	0.37+
2-34	Los Angeles	Avg. of mass rain curves - 5-min. highest intensities at common point	0.56
2-35	Chien & Sarikelle	Cleveland	0.50
2-8	Pilgrim & Cordery	Sydney, Australia	0.50

*See Fig. 2-8

TABLE 2-5

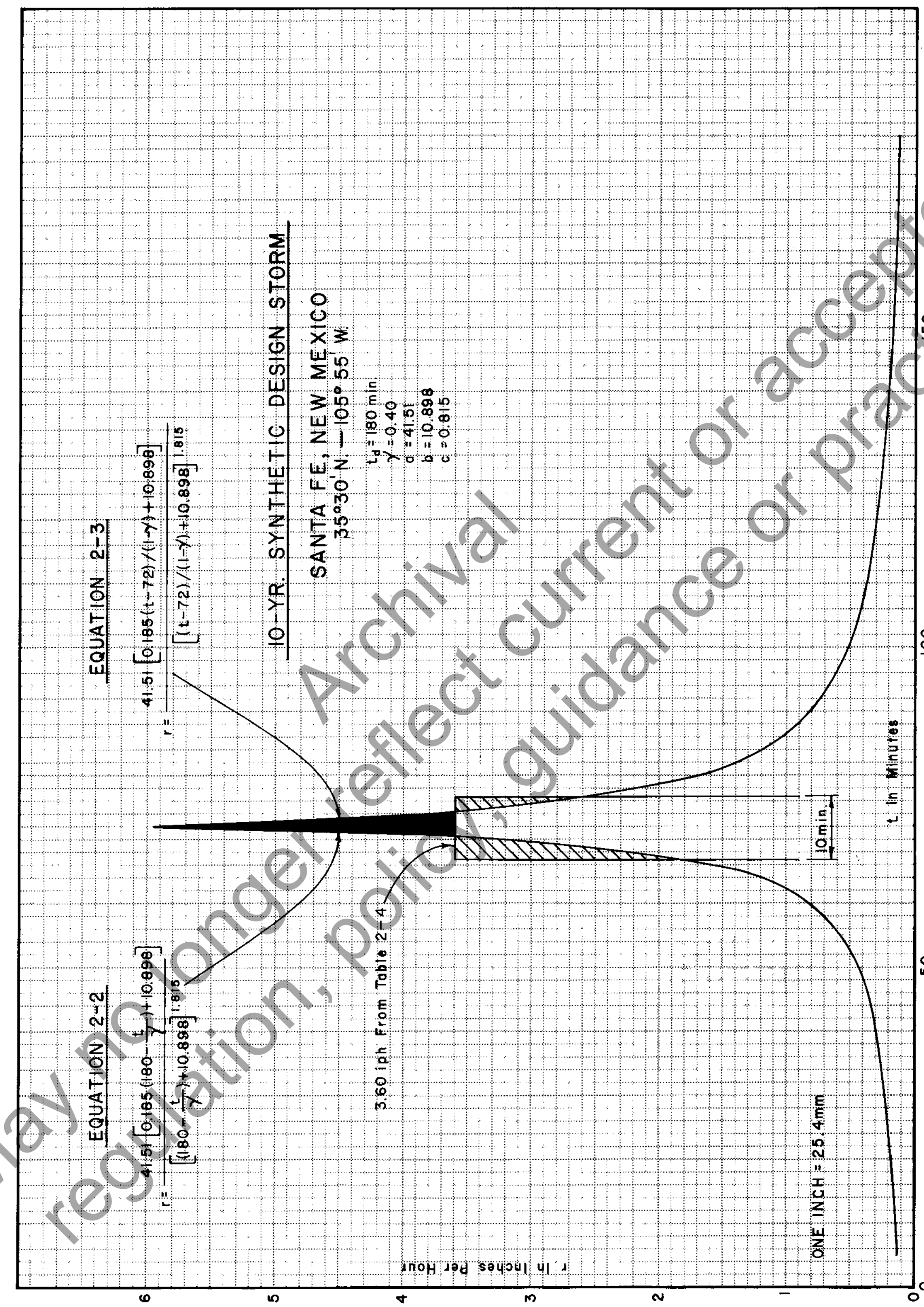

10-YR. SYNTHETIC DESIGN STORM
SANTA FE, NEW MEXICO
35°30' N — 105° 55' W
t_d = 180 min
γ = 0.40
a = 41.51
b = 10.898
c = 0.815
EQUATION 2-2
EQUATION 2-3
3.60 iph From Table 2-4
10 min.
ONE INCH = 25.4mm
t In Minutes
r In Inches Per Hour
0
50
100
150
1
2
3
4
5
6

FIG. 2-13

obtained from development of intensity-duration-frequency data such as are given in Tables 2-2 or 2-4 for a particular duration, is plotted as a uniform intensity centered below the peak of the synthetic curve, the area enveloped by the synthetic curve above the average uniform rate is exactly equal to the areas between the vertical lines denoting the beginning and ending of the uniform intensity and the synthetic curves. For example, on Fig. 2-13, the 10-minute uniform intensity burst with the area above within the synthetic curves filled in solidly and the areas outside the synthetic curves but below the uniform intensity cross-hatched shows that the two hatched areas are exactly equal to the solid area. Such a relationship is true for all durations encompassed by the synthetic curves; in each instance the solid and cross-hatched areas are equivalent.

Recognizing that no such storm probably ever occurs in nature, it appears to be a conservative, practical answer to the need for a temporal rainfall pattern with a reasonable relationship to the frequency desired for the design duration.

It is recommended that for most urban highway storm drainage design where hydrographs are needed, a synthetic storm hyetograph be developed for a total duration of 1 to 3 hours for the frequency desired. If the total critical time (time of concentration) of the entire urban subwatershed is known (of which the highway drainage is a part) that total time should be used. This approach has an added advantage: in many instances, the relatively short duration of rainfall critical for the highway's urban drainage, leaves in the chosen longer duration design hyetograph, additional rainfall which in many instances will be sufficient to satisfy the major initial abstractions, leaving as continuing abstractions only the steady minimum infiltration losses on pervious areas. Since this latter is often quite small, it can frequently be ignored without introducing significant error in the drainage design.

Occasionally, design needs cannot justify the effort necessary to develop a temporal pattern of rainfall in the detailed manner heretofore discussed. For short times of concentration and small areas in urban highway drainage, storm patterns may be formulated on more or less arbitrary temporal distributions of intensities, assumed either symmetrical in time or in some fashion that appears reasonable. Williams (1948), in discussing a paper (Ref. 2-13), indicated in Fig. 2-12 a striking similarity of pattern arrangements for short storms in Jacksonville, Florida, St. Louis, Missouri, Washington, D.C. and Miles City, Montana. Note the reasonable conformance of the relative magnitude of the pattern blocks irrespective of the widely separated geographical locations.

2.11 Probable Maximum Precipitation (PMP)

"Probable maximum precipitation (PMP) for a particular area represents an envelopment of depth-duration-area rainfall relations for all storm types affecting that area adjusted meteorologically to maximum conditions" (Ref. 2-36). It is used to check detention or other storage impoundments, the breaching of which might result in possible loss of life and great property damage. Under such conditions, the spillway hydrograph of outflow from the impoundment should be based upon the PMP. Table 2-1 lists the available NOAA National Weather Service publications containing probable maximum precipitation data. Figures 2-14 and 2-15 give additional guidance to the requisite rainfall data.

For lesser potential hazards and losses than those requiring the use of PMP, the emergency spillway hydrograph should be based upon precipitation data for the 100-year return period (Fig. 2-16) plus some fraction of the difference between the PMP and the precipitation for the 100-year return period.

Wherever an emergency spillway is required, the minimum rainfall for which it should be designed should be that for the 100-year return period. For very small detention storage in locations where an overtopping or breaching could not cause significant losses or damage, an emergency spillway may not be necessary.

2.12 Summary of Significant Design Information in Chapter 2

1. There are available precipitation data that can readily be used to develop intensity-duration-frequency curves for any locality in the United States including Alaska, Hawaii and Puerto Rico. Such rainfall intensity-duration-frequency information is essential in methods of design for the determination of peak runoff rates, e.g. the rational method.

2. A relatively simple method of computing the average rainfall intensities for various durations and frequencies for a specific locality is detailed and illustrated by example. This method requires the data from only three isopluvial charts of the total of 49 charts in U.S. Weather Service Technical Publication No. 40 (for all of the U.S. east of the 105th meridian). See Appendix A2.

3. Where storage or pumping require the development of inflow hydrographs, the time-distribution of rainfall within a storm becomes necessary. Chapter 2 develops equations for the determination of the rising and falling curves of a synthetic hyetograph based upon the available rainfall intensity-duration-frequency data for any chosen frequency. The synthetic storm hyetograph involves a determination of the time-location of the peak rainfall intensity. Preferably, the design storm including the location of its most intense period should be based upon a thorough analysis of about 20 years of historic rainfall. Lack of availability of suitable rainfall records in many locations together with disproportionate design costs as related to the magnitude of the problem to be solved, militate against the preferable approach in most instances. Unless readily determinable from pertinent historic rainfall records, the time-location of the synthetic storm peak should be assumed to be 0.33 to 0.50 of the time from the beginning of rainfall to its cessation.

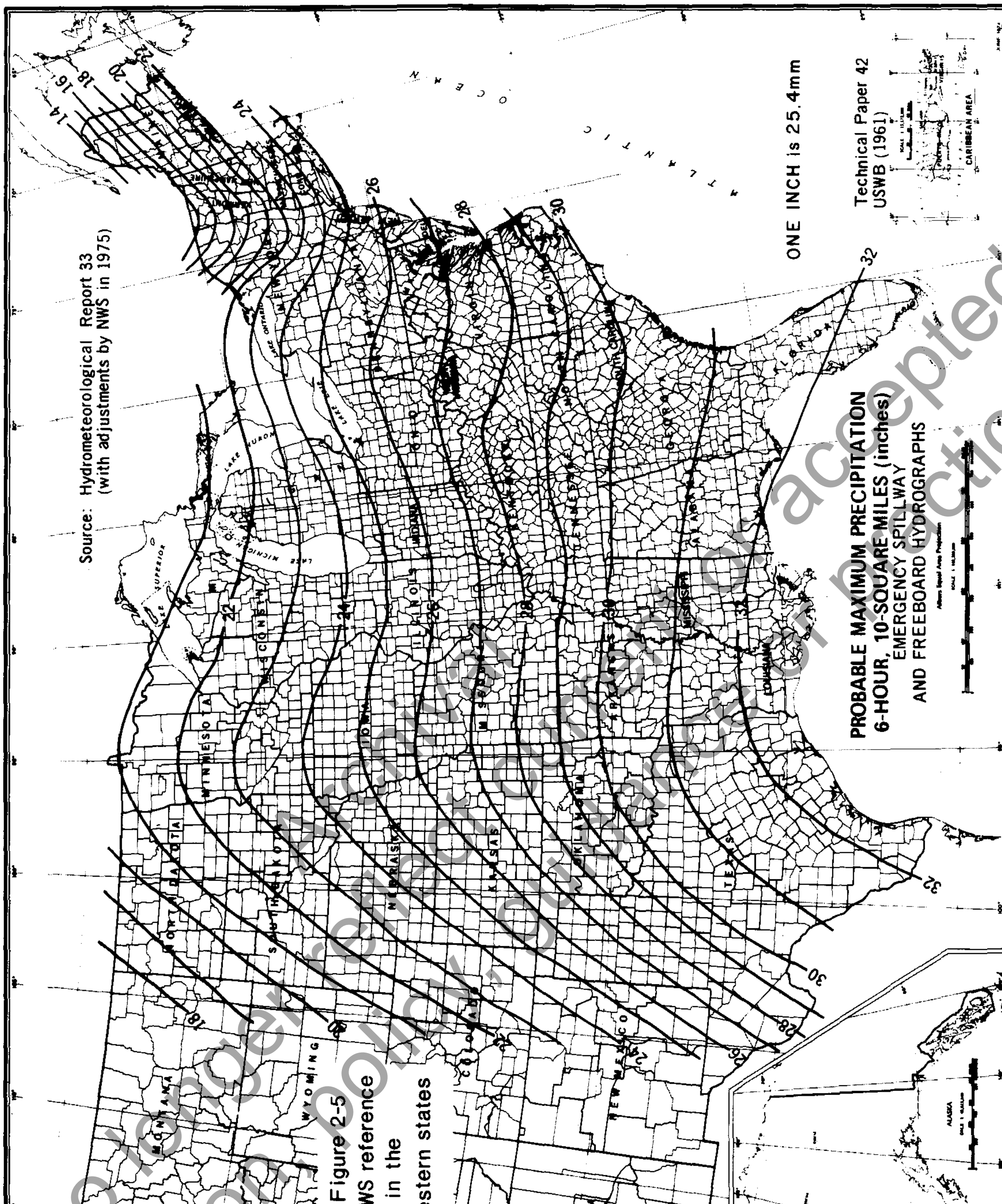


FIG. 2-14 *(After National Weather Service).*

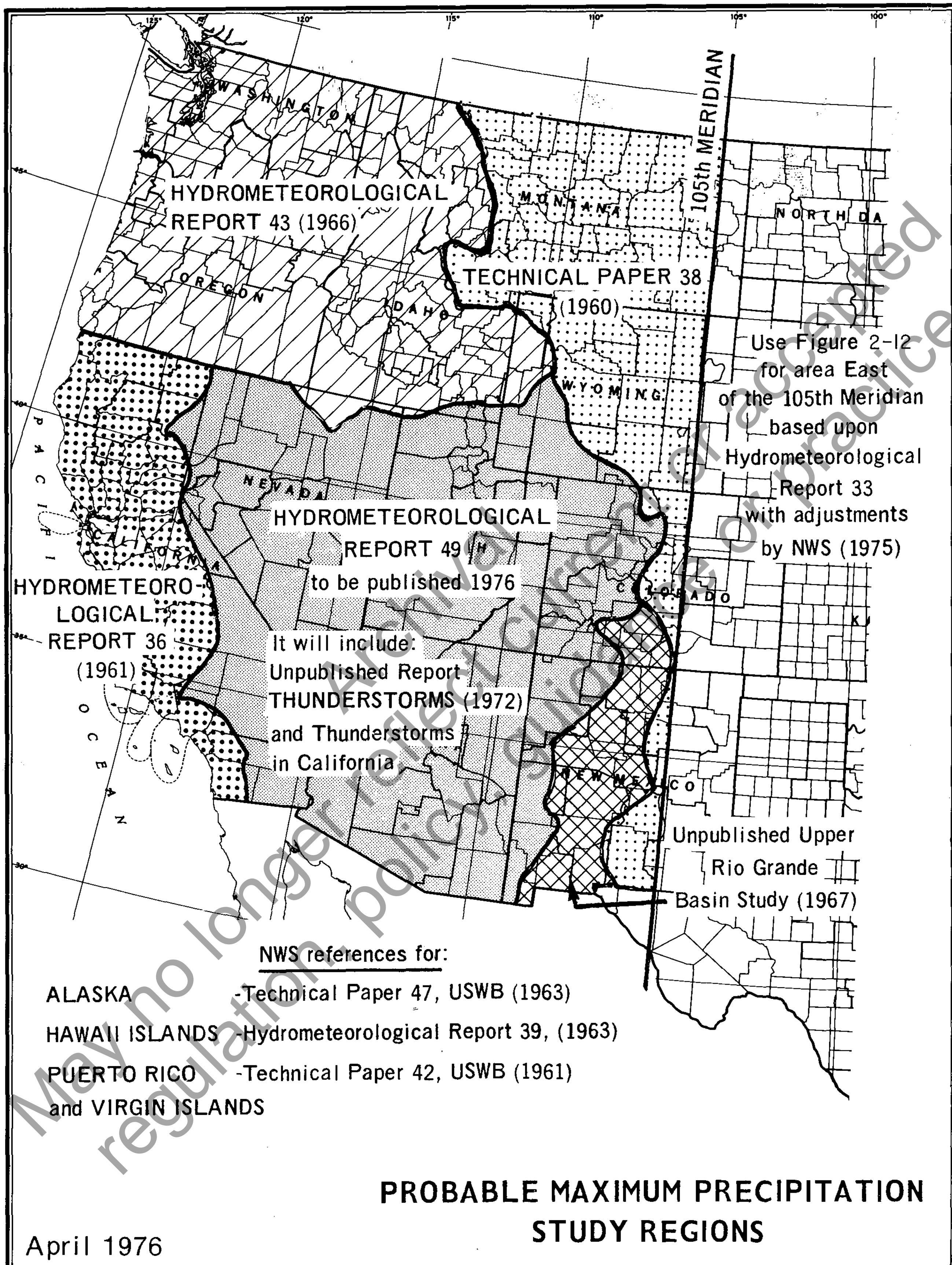


FIG. 2-15 *(After National Weather Service).*

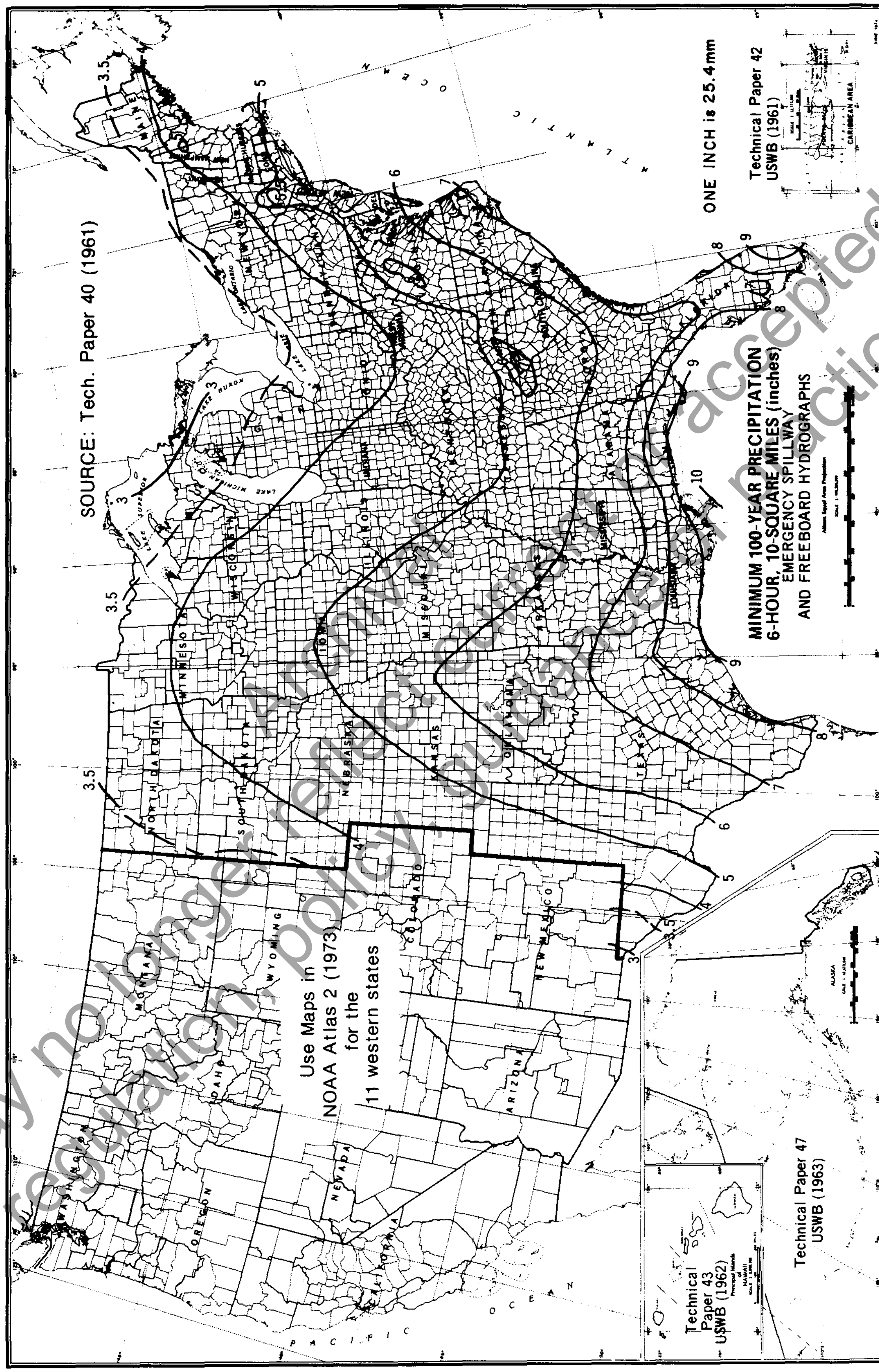


FIG. 2-16 *(After National Weather Service).*

The use of the rainfall data in a specific hydrograph problem is exemplified in Table 3-8 which determines the effective rainfall for a 10-year 2-hour precipitation at Boulder, Colorado. Column 2 lists the rainfall depths at 10-minute intervals (as obtained from "Rainfall Fig. 6--3" of Ref. 2-4). Column 4 rearranges the 10-minute increments with the highest rate placed at the 40-minute point and the other increments in descending amounts grouped either side of the peak rate. This is a judgement decision. The procedure for determining the net or effective rainfall starting with the rearranged incremental gross rainfall is discussed in Chapter 3. The effective rainfall is then used in applying the unit hydrograph to achieve the outflow hydrograph as given in Table 3-9.

If there had not been available a 10-year rainfall intensity-duration-frequency curve (from which to obtain the rainfall depths for column 2 of Table 3-8), such values could be computed from the appropriate equation comparable to equation A2-19 determined in the manner discussed in Appendix A2.

4. For runoff determinations where only the peak is of interest (e.g. the rational method), the rainfall value required (for the frequency and time of concentration involved) can be obtained from a rainfall intensity-duration-frequency curve developed from the isopluvial charts and formulas in the appropriate National Weather Service publication. Or the appropriate equation similar to equation A2-19 can be determined in the manner discussed in Appendix A2.

APPENDIX A2

Precipitation Formula Parameters a,b and c

A2.1 Chen (Ref. 2-20) adopts the general rainfall intensity formula:

$$r_{av} = \frac{Ka}{(t_d \pm b)^c} \quad ....(A2\text{-}1)$$

in which r_{av} = average rainfall intensity in inches per hour (mm per hour)

t_d = time duration of rainfall in minutes

a,b,c = storm parameters for a specific frequency; these depend on meteorological localities.

K = one (1) for English units; equals 25.4 for metric units.

"Because this equation can be expressed in logarithmic form and hence is linear in "log r_{av}" and "log $(t \pm b)$" for a given value of b, the determination of a, b and c values can be accomplished in a systematic manner by using the method of least squares and an optimization technique similar to the method of steepest descent for optimizing an unconstrained problem. The optimization problem formulated herein (Ref. 2-20) is tantamount to the one to find the a, b and c values for minimizing the expression

$$F(a,b,c) = \sum_{j=1}^{n} [\log r_{av}^{j} - \log a + c \log(t_d^j \pm b)]^2 \quad ...(A2\text{-}2)$$

The rainfall intensity-duration-frequency data obtained from Ref. 2-2 can be used for this computation."

To obtain these parameters for a variety of frequencies for a particular locality involves resorting to all 49 charts of Ref. 2-2. Consequently, Chen analyzed the manner in which the charts of Ref. 2-2 were constructed and developed the following less laborious yet satisfactory approach to the determination of standard storm parameters a_1, b_1 and c_1 which describe the ratios of various duration intensities to the 1-hour intensity for the same frequency. The reader is referred to the original Chen development in Ref. 2-20.

To determine a_1, b_1 and c_1 Chen proposes the following:

$$a = a_1 r_{av}^{T,1} \quad \text{(See Eq. A2-9)} \quad ....(A2\text{-}3)$$

$$b = b_1 \quad ....(A2\text{-}4)$$

$$c = c_1 \quad ....(A2\text{-}5)$$

$$r_{av}^{T,t_d} = \frac{a_1 r_{av}^{T,1}}{(t_d + b)^c} \qquad \text{(A2-6)}$$

$$x = \frac{r_{av}^{100,1}}{r_{av}^{10,1}} \qquad \text{(A2-7)}$$

$$r_{av}^{T,t_d} = \frac{a_1 r_{av}^{10,1} \log_{10} (10^{2-x} T^{x-1})}{(t_d + b)^c} \qquad \text{(A2-8)}$$

This equation (A2-8) is the general expression of the rainfall intensity-duration-frequency relationship. To make use of it, there must first be determined for the specific locality, the values of a, b, c and x from three basic isopluvial maps with the help of Fig. 2-17 which Chen prepared as described in Ref. 2-20. Use of Fig. 2-17 requires the ratio of the one-hour to 24-hour rainfall depth for the 10-year frequency. The value of x as expressed by equation (A2-7) is the ratio of the 100-year to 10-year rainfall intensity for 1-hour duration.

Equation A2-3 must be replaced by

$$a = a_1 r_{av}^{10,1} \log_{10} (10^{2-x} T^{x-1}) \qquad \text{(A2-9)}$$

The validity of Chen's shorter method using Figs. 2-4, 2-5, 2-6 and 2-17 plus Equation A2-8 was checked by comparing the rainfall intensities of various durations and frequencies obtained from the shorter method with those obtained from all 49 isopluvial maps of Ref. 2-2.

There follows an example of the use of Chen's shorter method.

The formulation of design storm patterns for New York (40.4°N, 74.0°W) requires the determination of the storm parameters a, b and c.

Step 1: From Figs. 2-4, 2-5 and 2-6 obtain the following:

10-year 1-hour rainfall	2.15 inches
10-year 24-hour rainfall	5.20 inches
100-year 1-hour rainfall	3.11 inches

Ratio 2.15/5.2 = 0.413

Ratio 3.11/2.15 = 1.447 = x

Step 2: From Fig. 2-17 for the ratio 0.413:

$a_1 = 23.9$

$b_1 = 7.85$

$c_1 = 0.75$

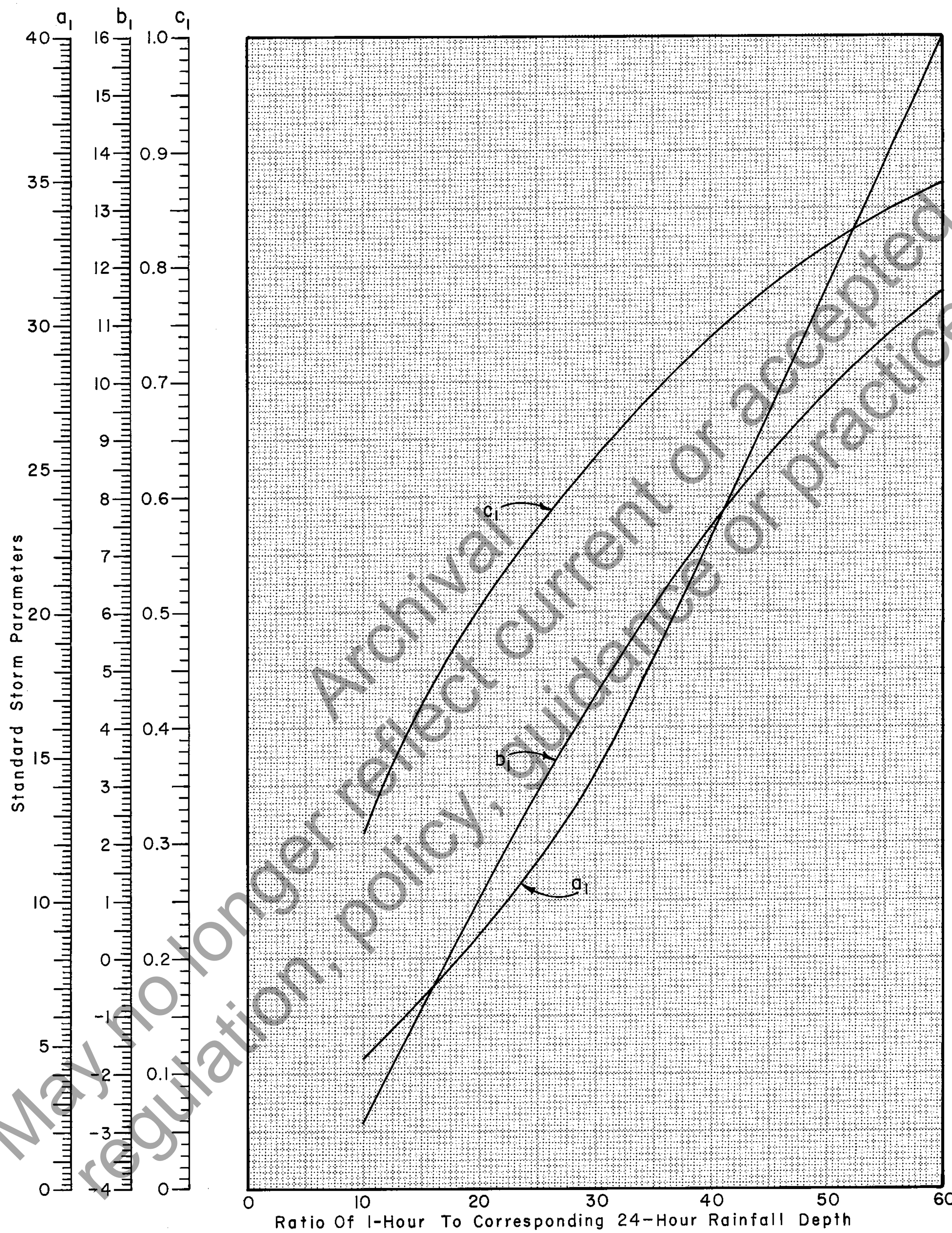


FIG. 2-17 Relationships between standard storm parameters and the ratio of 1-hour to corresponding 24-hour rainfall depth.
(After Chen, Utah State University).

Step 3: Substituting x = 1.447 in equation A2-9:

$$a = (23.9)(2.15)\ \log_{10}\ 10^{0.553}T^{0.447} \qquad \text{.............(A2-10)}$$

$$a = 51.39\ \log_{10}\ 10^{0.553}T^{0.447} \qquad \text{.............(A2-11)}$$

Step 4: For various return periods T(years) compute the corresponding values of a from equation A2-11 and substitute into equation A2-1:

$$r_{av} = \frac{28.42}{(t_d + 7.85)^{0.75}} \qquad T = 1 \qquad \text{..........(A2-12)}$$

$$r_{av} = \frac{35.33}{(t_d + 7.85)^{0.75}} \qquad T = 2 \qquad \text{..........(A2-13)}$$

$$r_{av} = \frac{44.48}{(t_d + 7.85)^{0.75}} \qquad T = 5 \qquad \text{..........(A2-14)}$$

$$r_{av} = \frac{51.39}{(t_d + 7.85)^{0.75}} \qquad T = 10 \qquad \text{..........(A2-15)}$$

$$r_{av} = \frac{60.53}{(t_d + 7.85)^{0.75}} \qquad T = 25 \qquad \text{..........(A2-16)}$$

$$r_{av} = \frac{67.45}{(t_d + 7.85)^{0.75}} \qquad T = 50 \qquad \text{..........(A2-17)}$$

$$r_{av} = \frac{74.36}{(t_d + 7.85)^{0.75}} \qquad T = 100 \qquad \text{..........(A2-18)}$$

Step 5: Using equations A2-12 through A2-18 and durations of 5 minutes to 24 hours, compute the rainfall intensities given at the left-hand side of each column in Table 2-6. The comparable intensities in the right-hand side of each double column are values obtained from the isopluvial charts of Ref. 2-2. A comparison of the intensities obtained from equations A2-12 through A2-18 with those obtained from the 49 isopluvial maps indicates that the former are within the tolerable accuracy.

This comparison leads to the conclusion that equation A2-8 or more specifically for New York City

$$r_{av} = \frac{51.39\ \log_{10}(10^{0.553}T^{0.447})}{(t_d + 7.85)^{0.75}} \qquad \text{.........(A2-19)}$$

Rainfall Intensity in Inches per Hour for Various Durations
and Frequencies at New York

Return Period (Years)

Duration	1 a	1 b	2 a	2 b	5 a	5 b	10 a	10 b	25 a	25 b	50 a	50 b	100 a	100 b
5 min.	4.19	4.08	5.21	5.04	6.55	6.48	7.57	7.56	8.92	8.52	9.94	9.72	10.96	10.68
10 min.	3.27	3.12	4.07	3.84	5.12	4.98	5.92	5.82	6.97	6.60	7.77	7.50	8.56	8.22
15 min.	2.71	2.64	3.38	3.24	4.26	4.20	4.92	4.88	5.79	5.56	6.45	6.32	7.11	6.92
30 min.	1.86	1.82	2.32	2.24	2.91	2.92	3.37	3.40	3.97	3.86	4.42	4.38	4.87	4.80
60 min.	1.20	1.23	1.49	1.43	1.88	1.86	2.17	2.15	2.56	2.48	2.85	2.78	3.15	3.11
2 hrs.	0.75	0.74	0.93	0.89	1.17	1.16	1.35	1.35	1.59	1.58	1.77	1.73	1.96	1.90
3 hrs.	0.56	0.54	0.70	0.66	0.88	0.85	1.01	0.96	1.19	1.13	1.33	1.27	1.47	1.43
6 hrs.	0.34	0.33	0.42	0.40	0.53	0.52	0.61	0.61	0.72	0.72	0.80	0.77	0.89	0.90
12 hrs.	0.20	0.20	0.25	0.24	0.32	0.31	0.37	0.36	0.43	0.42	0.48	0.46	0.53	0.53
24 hrs.	0.12	0.11	0.15	0.14	0.19	0.18	0.22	0.22	0.26	0.25	0.29	0.27	0.32	0.31

Figures in columns marked "a" are calculated by Chen "short" method.

Figures in columns marked "b" are from isopluvials of charts in Ref. 2-2

To obtain rainfall intensity in mm per hour multiply inches per hour by 25.4.

TABLE 2-6

can be used to compute the average rainfall intensity r_{av} (in/hr) for any duration t_d (minutes) and return period T (years). The a, b and c values so determined are believed to be as accurate as those computed directly from the 49 maps in Ref. 2-2.

Chen computed equations similar to A2-19 for the cities of Los Angeles, Chicago, Miami, Houston, Denver and Olympia (Washington) and calculated rainfall intensities for the same durations and frequencies as in Table 2-6. Comparison with information taken from the 49 pluviagraphs of Ref. 2-2 results in finding, in most cases, that the calculated are compatible with those obtained directly from the charts. It can be concluded that the values of the standard storm parameters a_1, b_1 and c_1 as calculated are adequate for each location studied.

Note that the ratio of the 1-hour to the corresponding 24-hour rainfall depth, for the 7 cities studied in detail, ranges over a broad spectrum of values.

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RUNOFF

3.1 General

Better judgement can be exercised in the design of facilities for the management of stormwater if there is understanding of what actually occurs from the time a runoff-producing storm starts until the storm and runoff cease. The principal phenomena of this part of the hydrologic cycle are:

3.1.1 Precipitation

The high intensity short-duration bursts of rainfall in thunderstorms are the usual type of precipitation contributing to critical urban runoff. For the rainfall frequency chosen by the designer, the necessary intensities for the durations of design interest can be obtained from intensity-duration-frequency curves developed as outlined in Chapter 2. If the design method requires hydrographs, the necessary temporal rainfall distribution may be developed as discussed in Chapter 2.

3.1.2 Interception

The part of rainfall that is retained by the leafy or aerial portion of the vegetation is termed the "interception loss". This is either absorbed by the leaf surfaces or returned to the atmosphere through evaporation. In general, between 0.02 and 0.10 inches of rain is held on foliage before appreciable drip takes place. In the quantitative sense, rainfall interception by vegetation is rarely of importance in connection with urban highway storm drainage and may properly be ignored in design.

3.1.3 Infiltration

Quantitatively, the most significant abstraction from rainfall before it becomes runoff is infiltration. For the purposes of storm drainage hydrology, infiltration capacity is the maximum rate at which water can enter the soil of a particular area under a given set of conditions. Actual infiltration (the passage of water through the soil surface into the soil) and percolation (the movement of water within the soil) are closely related with the lesser of the two governing the abstraction of rainfall through infiltration. Most field infiltration capacity curves approach a steady minimum rate after less than one hour. Relative minimum infiltration capacity for three broad soil groups are (Ref. 3-1):

Soil Group	Infiltration Capacity	
	In/Hr	mm/Hr
Sandy, Open-structured	0.50-1.00	12.7-25.4
Loam	0.10-0.50	2.5-12.7
Clay, Dense-structured	0.01-0.10	0.3-2.5

The Unified Soil Classification System (Ref. 3-2) gives the following expanded grouping of minimum infiltration rates for the more commonly encountered soil groups:

Description	Unified Soil Group Symbol	In./Hr.	mm/Hr.
Sand and gravel mixture	GW, GP SW, SP	0.8-1.0	20.3-25.4
Silty gravels & silty sands to organic silt & well developed loams	GM, SM ML, MH OL	0.3-0.6	7.6-15.2
Silty clay sand to sandy clay	SC, CL	0.2-0.3	5.1-7.6
Clays, inorganic & organic	CH, OH	0.1-0.2	2.5-5.1
Bare rock, not highly fractured	---	0.0-0.1	0 -2.5

The infiltration mean values are for uncompacted soils. For compacted soils, infiltration values will be decreased by percentages ranging from 25 to 75, the variation depending on the degree of compaction and the type soil encountered. The great influence of vegetal cover on infiltration capacity is evidenced by the fact that bare soil infiltration capacity can be increased 3 to 7.5 times with good permanent forest or grass cover. Little or no increase results with poor growth crops. Many factors influence infiltration capacity including soil type, moisture content, organic matter, vegetative cover and season. Antecedent precipitation such as high intensity rains of short duration coming after a dry period significantly affects soil infiltration capacity. Fig. 3-1 shows the variations to be expected due to the soil character as well as the effects of initial moisture content. It is noteworthy that for most soils, the infiltration capacity curve reaches a substantially constant ultimate infiltration capacity rate after a relatively short period, 30-45 minutes ordinarily.

3.1.4 Depression Storage

Some of the precipitation which reaches roofs, pavements and pervious surfaces is trapped in the many shallow depressions of varying size and depth present on practically all surfaces. Obvious difficulties in obtaining meaningful data have militated against measurement in the field of the specific magnitude of such depression storage.

Hicks (Ref. 3-5) in Los Angeles, used depression storage losses of 0.20, 0.15 and 0.10 inches (5.1, 3.8 and 2.5mm) for sand, loam and clay respectively, based upon analysis of periods of high rates of rainfall and runoff. Tholin and Keifer (Ref. 3-6) for Chicago, developed from analyses, overall depression storage of 1/4-inch (6.35mm) on pervious areas with a range of depth of specific depressions of up to 1/2-inch (12.7mm); and 1/16-inch (1.59mm) on paved areas with a range of depth up to 1/8-inch (3.18mm). Fig. 3-2 shows the to-be-expected correlation of depression storage with slope.

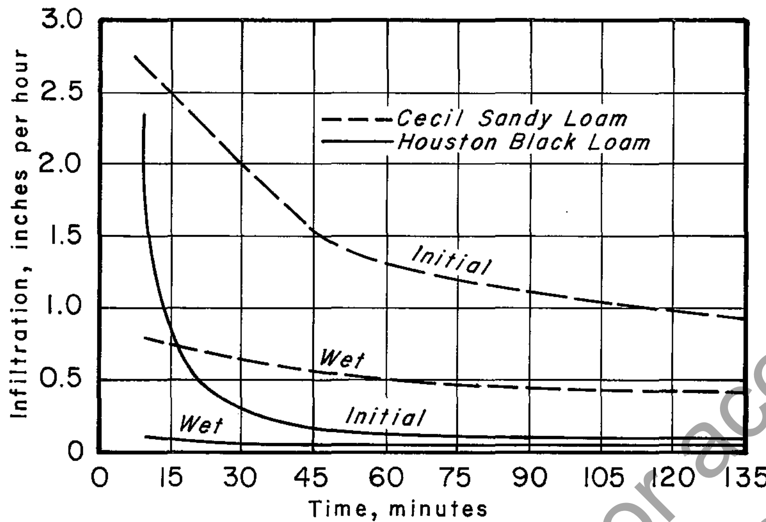


FIG. 3-1 Comparative infiltration rates during initial and wet runs. *(After Free, Browning and Musgrave.)*

ONE INCH is 25.4mm

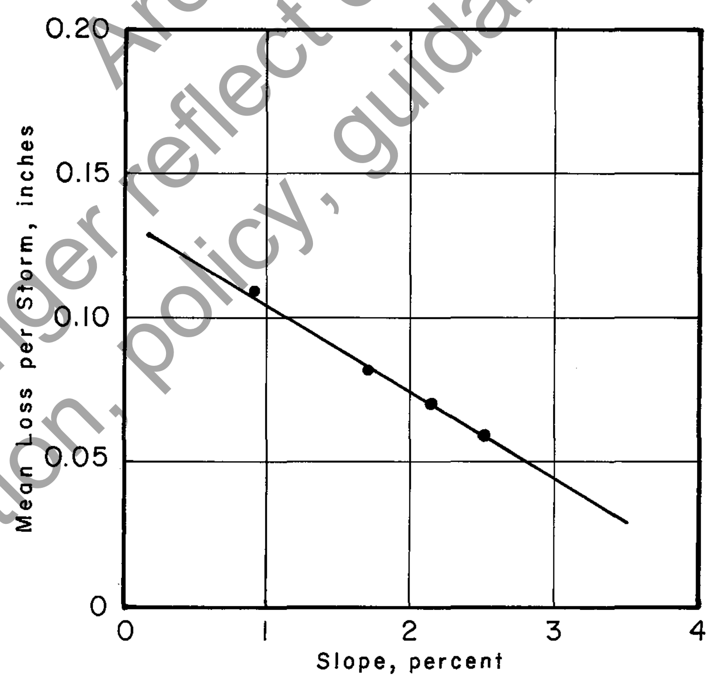


FIG. 3-2 Depression-storage loss versus slope for four impervious drainage areas. *(After Viessman.)*

3.1.5 Overland Flow

That portion of rainfall that exceeds a local infiltration rate develops a film of water on the surface until overland flow commences to travel over the ground surface to a channel. With each outflow rate at the lower end of a sloping plane surface, there is associated a detention depth which is a measure of the storage effect due to overland flow in transit. Horton (Ref. 3-4) stated that this initial detention "....commonly ranges from 1/8-inch (3.18mm) to 3/4-inch (19.05mm) for flat areas and 1/2-inch (12.7mm) to 1.5 inches (38.1mm) for cultivated fields and natural grasslands or forest". Fig. 3-3 indicates some experimental detention flow relationships.

Gallaway, Rose and Schiller (Ref. 3-9) analyzed water depth data obtained from experimental tests on different types of surfaces and developed Fig. 3-4 to indicate the effects of the principal variables on the water depth at 24 feet (7.315mm) from the crown line of the surface for varying rainfall intensities, average texture depths, drainage lengths and cross-slopes. Note that the sheet flow water depth on a pavement increases with rainfall intensity, drainage length and flatter cross-slopes. It decreases with increases in the average texture depth. The measured average texture depth for 9 different types of pavement surfaces shows a range of .02 inches (.51mm) to .03 inches (.76mm). See Table 3-1.

The information given by Figs. 3-3 and 3-4 is of particular significance with respect to the phenomenon known as "hydroplaning". The inability of the sheet flow on a pavement to move from directly beneath the tires of a moving vehicle is the basic cause for the sliding or hydroplaning. If the depth of the texture of the pavement surface does not permit water to flow out away from the tire contact surface and if the tire surface has no water escape routes between the high areas of the tread pattern, the water film under the tire contact lends itself to potential sliding or skidding. Loss of control of the vehicle can then occur.

As shown by Figs. 3-3 and 3-4 the sheet flow depth is increased significantly by higher rates of runoff (from greater rainfalls), by increased lengths of overland flow, by flatter overland flow slopes and by smaller texture depths. The highway designer can favorably influence some of these factors. For example, a 48-foot pavement can be crowned to have two 24-foot slabs draining away from a common crown or the entire 48 feet can be drained to one edge. In the absence of other compelling reasons, a design to have the sheet flow no deeper than that at the edge of a 24-foot wide pavement with a reasonably good cross-slope would be the preferable choice. The highway pavement design and specifications should achieve as durable a deep-textured pavement as practicable.

Utilizing the kinematic wave formulation nomograph for determining time of concentration for overland flow, Fig. 3-5, it is found that these vary for 1/8-inch per foot (1.04mm per metre) cross-slope as follows:

Description of Surfaces Tested:

Pavements: Max. Size of Aggregates and Average Texture Depths

Surface Type	Maximum Size of Aggregate (In.)	Average Texture Depth* (In.)
Portland cement concrete, rounded siliceous gravel, transverse drag**	3/4	0.035
Portland cement concrete, rounded siliceous gravel, longitudinal drag**	3/4	0.036
Clay-filled tar emulsion (jennite) seal, no aggregate		0.009
Hot-mixed asphalt concrete, crushed limestone aggregate terrazzo finish	1/2	0.003
Hot-mixed asphalt concrete, crushed siliceous gravel	1/4	0.019
Hot-mixed asphalt concrete, rounded siliceous gravel	5/8	0.039
Rounded siliceous gravel surface treatment, chip seal	1/2	0.141
Synthetic lighweight aggregate surface treatment, chip seal	1/2	0.164
Hot-mixed asphalt concrete, synthetic lightweight aggregate	1/2	0.020

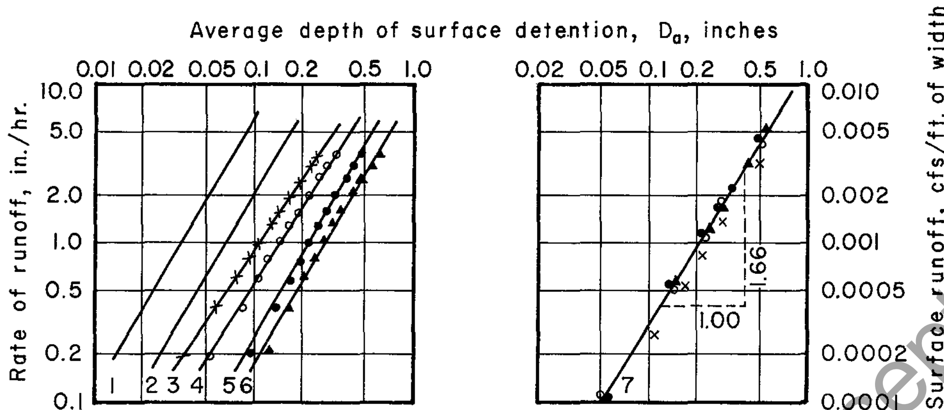
*Obtained by putty impression method.

**With respect to direction of vehicular travel.

One inch is 25.4mm

From: Gallaway, Rose & Schiller, 1972.

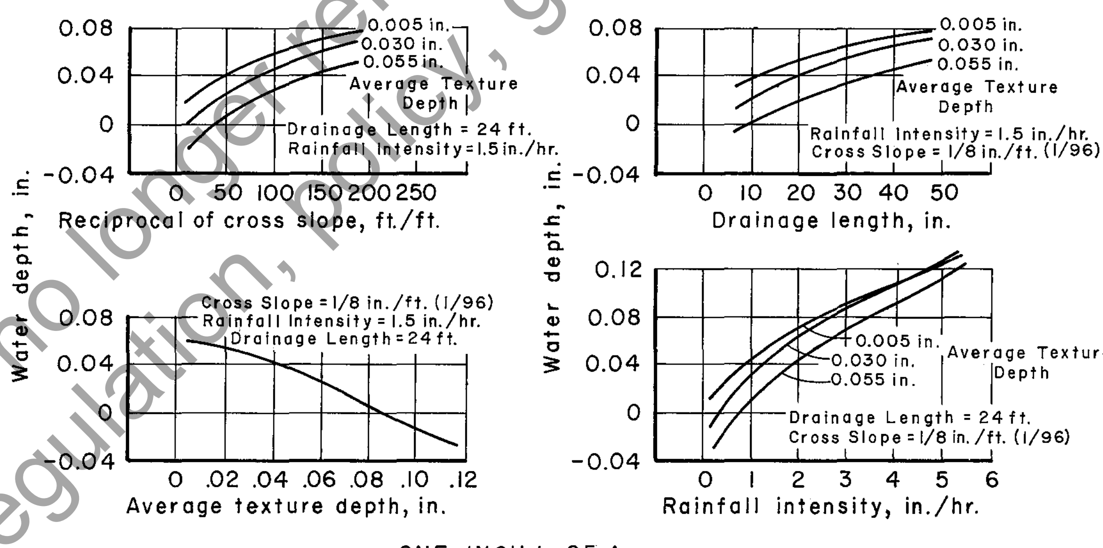
TABLE 3-1



ONE INCH is 25.4mm

Curve no.	Surface	Length, ft.
1	Paved surface	12
2	Paved surface	72
3	Bluegrass sod	12
4	Bluegrass sod	24
5	Bluegrass sod	48
6	Bluegrass sod	72
7	Bluegrass sod	12-72

FIG. 3-3 Detention-flow relationships of overland flow. (Infiltrometer data from 12 ft. plot by H.N. Holtan; other lengths by C.F. Izzard.) *From: Handbook of Applied Hydrology, Ven Te Chow, Editor-in-chief. Copyright 1964 by Mc Graw-Hill Book Co. Used by permission of Mc Graw-Hill Book Company.*



ONE INCH is 25.4mm

FIG. 3-4 Water depths versus variables for combined surfaces. *(After Galloway, et al.)*

Rainfall Rate in./Hr.	Rainfall Rate mm/Hr.	Overland Flow Length Feet	Overland Flow Length Metres	Time to Reach Equilibrium, Minutes
1.5	38.1	24	7.315	1.5
1.5	38.1	36	10.973	2.0
8.0	203.2	36	10.973	3.9
1.5	38.1	48	14.630	2.4
8.0	203.2	48	14.630	4.6

The significance of these figures is that for highway pavement runoff, the overland flow portion of the time of concentration almost always will be less than 5 minutes. To this must usually be added the time of flow in the gutter or swale to the first inlet. Such gutter flow time generally will be at most, 1 to 2 minutes. It is recommended that a minimum inlet time of 5 minutes be used for the upper most inlet. The relatively small mass run-offs involved for times less than 5 minutes taken together with consideration of minimum pipe size make it inadvisable from practical considerations to design for shorter inlet times. Reported inlet times for municipal urban drainage design vary from 5 minutes in densely developed steep areas to 10 to 15 minutes in well developed districts with relatively flat slopes. In very flat residential areas with widely spaced inlets, times of 20 to as much as 30 minutes are customary.

Heretofore, various formulas and nomographs (Refs. 3-14 and 3-33) have been presented for total time of concentration or for the overland flow portion of the time of concentration. A thorough study by the University of Maryland (Ref. 3-11) found that the soundest, most realistic formula for overland flow time of concentration T_c was the following kinematic wave equation:

$$T_c = \frac{K L_o^{.6} n^{.6}}{i^{.4} S_o^{.3}} \quad \text{..........(3-1)}$$

with T_c in seconds; L_o the overland flow length in feet or metres; n the Manning roughness coefficient of the pavement; i the rainfall rate in inches per hour or metres per hour; and S_o the overland flow slope in feet per foot or metres per metre. K is 56 for English units, 26.285 for metric units. Fig. 3-5 is a nomograph for the solution of the kinematic wave overland flow equation in English units.

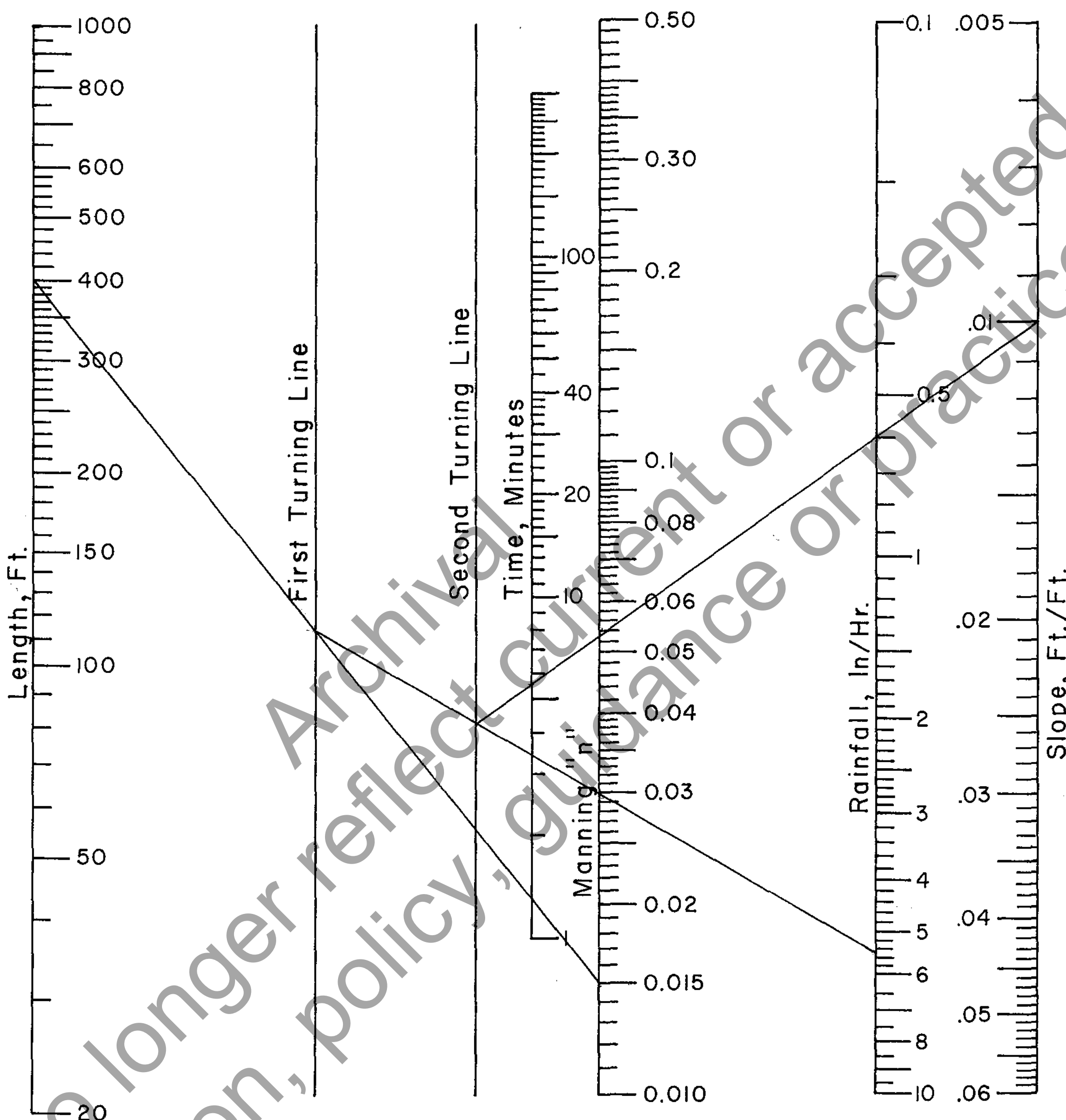
The kinematic wave theory nomograph is consistent with the latest concepts of fluid mechanics and considers all those parameters found important in overland flow when the flow is turbulent (where the product of the rainfall intensity and length of the slope is in excess of 500).

When using the nomograph, the following Manning roughness coefficients are recommended: 0.013 for concrete and 0.50 for turf. Since these values are in close agreement with normal flow data, Manning coefficients obtained from normal flow experiments on other surfaces are probably satisfactory for use.

In using the nomograph the designer has two unknowns as the time of concentration and the associated rainfall computations are started. The problem is one of iteration or trial and error. A value for i must be

Equation solved by nomograph:

$$t_c\ (\text{Sec}) = 56\,\frac{L_o^{.6}\ n^{.6}}{i^{.4}\ S_o^{.3}}$$



The initially assumed value of i and the nomograph value of t must be checked against the applicable intensity-duration-frequency curve by trial and error.

Example:

L_o= 400 ft.
n = 0.015
i = 5.5 in./hr.
S_o= 0.01
t = 5.5 min.

ONE INCH is 25.4mm
ONE FOOT is 0.3048m

FIG. 3-5 Nomograph for determining time of concentration for overland flow, Kinematic Wave Formulation. *(After Ragan.)*

assumed and Fig. 3-5 will give a related time of concentration. The assumed rainfall intensity must then be checked against the rainfall-intensity-duration curve for the frequency of recurrence chosen for the particular design problem.

An overland flow represented by L_o = 150 feet, S_o = 0.02, n = 0.500 (turf) and the 25-year intensity-duration-frequency curve is delivered to a swale 2100 feet long with an estimated average flow velocity of 5 feet per second or a swale time of flow of 7 minutes. The time of concentration at the lower end of the swale is determined as follows:

Assume i is 5 iph (Rational method); From Fig. 3-5, t_c = 21.1; T_c=28.1; from Fig. 2-7, i = 4.2

Assume i=4.2; from Fig.3-5, t_c =22.7; T_c = 29.7; from Fig.2-7, i=4.1

Assume i=4.0; from Fig.3-5, t_c =23.0; T_c = 30; from Fig. 2-7, i=4.05

Use T_c = 30 min. and i = 4.05 iph.

Thus, it is found that the time of overland flow concentration plus the time of flow in the swale is 30 minutes with a related 25-year rainfall intensity of 4.05 iph. At the head end of the swale, the overland flow t_c would be calculated as follows:

Assume i is 6 iph; from Fig. 3-5, t_c = 19.7 min., from Fig.2-7: i=5.1

Assume i = 5.1 iph; from Fig. 3-5, t_c = 21.0 min.; from Fig.2-7: i=4.92

This is satisfactory. Use i = 5.0.

The swale for this example would be designed for the greater design flow of either that at the upper end, using the appropriate C and A with i assumed 5.0 iph (or cfs/acre) or that at the lower end with its appropriate C_1 and A_1 with their related i of 4.05 iph.

In most instances, the total time of concentration includes flow times in swales, gutters and/or conduit reaches and it is then advantageous to estimate such latter times prior to evaluating total time of concentration.

3.1.6 Gutter Storage

The overland flow entering a gutter is zero at the upstream end and increases progressively downstream. The flow in the gutter is spatially varied and the longitudinal water surface profile is complex; it has been discussed in detail elsewhere (Refs. 3-6 and 3-10). Gutter storage generally has a greater peak-reducing influence than the surface detention of overland flow and requires a longer time to achieve equilibrium outflow. Long gutters sometimes provide a surplus of storage above that required to accomodate the rainfall excess. This, in turn, results in a gutter outflow rate at the inlet less than the equilibrium rate. Clearly, routing the overland flow hydrograph through storage in the gutter or channel leading to an inlet requires an evaluation of the instantaneous storage under the water surface profile for various rates of flow at the inlet.

For most practical design involving small tributary areas supplying gutter flow, there is practical recognition of the gutter storage in the use of the rational method wherein the time of concentration for the overland flow at the upper end of the gutter has added to it the time of flow in the gutter length to the inlet.

An approximate modified Manning equation (Ref. 3-10) computes uniform flow in shallow, wide, triangular channels such as swales and gutters:

$$Q = K\ (z/n)\ S^{.5} d^{8/3} \qquad \ldots\ldots\ldots\ldots\ldots (3\text{-}2)$$

where Q is the discharge in cfs or cubic metres per second, d is the depth of water in feet or metres, z is the ratio of water surface width to d, n is the Manning coefficient of roughness consistent with the constants in the equation and S is the longitudinal slope of the channel. The coefficient K is 0.56 for English units, 0.377 for metric units. A nomograph for this equation is available in English units as Fig. 5-1. From this equation or the nomograph there can be obtained maximum water depth in the gutter which will indicate the extent to which stormwater flowing along the edge of the pavement encroaches on the traffic lane. Also, there can be obtained the average velocity in the gutter which can be used to determine that part of the time of concentration involved in the flow from the upper end of the gutter to the inlet.

3.1.7 Conduit Storage

In the same basic way that any detention storage diminishes the height of an inflow hydrograph, the volume of detention in a conduit can effect a reduction in the peak rate of flow of the hydrograph. If satisfactory discharge-storage relationships are available, storage routing can be applied. Such relationships necessitate the computation of instantaneous backwater curves. Since only the rate of change in storage is necessary to solve the storage equation, it is considered expedient to assume a uniform flow condition for each discharge rate and compute the conduit volume occupied by the flow. This requires a knowledge of actual or assumed conduit cross-sections.

If the flow is in a pressure system no peak reduction factor is applicable since the conduit is usually full before peak flow is reached. The most common design practice is to have the storm sewer just full or lightly surcharged at design flow. Peak flow design methods used for the majority of urban highway storm drainage are not compatible with flow routing techniques. Coupled with the general accuracy of the methods and techniques of storm drainage design, these facts do not justify any reduction in design of peak flow rates due to conduit detention.

3.2 Rational Method

Currently (1978), and for the past 50 to 75 years, the overwhelming majority of storm sewer design has utilized what is termed the "Rational Method" to express the direct relationship between rainfall and runoff. (In the United Kingdom this method is known as the Lloyd-Davies Method). The traditional formula is expressed as:

$$Q = KCiA \quad \text{........................} (3\text{-}3)$$

Q is the peak runoff rate in cubic feet per second or cubic metres per second at a given point; C is a runoff coefficient representing the ratio of average rainfall to the peak runoff during a period termed the time of concentration; i is the average intensity of rainfall in inches per hour or mm per hour for a duration equal to the time of concentration and for a frequency of recurrence of that rainfall that has been chosen or is required for the design problem under scrutiny; A is the tributary area in acres or hectares; K is a coefficient equal to one for English units, equal to 0.00275 for metric units.

Time of concentration discussed more fully later, is defined as the time of flow from the hydraulically farthest point of the drainage area to the design point under consideration.

The peak runoff rates determined by careful use of the rational method have been found to be satisfactory for relatively small areas. Checks against observed rainfall-runoff information (unfortunately very scarce for urban areas) have indicated that generally, the rational method for small areas will give peak runoffs somewhat higher than those actually observed (Heimstra and Reich, Ref. 3-12, Missouri State Highway Department, 1972, Ref. 3-13). This publication recommends that the rational formula be used until the watershed area reaches approximately 500 acres (202 hectares). Current recommendations by others range from maximums of 200 acres (80.9 hectares)(Ref. 3-14) to one square mile (259 hectares)(Ref. 3-16); in some instances, the rational method is considered satisfactory for areas up to 1000 acres (405 hectares) (Ref. 3-15). It is recommended that for areas larger than 500 acres (202 hectares) but less than about 750 acres (304 hectares), the peak rate of runoff be estimated by both the rational method and by another means such as the unit hydrograph method. In that range in area sizes the method that produces the larger peak runoff should be used. Above 750 acres (304 hectares) up to several thousand acres (over 500 hectares) the hydrograph method of runoff determination is recommended.

3.2.1 Coefficient of Runoff

The runoff coefficient C, in the rational formula, is the parameter most fraught with the difficulties of precise determination since it lumps together an evaluation of several physical aspects of the runoff phenomenon. The runoff coefficient characterizes the following variables among others: antecedent precipitation, soil moisture, infiltration, detention, ground slope, ground cover, evaporation, the shape of the drainage area and overland flow velocity. Clearly, a high degree of engineering judgement and experience are desirable for viable estimates of the runoff coefficient for a particular set of circumstances.

The use of average coefficients for differing kinds of surfaces with such coefficients assumed not to vary through the duration of the storm, is common practice (Table 3-2). It is generally agreed, however, that the coefficient of runoff for any particular surface varies with respect to the length of time of prior wetting.

Horner (Ref. 3-18) suggested variations with time in two curves, one for completely impervious surfaces and the other for completely pervious surfaces of dense soils. These are characterized by rather rapid increases in the coefficient in the first 40 to 60 minutes followed by much slower increases to substantially constant values after about 120 minutes. Mitci (Ref. 3-19) has developed a general formula which substantially reproduces the Horner curves as well as intermediate ones for other percentages of imperviousness:

$$C = \frac{0.98t}{4.54 + t} P + \frac{0.78t}{31.17 + t} (1-P) \quad \ldots (3-4)$$

in which t is the time from beginning of rainfall in minutes and P is the percent of impervious surface. Fig. 3-6 graphs this formula for the range of 0% to 100% imperviousness. These curves cannot be used directly to determine the applicable runoff coefficient since the average rainfall intensity used in the rational method is not fixed in any time sequence of the rainfall. Experience has shown that in the great majority of significant storms the most intensive rainfall occurs appreciably after the beginning of precipitation. For this reason it is erroneous to assume the start of the time of concentration and the beginning of rainfall to be coincident. Usually, a substantial period of rainfall will have occurred before the beginning of the time of concentration and consequently, the low coefficients indicated at the beginning of rainfall are in no way representative of storm conditions when the average design intensity occurs.

To properly use the C values of Fig. 3-6 the following procedure is suggested using a Winnepeg, Canada example (Ref. 3-20):

1. Given: Residential Subdivision
   61 acres (24.69 hectares) total area
   Average surface slope less than 3% average
   Percent imperviousness 32% (roofs assumed draining onto grass)
   5-year rainfall defined at 5-minute intervals
   Longest time of travel of runoff from collector's headwaters to the main intercepting sewer in Winnepeg: 3 hours
   Point under design has a 12-minute time of concentration
   5-year rainfall intensity equation (Ref. 3-20):

$$i = \frac{K47.2}{(t_d + 8)^{.828}}$$

Runoff Coefficients - Range for Different
Kinds of Surfaces

Character of Surface	Runoff Coefficients
Pavement	
Asphaltic and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.05 to 0.10
Average, 2 to 7 percent	0.10 to 0.15
Steep, 7 percent	0.15 to 0.20
Lawns, heavy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

From: ASCE-WPCF "Design and Construction of Sanitary and Storm Sewers", ASCE Manuals and Reports on Engineering Practice No. 37, New York, 1969.

TABLE 3-2

RUNOFF COEFFICIENT AS A FUNCTION OF PERCENT IMPERVIOUSNESS AND ANTECEDENT RAINFALL

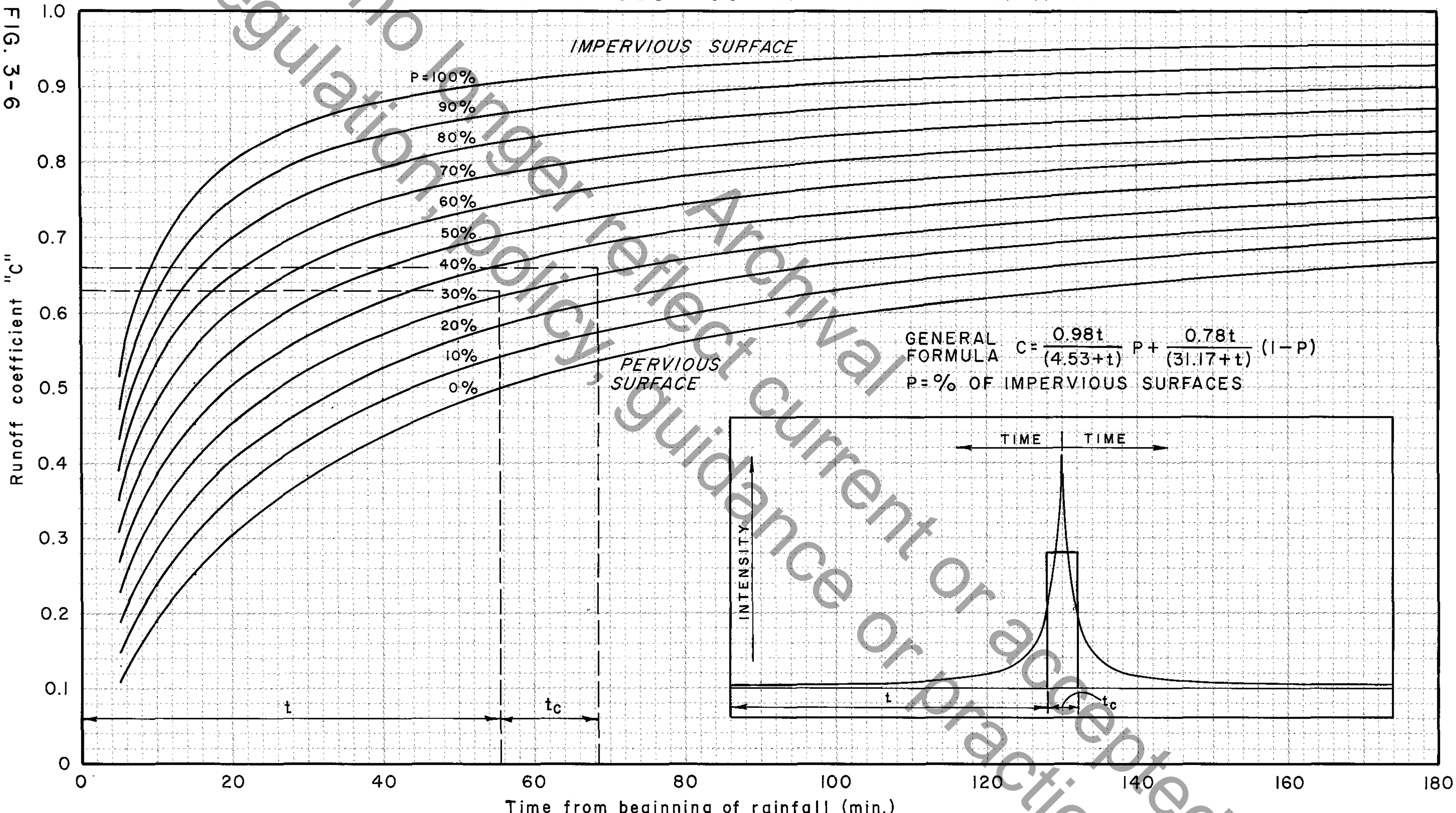


FIG. 3-6

in which i equals inches per hour or metres per hour; t_d is the time duration of rainfall in minutes; 47.2, 8 and 0.828 are storm parameters determined from precipitation data in English units; and K, a coefficient equal to one (1) for English units and equal to 0.0254 for metric units.

Synthetic rainfall peak at 0.33 x 180 or 60 minutes from start of rain.

2. Assume start of 12-minute design rainfall at (60 - .33 x 12) or 56 minutes from start of 3-hour storm.

3. Assume end of 12-minute design rainfall at 56 + 12 or 68 minutes from start of 3-hour storm.

4. From Fig. 3-6 for 32% imperviousness and times of 56 and 68 minutes, read C = .63 and .67 or .65 average. Attention is directed to the fact that if the C value is obtained from the first 12 minutes of Fig. 3-6, it would have an average value of about 0.26 or 40% of the more realistic C of 0.65. Even at 12 minutes, the C value is only 0.38 or 58% of 0.65.

Chicago (Ref. 3-6) used a 3-hour total duration of the synthetic design rainstorm "....to cover the time of travel from headwater to outlet in the largest individual sewer systems within Chicago". Montreal (Ref. 3-19) selected three hours for the same reason "....to cover all intermediate periods". Winnipeg (Ref. 3-20) also uses three hours presumably for the same reasons. For urban areas with largest sewer system travel time less than three hours, use the computed travel time but in no instance use less than two hours. The sole purpose of assuming a reasonable enveloping time of travel is to ensure the placement of the critical short duration rational method rainfall intensity within a longer storm for suitable choice of the C value from the curves of Fig. 3-6. Where a careful study of record storms is not made to determine the position of the most intense periods of rainfall, assume the critical short duration intensity (that conforming to the time of concentration of the point under design) to be symmetrically placed either side of the midpoint of the longer system storm. This will then give the time positions which, together with the design imperviousness of the tributary area, permits a determination of C from Fig. 3-6.

The foregoing describes the choice of a rational method C value whenever the rainfall design frequency is a 10-year or less recurrence interval. For longer recurrence intervals modification of the 10-year C shall be made as follows. For the chosen design frequency and percent of imperviousness, enter the chart of Fig. 3-7 and determine the ratio of the C for the design point to the maximum C for the 100-year frequency (which maximum is assumed to be 1). It then becomes necessary to determine the ratio for the 10-year frequency. The 10-year C value is then modified by the quotient of the chosen design frequency ratio divided by the 10-year frequency ratio. The procedure is illustrated by example:

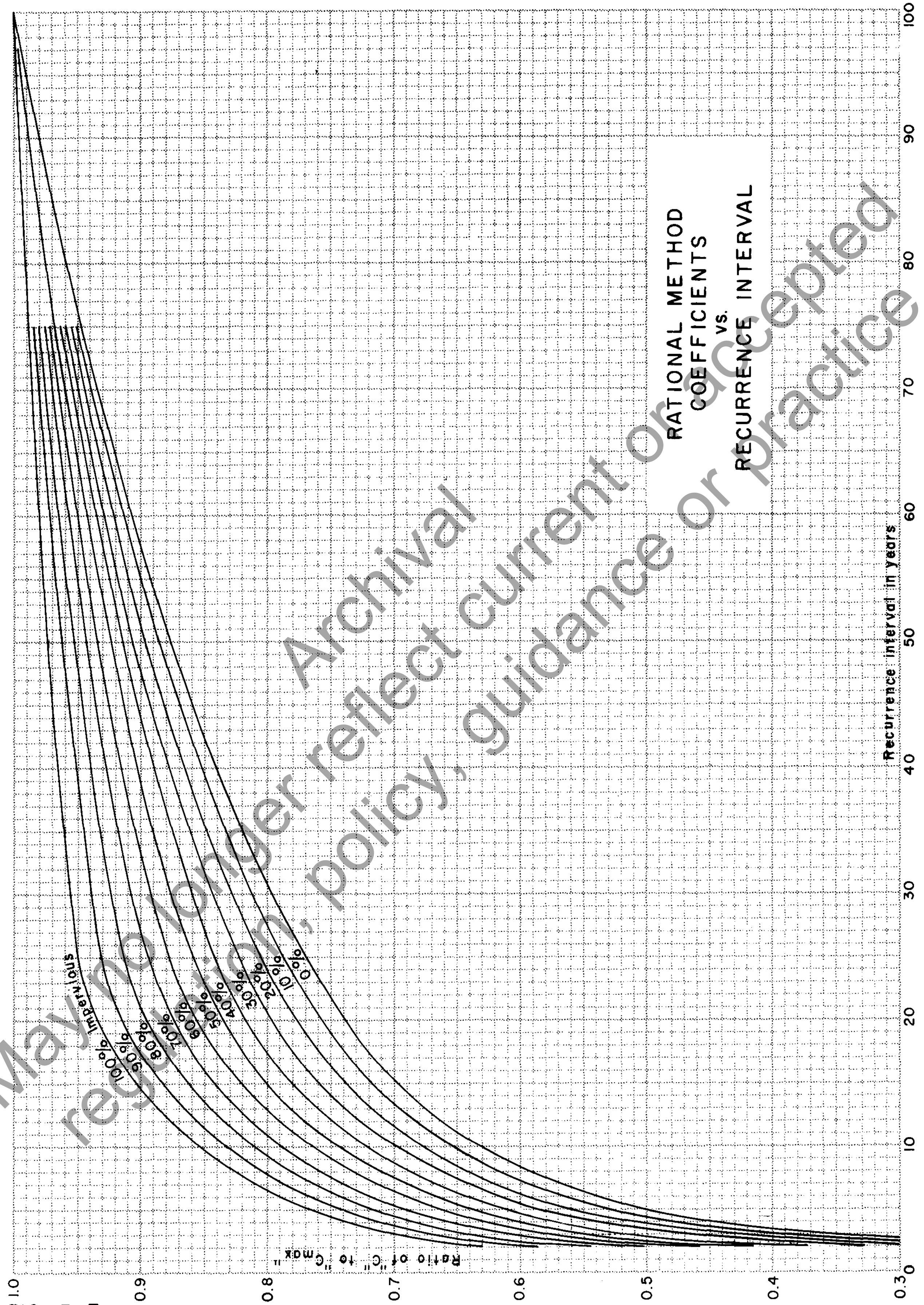

RATIONAL METHOD
COEFFICIENTS
vs.
RECURRENCE INTERVAL
Recurrence interval in years
Ratio of "C" to "Cmax"
100% Impervious
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
10
20
30
40
50
60
70
80
90
100
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3

FIG. 3-7

Given: Commercial Subdivision, 15 acres (6.07 hectares), Clayton, Missouri
65% imperviousness
25-year design frequency
Longest time of travel of runoff from collector's headwaters to point of discharge: 2 hours
Point under design has a 10-minute time of concentration
10-year rainfall intensity (from Chapter 2) for 10-minute time of concentration: 0.97 inches or 5.82 iph (147.8 mm per hour)
25-year rainfall intensity for 10 minutes time of concentration: 1.13 inches or 6.78 iph (172.2 mm per hour)

Solution: From Fig. 3-7 for 65% imperviousness, find for 10-year recurrence interval ratio of 0.776 and for 25-year recurrence interval a ratio of 0.886. The 10-year C value should then be multiplied by 0.886/0.776 or 1.14 to obtain the C for the 25-year recurrence interval.

The time of concentration would be between the 2-hour storm time of 55 minutes and 65 minutes. From Fig. 3-6, for 65% impervious area for these times, C is 0.76 to 0.78 or an average of 0.77. Thus, the rational method gives the 10-year peak runoff as:

$$Q = (0.77)(5.82)(15) = 67.2 \text{ cfs } (1.903 \text{ m}^3/\text{s})$$

The 25-year rational method peak is:

$$Q = (0.77 \times 1.14)(6.78)(15) = 89.3 \text{ cfs } (2.529 \text{ m}^3/\text{s})$$

If the 10-year C value is used unchanged to determine the 25-year peak, the latter becomes 78.3 cfs (2.217 m^3/s). The influence of frequency is significant. The development of Fig. 3-7 is described in Appendix A-3.

3.2.2 Time of Concentration

It is assumed that the maximum rate of flow resulting from a certain rainfall intensity over the watershed area is produced by that rainfall maintained for a time equal to the period or time of concentration of flow at the point under consideration. This is generally described as that time required for surface runoff from the hydraulically most distant part of the drainage basin to reach the point being studied. The estimation of the short times of concentration usual in urban drainage is of considerable importance in the application of the rational method. This is so because the average rainfall rate for a duration corresponding to the time of concentration must be determined from the rainfall intensity-duration-frequency curves (Fig. 2-7). These curves show a relatively greater drop in intensity values with increased duration for the shorter duration rainfalls. Clearly, if a time of concentration is estimated that is actually longer than that which realistically occurs, the rainfall intensity obtained from the intensity-duration-frequency curve will be lower than that which actually should be used in the rational formula.

In urban storm drainage, the time of concentration consists of an inlet time which usually is made up of the time required for overland flow runoff to reach a collecting swale or gutter plus the time of flow in the swale or gutter to the uppermost inlet in a storm drainage system. If point of design interest is below the uppermost inlet, there necessarily would be added time of flow in the drain from the inlet or inlets above the point under design. The inlet time varies with the surface slope, depression storage, surface cover, antecedent rainfall and infiltration capacity of the soil, as well as the distance or length of overland flow. As earlier discussed, the last of these items for paved surfaces suggests a practical minimum inlet time of 5 minutes for roadway pavements and paved swales and it is recommended that that be the minimum inlet time for roadway drainage. For inlets picking up water from a grassed swale, a minimum inlet time of 10 minutes is suggested. This latter assumes that in some instances, real inlet time may be somewhat less than 10 minutes but the total volumes of runoff involved between the real inlet time and the assumed 10-minute time are such that only minor ponding in the swale might exist for very short periods of time at the inlet itself. Gutter, swale, channel and conduit flow times can be closely estimated from their hydraulic properties.

The principal need for determination of a time of concentration is to select the average rainfall intensity for a duration equal to that time for the frequency of recurrence that has been decided upon. The values in rainfall intensity-duration-frequency curves are made up such that generally speaking, the short times of concentration involved in urban drainage design, occur at some point in a storm after some prior rainfall has occurred. It is re-emphasized here that the values given by intensity-duration-frequency curves bear no relation to the position of the period or duration in the storm event for which average rate of rainfall is needed for design purposes. That is, if a 30-minute average rainfall rate for a 10-year frequency is desired, it would be picked off from the extreme left part of the chart. This does not, in any sense, mean that the average intensity of rainfall given by the curves occurred in the first 30 minutes of any specific rainfall period.

3.3 The Unit Hydrograph Method

3.3.1 Introduction

For urban watersheds larger than about 500 acres (202 hectares) and smaller than about 2000 square miles (518,000 hectares) (Ref. 3-34), or where storage of significant character is involved, it is recommended that the design storm runoff be developed by the unit hydrograph method. The upper drainage area limit is of no practical concern for urban storm drainage except as a perennial stream may border or traverse a populous area.

A graph showing the discharge of flowing water with respect to time is a hydrograph. This visually integrates all the climatic and physiographic characteristics of a drainage basin as such characteristics govern the relation between rainfall and runoff. The complexities of the basin characteristics are reflected in the time distribution of runoff at the point of interest. Concentrated storm rainfall usually produces a typical single-peak distribution curve as a hydrograph. When there is abrupt variation in rainfall intensity and abnormal groundwater recession or a succession of closely spaced storm rainfalls, multiple peaks may appear on a hydrograph. The reflection of time-related rainfall and flow as shown by a hydrograph is invaluable in understanding the processes that determine runoff.

3.3.2 Types of Hydrographs

In watershed work there are four types of hydrographs suitable for use:

1. Natural hydrographs are those obtained directly from the flow records of a gaged stream channel or conduit.

2. Synthetic hydrographs obtained through the use of watershed parameters and storm characteristics to simulate a natural hydrograph.

3. A unit hydrograph is defined as a hydrograph of a direct runoff resulting from 1 inch (25.4mm) of effective rainfall generated uniformly over the basin area during a specified period of time or duration.

4. A dimensionless hydrograph is one made to represent many unit hydrographs by using the time to peak and the peak rates as basic units in plotting the hydrographs in ratios of these units; sometimes this is called the "index hydrograph".

As defined above, the unit hydrograph can be used to develop the hydrograph of runoff for any quantity of effective rainfall.

The unit hydrograph theory depends upon the above definition and the following assumptions:

1. Within its duration the effective rainfall is uniformly distributed throughout the entire area of the basin.

2. At any point on a stream the discharge ordinates of different unit graphs are directly proportional to the total amount of direct runoff represented by each hydrograph. That is, a rainfall excess (direct runoff) of 2 inches (50.8mm) within the unit duration will produce a surface runoff hydrograph having ordinates twice as great as those of the 1-inch (25.4mm) effective rainfall.

3. The base or time duration of the direct runoff hydrograph due to an effective rainfall of unit duration is constant.

4. The effects of all of the combined physical characteristics of a given drainage basin due to a given period of rainfall are reflected in the shape of the hydrograph of runoff. This includes, for the specific basin, the shape, slope, surface detention, permeability, drainage pattern and channel storage.

Use of the unit hydrograph is limited in the following manner:

a. The principle of the unit hydrograph is applicable to basins of any size. To derive unit graphs it is desirable to use storms well distributed over the entire basin which will produce runoff nearly concurrently from all parts of it. Rarely do such storms occur over large areas. The areal extent of rainfalls that have been observed for a region of interest, will therefore determine the extent of the basin for which a unitgraph may be derived from observed data. This limitation has little practical meaning for highway drainage in urban areas.

b. Relatively small amounts of snowmelt runoff in actual hydrographs make them unsuitable sources for unit hydrographs.

c. Rainfall upon extensive snow cover retards the runoff and increases the time of concentration such that unitgraphs cannot be derived from such rainfalls.

d. The physical characteristics of a watershed remain relatively constant but the variable nature of rainfall cause variations in the shape of the resulting hydrographs. Rainfall duration, time-intensity pattern, areal distribution and amount of rainfall each can affect hydrograph shape. Each possible duration of rain which results in 1 inch (25.4mm) of runoff from rainfall generated uniformly over the area produces a separate unitgraph. In reality, "the effect of small differences in duration is not large and a tolerance of $\pm$ 25% from the established duration is ordinarily acceptable. Further, a unit hydrograph for a short duration of rainfall can be used to develop hydrographs for storms of longer duration" (Ref. 3-34).

Practically, a unit hydrograph is based on the assumption of a uniform intensity of runoff for the unit duration and the time-scale of intensity variations that are critical depend principally on basin size. "If the unit hydrographs for a basin are applicable to storms of shorter duration than the critical time for the basin, hydrographs of longer storms can be synthesized quite easily. A basic duration of about one-fourth of the basin lag is generally satisfactory" (Ref. 3-34).

Areal distribution of rainfall is unimportant for urban highway drainage since virtually all such drainage involves areas too small to be significantly influenced by areal rainfall distribution; no major changes in hydrograph shape would result.

A basic assumption of the unit hydrograph is that the ordinates of flow are proportional to the volume of runoff from any storm of the same duration. Actually, it is known that the duration of the hydrograph recession is a function of the peak flow. For practical use, the assumptions of a constant hydrograph base and ordinates proportional to runoff volume are satisfactory for engineering purposes. The principal underlying these assumptions is that modifications of the discharge hydrograph due to storage are independent of the magnitude of the runoff. This is not rigorously true but for practical purposes, it s condition approximated in natural channels or cross-sections for bankfull stage or less but is not applicable to abrupt changes in section properties such as those which accompany floodplain storage or overbank flows. The unit time of the unit hydrograph is the actual duration of the precipitation excess which, of course, varies with the actual storm. It should not be confused with the unit hydrograph duration. Experience has developed that in general, this unit time is approximately 20% of the time interval between the beginning of runoff from a short, high intensity storm and the peak discharge of the corresponding runoff.

3.3.3 Base Flow

It should be borne in mind that the unit hydrograph represents surface runoff only. If the watershed under study has a persistent low flow at the design point in between rains, it may be necessary to separate such base flows from the total flows to construct a proper unit hydrograph. Fig. 3-8 (Ref. 3-35) shows typical actual hydrographs with dashed lines illustrating methods of separating base flow from surface flow. These graphs indicate the ideal conditions of isolated storms occurring at times of low flow; the separation procedure is therefore relatively simple. The first separation was made as indicated by line a. Consideration of the manner in which groundwater built up during rains in similar basins suggested that a line such as b was more nearly correct. The permeability of the watershed soils is the principal

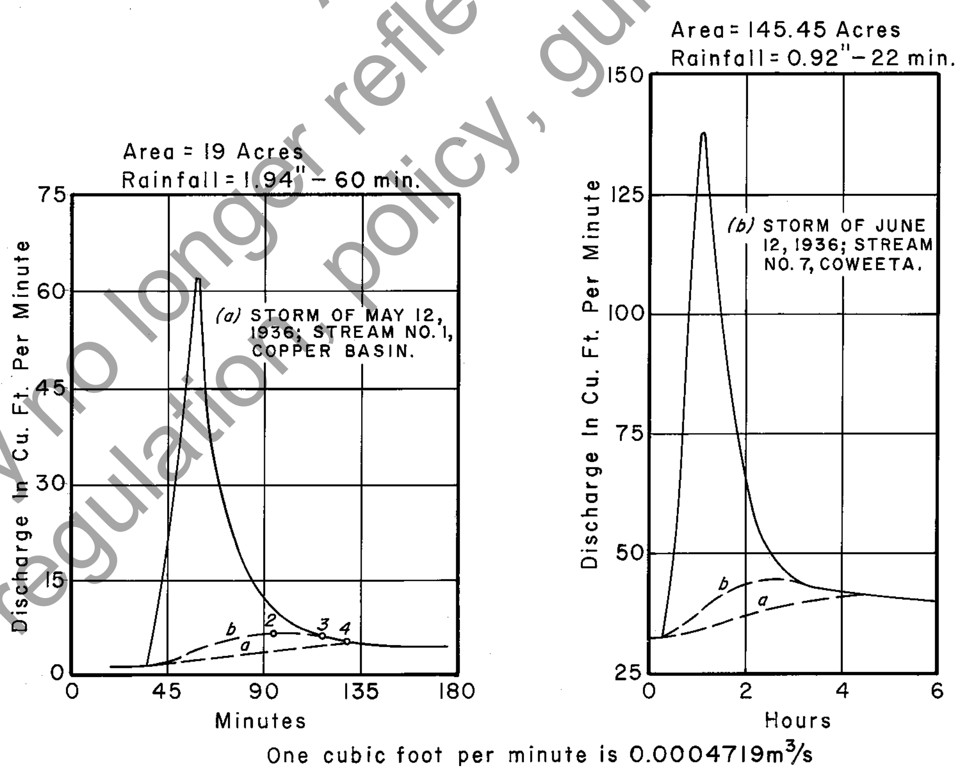


FIG. 3-8 Typical Hydrographs. *(After E. F. Brater).*

factor in whether the groundwater elevations adjacent to the stream rise more or less rapidly than the stream elevations. Judgement based upon the best available knowledge must decide which circumstances probably govern. If the groundwater rises more rapidly than the stream, the slope of the water table toward the stream increases and line a of Fig. 3-8 should be modified as shown by line b. Groundwater discharge does not act in an erratic or jerky manner and consequently, the line of separation must be a smooth curve tangent to the actual hydrograph both where it leaves it and where it rejoins it. The surface water hydrograph has its ordinates above the separation line. In developing a unit hydrograph from actual record rainfall and related flow records, a base flow separation such as described above should be made if the circumstances warrant.

For the usual circumstances governing urban storm drainage: some if not all closed drains, often supplemented by shallow swales (which intercept no groundwater), and significant amounts of impervious areas - base flow is rarely a practical consideration in developing a pertinent unitgraph. Another fact in urban storm drainage is the usually very short response times. Only under quite unique circumstances would the infiltrated rainfall flow through the soils so rapidly as to significantly augment the rapid collection of surface flows. The logical, practical conclusion is that where the urban highway drainage involves using the unit hydrograph, the theoretical need to consider base flow in detail can be ignored.

3.3.4 Basin Lag

Studies of the unit hydrograph have found a principal parameter to be basin lag which is defined herein as the time from the centroid of the effective rainfall to the runoff peak. The lag time reflects the effects of basin shape, slope, roughness, etc. Snyder (Ref. 3-25), for a similarly defined lag, found for the basins of the Appalachian Mountain area that the lag t_p in hours can be

$$t_p = K\, C_t (L L_{ca})^{0.3} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (3\text{-}5)$$

where L is the length of the mainstream from the point of interest to the watershed divide in miles or kilometres; L_{ca} is the distance in miles or kilometres from the same point of interest measured along the mainstream to a point opposite the centroid of the basin. The coefficient K is one (1) for English units, it is 0.75 for metric units. Snyder found the coefficient C_t to vary from 1.8 to 2.2 with some indication of lower values for steeper sloped basins. Eagleson (Ref. 3-26) calculated C_t for five sewered areas in Louisville, Kentucky as listed in Table 3-3. There are two Houston, Texas and one Illinois sewered urban areas also listed in the table (Ref. 3-27). Note that for the sewered areas with considerable channel improvements (the usual suburban condition) the average coefficients for areas under 10 square miles (2590 hectares) are: C_t 0.25 and C_p 2.06. The influence of urbanization on these coefficients

F. F. SNYDER SYNTHETIC UNIT HYDROGRAPH COEFFICIENTS FOR SEWERED URBAN AREAS

Sewered Area No.	Drainage Area Sq.Mi.	Percent Impervious	C_t* Hr. Per Mi.$^{3/5}$	K**	K/C_t	C_p	Mean Basin Slope Ft. Per Ft.
2	0.22	83	0.22	298	1355	0.45	.00923
3	1.90	50	0.27	393	1455	0.61	.00361
4	2.77	70	0.21	153	730	0.24	.00210
5	6.44	48	0.32	402	1255	0.63	.00244
6	7.51	33	0.21	383	1825	0.60	.00355
White Oak	92.0		0.45	73	162	0.11	
Brays	100.0		0.29	69	238	0.11	
Boneyard	4.64	37.4	0.54	187	345	0.29	

One square mile is 2.59km^2; One foot is 0.3048m

All information for areas 2-6 inclusive for Louisville, Kentucky sewered areas - from Ref. 3-26.

White Oak and Brays, Houston, Texas areas from Ref. 3-27.

Boneyard Creek, Urbana, Illinois - from Ref. 3-27.

All areas except Boneyard have extensive urban development with storm sewers and considerable channel improvements.

Boneyard area has extensive urban development with storm sewers but no channel improvement.

* F.F. Snyder Coefficient - Ref. 3-25.

**K = $q_{max} \times t_p$ = cfs per square mile hour.

TABLE 3-3

is clear; Snyder found for natural Appalachian watersheds: C_t of 1.8 to 2.2 and C_p from 0.56 to 0.69. See later discussion under the "Colorado Urban Hydrograph". These average urban coefficients assume the area under design is less than 10 square miles (2590 hectares) and more than 100 acres (40.47 hectares) with a virtually complete storm drainage system consisting of sewers supplemented with considerable channel improvements.

3.3.5 Effective, Excess or Net Rainfall

A necessary and critical first step in the development of a hydrograph based upon the unit hydrograph is a determination of the net or excess rainfall. The total volume of runoff resulting from a storm rainfall is that portion of the precipitation that produces direct runoff and is often called "net", "excess" or "effective rainfall". The amount of runoff from a storm event largely depends on detention, infiltration, evapo-transpiration, etc. or what are sometimes termed "losses" or "abstractions". These are related to the soil type, antecedent rainfall, type of vegetation and the amount of impervious cover. The pervious areas will abstract depression storage and infiltration and there will be depression storage on the impervious areas.

The Soil Conservation Service (Ref. 3-23) has developed a methodology for determining the amount of net or effective rainfall through the use of runoff curve numbers. These curve numbers (CN) reflect the effect of the hydrologic soil-cover complex on the amount of rainfall that runs off.

3.3.5.1 Hydrologic Soil Groups

SCS has classified for hydrologic purposes four soil groups defined as follows:

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.

B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

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.

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 clay pan 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.

More than 4000 soils have been given a hydrologic soil group classification (Ref. 3-23). Some of these classifications were based on the use of rainfall-runoff data from small watersheds or infiltrometer plots of the SCS but the majority are based on the judgements of soil scientists and correlators who used physical properties of the soil in making their decisions. To use the classification in estimating runoff it is necessary to know the approximate area of each soil and for large watersheds, its location by hydrologic unit (each hydrologic unit is the drainage area of a minor tributary flowing into the mainstream or a major tributary). Areas between minor tributaries are combined and also assumed to be hydrologic units. The state soil scientist can be a primary help in classifying the soils of the particular watershed under study. Fig. 3-9 indicates the steps required to determine percentages of hydrologic soil groups.

3.3.5.2 Runoff Curve Numbers

SCS (Ref. 3-24) has runoff curve numbers which can be used to determine effective runoff for areas expected to become urban, those under development and those already completely urbanized. Table 3-4 lists the proper curve numbers to be used for the land use description noted. For areas in which the values in this table do not directly apply, it is suggested that separate curve numbers for each pervious condition be weighted in accordance with the applicable area. Such a weighted CN for the total pervious area can then be weighted with the imperviousness CN for the entire area to obtain a composite runoff curve number. Fig. 3-10 (Ref. 3-24) which assumes a CN of 98 for 100% impervious areas can be used to choose a composite CN. An example follows:

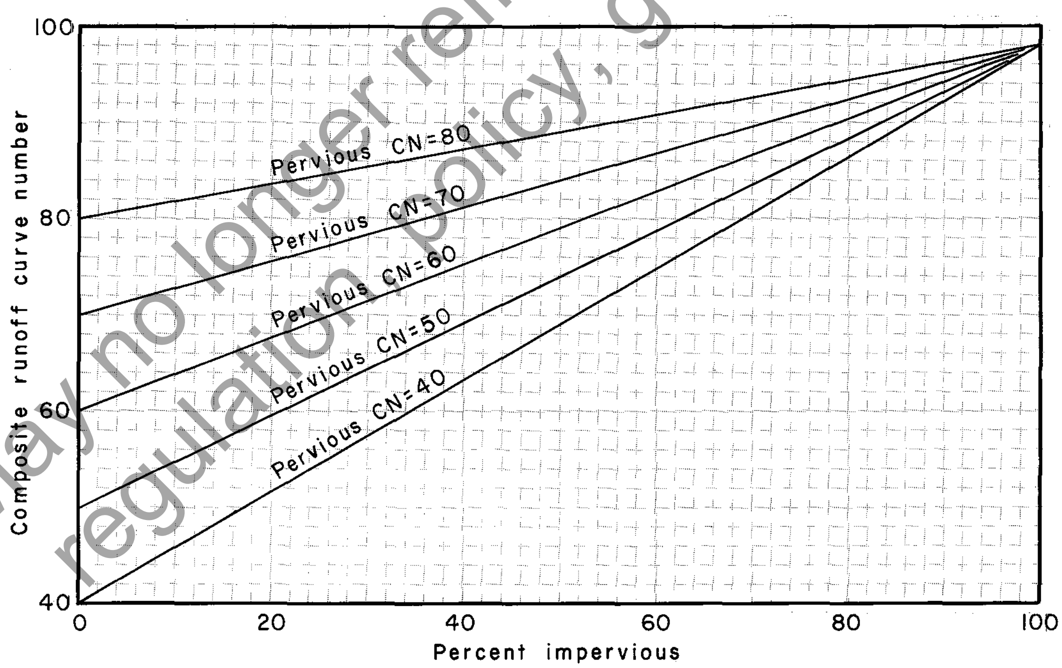
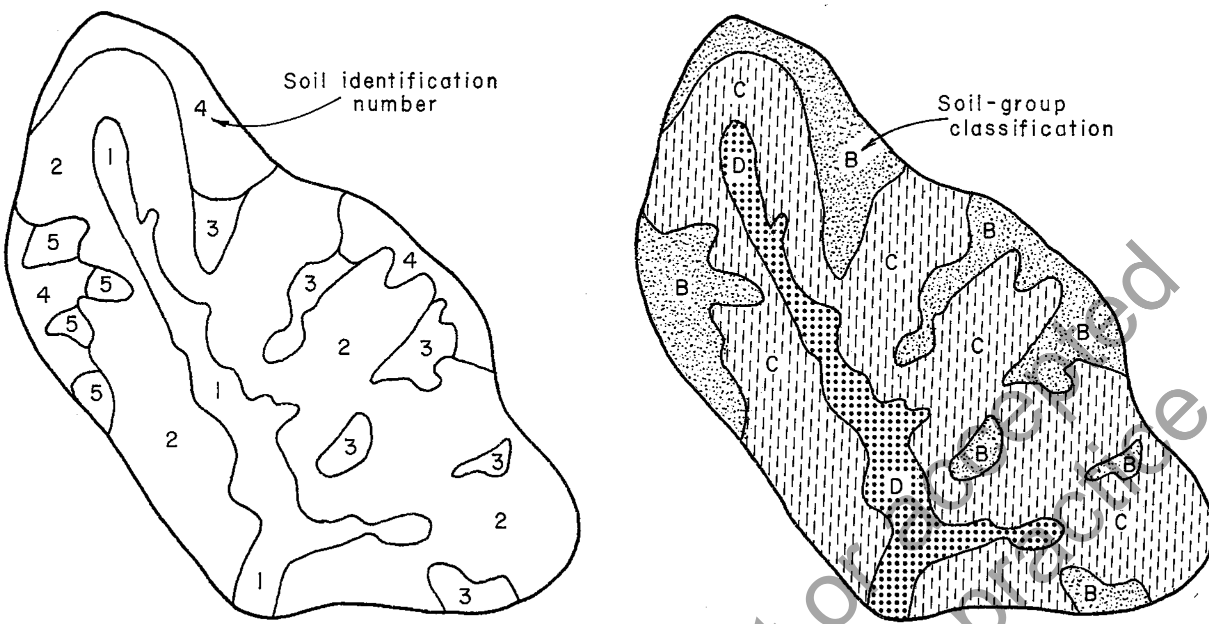
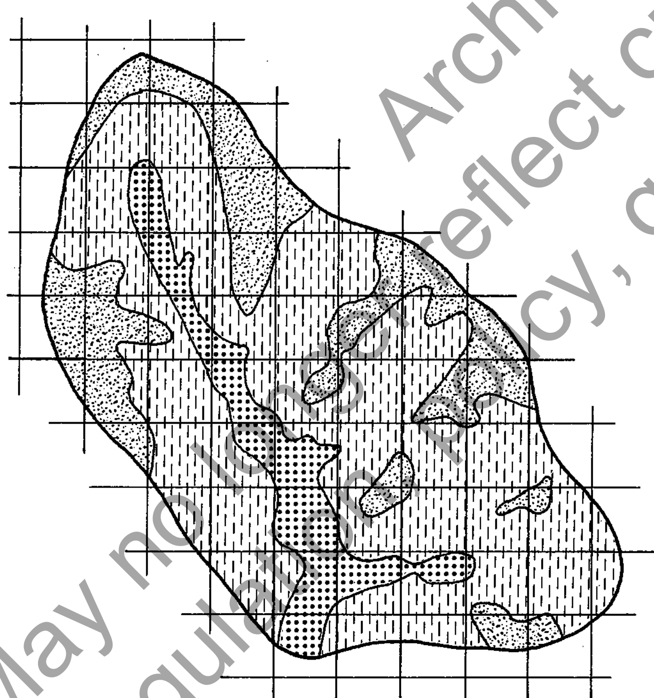


FIG. 3-10 Percentage of impervious areas vs. composite CN's for given pervious area CN's. *(After Soil Conservation Service).*



(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

FIG. 3–9 Steps in determining percentages of soil groups.
(After Soil Conservation Service).

Runoff curve numbers for selected agricultural, suburban and urban land use. (Antecedent moisture condition II, and $I_a = 0.2S$)

Land Use Description	Hydrologic Soil Group			
	A	B	C	D
Cultivated land[1/]: without conservation treatment	72	81	88	91
: with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
: good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or forest land: thin stand, poor cover, no mulch	45	66	77	83
good cover[2/]	25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential:[3/]				
Average lot size — Average % 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
Paved with open ditches	81	89	92	94
Gravel	76	85	89	91
Dirt	72	82	87	89

1/ For a more detailed description of agricultural land use curve numbers refer to S.C.S. National Engineering Handbook, Sec. 4, Hydrology, Chap. 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.

TABLE 3-4

(After Soil Conservation Service)

Compute the runoff curve number for a 1000-acre (404.7 hectares) watershed. The hydrologic soil group is 50 percent B and 50 percent C interspersed throughout the watershed. The land use is:

- 40% residential area that is 30% impervious
- 12% residential area that is 65% impervious
- 8% paved roads with open ditches
- 10% paved roads with curbs and storm sewers
- 16% open land with 50% fair cover and 50% good cover
- 14% parking lots, plazas, schools, etc. (all impervious)

Using Table 3-4 and Fig. 3-10, display the data given and compute the runoff curve number.

	Hydrologic Soil Group					
		B			C	
Land Use	Pct.	CN	Product	Pct.	CN	Product
Residential (30% impervious)	20	72	1,440	20	81	1,620
Residential (65% impervious)	6	85	510	6	90	540
Roads with open ditches	4	89	356	4	92	368
Roads with curbs and sewers	5	98	490	5	98	490
Open Land:						
Fair cover	4	69	276	4	79	316
Good cover	4	61	244	4	74	296
Parking Lots, plazas, etc.	7	98	686	7	98	686
	50		4,002	50		4,316

Thus

$$\text{Weighted CN} = \frac{4{,}002 + 4{,}316}{100} = 83.18 \text{ (use 83)}$$

3.3.5.3 SCS Mass Runoff Equation

Fig. 3-11 (Ref. 3-24) shows schematically, curves of accumulated storm rainfall P, runoff Q and infiltration plus initial abstraction ($F + I_a$). The initial abstraction consists principally of interception and surface storage all of which occur before runoff begins. For convenience in estimating runoff, initial abstraction includes all storm rainfall occurring before surface runoff begins.

For the simpler storm the relation between rainfall, runoff and retention (the rain not converted to runoff) at any point on the mass curve, can be expressed as

$$\frac{F}{S} = \frac{Q}{P_e} \qquad \text{..............(3-6)}$$

where F is the infiltration or actual retention occurring after runoff begins in inches, S is the potential retention in inches or mm, Q is the actual direct runoff in inches or metres and P_e is the potential runoff or effective storm rainfall (storm rainfall, P, minus the initial abstraction) in inches or mm. With $F = P_e - Q$, equation 3-6 can be written as

$$Q = \frac{P_e^2}{P_e S} \qquad \text{.............(3-7)}$$

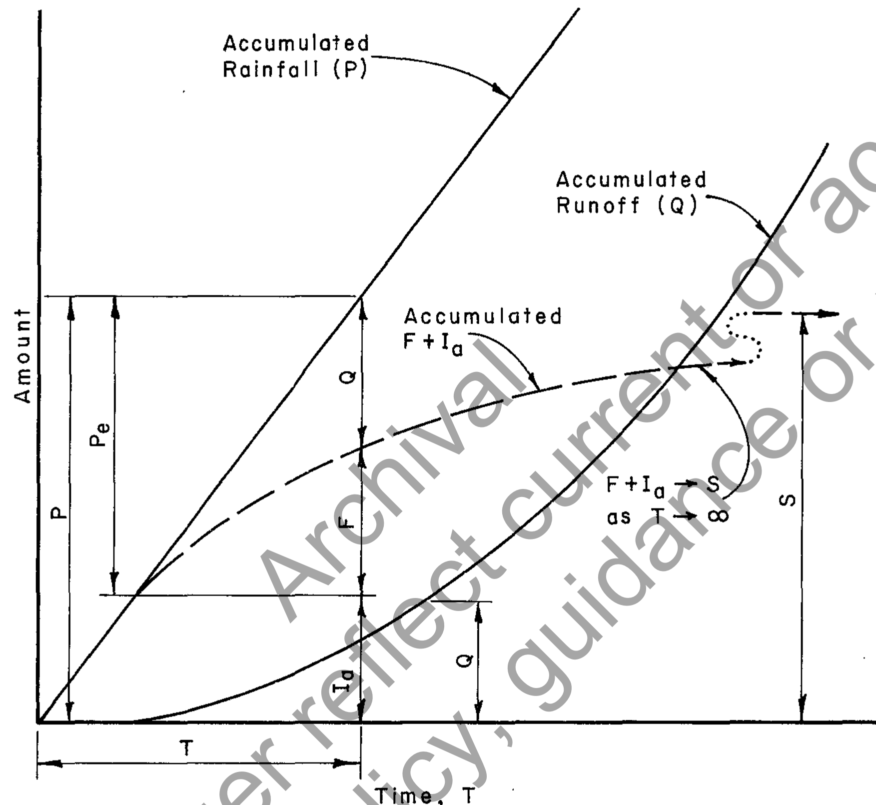


FIG. 3-11 Schematic curves of accumulated rainfall (P), runoff (Q), and infiltration plus initial abstraction ($F + I_a$) showing the relation expressed by equation 3-9. *(After Soil Conservation Service).*

The initial abstraction (I_a) in inches or mm estimated from an empirical relation based on SCS data from small watersheds is

$$I_a = 0.2S \qquad \text{................(3-8)}$$

Substituting, develop the basic SCS equation

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad \text{................(3-9)}$$

The potential retention S in inches or mm is related to the soil and cover conditions of a watershed. This, in turn, is related to the runoff curve number by the equation

$$CN = \frac{1000}{S + 10} \qquad \ldots\ldots\ldots\ldots\ldots(3\text{-}10)$$

from which

$$S = \frac{1000K}{CN} - 10 \qquad \ldots\ldots\ldots\ldots\ldots(3\text{-}11)$$

The coefficient K is one (1) for English units, for metric units it is given by the equation

$$K = \frac{(S + 10)CN}{1000} \qquad \ldots\ldots\ldots\ldots\ldots(3\text{-}11a)$$

with S in metres.

The basic runoff equation is solved in Table 3-5 (Ref. 3-24) for a range of curve numbers and rainfall depths in inches with interpolation feasible for intermediate values of either factor. The Q thus determined is the effective or net rainfall mass which becomes direct runoff.

From the CN for the watershed under design and the applicable rainfall depth for the chosen design frequency, Table 3-5 and Fig. 3-12 give the runoff depth in inches (a solution to equation 3-9).

For a specific design problem, after having determined the runoff mass, it becomes necessary to develop the hydrograph of runoff.

Having developed the mass effective rainfall which is equivalent to the mass direct runoff, it is necessary to determine its time distribution. For most urban drainage problems it is necessary that the time distribution be at 5-minute or 10-minute intervals. The frequency of the design rainfall to be used for the storm drainage under consideration will have been chosen. From intensity-duration-frequency data, determine the 5-, 10-, 15-minute, etc. rainfall mass values, ascertaining the mass added by each 5 minutes until the total mass equals the previously developed mass net rainfall. Then rearrange the 5- or 10-minute rainfall rates, placing the highest intensity centered about the assumed highest point of the rainfall distribution curve (see Precipitation chapter) with a stepped succession of lesser 5- or 10-minute intensities on either side of the highest, until the proper mass net rainfall is developed.

The resulting time-distribution of the net or effective rainfall is then ready for translation to a hydrograph of runoff.

3.3.5.4 Synthetic Unit Hydrograph

While it is preferable to derive unit hydrographs from actual rainfall-runoff measurements, the paucity of usable data from urban areas makes it

Runoff depth in inches for selected CN's and rainfall amounts.

Rainfall (inches)	Curve Number (CN)[1/]								
	60	65	70	75	80	85	90	95	98
1.0	0	0	0	0.03	0.08	0.17	0.32	.56	.79
1.2	0	0	0.03	0.07	0.15	0.28	0.46	.74	.99
1.4	0	0.02	0.06	0.13	0.24	0.39	0.61	.92	1.18
1.6	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5	0.17	0.30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.33	0.51	0.72	0.96	1.25	1.59	1.98	2.45	2.78
4.0	0.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
5.0	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	1.92	2.35	2.80	3.28	3.78	4.31	4.85	5.41	5.76
7.0	2.60	3.10	3.62	4.15	4.69	5.26	5.82	6.41	6.76
8.0	3.33	3.90	4.47	5.04	5.62	6.22	6.81	7.40	7.76
9.0	4.10	4.72	5.34	5.95	6.57	7.19	7.79	8.40	8.76
10.0	4.90	5.57	6.23	6.88	7.52	8.16	8.78	9.40	9.76
11.0	5.72	6.44	7.13	7.82	8.48	9.14	9.77	10.39	10.76
12.0	6.56	7.32	8.05	8.76	9.45	10.12	10.76	11.39	11.76

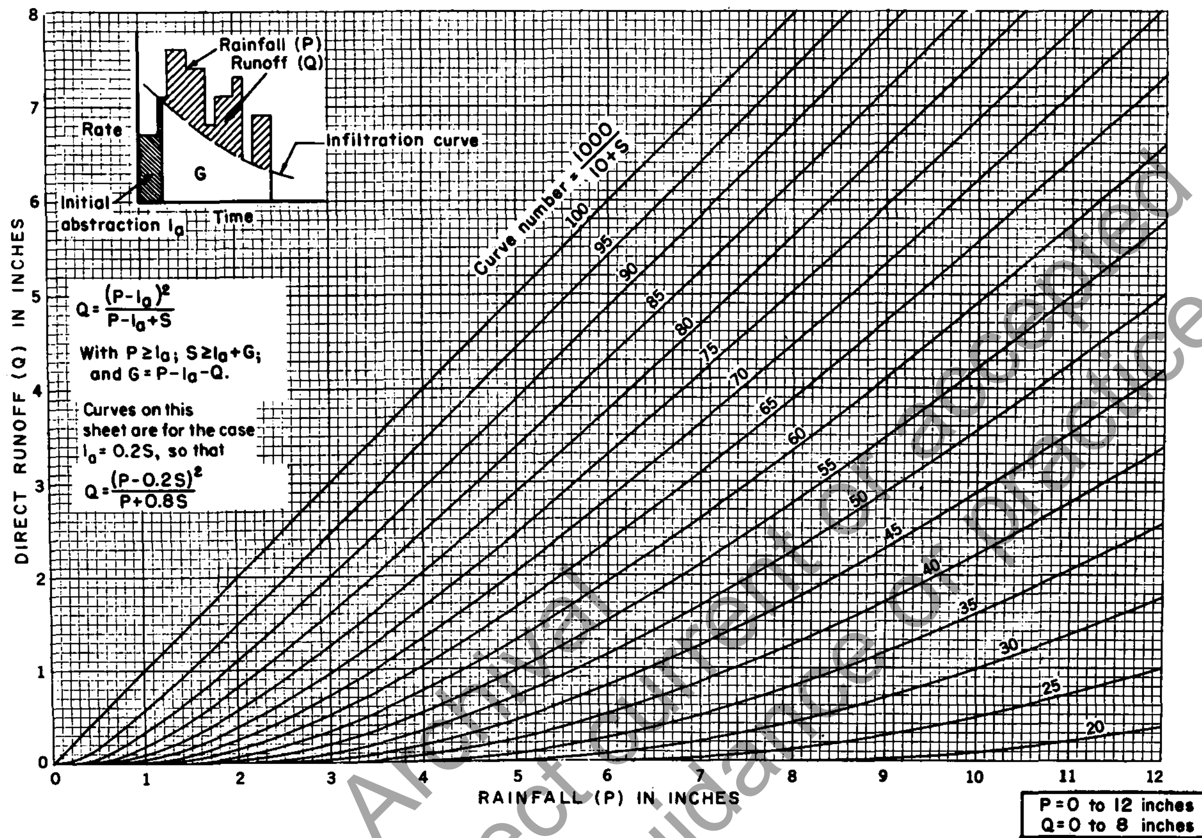
One inch is 25.4mm

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

TABLE 3-5

(After Soil Convervation Service)

SOLUTION OF EQUATION $Q = \frac{(P-0.2S)^2}{P+0.8S}$



ONE INCH is 25.4mm

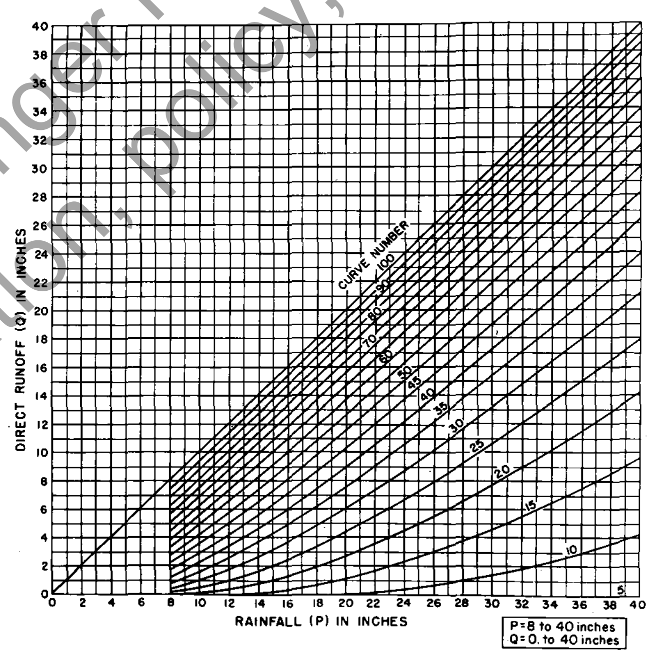


FIG. 3-12 *(After Soil Conservation Service).*

necessary to utilize formulas relating the physical geometry and characteristics of a watershed to the hydrographs resulting from known or assumed rainfall. The synthetic unit hydrograph is a reasonable approach to the determination of runoff.

3.3.5.5 Snyder's Synthetic Unit Hydrograph Relation:

Franklin F. Snyder (Ref.3-25) developed empirical relations correlating the dependent variables of lag time and peak discharge with various physiographic watershed characteristics. Basin lag is defined as the time from the centroid of effective rainfall to the hydrograph peak. This is the preferred definition over the more rigorous time difference between the centroid of effective rainfall and the centroid of runoff. Clearly, the first definition is simpler to apply. Snyder found, from his studies of basins in the Appalachian Mountain region, basin lag t_p in hours could be expressed by equation 3-5.

To develop an equation for peak flow, it is necessary to adopt a standard unit duration of excess rainfall t_r. Snyder found $t_r = t_p/5.5$ a workable assumption. (3-12)

For rains of this duration, the unit hydrograph peak q_p (cfs/sq.mi.) can be obtained by:

$$q_p = \frac{KC_p}{t_p} \qquad \text{.............(3-13)}$$

K is a conversion factor of 640 to give q_p in cubic feet per second per square mile; and equal to 7 to give q_p in cubic metres per second per square kilometre. C_p is a coefficient ranging from 0.56 to 0.69 for Snyder's data which are for natural watersheds. C_p is discussed for urban areas under "Basin Lag" and "The Colorado Urban Hydrograph".

The synthetic approach represented by equations 3-5, 3-12 and 3-13 always gives an initial unit hydrograph with an excess rainfall duration, t_r equal to $t_p/5.5$. With changes in duration of the unit hydrograph, changes in lag time do occur. For other durations t_R (hours), the modified lag becomes

$$t_{pR} = t_1 + 0.25(t_R - t_r) \qquad \text{.............(3-14)}$$

with t_{pR} the adjusted lag time in hours

t_1 the original lag time in hours

This modified lag is then used in equation 3-13.

Successful use of the Snyder synthetic unit hydrograph formulas depends upon a determination of the coefficients C_t and C_p. Where a gauged basin of similar characteristics to those of the problem area is not available for direct determination of the applicable coefficients, it is suggested that the coefficients be chosen as discussed in the subsequent paragraphs.

As additional data from urban areas accumulate, it may well be that an orderly relationship can be found between such major parameters of urbanization as imperviousness and main channel improvement as well as general topographic slopes.

3.3.5.6 The Colorado Urban Hydrograph

In 1969, the Denver Regional Council of Governments issued a two-volume "Urban Storm Drainage Criteria Manual" (Ref. 3-14). In this manual, utilizing the Snyder Synthetic Unit Hydrograph Method, there was developed a method of computing the hydrograph of runoff based upon some rainfall-runoff measurements. In May 1975 and July 1977, the material covering the CUHP (Colorado Urban Hydrograph Procedure) was revised to reflect the analysis of accumulated data between 1967 and 1973 on 19 different urban watersheds in the Denver-Boulder metropolitan region. The statistical analysis involved ninety-six 5-minute hydrographs derived from flood events on those watersheds from the derived unit hydrographs. The Snyder time and peak coefficients C_t and C_p were obtained. The equations follow:

$$C_t = \frac{7.81}{(P_a)^{0.78}} \quad : \quad \text{for } P_a \geqslant 30 \text{ percent} \qquad \text{..........(3-15)}$$

in which P_a is the percentage of the watershed which is impervious.

The equation for C_p is:

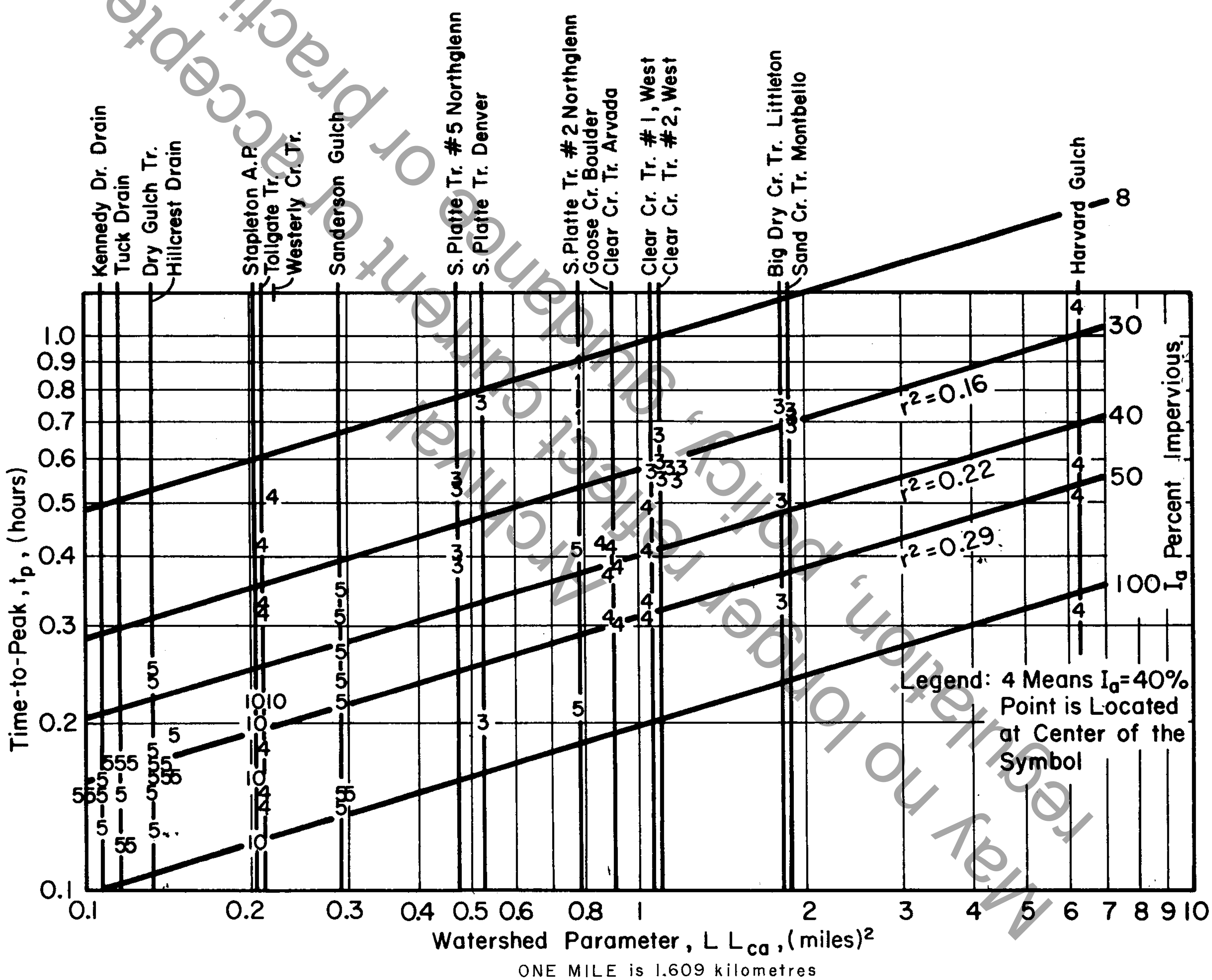
$$C_p = 0.89 C_t^{0.46} \qquad \text{..........(3-16)}$$

Fig. 3-13 from the Denver studies shows the lag or time to peak, T_p, in hours against the watershed parameter LL_{ca} in square miles. The 50% imperviousness line on this figure was drawn first because more data were available over a larger range of the watershed parameter LL_{ca}. The lines for other percentages of imperviousness were subsequently drawn parallel to the 50% line on the lag curve.

The Denver studies state: "The scatter of the data on Fig. 3-13 is attributed to the fact that the runoffs observed during the 1967-1973 period were mainly small floods. Based on unit hydrograph research in this field (Eagleson, Ref. 3-26; Minshall, Ref. 3-28; Schulz and Lopez, Ref. 3-29; and Vansickle, Ref. 3-30), there is a tendency for non-linearity and scatter to exist among the unit hydrograph parameters when the unit hydrographs are derived from small amounts of rainfall excess".

Fig. 3-14 has accompanying it a list of small urban watersheds for which data exists concerning unit hydrographs and such data are shown on Sheet 2 of Fig. 3-14. These non-Denver data support the validity of the curve for equation 3-15.

FIG. 3-13 Lag Curve For Denver Urban Watersheds.
(After Denver Regional Council of Governments).



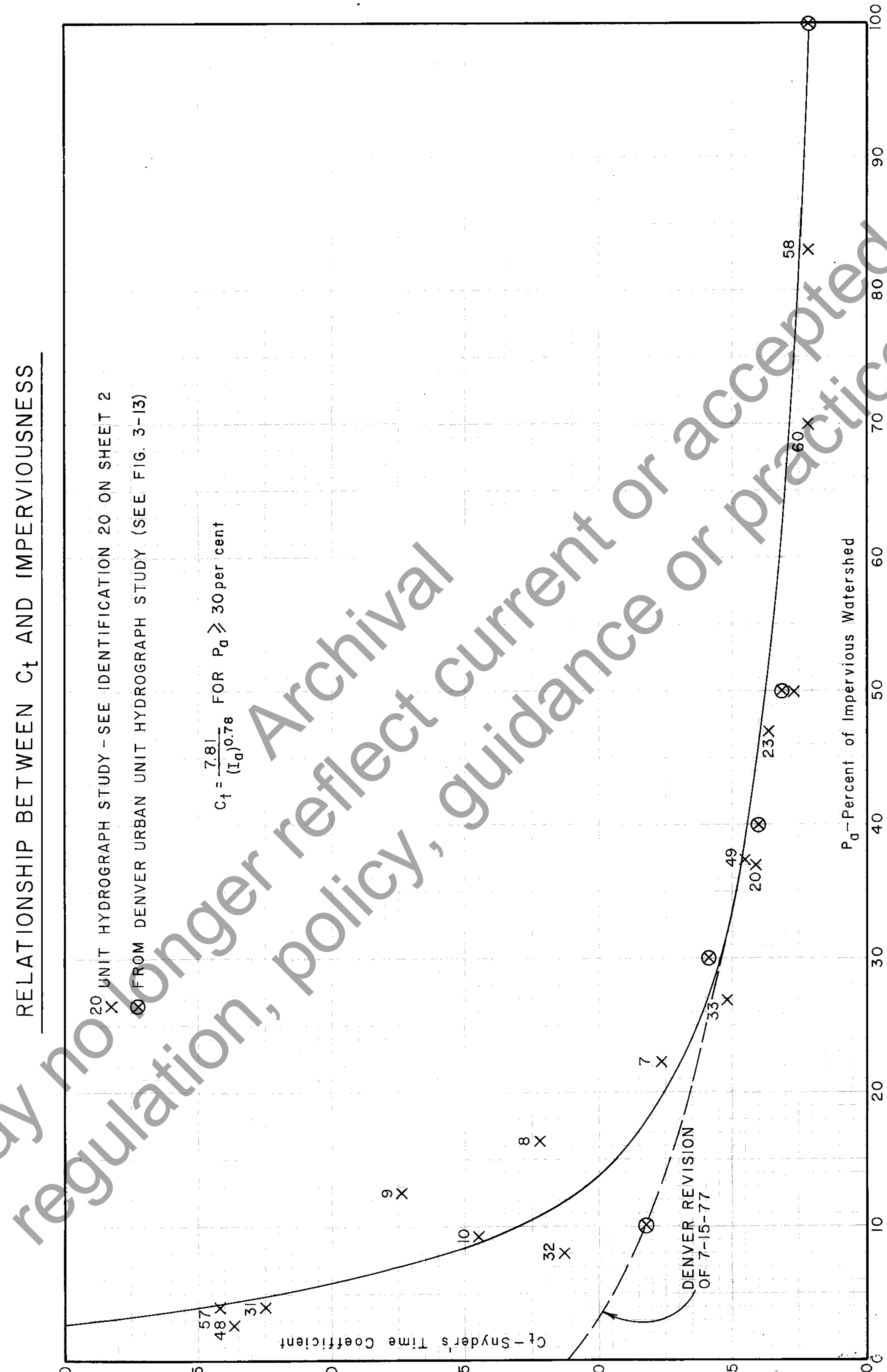


FIG. 3-14 (Sheet 1). *(After Denver Regional Council of Governments).*

UNIT HYDROGRAPHIC STUDY IDENTIFICATION

Ident. Number	Stream	Drainage Area (sq.mi.)	P_a (%)	C_t	E*
7	Wooden Bridge Run, Philadelphia, PA	3.35	22.1	0.76	8.50
9	Poquessing Cr. at Trevose Rd., Philadelphia, PA	5.1	12.5	1.74	12.48
8	Wissahichon Cr. at Bells Mill Rd., Philadelphia, PA	53.6	16.3	1.22	10.76
10	Pennypack Cr. at Pine Rd., Philadelphia, PA	37.9	9.1	1.45	8.12
20	Brushy Cr. at Highway 311, Winston-Salem, NC	0.55	37	0.41	6.85
23	Turtle Cr., Dallas, TX	7.98	47	0.37	7.45
31	Cole Cr. at Guhn Rd., Houston, TX	7.05	4	2.25	6.63
32	Brickhouse Gully at Costa Rica St., Houston, TX	10.5	8	1.13	5.72
33	Waller Cr. at 38th St., Austin, TX	2.31	27	0.51	6.67
48	Anacostia Cr., IL	72.4	2.7	2.36	5.12
49	Boneyard Cr. at Urbana, IL	4.45	37.4	0.45	7.59
57	Salt Fork, West Branch, IL	71.4	4	2.42	7.14
58	Louisville at 17th St., KY	0.22	83	0.22	6.91
59	Louisville, North Trunk Sewer	1.9	50	0.27	5.71
60	Louisville, West Outfall, KY	2.77	70	0.21	5.77
61	Louisville, South Outfall, KY	6.44	48	0.32	6.55
62	Louisville, Southwest Outfall, KY	7.51	33	0.21	3.21
	Stapleton Airport, Denver, CO		100	0.21	7.44
	Clear Creek, Tr.#2 West, CO		30	0.53	8.23
	Clear Creek, Tr.#1 West, CO		40	0.40	7.11
	S. Platte Tr.#2, Northglenn, CO		50	0.31	6.56

$*E = C_t P_a^{0.78}$

ONE SQUARE MILE is 2.59 SQUARE KILOMETRES

FIG. 3-14 (Sheet 2) *(After Denver Regional Council of Governments).*

The Denver studies make the following comments with respect to determination of P_a: "The percent of the impervious watershed, P_a, for an urban watershed in the early stages of planning, may be estimated using the values suggested in Table 3-6. Alternatively, the percent of the impervious watershed could be estimated from aerial photographs of an existing urban watershed having a similar plan of development, adjacent to the planned watershed.

TABLE 3-6

Percent Imperviousness - Range for Various Land-Use Characteristics

Description of Area	Percent Imperviousness
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.20 to 0.50
Multi-family units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.80
Residential (suburban)	0.15 to 0.40
Apartment	0.40 to 0.65
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, Cemeteries	0.05 to 0.25
Playgrounds	0.20 to 0.35
Railroad Yard	0.20 to 0.35
Unimproved	0.10 to 0.30

For estimating C_t: Add 10% for sparsely sewered areas. Subtract 10% for fully sewered areas.

Correct for slope using following equations:

For $S_e < 0.01$ ft/ft ; $C_t = 0.40\, C_{t_o}\, S_e^{-0.2}$.

For $S_e > 0.025$ ft/ft ; $C_t = 0.48\, C_{t_o}\, S_e^{-0.2}$.

For $0.01 \leqslant S_e \leqslant 0.025$ ft/ft ; $C_t = C_{t_o}$.

Where, S_e = Effective main water course computed using downstream 80% of channel

C_{t_o} = The C_t coefficient from Figure 3-14 or equation 3-15.

For estimating C_p: Use the slope corrected C_t with equation 3-16 or Figure 3-15. Subtract 10% for sparsely sewered areas; add 10% for fully sewered areas.

The foregoing instructions for modifying the results of equations 3-15 and 3-16 are made because the constant in each equation varies with the degree of sewering of the area, the steepness or flatness of the topography. The equation's results reflect a partially sewered area of moderate slope.

Equations 3-15 and 3-16 or Figs. 3-14 and 3-15 can be used to estimate C_t and C_p for a specific problem. Note that the figures utilize data from watersheds in Denver, Philadelphia, Winston-Samel (N.C.), Dallas, Houston and Austin (TX), Anacostia Creek (IL), Boneyard Creek (IL), Salt Fork (IL) and five sewered watersheds in Louisville (KY).

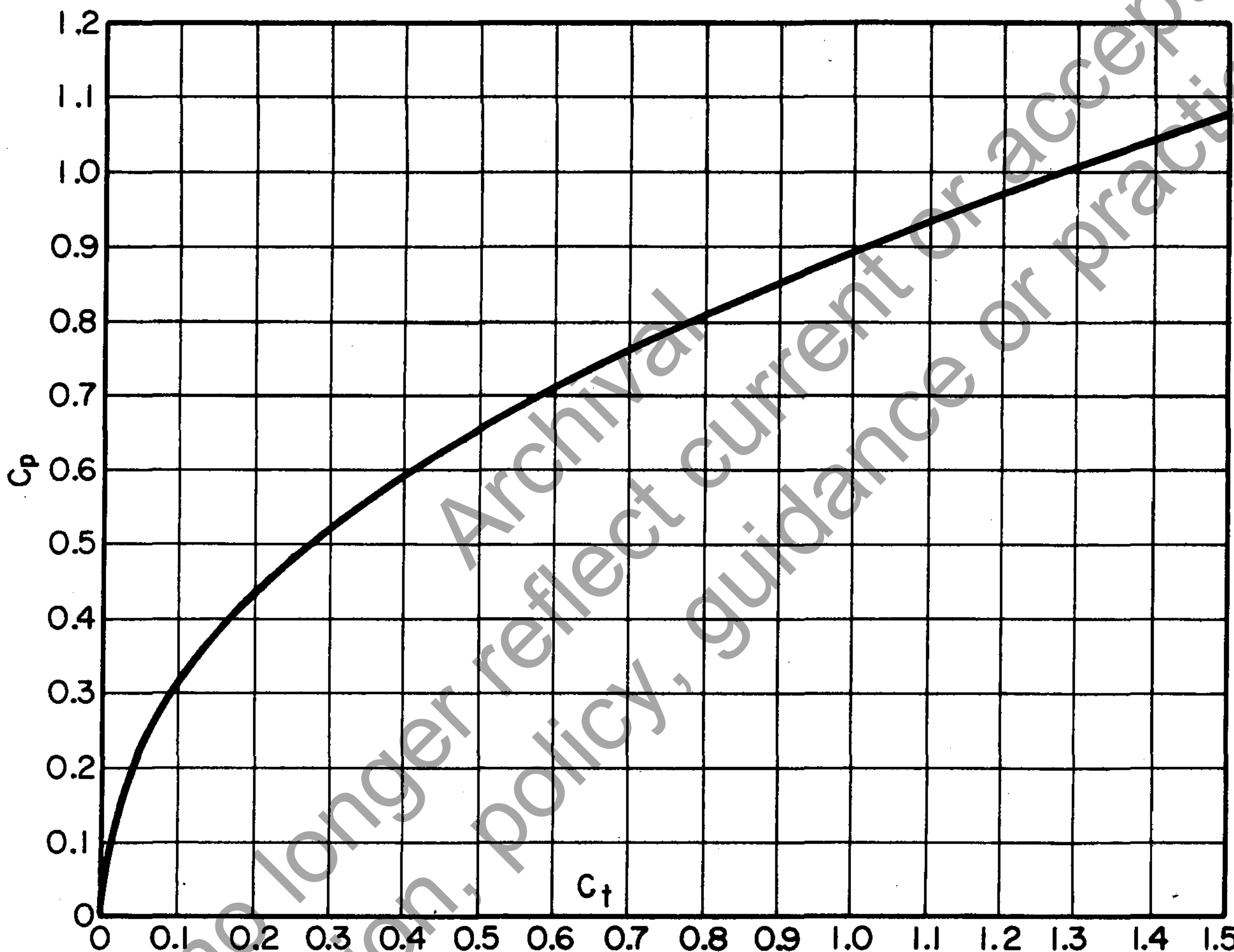


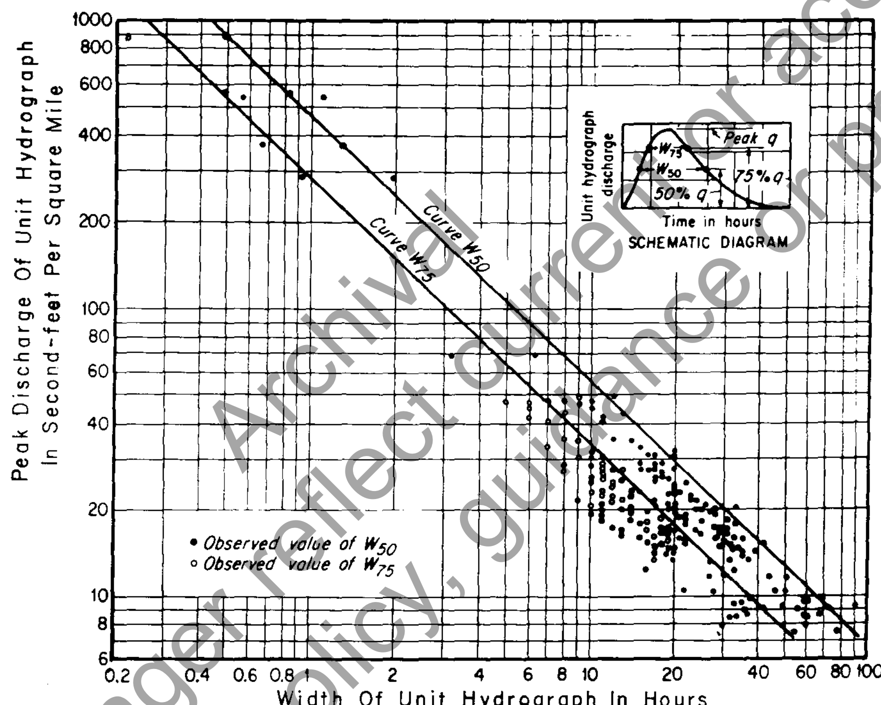
FIG. 3—15 Relationship Between C_p and C_t. *(After Denver Regional Council of Governments).*

3.3.5.7 Unit Hydrograph Shape

The physical characteristics of a watershed determine the shape of the unit hydrograph. From the Snyder equation we can develop the lag time, duration of unit rainfall excess and the peak discharge. The United States Army Corps of Engineers (Ref. 3-31) analyzed a great many unit hydrographs from various parts of the United States and developed the curves shown on Fig. 3-16. These curves give additional assistance in plotting time widths for points on the unit hydrograph located at 50% and 75% of peak discharge. With the computed peak discharge, the time widths can be read from this graph or computed by the following empirical formulae:

$W_{75}=3.352/q_p^{1.08}$ in metric units; $W_{75} = 440/q_p^{1.08}$ English units (3-17)

$W_{50}=5.87/q_p^{1.08}$ in metric units; $W_{50} = 770/q_p^{1.08}$ English units (3-18)



One second-foot is 0.02832 m³/s — One square mile is 2.59 square kilometres

FIG. 3-16 Unit-hydrograph width at 50 and 75 percent of peak flow. *(After U. S. Corps of Engineers).*

In plotting these time widths, it is suggested that in general, the ordinates be proportioned each side of the hydrograph peak in a ratio of about 0.4 to 0.6 with the short time side on the left of the synthetic unit hydrograph peak. The base time for the unit hydrograph can be estimated by multiplying the lag time by 5. This latter multiplier is that used in the SCS dimensionless hydrograph (Ref. 3-23) which is based upon analyses of many studies of experimental plot runoffs as well as actual watershed data.

Another factor utilized in sketching a synthetic unit hydrograph is the fact that total direct runoff amounts to one inch.

Drawing the synthetic unit hydrograph can be done using the peak rate of flow, the time to peak and the time widths at the 75% and 50% of peak to sketch in the computed hydrograph. The area under this, when planimetered, should equal one inch of runoff from the tributary area under study. For most problems, one or two adjustments to the initially sketched hydrograph will bring the planimetered area into close enough agreement with the theoretical total surface runoff.

3.3.5.8 Dimensionless Hydrograph

An alternate method of obtaining a satisfactory unit hydrograph is based upon the SCS dimensionless unit hydrograph (Ref. 3-23).

The dimensionless hydrograph is essentially a unit hydrograph for which the discharge is expressed by the ratio of discharge to the peak discharge as related to the ratio of time to the lag time. The peak rate of flow, the time to peak and the time from beginning of the unit rainfall to peak are computed as indicated in the following example. Then the time and discharge ratios of the SCS dimensionless hydrograph (as given in columns 1 and 3 of Table 3-7) are applied to the appropriate factors to obtain the coordinates of the unit hydrograph given in columns 2 and 4. The mass curve ratios in column 6 are presented for such use as the designer may find desirable. The example given utilizes the dimensionless hydrograph to develop the unit hydrograph for the illustrative example. The choice between the two methods depends upon the designer's preference since the synthetic hydrographs are quantitatively quite similar. The dimensionless hydrograph is slightly thicker in the upper section and somewhat thinner in the lower third. The dimensionless graph eliminates much of the effect of basin shape and the effect of basin size.

There follows a step-by-step description of the development of a design storm hydrograph for an assumed watershed.

3.3.5.9 Example 3-1

The example watershed has the following characteristics:

Area: 544 acres = 0.85 square miles
L = 1.21 miles
L_{ca} = 0.85 miles
Pervious Area = 60%
Impervious Area = 40%
Assume Unit duration = 10 minutes

The unit duration for most developed areas should be 5 or 10 minutes. A general rule is that the duration of unit excess rainfall shall

Computation of Coordinates
for Unit Hydrograph for Use
in Example

Cols. 1 and 3 are S.C.S. Ratios for Dimensionless Unit Hydrograph (Ref. 3-23)

t/T_p Time Ratios	t Time Hours Col. 1 x 0.53	q/q_p Discharge Ratios	q_p Discharges Col. 3 x 750 cfs	Q_o Σq_p	Q_o/Q Mass Curve Ratios
(1)	(2)	(3)	(4)	(5)	(6)
0	0	0	0	0	
0.1	0.053	0.030	23	23	.003
0.2	0.107	0.100	75	98	.012
0.3	0.160	0.190	143	241	.029
0.4	0.213	0.310	233	474	.057
0.5	0.267	0.470	353	827	.099
0.6	0.320	0.660	495	1322	.158
0.7	0.373	0.820	615	1937	.232
0.8	0.426	0.930	698	2635	.315
0.9	0.480	0.990	743	3378	.404
1.0	0.533	1.000	750	4128	.494
1.1	0.586	0.990	743	4871	.582
1.2	0.640	0.930	698	4569	.546
1.3	0.693	0.860	645	5214	.623
1.4	0.746	0.780	585	5799	.693
1.5	0.800	0.680	510	6309	.754
1.6	0.853	0.560	420	6729	.805
1.7	0.906	0.460	345	7074	.846
1.8	0.960	0.390	298	7372	.881
1.9	1.013	0.330	248	7620	.911
2.0	1.066	0.280	210	7830	.936
2.2	1.173	0.207	155	7985	.956
2.4	1.280	0.147	110	8095	.968
2.6	1.385	0.107	80	8175	.977
2.8	1.495	0.077	58	8233	.984
3.0	1.600	0.055	40	8273	.989
3.2	1.710	0.040	30	8303	.993
3.4	1.820	0.029	22	8325	.995
3.6	1.920	0.021	16	8341	.997
3.8	2.020	0.015	11	8352	.9985+
4.0	2.132	0.011	8	8360	.9995+
4.5	2.400	0.005	4	8364	1.0000
5.0	2.665	0.000	0	8364	

One cfs is 0.02837m^3s

$\Sigma q_p = 8364$

TABLE 3-7

preferably be about 0.2 of the time from the center of the excess rainfall to the unit hydrograph peak but, in general, it shall not exceed 0.25 of the lag time. Another consideration is the plotting accuracy of the final hydrograph; if the interval (the unit time) is long, fewer points are calculated on the hydrograph. If too few points are determined to draw a good hydrograph, a shorter interval should be chosen.

3.3.5.10 Step-by-Step Computations

Determine 10-year design runoff hydrograph from basin assumed in Denver metropolitan area.

Step 1: Obtain C_t using equation 3-15

$$C_t = \frac{7.81}{(40)^{0.78}} = 0.44$$

Step 2: Calculate t_p using equation 3-5

$$t_p = C_t(LL_{ca})^{.3} = 0.44(1.21 \times 0.85)^{0.3} = 0.44 \text{ hours (or 27 minutes)}$$

Step 3: Calculate C_p using equation 3-16

$$C_p = 0.89(C_t)^{0.46} = 0.89 \times (.44)^{.46} = 0.61$$

Step 4: Calculate q_p using equation 3-13

$$q_p = \frac{640C_p}{t_p} = \frac{640(0.61)}{.44} = 887 \text{ cfs/sq.mi.}$$

Step 5: Determine

$$Q_p = q_pA = 887(.85) = 754, \text{ say } 750 \text{ cfs}$$

Step 6: Calculate the time to peak (T_p) from beginning of rainfall

$$T_p = 60t_p + \frac{t_u}{2} = 27 + \frac{10}{2} = 32 \text{ minutes (or 0.53 hr.)}$$

with t_u being the unit rainfall interval.

Step 7: Using the dimensionless unit hydrograph ratios from Table 3-7 and the unit hydrograph peak rate of 750 cfs as determined in Step 5, develop the unit hydrograph discharges for the unit duration T_p of 32 minutes or 0.533 hours. Column 2 of Table 3-7 is the time ratios in column 1 multiplied by 0.533. The dimensionless hydrograph and mass curve are given by the ratios of columns 1, 3 and 6. Fig. 3-17 is the unit hydrograph for the conditions of the example.

COMPARISON OF UNIT HYDROGRAPHS DEVELOPED BY TWO METHODS

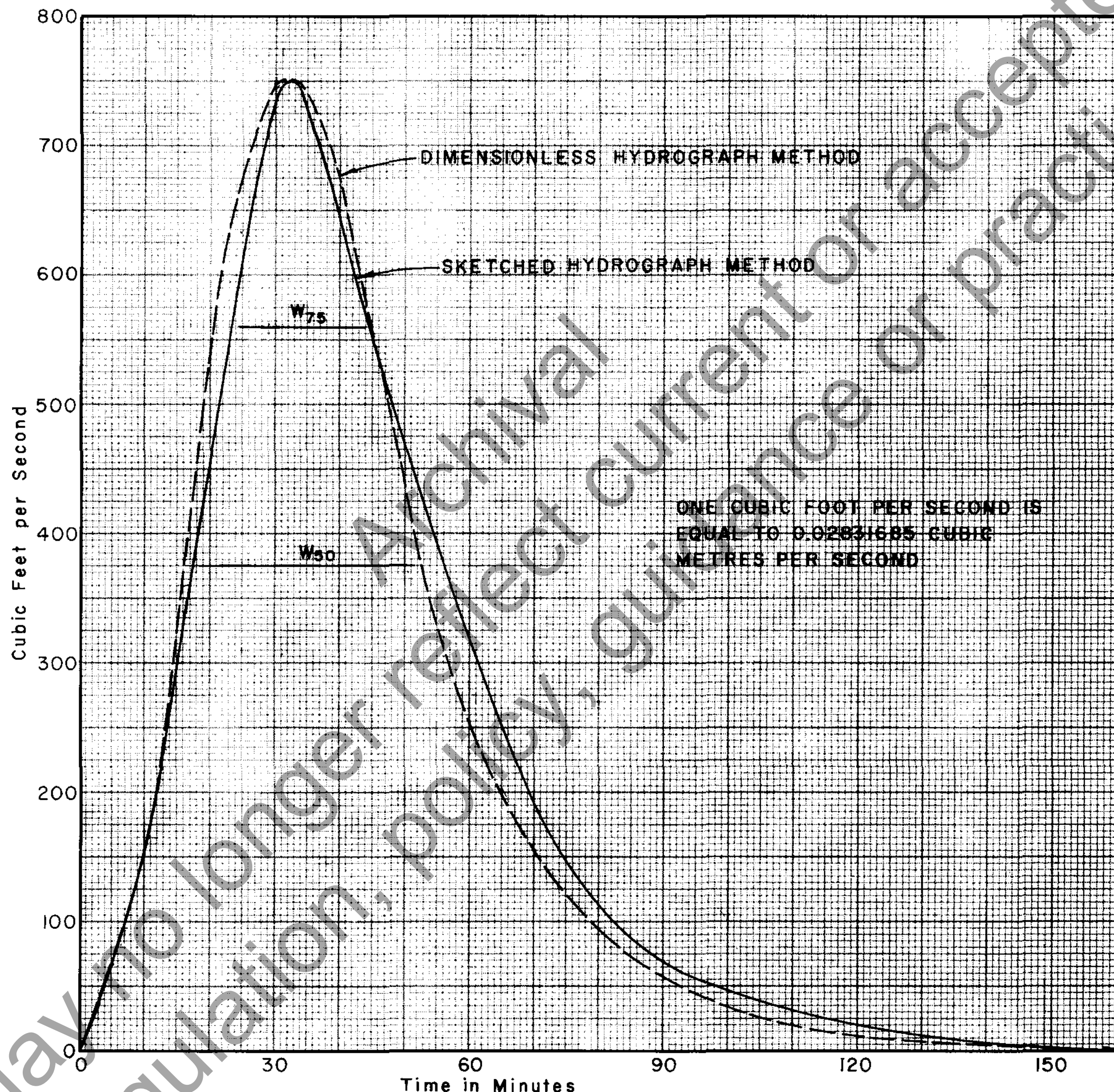


FIG. 3-17

Step 8: Calculate the mass or total effective net rainfall (equal to the surface runoff under the design hydrograph) as follows:

a. From the rainfall intensity-duration-frequency curves for the Denver metropolitan area, obtain the rainfall values in inches for the 10-year frequency for the 10-,20- and 30-minute etc. durations and enter in column 2 of Table 3-8.

b. In column 3 enter the incremental precipitation for each 10-minute period.

c. Exercising judgement, rearrange the 10-minute rainfalls to achieve a synthetic precipitation pattern. For most of the United States, the most intense unit rainfall for urban areas can be placed close to the 30- or 40-minute interval of the storm with increasing intensity increments prior to the peak and decreasing ones after the peak. Avoid rearrangements that involve high-low-high sequences.

Step 9: For the pervious area there will be an infiltration abstraction for each time period. The Denver Drainage Criteria Manual gives an arbitrary infiltration rate to be used of 1/2-inch per hour. Because of the unknown temporal and spatial variation of the input rainfall as well as of the watershed properties, it is impractical to make a more precise approximation than the assumption that the 1/2-inch per hour loss rate involves 0.08 inch in each 10-minute period.

If a specific design area has data on the actual infiltration characteristics of its soils, such information should be used insofar as feasible. Some design problems justify field testing for specific infiltration rates. The United States Geological Survey in 1963 published "A Field Method for Measurement of Infiltration" (Water Supply Paper 1544-F, Ref. 3-32) which discusses several acceptable methods. The simplest involves driving into the soil an 18-inch diameter infiltration ring. Water is placed into the ring and the drop in water level measured at various time intervals. Additional water is added from time to time and readings are continued. The tests should continue until the infiltration rate is virtually constant.

Step 10: The total depression storage must be estimated and entered in column 6 for the pervious areas and in column 9 for the impervious areas. From the prior discussion of depression storage, it is assumed for this example that the total pervious area depression storage will be 0.25-inch; for the impervious area 0.1 inch.

For the pervious area the first rainfall available for depression storage is 0.02-inch (.10-inch rainfall less 0.08-inch infiltration) in the 20-minute time. In the following 10 minutes, 0.05-inch becomes available for depression storage (0.13-inch rainfall less 0.08-inch infiltration). And the subsequent 10-minute rainfall (0.89-inch) supplies enough excess

DETERMINATION OF EFFECTIVE RAINFALL

TIME (MIN)	PRECIPITATION TOTAL IN.	PRECIPITATION INCREMENTAL IN.	PRECIPITATION REARRANGED INCREMENTAL IN.	PERVIOUS AREA 60% MAXIMUM INFILTRATION IN.	PERVIOUS AREA 60% DEPRESSION STORAGE IN.	PERVIOUS AREA 60% EFFECTIVE PRECIP. IN.	PERVIOUS AREA 60% 60% EFFECTIVE PRECIP. IN.	IMPERVIOUS AREA 40% DEPRESSION STORAGE IN.	IMPERVIOUS AREA 40% EFFECTIVE PRECIP. IN.	IMPERVIOUS AREA 40% 40% EFFECTIVE PRECIP. IN.	TOTAL AVG. EFFECTIVE PRECIP. IN.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0	0	0	0	0	0	0	0	0	0	0	0
10	0.89	0.89	0.05	0.05	0	0	0	0.05	0	0	0
20	1.18	0.29	0.10	0.08	0.02	0	0	0.05	0.05	0.02	0.02
30	1.40	0.22	0.13	0.08	0.05	0	0		0.13	0.05	0.05
40	1.53	0.13	0.89	0.08	0.18	0.63	0.38		0.89	0.36	0.74
50	1.64	0.11	0.29	0.08	0	0.21	0.13		0.29	0.11	0.24
60	1.74	0.10	0.22	0.08	0	0.14	0.08		0.22	0.09	0.17
70	1.82	0.08	0.11	0.08	0	0.03	0.02		0.11	0.04	0.06
80	1.89	0.07	0.08	0.08	0	0	0		0.08	0.03	0.03
90	1.95	0.06	0.07	0.07	0	0	0		0.07	0.03	0.03
100	2.00	0.05	0.06	0.06	0	0	0		0.06	0.02	0.02
110	2.04	0.04	0.04	0.04	0	0	0		0.04	0.02	0.02
120	2.08	0.04	0.04	0.04	0	0	0		0.04	0.02	0.02
TOTALS			2.08	0.82	0.25	1.01	0.61	0.10	1.98	.79	1.40

One inch is 25.4mm

TABLE 3-8

over the 0.08-inch infiltration to satisfy the remaining 0.18-inch of depression storage.

The depression storage on the impervious area is assumed satisfied 0.05-inch in each of the first two 10-minute periods.

Step 11: Having entered in Table 3-8 the infiltration and depression storage abstractions for the pervious and impervious areas, the effective precipitation for each 10-minute period is computed and entered into columns 7 and 10. For the 60% pervious area, each 10-minute pervious area effective rainfall is multiplied by 0.60 and the weighted effective rainfall entered in column 8. Similar weighting for the 40% impervious area is entered in column 11.

The sum of the net precipitations in columns 8 and 11 is entered in column 12 as the total average effective precipitation.

Step 12: As a check on the overall validity of the determination of effective precipitation, use the SCS equation

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad \ldots\ldots\ldots\ldots (3\text{-}19)$$

Table 3-8 gives P = 2.08 inches
Pervious: 2.08 - 1.01 = 1.07 x .60 = 0.642 inches
Impervious: 2.08 - 1.98 = 0.10 x .40 = <u>0.040</u> inches
S = 0.682 inches

$$\text{SCS calculated:} \quad Q = \frac{(2.08 - 0.2 \times .682)^2}{2.08 + 0.8 \times .682} = 1.44 \text{ inches}$$

Table 3-8 gives Q as 1.40 which is within less than 3% of that calculated with the SCS formula. This indicates that the assumptions concerning infiltration and depression storage are reasonable and the total effective precipitation is realistic.

Step 13: Table 3-9 gives the computations involved in developing the hydrograph for the problem area, utilizing the unit hydrograph as determined in Step 7. In column 2, place the 10-minute ordinates of the unit hydrograph (from Table 3-8 and Fig. 3-17). Across the top under "Excess Precipitation in Inches" place at the top of columns 3 to 13 the effective rainfall amounts determined in column 12 of Table 3-8.

Multiply each 10-minute unit hydrograph amount in column 2 by the excess precipitation placed at the top of each of columns 3 to 13. Record the products in each column as shown, starting each vertical set of numbers one time interval below the start of the prior set. With all vertical columns 3 to 13 filled in, total each horizontal line's values and record in column 14. This column then gives in sequence the 10-minute ordinates of the hydrograph resulting from a 10-year 2-hour storm on the example watershed.

EXAMPLE OF DETERMINATION OF STORM HYDROGRAPH

TIME (MIN)	UNIT HYDROGRAPH (CFS)	EXCESS PRECIPITATION IN INCHES											HYDROGRAPH (CFS)
		.02	.05	.74	.24	.17	.06	.03	.03	.02	.02	.02	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
0													
10	152	3.0											
20	528	10.6	7.6										
30	748	15.0	26.4	112.5									154
40	676	13.5	37.4	390.7	36.5								478
50	450	9.0	33.8	553.5	126.7	25.9							749
60	253	5.1	22.5	500.2	179.5	89.7	9.1						806
70	155	3.1	12.7	333.0	162.2	127.2	31.6	4.6					674
80	95	1.9	7.7	187.2	108.0	115.0	44.8	15.8	4.6				485
90	58	1.2	4.8	114.7	60.7	76.5	40.5	22.4	15.8	3.0			340
100	35	0.7	2.9	70.3	37.2	43.0	27.0	20.3	22.4	10.6	3.0		237
110	21	0.4	1.7	42.9	22.8	26.4	15.2	13.5	20.3	15.0	10.6	3.0	172
120	11	0.2	1.1	25.9	13.9	16.2	9.3	7.6	13.5	13.5	15.0	10.6	127
130	7	0.1	0.6	15.5	8.4	9.9	5.7	4.6	7.6	9.0	13.5	15.0	90
140	4	0.1	0.4	8.1	5.0	5.9	3.5	2.8	4.6	5.1	9.0	13.5	58
150	1	0.0	0.2	5.2	2.6	3.6	2.1	1.7	2.8	3.1	5.1	9.0	35
160	0		0.1	3.0	1.7	1.9	1.3	1.0	1.7	1.9	3.1	5.1	21
170			0.0	0.7	1.0	1.2	0.7	0.6	1.0	1.2	1.9	3.1	11
180				0.0	0.2	0.7	0.4	0.3	0.6	0.7	1.2	1.9	6
190					0.0	0.0	0.2	0.2	0.3	0.4	0.7	1.2	3
200						0.0	0.0	0.1	0.2	0.2	0.4	0.7	2
210							0.0	0.0	0.1	0.1	0.2	0.4	1
220								0.0	0.0	0.1	0.1	0.1	0.3
230									0.0	0.0	0.1	0.1	0.2
240										0.0		0.0	0.0
												0.0	0.0

One inch is 25.4mm

TABLE 3-9

3.3.5.11 Alternate Procedure Following Step 7

Step 7A: In lieu of using the dimensionless unit hydrograph, develop the unit hydrograph as follows:

From Step 6: T_p = 32 minutes (or 0.53 hr.)

From Step 5: q_p = 750 cfs

Assumed duration of unit excess rainfall = 10 m = 0.17 hr.

Step 8A: From equations 3-17 and 3-18:

$$W_{75} = 440/750^{1.08} = 0.345 \text{ hr. } (20.7 \text{ min.})$$

$$W_{50} = 770/750^{1.08} = 0.605 \text{ hr. } (36.3 \text{ min.})$$

Step 9A: Plot on rectangular coordinates: the peak flow of 750 cfs at time of 32 minutes after start of excess rainfall; at 75% of the peak or at 562.5 cfs plot points at (32 - .4 x 20.7) or 23.7 minutes and (23.7 + 20.7) or 44.4 minutes; at 50% of the peak or 375 cfs plot points at (32 - .4 x 36.3) or 17.5 minutes and (17.5 + 36.3) or 53.8 minutes.

Assume hydrograph will terminate at five times the time from beginning of excess rain to the peak or 5 x 32 = 160 minutes.

Step 10A: Sketch the unit hydrograph as shown on Fig. 3-17. Planimeter the area under the sketched hydrograph which should equal 1 inch of runoff from the 544 acres of the example watershed which is 1,974,720 cubic feet. The planimetered area is 1,970,300 cubic feet. This is remarkably close agreement. The dimensionless-based hydrograph has a satisfactory planimetered area of 1,908,600 cubic feet. If the check had been off significantly (more than 5% $\pm$) the recession of the hydrograph could be modified and the enveloped area again planimetered until acceptable agreement was reached.

3.3.6 The Isochronal Method

For small watersheds an alternate method for determining the hydrograph for a specific area utilizes a time-area diagram and the net or effective rainfall pattern for intervals of the same unit duration as assumed in the time-area distribution.

The time-area histogram (Fig. 3-18) for a watershed is determined by estimating lines of equal travel time (isochrones) from a design point in a watershed and plotting the areas between isochrones against time. The time-area diagram is a representation of the time distribution of an instantaneous input of rainfall excess. For example, for an instantaneous input of 1 inch of storm excess, the summation of the areas multiplied by the appropriate conversion coefficient would equal 1 inch of total runoff, hence it is comparable to an instantaneous unit hydrograph (IUH).

The Denver studies make the following comments with respect to determination of P_a: "The percent of the impervious watershed, P_a, for an urban watershed in the early stages of planning, may be estimated using the values suggested in Table 3-6. Alternatively, the percent of the impervious watershed could be estimated from aerial photographs of an existing urban watershed having a similar plan of development, adjacent to the planned watershed.

TABLE 3-6
Percent Imperviousness - Range for
Various Land-Use Characteristics

Description of Area	Percent Imperviousness
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.20 to 0.50
Multi-family units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.80
Residential (suburban)	0.15 to 0.40
Apartment	0.40 to 0.65
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, Cemeteries	0.05 to 0.25
Playgrounds	0.20 to 0.35
Railroad Yard	0.20 to 0.35
Unimproved	0.10 to 0.30

For estimating C_t: Add 10% for sparsely sewered areas. Subtract 10% for fully sewered areas.

Correct for slope using following equations:

For $S_e < 0.01$ ft/ft ; $C_t = 0.40\ C_{t_o}\ S_e^{-0.2}$.

For $S_e > 0.025$ ft/ft ; $C_t = 0.48\ C_{t_o}\ S_e^{-0.2}$.

For $0.01 \leq S_e \leq 0.025$ ft/ft ; $C_t = C_{t_o}$.

Where, S_e = Effective main water course computed using downstream 80% of channel

C_{t_o} = The C_t coefficient from Figure 3-14 or equation 3-15.

For estimating C_p: Use the slope corrected C_t with equation 3-16 or Figure 3-15. Subtract 10% for sparsely sewered areas; add 10% for fully sewered areas.

The foregoing instructions for modifying the results of equations 3-15 and 3-16 are made because the constant in each equation varies with the

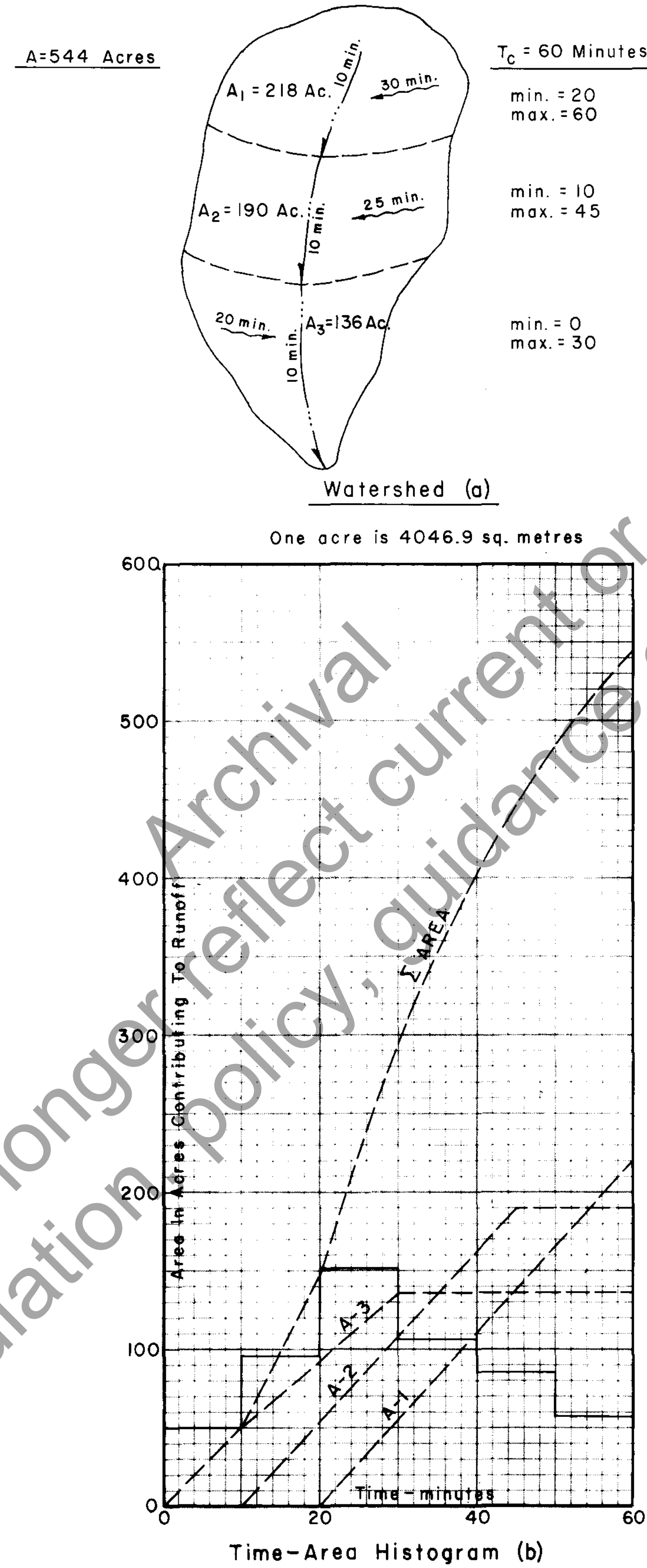


FIG. 3-18 Isochronal example Watershed and Time-Area Histogram

The latter has been defined as the unit hydrograph resulting from the assumption that the duration of the effective precipitation becomes infinitesimally small. Put another way, for an IUH, the effective precipitation is applied to the drainage basin in zero time. This is clearly a fictitious situation and a concept used in hydrograph analysis. It can be demonstrated (Ref. 3-8) that the time-area diagram can result in an estimated instantaneous unit hydrograph. From this there can be developed a finite time unit hydrograph.

This isochronal method assumes that the translation of the watershed response to rainfall is a function of watershed travel time. Given a hydrograph or rainfall histogram of various rainfall excesses having durations equal to the time interval between isochrones, the flow at the basin outlet can be estimated by converting the time-area histogram to time-discharge diagrams, lagging, superposing and adding.

The determination of a time pattern of gross rainfall and from it, a histogram of rainfall excess at intervals of 5 or 10 minutes (as the circumstances of a specific problem may suggest) is discussed in Chapters 2 and 3.

To develop a time-area histogram for a specific watershed it is desirable where physiographic data are available, to estimate channel and overland flow velocities such that there can be drawn on a topographic map of the basin, lines of equal travel time from the basin outlet (the point under design). Generally, the time contours are likely to be very irregular since they are affected by surface slopes, surface irregularities, location of inlets, length and slope of closed sewers and other factors. For practical reasons it is deemed sufficient to assume the entire basin as approximating a regular geometric figure such as a square, rectangle, triangle or sector. The time zones would be assumed as areas of equal width between arcs of concentric circles (centered at the design point or outlet). Fig. 3-19 from Ref. 3-36 shows the time-area curves for various geometric-shaped watersheds assuming constant velocity. Most urban watersheds served by closed drains approximate rectangles in effective shape. For the usually small watersheds involved in urban highway drainage, the upper reaches with slightly steeper gradients are not sufficiently time influential to change the time-area relationships of Fig. 3-19.

Where field or map data are unavailable it is recommended that the total area of the specific problem be assumed to be time-area distributed as shown for the rectangle on Fig. 3-19.

The isochronal method is illustrated by using the same example used to demonstrate the development of the unit hydrograph.

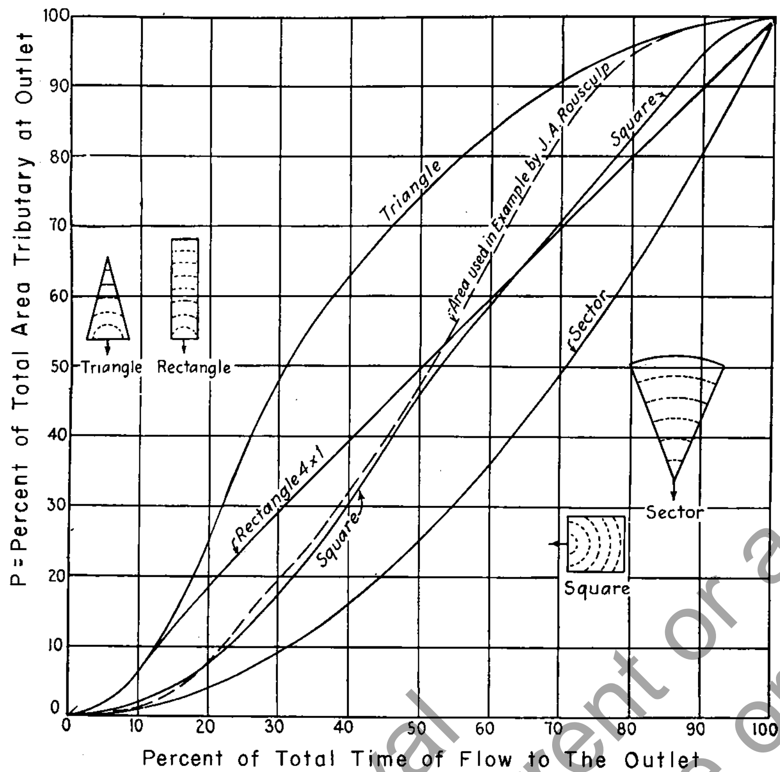


FIG. 3-19 Ratio of area tributary at outlet to time of flow for regular areas with constant velocity.
From: Sewerage and Sewage Disposal - A textbook by Leonard Metcalf and Harrison P. Eddy. Copyright 1922, 1930 by the Mc Graw-Hill Book Co., Inc. Used with permission of Mc Graw-Hill Book Company.

3.3.6.1 Example 3-2

Given: 544 acres
Pervious area 60%
Impervious area 40%
Effective rainfall at 10-minute intervals assumed same as developed in Table 3-8; based upon 10-year rainfall in Denver, Colorado.
Time-area distribution as shown on Fig. 3-18.

Step 1: Enter consecutive 10-minute intervals in column 1 of Table 3-10.

Step 2: Enter in column 2 the 10-minute effective rainfall rates in inches per hour.

Step 3: Enter at the top of columns 3 through 8 inclusive, the 10-minute incremental areas in acres.

Step 4: Multiply the effective rainfall rates in column 2 by each incremental acreage in columns 3 through 8, offsetting each column's products by one time interval beyond that of the previous column.

ISOCHRONAL COMPUTATION OF RUNOFF HYDROGRAPH

Time Min.	Effect. i iph	Incremental Acres 50	95	151	105	85	58	Hydrograph cfs
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0	0						0
10	0.12	6	0					6
20	0.30	15	11	0				26
30	4.44	222	29	18	0			269
40	1.44	72	422	45	13	0		552
50	1.02	51	137	670	32	10	0	900
60	0.36	18	97	217	466	26	7	831
70	0.18	9	34	154	151	377	17	742
80	0.18	9	17	54	107	122	257	566
90	0.12	6	17	27	38	87	84	259
100	0.12	6	11	27	19	31	59	153
110	0.12	6	11	18	19	15	21	90
120	0	0	11	18	13	15	10	67
130			0	18	13	10	10	51
140				0	13	10	7	30
150					0	10	7	17
160						0	7	7
170							0	0

TABLE 3-10

One acre is $4046.9m^2$; One cfs is $0.0283m^3/s$

Step 5: Accumulate each horizontal line from column 3 to 8, putting total in column 9.

Step 6: Plot the hydrograph values of column 9 against time on Fig. 3-20.

Comparing the hydrographs determined by the unit hydrograph method and the isochronal method suggests that for the example, the differences are within acceptable limits. If the physiographic and hydraulic data are available, the isochronal method appears preferable, particularly for very small areas (less than 100 acres). For especially important circumstances, it might be desirable to develop the hydrograph for design purposes by each of the two suggested methods. A careful evaluation of each would suggest which of the two results should be used in the further design work (storage, pumping station design, etc.).

COMPARABLE HYDROGRAPHS OBTAINED BY THE UNIT HYDROGRAPH AND ISOCHRONAL METHODS

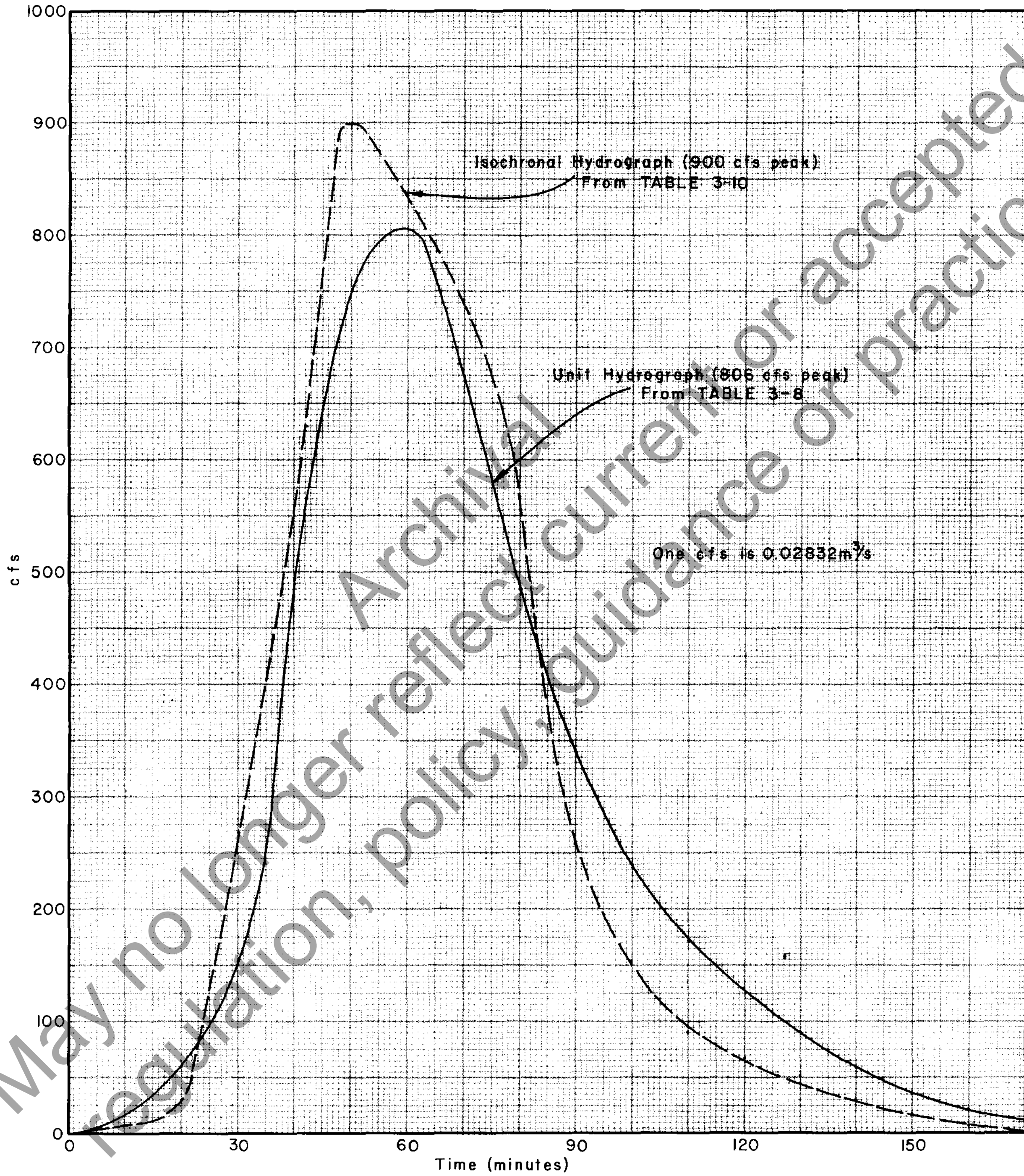


FIG. 3–20

3.4 Summary of Significant Design Information in Chapter 3

1. Improvement in the rational method for the determination of peak design flows can be accomplished by more realistic determination of the C value. Instead of the usually erroneous assumption that the relatively short time of concentration associated with urban drainage occurs at the beginning of a storm, it is recommended that the critical time be assumed to be between the one-third to two-thirds points of a longer storm assumed to encompass a total time of one to three hours. Since the C values increase rapidly in the first 40 to 60 minutes of a rainfall, followed by a relatively slow increase thereafter, the C chosen later in the total storm is a greater value; see Fig. 3-6.

2. In addition to the considerations of item 1, it is desirable for rainfall frequencies rarer than once in 10 years to increase the C value in accordance with the graph of Fig. 3-7. The values obtained from the use of Fig. 3-6 are assumed to be valid for all frequencies of once in 10 years or of lesser recurrence intervals.

3. When circumstances require the preparation of a hydrograph, it is necessary to determine the mass excess or net rainfall and then assume its time distribution. The total excess or net rainfall can be determined by the SCS method utilizing runoff curve numbers which reflect the infiltration capabilities of the soil. The text illustrates how this can be accomplished.

Once the time-distributed net rainfall is determined for short intervals (5- or 10-minute durations usually are applicable), the actual hydrograph can be developed by means of the unit hydrograph or isochronal method.

4. If the circumstances suggest the unit hydrograph method, whether it is developed from available rainfall-runoff data for a watershed comparable to that under consideration, depends upon the availability of such data. In the usual absence of such information for small urban areas, a synthetic hydrograph should be developed. The F.F. Snyder equations for synthetic unit hydrographs should be used with values for coefficients C_t and C_p taken from Figs. 3-14 and 3-15. For imperviousness of less than about 20% this should be used with caution, particularly where the watershed has relatively few closed storm drains and few improved channels. These latter two conditions would tend to increase the numerical value of C_t for any and all impervious conditions.

5. The actual unit hydrograph can be developed by either of two means. Use of the SCS dimensionless unit hydrograph eliminates much of the effects of basin shape and size. The alternate method sketches in the unit hydrograph drawn through the peak, the points at the 50% and 75% widths (the time locations of 50% and 75% of the peak), the assumption of the base of the hydrograph being five times the time to peak from beginning of excess rainfall; and the basic assumption that

the total area under the unit hydrograph represents the runoff volume from 1 inch (25.4mm) of effective rainfall on the watershed. Usually a couple of attempts at sketching the recession side of the unit hydrograph will meet the 1-inch (25.4mm) volume requirement in satisfactory fashion. There are no compelling reasons to choose either of the unit hydrograph methods over the other.

6. The isochronal method of developing a hydrograph from the time-patterned net rainfall is an attractive and preferable method when watershed data enable the construction of a suitable time-area histogram with effective rainfall intensities for each 5 to 10 minutes of the net storm volume.

7. For some problems, it may be desirable to develop the desired hydrograph by both the unit graph method and the isochronal method with judgement dictating which of the two should be adopted for design (or an average of the two). Such use of both methods might be useful for determining the requirements of a pumping station in the sag of a grade separation.

APPENDIX A3

The Rational C and Frequency of Recurrence

Several studies, over the years, have suggested that one of the dominant parameters insofar as it influences the rational C, is the frequency of the rainfall. In 1938, Merrill Bernard (Ref. 3-21) in developing his modification of the rational method, suggested that the rational C varied in a predictable manner as related to the maximum value C might have. This latter he assumed to be that C value related to the 100-year frequency. He suggested that C would vary in accordance with the following equation:

$$C = C_{max}\left(\frac{T}{100}\right)^{x} \qquad \text{..........(3-20)}$$

T is the recurrence interval in years. Bernard developed this from the following reasoning: "When for either rainfall or streamflow, frequency is plotted logarithmically against magnitude, the slope of the plot is consistently between 0.15 and 0.23" and he is speaking of basic data rainfall or streamflow. He goes on to say: "This slope is the exponent x in the rainfall equation

$$i = \frac{KT^{x}}{T_{c}} \qquad \text{..........(3-21)}$$

For use in the rational method, it is proposed to reduce the value of the limiting coefficient to that of the selected frequency by a similar equation". Then Bernard gives the equation 3-20 relating the limiting coefficient C to C_{max}. Bernard also presented a map which gives values of the exponent x in the foregoing equation for all the area of the United States east of the 11 western mountain states.

Since Bernard's work was concerned with rural undeveloped watersheds, it can be assumed the variation in C value with frequency represents the relationship of such values for a substantially zero percentage of imperviousness. On Fig. 3-7 the curve marked 0% imperviousness follows the values of the averages given for the 10-, 25- and 50-year frequencies listed by Bernard. Note that Fig. 3-7 plots the 100-year value of C as 1.0 and those for the other frequencies of recurrence intervals as ratios of the pertinent C to C_{max}.

In 1960, ASCE-WPCF (Ref. 3-17) in connection with tabulations of rational method runoff coefficients stated: "The coefficients in these two tabulations are applicable for storms of 5- to 10-year frequency. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff". These same statements are repeated in the second edition (1969) of the same ASCE-WPCF publication.

Denver (1969, Ref. 3-14) makes the following statement: "The adjustment of the rational method for use with major storms can be made by multiplying the right side of the rational method by a frequency factor C_f which is used to account for antecedent precipitation conditions. The rational formula now becomes:

$$Q = CiAC_f \qquad \text{.............(3-22)}$$

The following table of C_f values can be used. The product of $C \times C_f$ should not exceed 1.0". Then follows from the Denver material:

Frequency Factors for Rational Formula

Recurrence Interval (Years)	C_f
2-10	1.0
25	1.1
50	1.2
100	1.25

Fig. 3-7 assumes the 100-year value as 1.0 which makes the other Denver values: 0.96, 0.88 and 0.80, respectively, and it is these values which could be placed on Fig. 3-7 to indicate the general, reasonable sequence of values suggested by the Denver criteria.

Santa Barbara, California (Ref. 3-22) developed the Santa Barbara Urban Hydrograph Method, a relatively simple, practical, mathematical simulation model to be used in local storm drainage planning and design. After calibrating and verifying the model on a 388-acre (157.0 hectares) area with 22% imperviousness, the results of the SBUH Method were applied to the rational method to study derived C values versus the average rainfall for the time of concentration.

"A plot showed widely scattered points with no line of good fit apparent. The coefficients were relatively low for short duration, high intensity storms on wet watersheds. It was not possible to obtain single value coefficients for observed rainfall intensities because of the wide variation in rainfall distribution and antecedent moisture conditions. Runoff coefficients were then calculated for various return periods by using the results of the frequency analyses of rainfall and runoff from the SBUH results. Peak runoff rates were divided by t(c) rainfall intensities for the same return periods".

The results give the ratios of C to C_{max} with the values .43, .63, .80, .88, .92, .97 and 1.00, respectively, for recurrence intervals of 1.5, 2.33, 5, 10, 25, 50 and 100 years, respectively. These ratios compared to the curves of Fig. 3-7 fit reasonably well into the general shape of the curves of Fig. 3-7 with the exception that the more frequent storms for the Santa Barbara study give somewhat higher values than comparable impervious percentages would appear to, considering the other guidance information utilized in developing Fig. 3-7.

The uppermost values of C to C_{max} from the several studies involving C versus frequency have been used for guidance in developing the 100% imperviousness curve of Fig. 3-7 and the Merrill Bernard values for placing the 0% imperviousness curve. The intermediate curves have all been determined by weighting the ratios obtained from the 0% and 100% curves. It is believed that Fig. 3-7 offers a practical guide to modifying the coefficients of runoff obtained from Fig. 3-6.

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STORAGE

4.1 General

Since the primary objective of stormwater management is to mitigate the changes in runoff brought about by changed uses of land, any successful efforts to lessen the quantity or rate of runoff as each of these is increased by urban highways can be a part of good stormwater management. Simplistically stated, such management is a space-allocation problem. At any given time and place, during or immediately after a storm, there is a given amount of rainfall in storage or in transit.

In all runoff situations, there are natural phenomena operating to lessen the quantity and rate of runoff. Interception by vegetation abstracts some of the rainfall which consequently never becomes runoff. Generally, between 0.02 and 0.10 inches of rain is held on foliage before appreciable drip takes place. Infiltration into pervious areas varies with the condition and character of the soil. The many minor bird-bath-like depressions that exist in all surfaces both permeable and impermeable, fill with rainfall which infiltrates into the permeable soils. For conveyance of runoff in overland flow, gutters, swales, open channels or closed conduits, appreciable depths must be developed. Natural ponds, marshes, large depressions each capture some of the runoff and reduce the peak rates as well as abstract significant quantities of rainfall. Each of these factors and occasional others influence the amount and rate of runoff.

Urban highways and arterial streets replace varying amounts of permeable areas with hard surfaces. In the older portions of large cities, the pavements may represent over 50% of the total urban impervious area. The diminution of permeable surfaces lessens the depression storage and infiltration. The paved surfaces speed up the conveyance of runoff. Thus, urban highways result in greater quantities of runoff at higher rates than would occur under pre-highway conditions. Stormwater management aims at minimizing, or preferably eliminating entirely, these development-caused increases in runoff.

4.2 Storage Characteristics

Storage of excess storm runoff is one of the most promising methods to lessen the impact of development. The reduced outflow rates made possible by storm runoff storage can hold downstream flows to within the safe conveyance capacity of downstream storm drainage facilities. The costs of such storage must be compared to those involved in increasing the downstream conveyance capacity or to the potential damages to servient property if storm drainage after development delivers increased sedimentation and increased water pollution (both intensified by the increased storm runoff resulting from development).

The three basic types of stormwater storage are retention, detention and conveyance storage.

A retention facility is characterized by a several-day storage period and a low release rate both during and subsequent to the rainfall. Such storage often has a permanent pool and may be multi-purpose, i.e. recreational, esthetic, etc. The flood storage volume is superposed above the permanent pool and may accomodate the entire runoff from a certain design rainfall event. Because retention inherently involves large impoundment volumes, its use in stormwater management may be limited to small scale runoff situations.

Detention storage usually reduces outflow to a rate less than that of the peak inflow. Frequently, the goal is to limit the peak outflow rate to that which existed from the same watershed before development. Normally, the detention site drains completely in less than a day. Consequently, the usually dry detention storage facility can often be used for sports fields, car parking, etc. Fig. 4-1 illustrates the general distinctions between retention and detention storage.

Both detention and retention storage present great potential for reducing drainage costs.

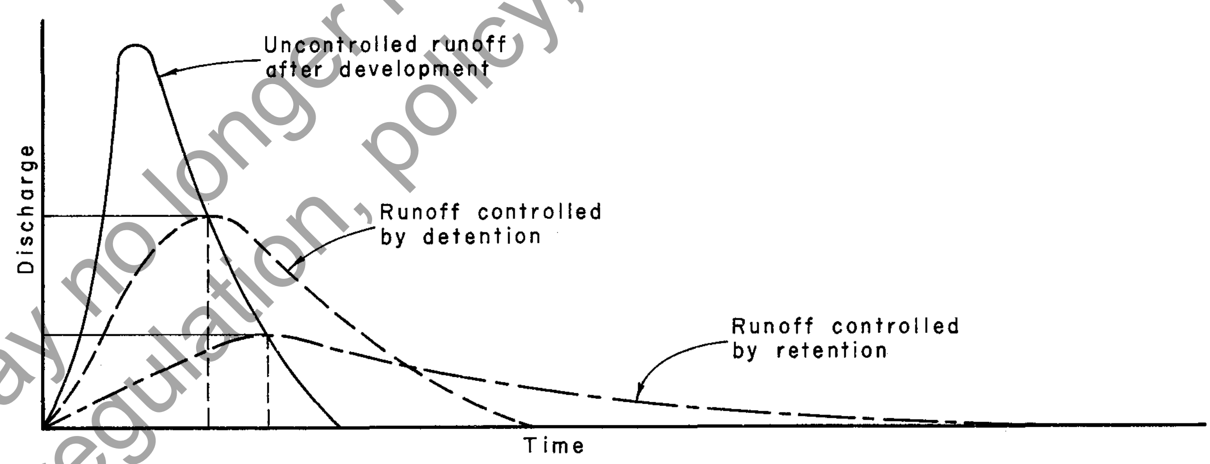


FIG. 4-1 Typical detention and retention hydrographs
(After Wiswall and Shumate)

Conveyance storage is inherent in overland flow and in swales, channels and conduits. The volume required to sustain the movement of water is stored in a transient form. Consequently, it is advantageous in the management of stormwater to increase such transient storage. Overland flow storage can be increased by discharging flows from pavements onto turf-covered surfaces. The greater the extent of the latter or the longer the flow path across turf, the greater the overland flow storage (and the longer the opportunity for infiltration into the underlying soils). If concentrated storm runoff can be routed via large cross-sectional channels (hence slow velocity), significant conveyance storage can be designed into the system. Meandering an open stormwater channel can create added storage.

4.3 Storage Size and Location

Any one or more of the three basic types of storage can be designed in a stormwater management system in a wide range of sizes at a variety of locations in the watershed.

As later discussed in detail, the size of a storage facility is directly related to the objectives of the flow-management scheme for a particular subwatershed. The more frequent purpose is the reduction of the increased rate of runoff from development within the watershed to that which prevailed prior to the urbanization. Controlling the outlet discharge to a rate less than the maximum inflow rate involves a specific volume of detention storage for chosen quantities and rates of inflow and established maximum outflows. Thus, the extent of the to-be-controlled watershed and the character of its development determine the size of a detention facility.

A Canadian study (Ref. 4-2) recommended that, in general, watersheds with undeveloped headwater areas can use detention techniques to control runoff problems, while watersheds with undeveloped areas close to a receiving body of water can continue to utilize conventional techniques. This is in recognition of timing of peak flows in the watercourse. Runoff from a new development near the mouth of a watershed could use conventional design techniques to ensure releases far in advance of the peak flow from the balance of the watershed. In fact, it may aggravate flood and channel erosion conditions if runoff from the downstream areas is detained and released when the upstream flood peak does arrive.

Storage can be classified by location as on-site, off-site, upstream, downstream, channel (or onstream) and off-stream. Based on function, storage facilities may be for single or multipurpose use and temporary (detention) or permanent (retention), open or closed (surface or subsurface). Most urban highway stormwater storage is on-site. Storage with its primary purpose the replenishment of groundwater is well exemplified in Refs. 4-3 and 4-4.

4.4 Place of Storage in Urban Highway Drainage

There are few circumstances of urban highway drainage that justify the separate provision of detention storage for the runoff from the roadway itself. In most instances, the runoff from the roadway is part of a larger amount of runoff from the subwatershed traversed by the highway. For such conditions, if detention storage is indicated, it is most economically provided for the whole subwatershed; this often means a cooperative project with the local storm drainage authority.

Pumping of stormwater is sometimes unavoidable at sags or sumps where gravity drainage is impossible or uneconomic. The high initial cost, maintenance expense, power costs, can all be lessened if suitable storage can be incorporated in the design to reduce the maximum outflow from the storage to an acceptable low rate as compared to the peak inflow rate. A mass inflow curve taken from the hydrograph of inflow to the sump will permit consideration of various constant-capacity pumps to select that installation most economically suitable. In urban situations, it is possible that the required storage might have to be a buried structure.

For occasional suburban highways with ample rights-of-way and large interchanges there may be opportunities for on-site detention storage of roadway runoff; usually urban highways would need to acquire special land parcels for any on-site detention storage. Such highway detention storage should be designed as an acceptable part of the stormwater management of the larger subwatershed of which it is a part.

4.5 Determination of Storage Volume

It is assumed that the storage which usually can be involved in the urban drainage of highways will be relatively small in magnitude and the methods for determining its volume as discussed herein are pertinent only under such circumstances. The intent is to reduce the peak runoff, i.e. the increase in the hydrograph due to urbanization. The stored water re-enters the drainage system later.

The permissible discharge rate from a storm management storage facility must be known to establish the required volume in the impoundment. The most usual requirement is that the maximum discharge rate shall not exceed that which would occur under the same assumed design conditions of rainfall and soil conditions before development or under natural conditions of the watershed tributary to the storage facility. Occasionally, the flow capacity of storm drainage facilities immediately downstream from the to-be-developed area will determine the permissible discharge from the detention storage facility.

The required storage depends on:

1. The time distribution and volume of inflow.

2. The maximum allowable discharge rate and variation of discharge with depth of ponding.

3. The configuration of the detention facility.

4. The costs as related to the benefits.

The required volume of storage will be the maximum difference between the cumulative distribution of inflow and the cumulative distribution of outflow when the maximum allowable discharge is not exceeded. An inflow hydrograph of pre-selected duration and frequency; reservoir stage-volume and stage-discharge curves for the detention structure are essential prerequisites to a determination of the required storage. This latter is obtained by routing the inflow hydrograph through the detention facility. Maximum allowable discharge may be determined by the rational method for tributary areas of less than 500 acres.

4.6 Outlet Hydraulics

The usual outlets for small detention or retention storage include:

1. A pipe or culvert conduit through the impounding dam, placed to drain the lowest level in the impoundment area.

2. A vertical riser with or without perforations depending upon whether the storage is to include a permanent pool or not, with the riser connected by an elbow (or tee) to a sloping (almost horizontal) pipe or conduit through the dam.

3. A supplemental emergency spillway, usually a broad-crested weir designed to limit the elevation of impounded water and safely pass downstream excess runoff from storms rarer than those which the facility is normally expected to handle satisfactorily. The latter spillway design storms are usually the 100-year recurrence interval event or such rarer rainfall event as circumstances of potential risk may indicate.

4.6.1 Culvert Outlets

A pipe through the damming structure or fill can be the simplest discharge control where the design has a small permissible outflow or release rate and the storage facility is to be dry between storms. The principles and charts of FHWA Hydraulic Engineering Circulars Nos. 5 (Ref. 4-5) and 10 (Ref. 4-6) can be used to determine pipe size. Careful attention is needed with respect to the inlet end of the pipe to minimize blockage by sediment or debris. And some erosion protection may be required at the control pipe outlet (Ref. 4-13). Average inflow rates should be at 5-minute intervals for the rapidly changing portions of the hydrograph with 10-minute or longer intervals where the inflow rates are changing more gradually.

4.6.2 Drop Pipe Discharge Control

Under circumstances calling for the use of a detention storage facility as a sedimentation trap (in addition to its primary purpose of attenuating the outflow hydrographs), the vertical riser is provided with perforations. Consequently, the flow through such a vertical riser consists of two components, the first through the perforations while the other is flow over the top edge of the riser.

4.6.2.1 Perforated Risers

Flow through perforated risers is treated as flow through circular orifices which can be determined by

$$Q_o = KC_oA_o(2gH_o)^{.5} \qquad \text{..............(4-1)}$$

where Q_o = flow rate for one orifice in cfs or m^3/s, C_o = discharge coefficient, A_o = area of orifice in square feet or square metres, H_o = effective head at each orifice in feet or metres, K is a coefficient to account for units English or metric, and g = acceleration of gravity in feet or metres per second per second. If D_o = diameter of circular orifice in inches or metres

$$Q_o = KC_oD_o^2H_o^{.5} \qquad \text{..............(4-2)}$$

with K equal to 0.0438 for English units and equal to 3.4821 for metric units.

If the holes are cleanly cast or drilled and burrs removed to give sharp edges to the holes, a discharge coefficient C_o of 0.6 to 0.7 is appropriate; in the absence of specific knowledge, use 0.65. Then, for any horizontal series of circular holes, under the same effective head,

$$Q_i = KN_iD_i^2H_i^{.5} \qquad \text{.............(4-3)}$$

with K equal to 0.02847 for English units and equal to 2.2634 for metric units with Q_i = flow in cfs or cms through the i^{th} set of holes of diameter D_i in inches or metres and number N_i under effective head H_i in feet or metres. Effective head is to the centroid of the area which for a circular hole is its center.

In the field, sometimes for a corrugated metal riser, an acetylene torch is used to burn the perforations in the metal. In such instances, the orifice coefficient should be 0.4 to reflect the corrugated pipe and the jagged edges of the holes. With the coefficient of 0.4 the orifice flow is

$$Q_i = KN_iD_i^2H_i^{0.5} \qquad \text{...............(4-4)}$$

with Q_i, N_i, D_i and H_i as defined for equation 4-3; K equal to 0.01752 for English units and 1.3928 for metric units. Holes should be a minimum of $3D_i$ center to center.

4.6.2.2 Flow Over Top of Riser

Flow over the top edge of the riser, Fig. 4-2a, is assumed as flow over a sharp crested weir with

$$Q_w = C_w L_w H_w^{1.5} \quad \ldots\ldots\ldots\ldots\ldots (4\text{-}5)$$

Q_w is flow rate over weir in cfs or m^3/s; C_w = discharge coefficient; L_w = length of weir in feet or metres = πD_w in which D_w is the riser diameter in feet or metres and H_w is the effective head above the top of the riser in feet or metres. With $C_w = 3.0$, the equation becomes

$$Q_w = K D_w H_w^{1.5} \quad \ldots\ldots\ldots\ldots\ldots (4\text{-}6)$$

with K equal to 9.4248 for English units and 5.2033 for metric units.

If D_w is in inches, this becomes

$$Q_w = 0.785\, D_w H_w^{1.5} \quad \ldots\ldots\ldots\ldots\ldots (4\text{-}7)$$

The total flow through the riser, Q_r in cfs, then becomes

$$Q_r = Q_o + Q_w \quad \ldots\ldots\ldots\ldots\ldots (4\text{-}8)$$

Fig. 4-2b indicates the details of a trash-rack and anti-vortex plate suggested for the top of a drop-pipe spillway such as sketched on Sheet 1 of Fig. 4-2a. For concrete or other pipe risers, a comparable arrangement should be installed. Note that the anti-vortex device should be installed normal to the centerline of the dam. Laboratory experiments (Ref. 4-7) indicate that a strong vortex can reduce the flow through an orifice by as much as 75%. Blaisdell (Ref. 4-8, Jan. 1952), describes the theory of the hydraulics of closed conduit spillways and discusses vortices in detail.

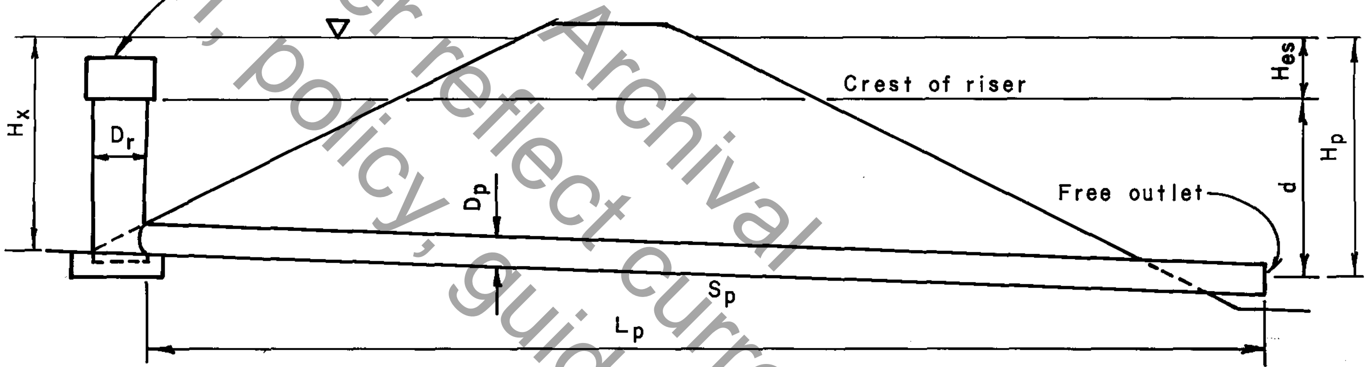
4.6.2.3 Flow Through Pipe

Under some conditions, the flow through the vertical riser may be great enough so that the pipe from the base of the riser passing under the dam may control flow instead of the riser. The pipe capacity then is

$$Q_p = \frac{A_p (2gH_p)^{.5}}{(1 + K_e + K_c L_p)^{0.5}} \quad \ldots\ldots\ldots (4\text{-}9)$$

in which Q_p = flow rate in pipe in cfs or cubic metres per second; A_p = cross-sectional area of pipe in square feet or square metres; H_p = effective head on outfall in feet or metres as measured between the elevations of the pond surface and the center of the pipe cross-section at the outfall; K_e is an entrance coefficient; L_p = the length of pipe in feet or metres; and

PIPE SPILLWAY DESIGN



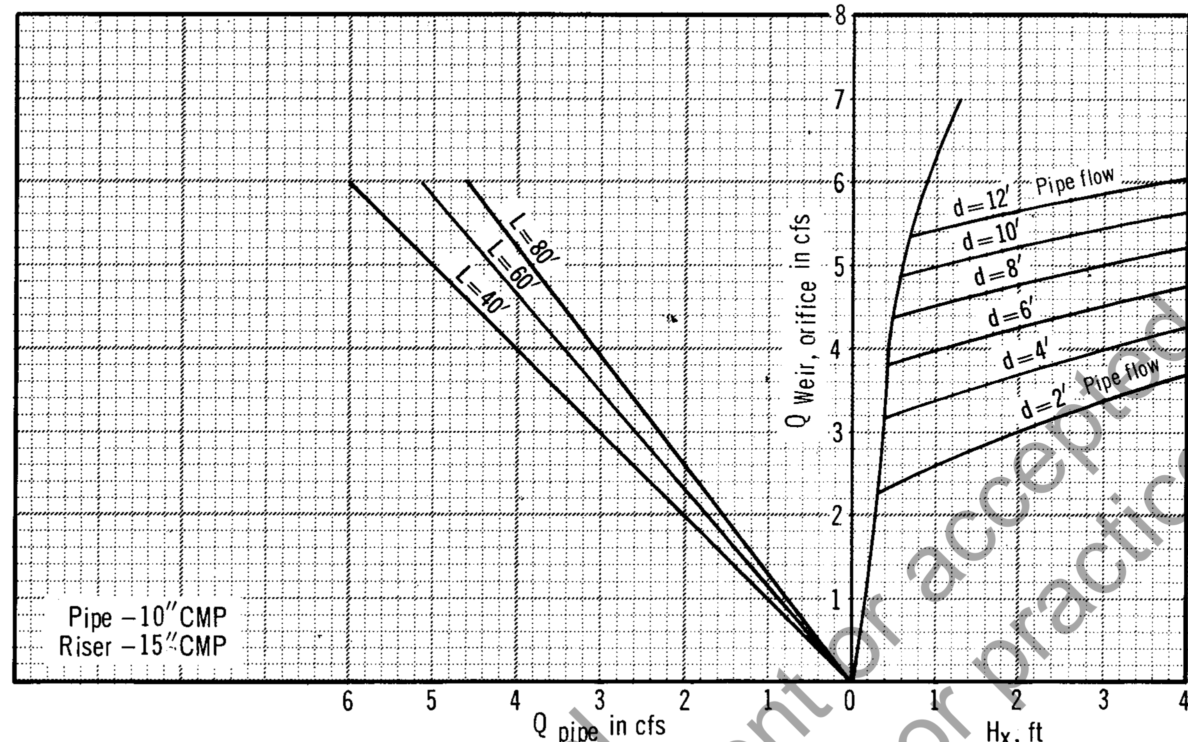
H_{es} = Head over crest of riser in feet or metres.

L = Length of pipe in feet or metres.

d = Difference between crest of riser and outlet water surface elevations but not greater than vertical distance from riser crest to centerline of outlet.

Spillway discharge capacities must be reduced 50 % if no anti-vortex device is used.

FIG. 4-2(a) Sheet 1 of 7 *(From S.C.S.)*



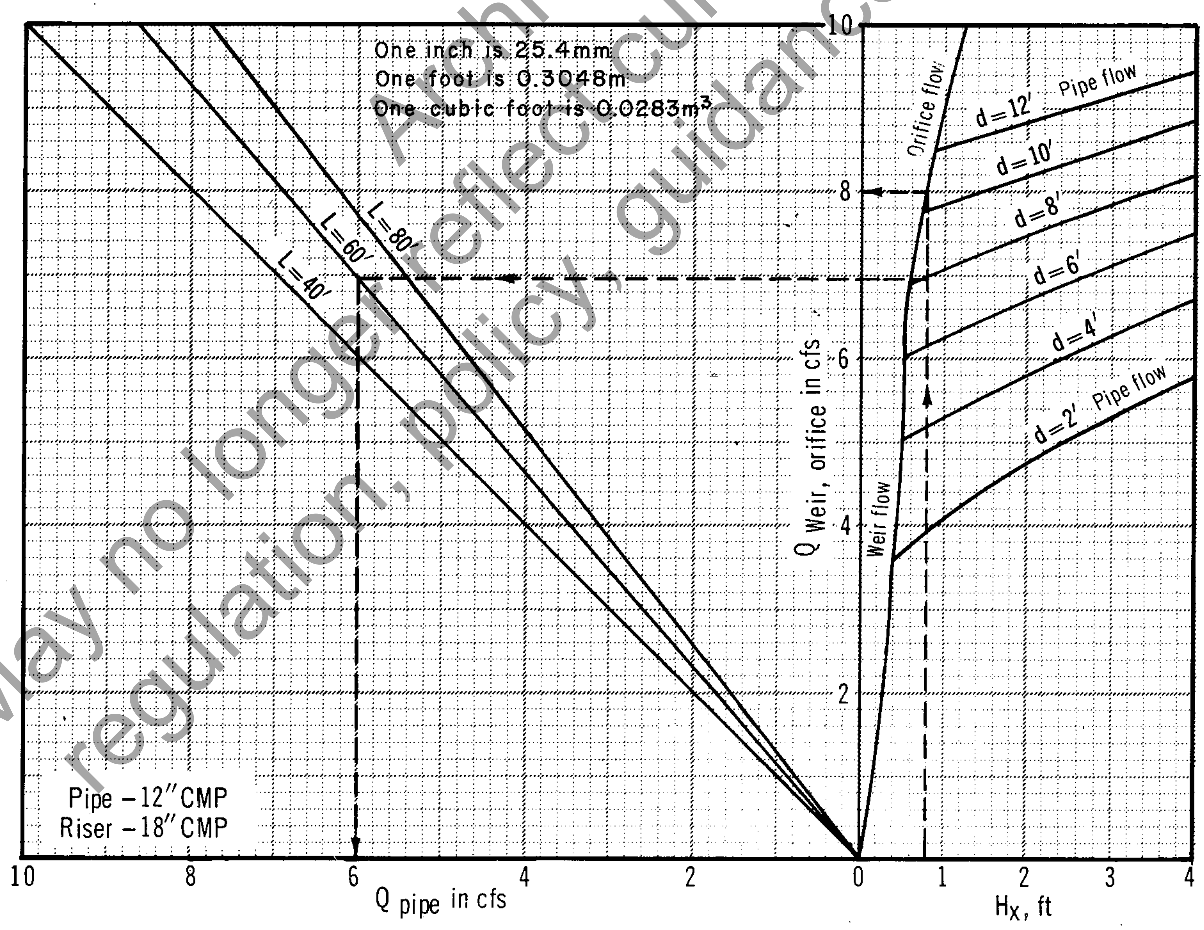
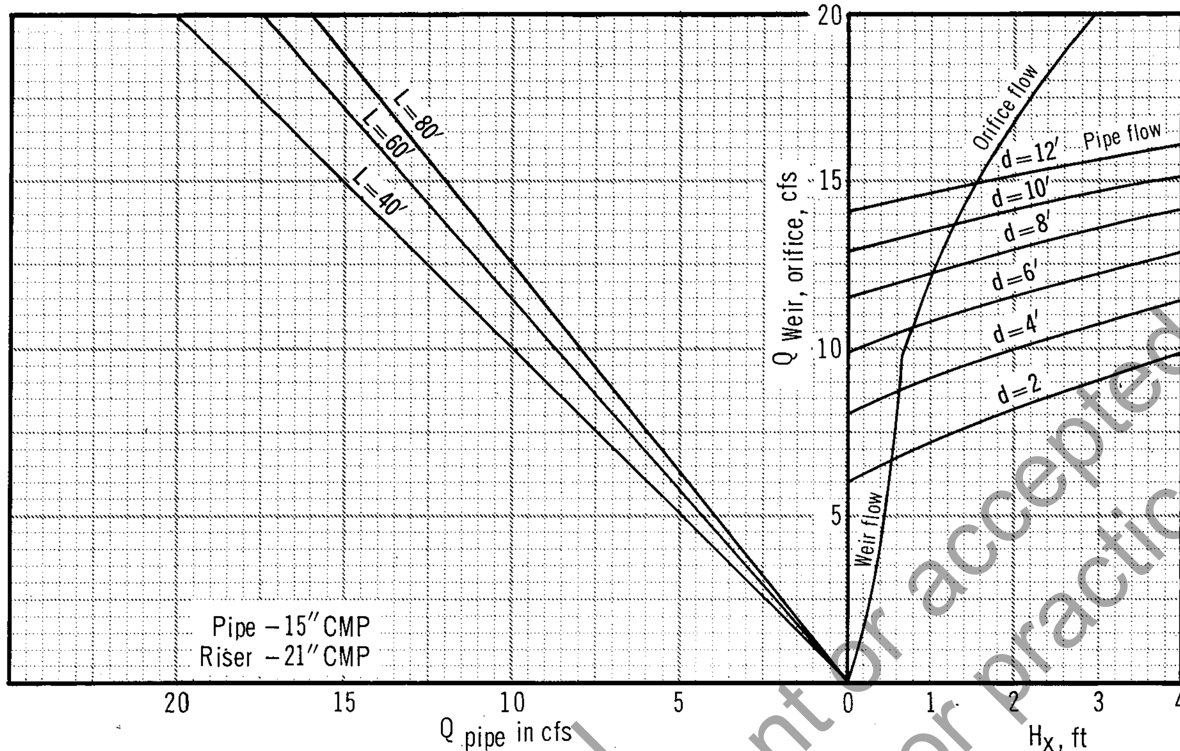


FIG. 4-2(a) Sheet 2 of 7



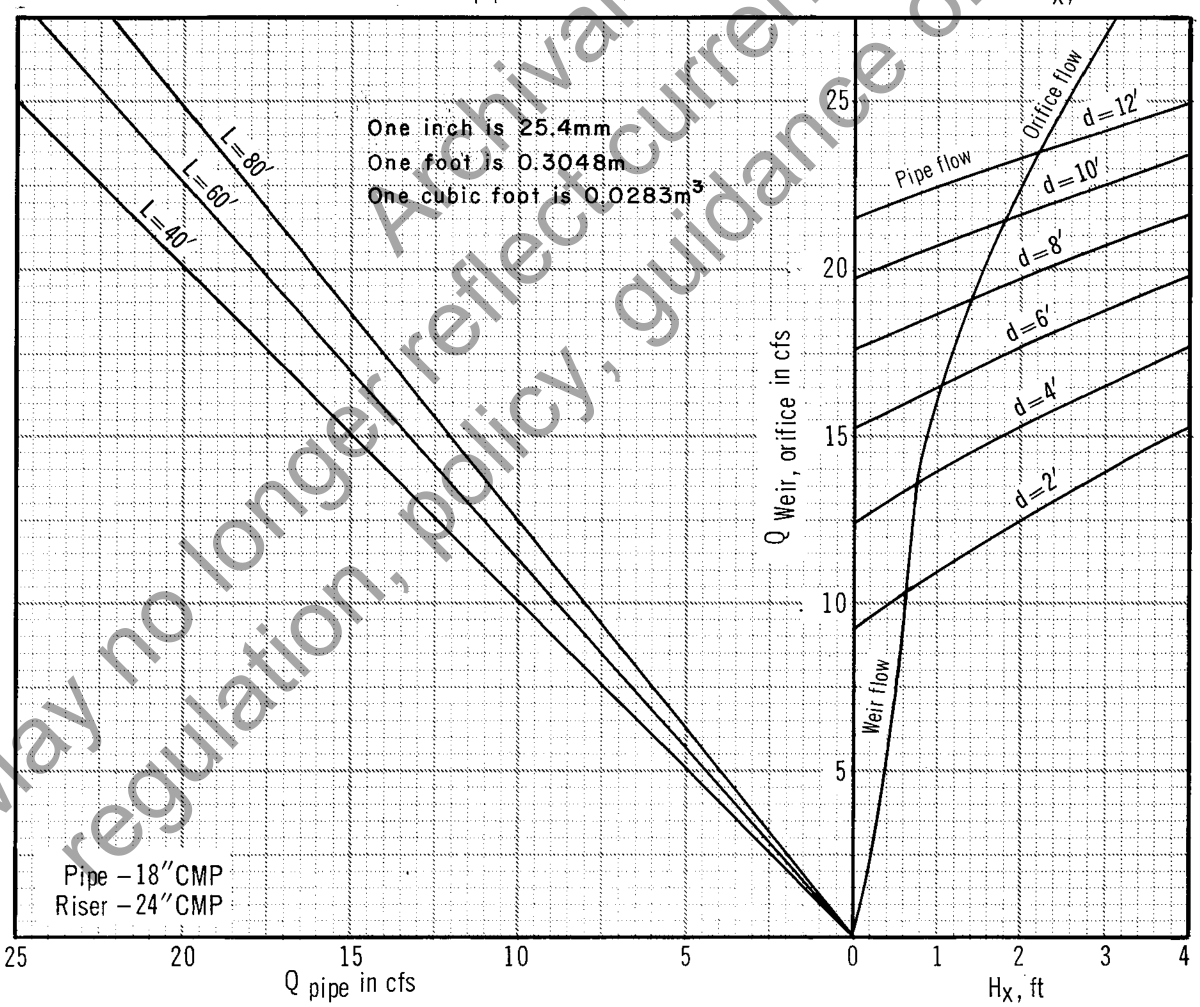
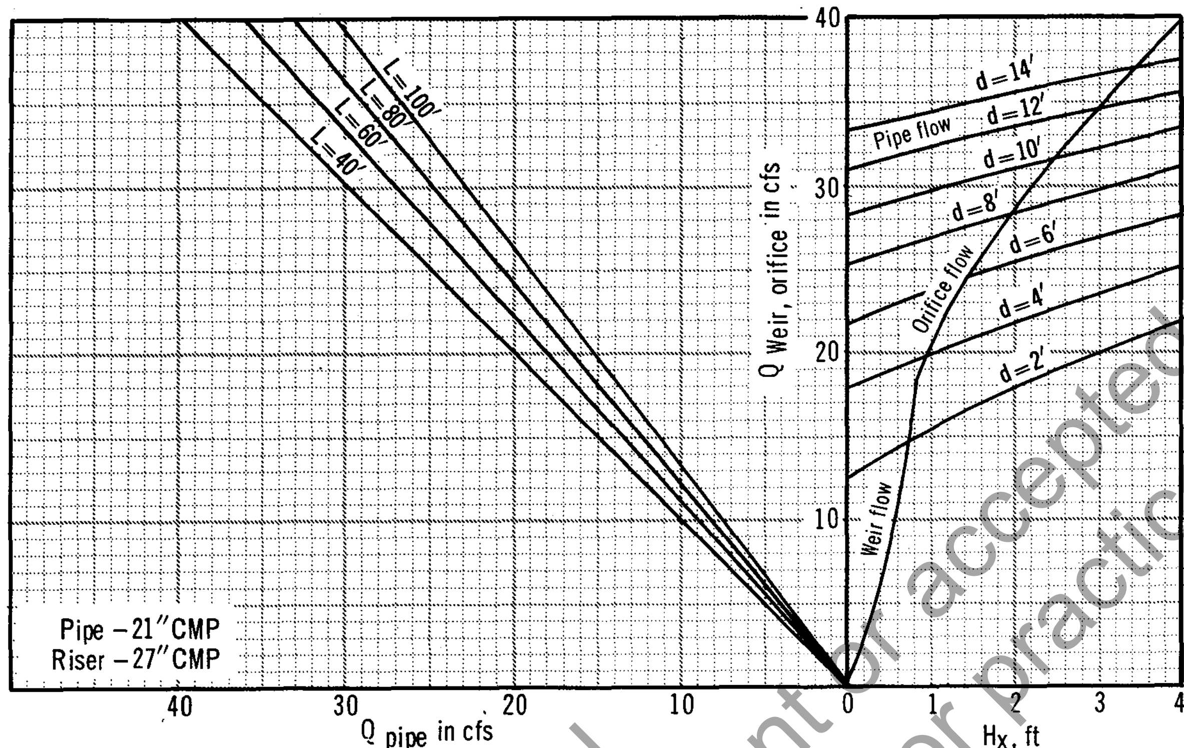


FIG. 4-2(a) Sheet 3 of 7



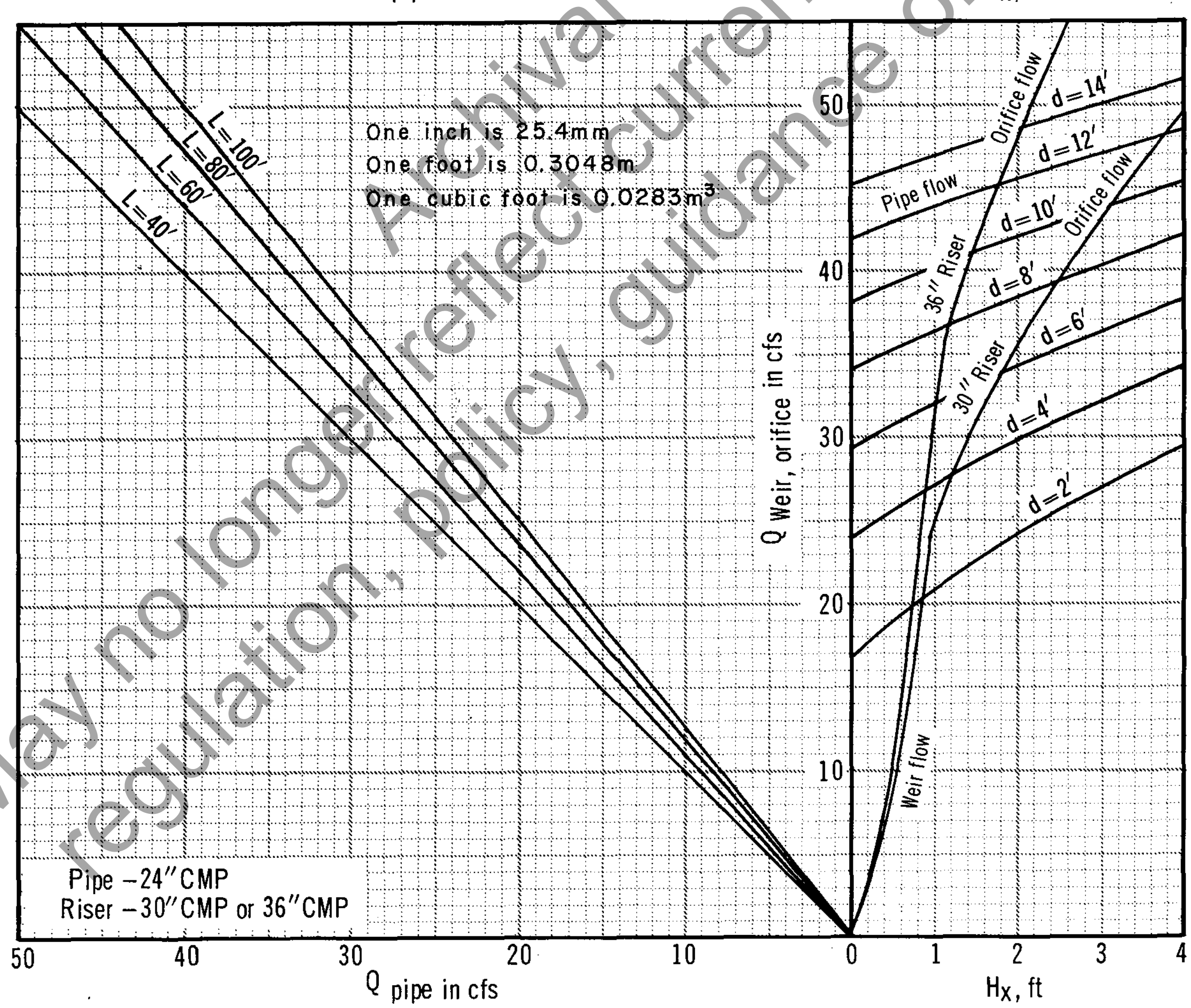


FIG. 4-2(a) Sheet 4 of 7

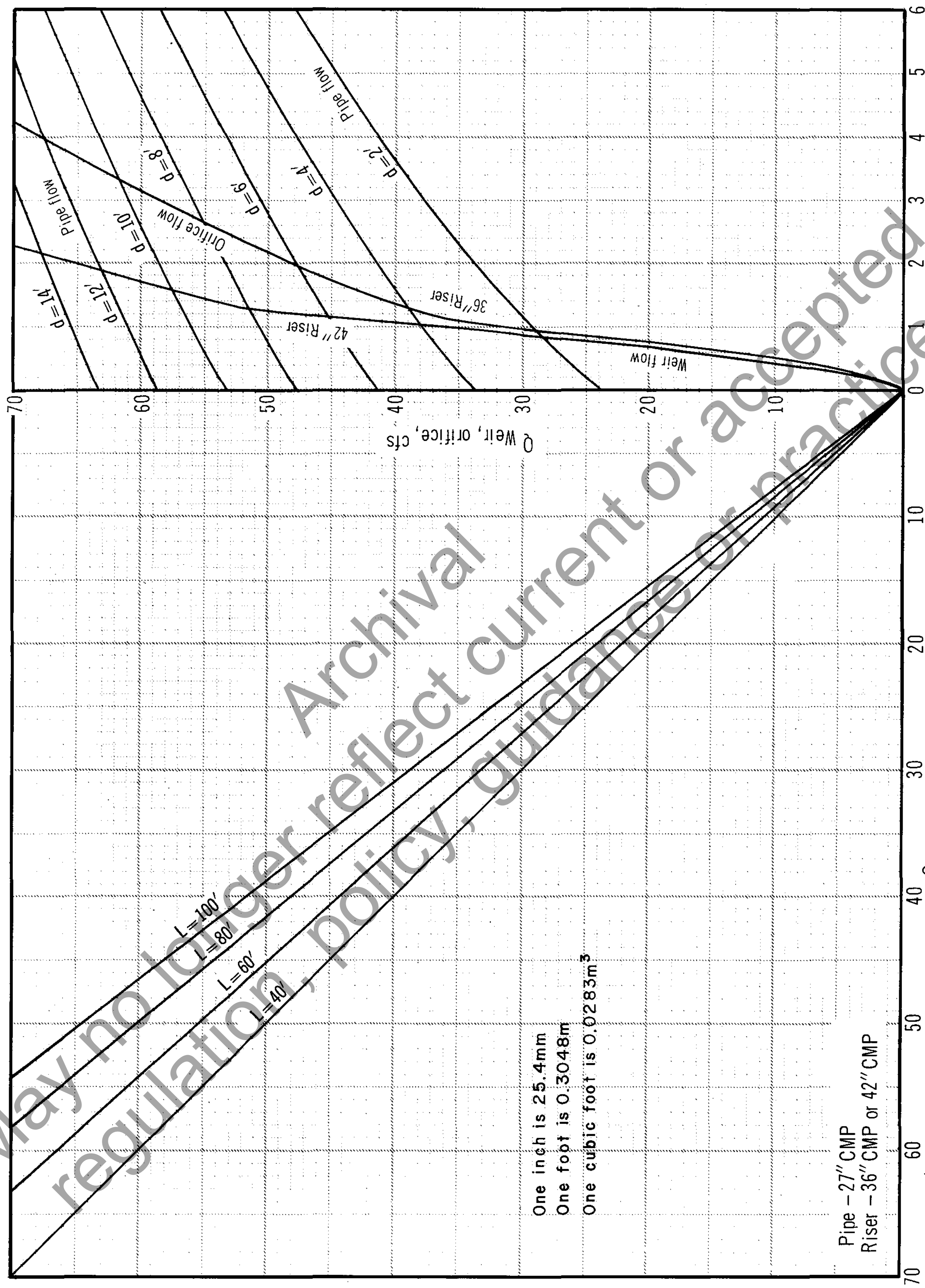


FIG. 4-2(a) Sheet 5 of 7

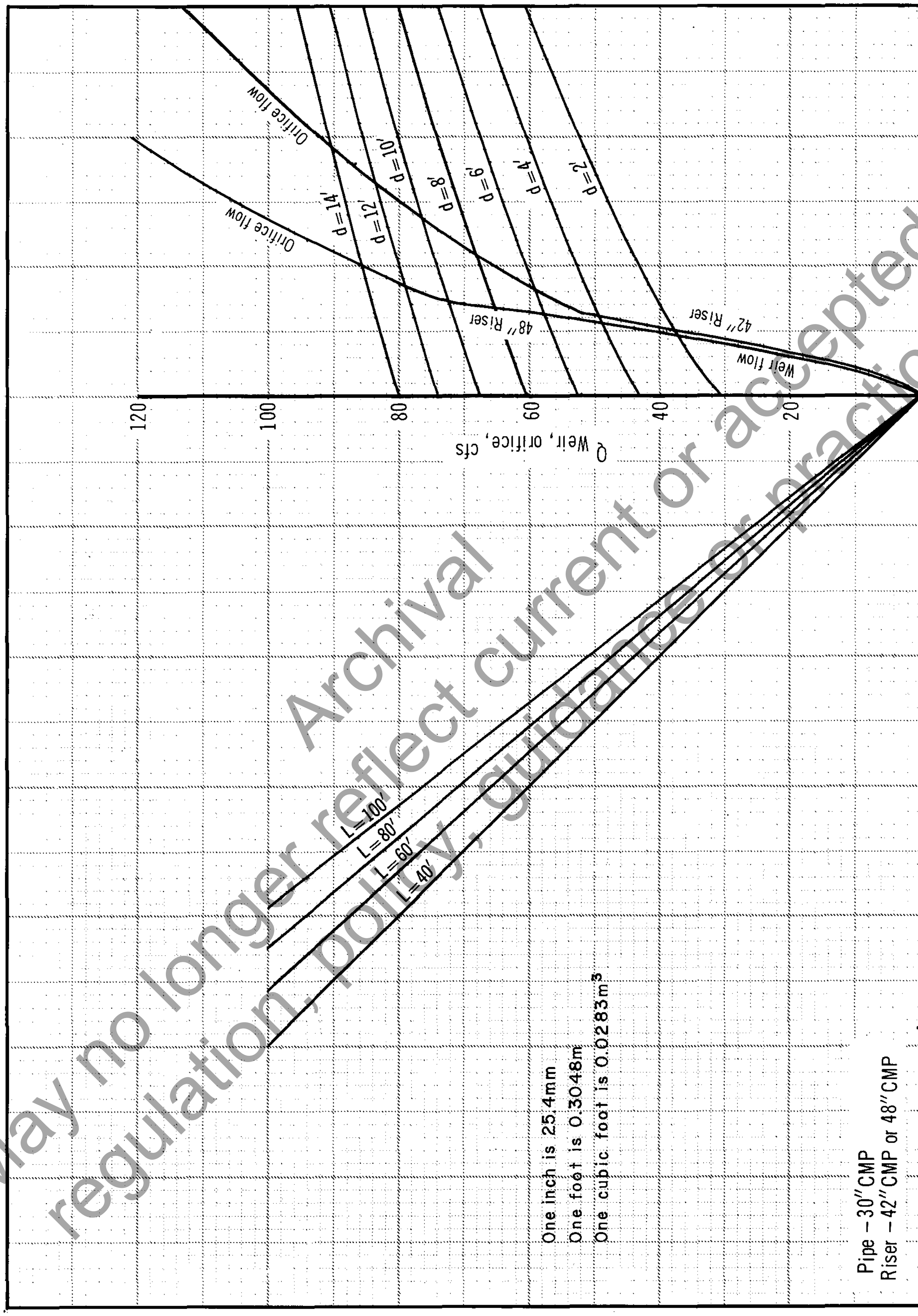


FIG. 4-2(a) Sheet 6 of 7

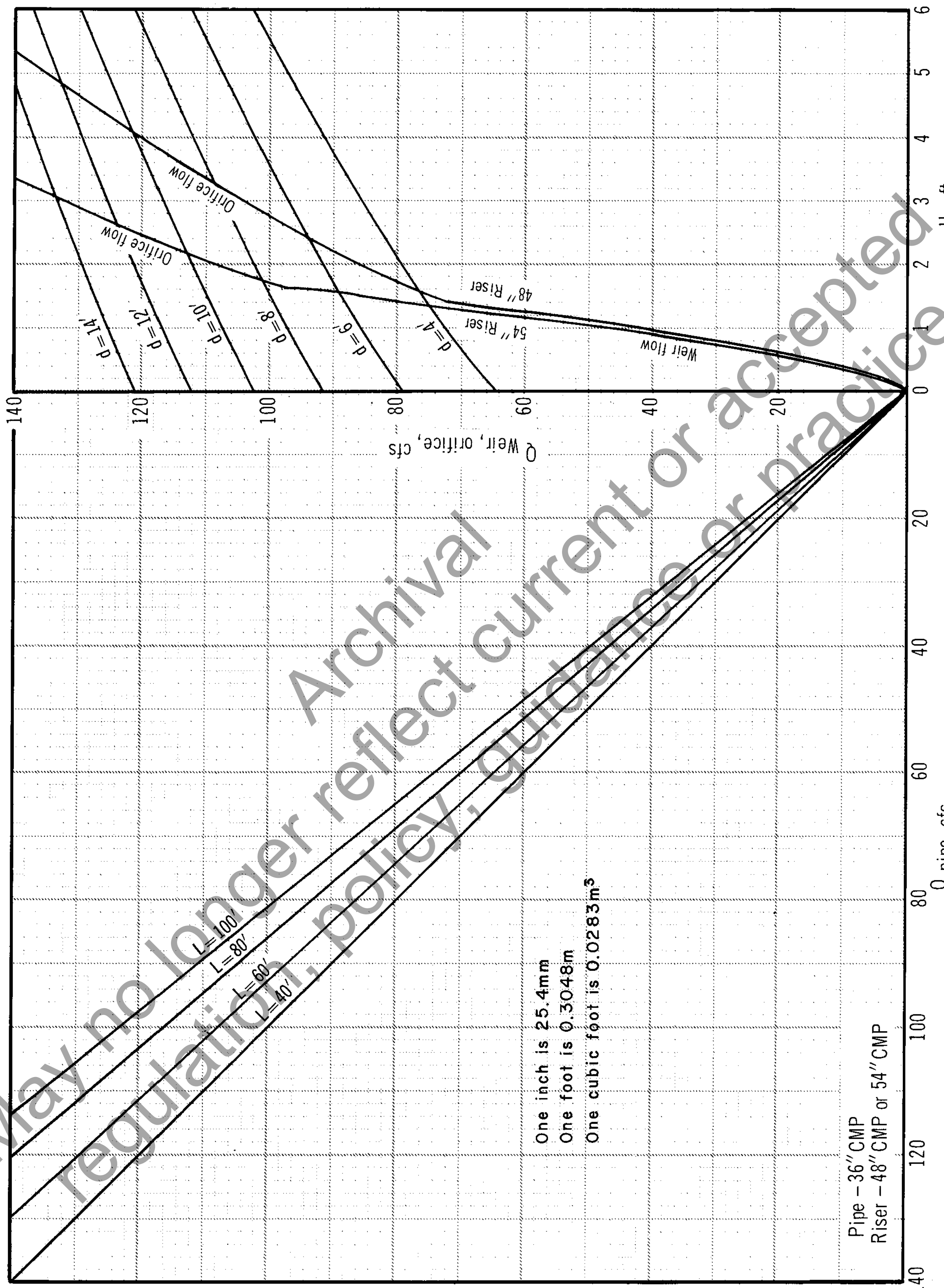
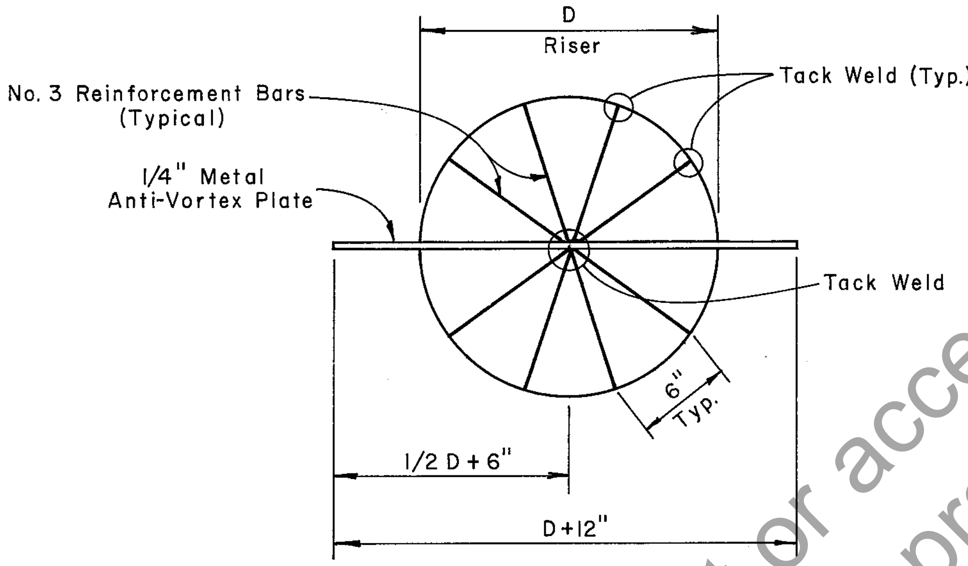
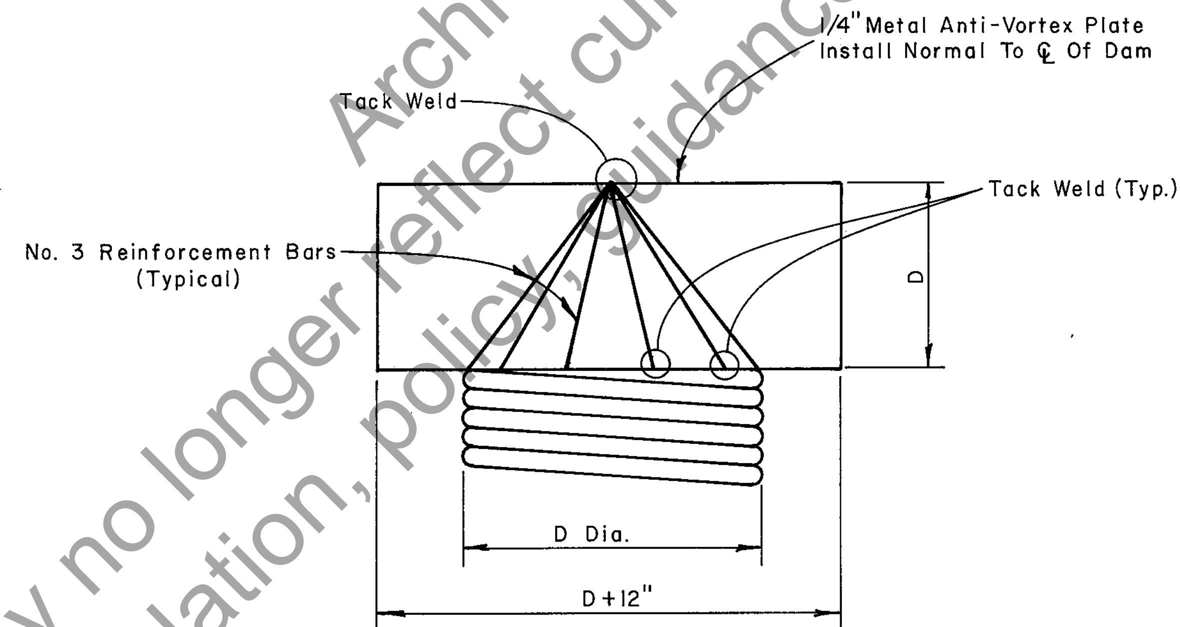


FIG. 4-2(a) Sheet 7 of 7



PLAN

One inch is 25.4mm



SECTION

DETAIL OF TRASH RACK AND ANTI-VORTEX DEVICE

FIG. 4–2(b)

$$K_c = \frac{Bn^2}{D_p^{1.33}} \qquad \ldots\ldots\ldots\ldots\ldots(4\text{-}10)$$

n = Manning's coefficient; D_p = pipe diameter in feet or metres; B is 185.2 for English units and 137.2 for metric units.

With D_p in inches, this becomes

$$K_c = \frac{5088n^2}{D_p^{1.33}} \qquad \ldots\ldots\ldots\ldots\ldots(4\text{-}11)$$

H_p can be calculated as (see Fig. 4-2)

$$H_p = H_x + S_p L_p - 0.5D_p \qquad \ldots\ldots\ldots\ldots\ldots(4\text{-}12)$$

with S_p = flowline slope of pipe.

If K_e is assumed as 0.5, n as .013, D_p in inches or metres; H_p and L_p in feet or metres, equation 4-9 becomes

$$Q_p = \frac{M D_p^2 H_p^{0.5}}{\left[1.5 + (NL_p)/D_p^{1.33}\right]^{0.5}} \qquad \ldots\ldots\ldots(4\text{-}13)$$

M is 0.044 for English units and 3.478 for metric units. N is 0.86 for English units and 0.02104 for metric units.

Of the computed flows for the riser only and for the pipe only, the lesser determines the outflow.

4.6.3 Emergency Spillway

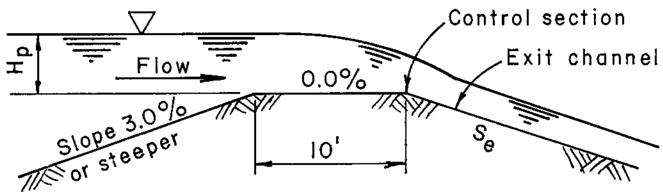
For most small detention storage facilities, a suitable emergency spillway can be a broad-crested overflow weir cut through the top of the containing embankment with its horizontal top at an elevation one to two feet above maximum design storage elevation. (It is preferable to have a freeboard of 2 feet minimum but for very small impoundments, say less than 1 or 2 acres (0.4 to 0.8 hectare) maximum water surface, an absolute minimum freeboard of 1 foot (0.305m) should be provided.)

For ease of construction the transverse cross-section of the weir cut can be trapezoidal. To avoid the complexities this would inject into the hydraulic computations, it is suggested it be assumed that the emergency flow passes through a broad-crested weir with vertical sidewalls. The equation for discharge is

$$Q_{es} = C_{es} b H_p^{0.5} \qquad \ldots\ldots\ldots\ldots\ldots(4\text{-}14)$$

with Q_{es} the flow in cfs or m^3/s; b the width of the emergency spillway in feet or metres and H_p the effective head on the emergency spillway in feet or metres.

EMERGENCY SPILLWAY DESIGN



Side slopes = 2.1
n = 0.04 (Manning's)
Q = Discharge, cfs
V_c = Critical velocity, fps
S_c = Critical slope, %
H_p = Height of pool above emergency spillway control section

One foot is 0.3048m; One cubic foot is 0.0283m^3

Hp, ft.		Spillway Bottom Width, b, feet											
		8	10	12	14	16	18	20	22	24	26	28	30
0.8	Q	14	18	21	24	28	32	35	-	-	-	-	-
	V_c	3.6	3.6	3.6	3.7	3.7	3.7	3.7	-	-	-	-	-
	S_c	3.2	3.2	3.2	3.2	3.1	3.1	3.1					
1.0	Q	22	26	31	36	41	46	51	56	61	66	70	75
	V_c	4.1	4.1	4.1	4.1	4.1	4.1	4.2	4.2	4.2	4.2	4.2	4.2
	S_c	3.0	3.0	3.0	3.0	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9
1.2	Q	31	37	44	50	56	63	70	76	82	88	95	101
	V_c	4.5	4.5	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.6	4.6	4.6
	S_c	2.8	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.6
1.4	Q	40	48	56	65	73	81	90	98	105	113	122	131
	V_c	4.9	4.9	4.9	4.9	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
	S_c	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
1.6	Q	51	62	72	82	92	103	113	123	134	145	155	165
	V_c	5.2	5.2	5.3	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4	5.4
	S_c	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.4
1.8	Q	64	76	89	102	115	127	140	152	164	176	188	200
	V_c	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S_c	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3
2.0	Q	78	91	106	122	137	152	167	181	196	211	225	240
	V_c	5.8	5.8	5.8	5.9	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
	S_c	2.5	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3

NOTE: For a given H_p, decreasing exit slope from S_c decreases spillway discharge, but increasing exit slope from S_c does not increase discharge.

If a slope (S_e) steeper than S_c is used, velocity (V_e) in the exit channel will increase according to the following relationship:

$$V_e = V_c \left(\frac{S_e}{S_c}\right)^{0.3}$$

TABLE 4-1
(After Maryland SCS)

The Maryland office of the SCS (Ref. 4-9) has developed the figures of Table 4-1 for emergency spillway design. The coefficient is not a fixed value, varying from 2.45 for the lowest head (0.8 feet=0.244 m) on the narrowest spillway (8 feet=2.438m) to 2.83 for the highest head (2.0 feet= 0.610m) and the broadest spillway (30 feet=9.144m). The critical slopes of Table 4-1 are based upon an assumed n = 0.04 for turf cover of the spillway. For a paved spillway, the n should be assumed as 0.015.

Critical velocity can be computed as follows from Ref. 4-10:

$$d_c = \left[Q_i^{\,2}/g\right]^{0.33} \qquad \text{.........(4-15)}$$

with d_c = critical depth in feet or metres; Q_i = discharge per foot or metre width of channel.

$$v_c = Q_i/d_c \qquad \text{.........(4-16)}$$

substituting in equation (4-16)

$$v_c = g^{1/3}Q_i^{.33} = KQ_i^{.33} \qquad \text{.........(4-17)}$$

with K equal to 3.18 for English units and equal to 2.140 for metric units.

In virtually all instances, the hydraulic radius of the assumed rectangular weir down the slope can be assumed to be equal to the mean depth of the overflow down the slope or d_c. Then (Ref. 4-10):

$$S_c = Kn^2/d_c^{.33} \qquad \text{............(4-18)}$$

by substitution of equation 4-16 this becomes

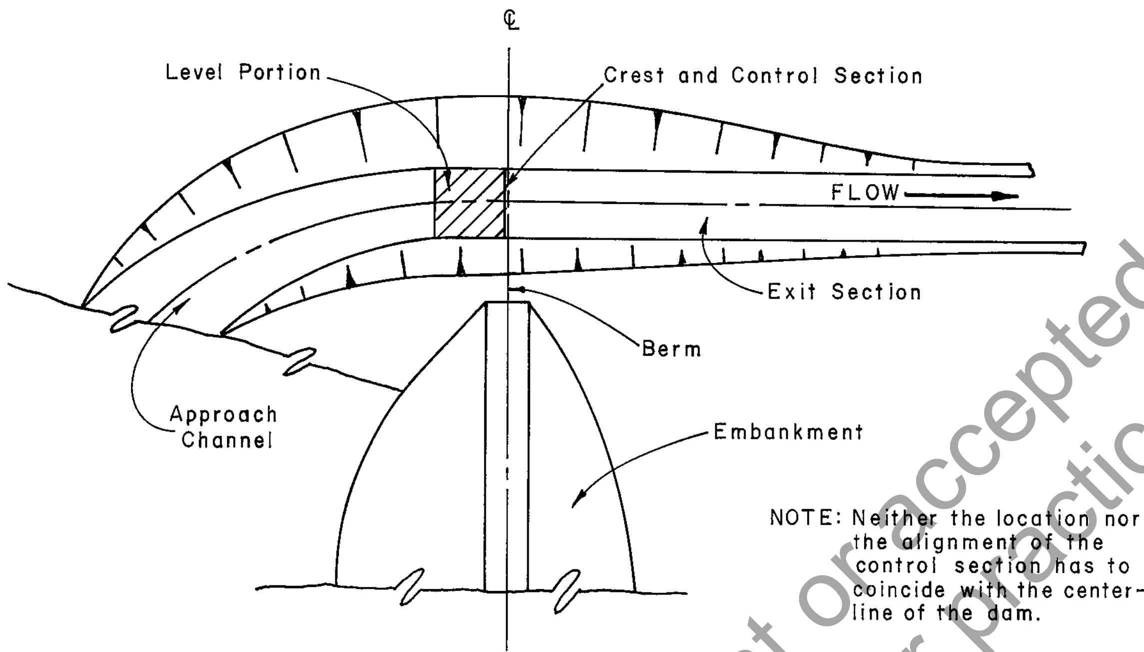
$$S_c = Kn^2 v_c^{0.33}/Q_i^{0.33} \qquad \text{............(4-19)}$$

with K equal to 14.56 for English units; and equal to 9.8375 for metric units.

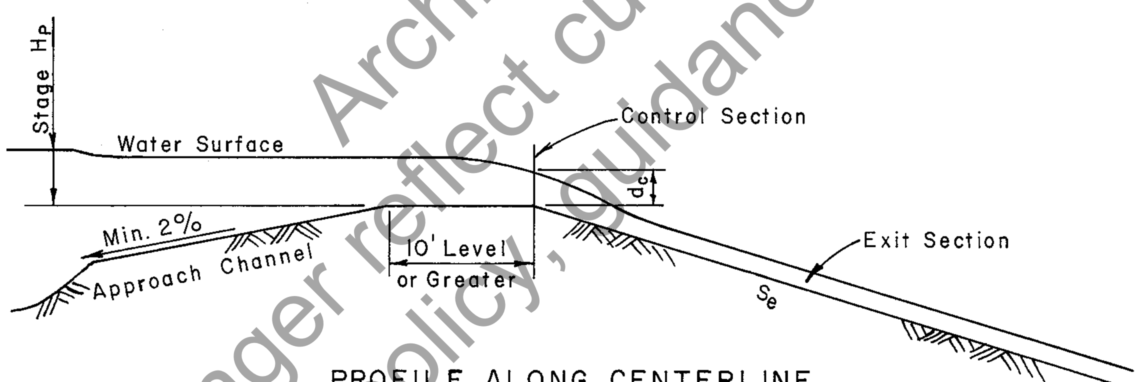
Fig. 4-3 sketches a typical emergency spillway weir and Table 4-1 lists a range of outflows up to 240 cfs (6.796 cms) with their related critical velocities and critical slopes for grass-lined spillways.

4.6.4 Routing Procedure

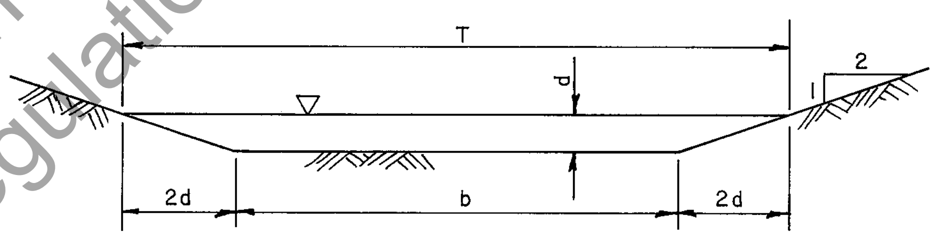
For the storage facilities involved in urban highway drainage, it will be assumed that the water surface in the impoundment is horizontal, that the overflow and storage are each functions of the stored water elevation. Under such circumstances, the continuity equation becomes (for short



PLAN VIEW OF EXCAVATED EMERGENCY SPILLWAY



PROFILE ALONG CENTERLINE



CROSS-SECTION AT CONTROL SECTION

EMERGENCY SPILLWAY

FIG. 4-3

finite time periods Δt, in minutes, the hydrograph may be taken as a straight line):

$$\frac{I_1 + I_2}{2}\ 60\ \Delta t - \frac{O_1 + O_2}{2}\ 60\ \Delta t = S_2 - S_1 \qquad (4\text{-}20)$$

in which I_1 and I_2 are the inflow rates in cfs at the beginning and end of the time period Δt in minutes; O_1 and O_2 are the outflow rates at the beginning and end of the same time period Δt (the factor 60 converts the time period to seconds); S_2 and S_1 are the storage volumes in cubic feet at the beginning and end of the time period Δt. In short: inflow mass less outflow mass equals change in storage. This can be rearranged as follows:

$$\frac{I_1 + I_2}{2} + \frac{S_1}{60\ \Delta t} - \frac{O_1}{2} = \frac{S_2}{60\ \Delta t} + \frac{O_2}{2} \ \dots\dots\dots\dots (4\text{-}21)$$

At the beginning of any routing period t, all parameters on the left side of the equation are known. Fortunately, the right side parameters are directly related in the storage-discharge curve, if it is assumed that the water surface of the impoundment is horizontal (with no significant backwater). I_1 and I_2 can be obtained from the inflow hydrograph and S_1 is known for the starting depth; the outflow for the amount of storage at the starting depth can be calculated or obtained from the discharge-storage curve or relationship.

The following examples illustrate the relative simplicity of solving equation 4-21:

4.6.4.1 <u>Example 4-1</u>

Given: Area = 210 acres or 0.33 square miles

L = 0.85 mile

L_{ca} = 0.59 mile

Traversed by highway 2000 feet long, 240 feet wide; 11-acre right-of-way;

5.1-acre pavement;

Impervious area = 40%

Pervious area = 60%

Assume unit duration = 10 minutes

Storage basin equivalent to one with bottom dimensions of 170 feet by 340 feet and 2:1 side slopes; a single outlet draining the lowest point; and a broad-crested overflow spillway.

Determine outflow hydrograph with peak no greater than 60% of the maximum inflow with inflow hydrograph based upon 10-year 30-minute rainfall at Denver, Colorado. Excess precipitation the same as was used in Table 3-9.

Step 1: Using the Denver Synthetic Unit Hydrograph formulas (Equations 3-15 and 3-16), develop the 10-minute unit hydrograph

$$C_t = 7.81/40^{.78} = 0.44$$

$$t_p = 0.44(0.85 \times 0.59)^{.3} = .36 \text{ hour} = 22 \text{ minutes}$$

$$C_p = 0.89 \times .44^{.46} = 0.61$$

$$q_p = \frac{640 \times 0.61}{.36} = 1084 \text{ cfs/sm}$$

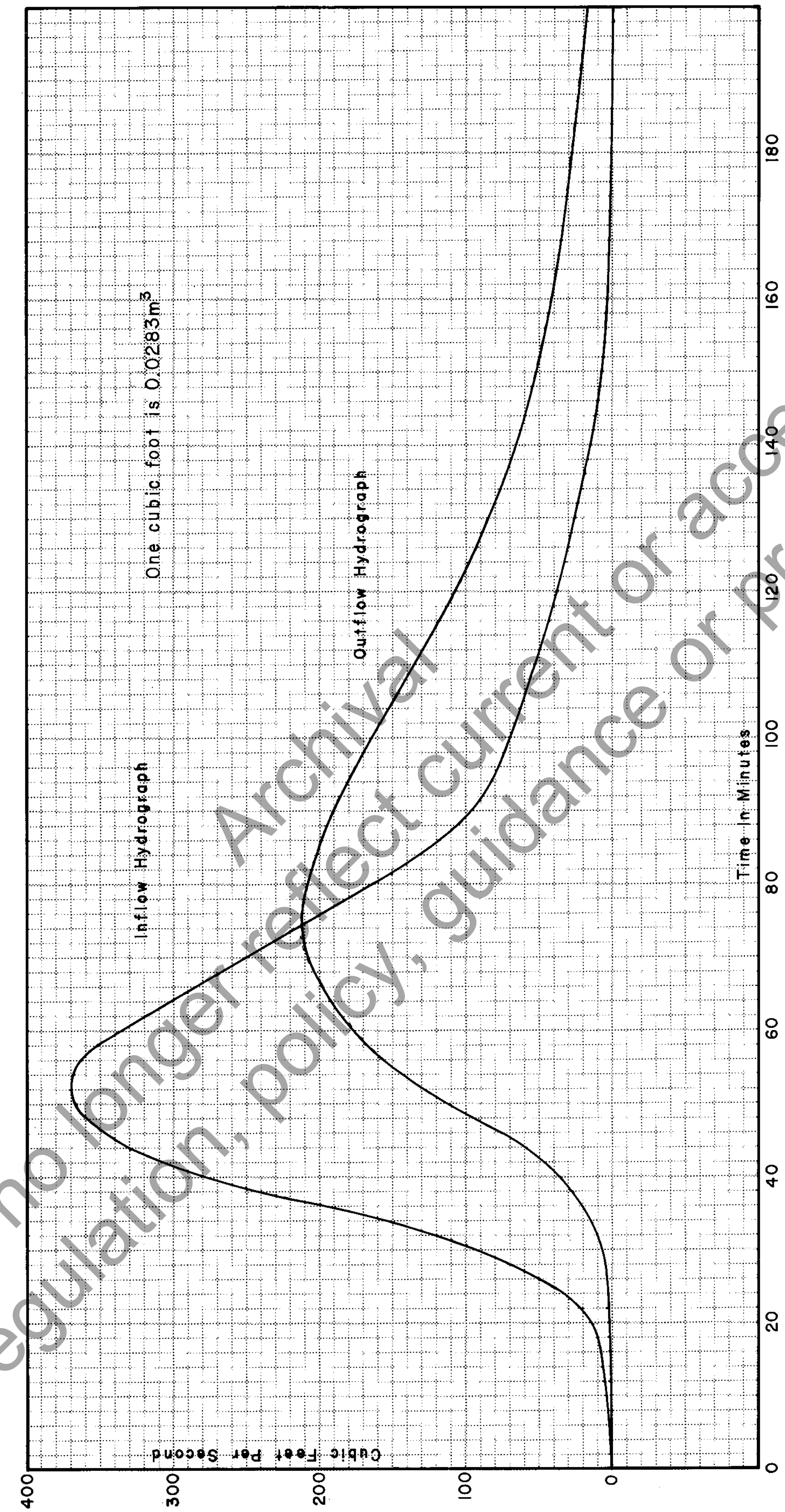
$$Q_p = 1084 \times 0.33 = 358 \text{ cfs}$$

$$T_p = 22 + 10/2 = 27 \text{ minutes} = 0.45 \text{ hour}$$

Step 2: With unit hydrograph Q assumed as 358 cfs and T_p as 0.45 hour, the CSC dimensionless unit hydrograph results in the inflow hydrograph of Fig. 4-4 with 10-minute ordinates as given in column 14 of Table 4-2.

Step 3: Using FHWA HEC-5 and HEC-10 select an outlet pipe size and assuming a maximum outflow of 220 cfs with a maximum water depth of 8 to 10 feet (and a culvert length of 70 feet), determine the head-discharge relationship. Chart 2 of HEC-5 indicates a 60-inch pipe under 8 feet total head above its invert would handle about 220 cfs, so it is decided to develop the head-discharge curve for a 60-inch pipe. Fig. 4-5 and column 2 of Table 4-3 give the curve and tabulation of the data.

Step 4: Assume a detention basin with a depth-volume curve such as would result from a rectangular basin with a bottom 170 feet by 340 feet and 2:1 side slopes. Such a curve is given by Fig. 4-5. The surface area of the pond structure at 10-foot depth would be 79,800 square feet or 1.8 acres; at 8-foot depth 75,144 square feet or 1.7 acres; and 70,616 square feet (almost 1.6 acres) at 6-foot depth. Table 4-3 column 3 gives the total storage below each foot of elevation up to a depth of 10 feet. Note that storage is given in cfs-minutes which is the actual cubic feet of storage divided by 60.



INFLOW AND OUTFLOW HYDROGRAPHS

STORAGE ROUTING EXAMPLE NO. 4-1

FIG. 4-4

DETERMINATION OF STORM HYDROGRAPH (EXAMPLE 4-1)

Time (Min.) (1)	Unit Hydrograph (cfs) (2)	Excess Precipitation in Inches .02 (3)	.05 (4)	.74 (5)	.24 (6)	.17 (7)	06 (8)	03 (9)	03 (10)	02 (11)	02 (12)	02 (13)	Hydrograph (cfs) (14)
0	0												0
10	98	2.0											2.0
20	309	6.2	5.0										11.2
30	350	7.0	15.4	73									95.4
40	250	5.0	17.5	229	23.5								275
50	129	2.6	12.5	259	74	16.7							365
60	72	1.4	6.5	185	84	52	5.9						334.8
70	38.3	0.8	3.6	95	60	60	18.5	2.9					259
80	21.2	0.4	1.9	53	31	43	21.0	9.3	2.9				162
90	11.7	0.2	1.1	28	17.3	22	15.0	10.3	9.3	2.0			95
100	6.4	0.1	0.6	15.6	9.2	12.2	7.7	7.5	10.3	6.2	2.0		71
110	5.1	0.1	0.3	8.7	5.1	6.5	4.3	3.9	7.5	7.0	6.2	2.0	52
120	3.7	0.1	0.3	4.7	2.8	3.6	2.3	2.2	3.9	5.0	7.0	6.2	38
130	0	0	0.2	3.8	1.5	2.0	1.3	1.1	2.2	2.6	5.0	7.0	26
140			0	2.7	1.2	1.1	0.7	0.6	1.1	1.4	2.6	5.0	16
150				0	0.9	0.9	0.4	0.4	0.6	0.8	1.4	2.6	8
160					0	0.6	0.3	0.2	0.4	0.4	0.8	1.4	4
170						0	0.2	0.2	0.2	0.2	0.4	0.8	2
180							0	0.1	0.2	0.1	0.2	0.4	1
190								0	0.1	0.1	0.1	0.2	0.5
200									0	0.1	0.1	0.1	0.3
210										0	0.1	0.1	0.2
220											0	0.1	0.1
230												0	0

One inch is 25.4mm. One cubic foot is $0.0283 m^3$

TABLE 4-2

EXAMPLE 4-1
HEAD-STORAGE & HEAD-DISCHARGE
STORAGE ROUTING COMPUTATIONS

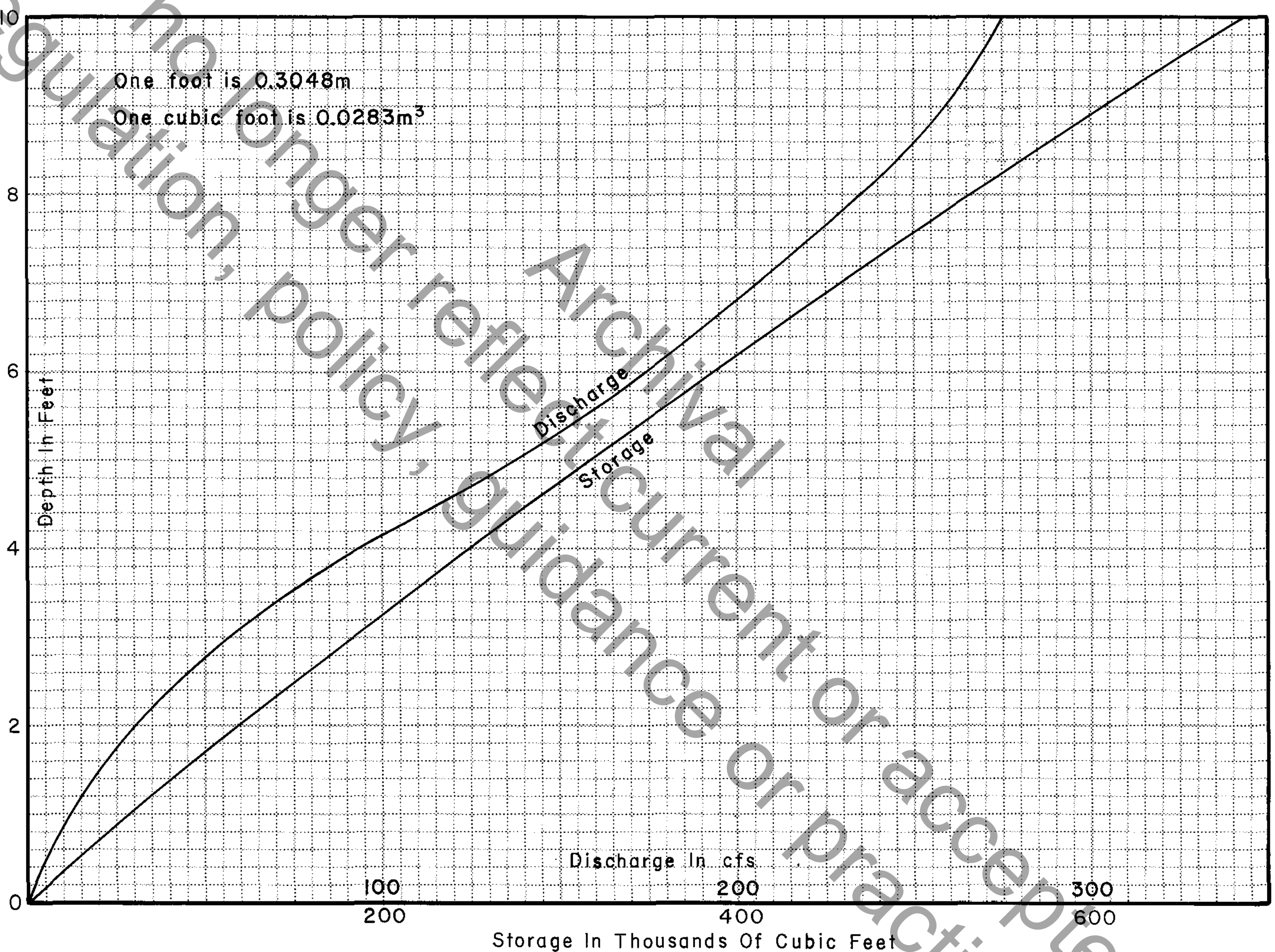


FIG. 4-5

STORAGE-INDICATION COMPILATION TABLE
EXAMPLE 4-1

Elevation (ft.) (1)	Discharge O_2 (cfs) (2)	Storage S_2 (cfs-min) (3)	$\frac{O_2}{2}$ (cfs) (4)	Δt = 10 min. $\frac{S_2}{\Delta t}$ (cfs) (5)	$\frac{S_2}{\Delta t}+\frac{O_2}{2}$ (cfs) (6)
0	0				
1	14	980.5	7	98.0	105.0
2	32	1995.7	16	199.6	215.6
3	57	3046.6	28.5	304.7	333.2
4	94	4167	47	416.7	463.7
5	137	5269	68.5	526.9	595.4
6	175	6422	87.5	642.2	729.7
7	205	7622	102.5	762.2	864.7
8	235	8863	117.5	886.3	1003.8
9	260	10144	130	1014.4	1144.4
10	275	11467	137.5	1146.7	1284.2

TABLE 4-3

One cubic foot is $0.0283 m^3$
One foot is 0.3048m

STORAGE ROUTING COMPUTATIONS
EXAMPLE 4-1

Routing Interval (1)	Time min. (2)	Inflow cfs (3)	Avg. Inflow cfs (4)	$S_1/\Delta t+O_1/2$ cfs (5)	O_1 cfs (6)	$S_2/\Delta t+O_2/2$ cfs (7)	O_2 cfs (8)
1	0	0	0	0	0	0	0
2	10	2.0	1.0	0	0	1.0	0.1
3	20	11.2	6.6	1.0	0.1	7.5	1
4	30	95	53.1	7.5	1	59.6	7.7
5	40	275	185	59.6	7.7	236.9	36
6	50	365	320	236.9	36.0	520.9	112
7	60	335	350	520.9	112	758.9	175
8	70	259	297	759	175	881	208
9	80	162	210.5	881	208	883.5	209
10	90	95	128.5	883.5	209	803	191
11	100	71	83	803	191	695	164
12	110	52	61.5	695	164	593	136
13	120	38	45	593	136	502	106
14	130	26	32	502	106	428	84
15	140	16	21	428	84	365	65
16	150	8	6	365	65	312	52
17	160	4	3	312	52	266	42
18	170	2	1.5	266	42	227	34
19	180	1	0.75	227	34	191	28
20	190	0.5	0.4	191	28	164	26
21	200	0.3	0.25	164	26	138	19
22	210	0.2	0.15	138	19	119	16
23	220	0.1	0.05	119	16	103	14
24	230	0	0	103	14	89	12
25	240			89	12	77	10
26	250			77	10	67	8.8
27	260			67	8.8	58.2	7.5
28	270			58.2	7.5	50.7	6.4
29	280			50.7	6.4	44.3	5.7
30	290			44.3	5.7	38.6	4.8
31				38.6	4.8	33.8	4.2

TABLE 4-4

Step 5: Complete Table 4-3 by computing the proper values to be inserted in columns 4, 5 and 6. Then plot on Fig. 4-6 the curve of $S_2/\ \Delta t + O_2/2$ against O_2.

Step 6: Set up and complete storage routing, Table 4-4.

Column 1 - routing interval sequence for ease of reference.

Column 2 - insert cumulative time in 10-minute intervals.

Column 3 - from inflow hydrograph as developed in Step 2.

Column 4 - average inflow in each 10-minute interval.

Column 5 - start with 0 in routing interval No. 1. Each subsequent figure in this column is the same as that in column 7 on the line immediately preceding.

Column 6 - start with 0 in routing interval No. 1. Each subsequent figure in this column is the same as that in column 8 of the preceding line.

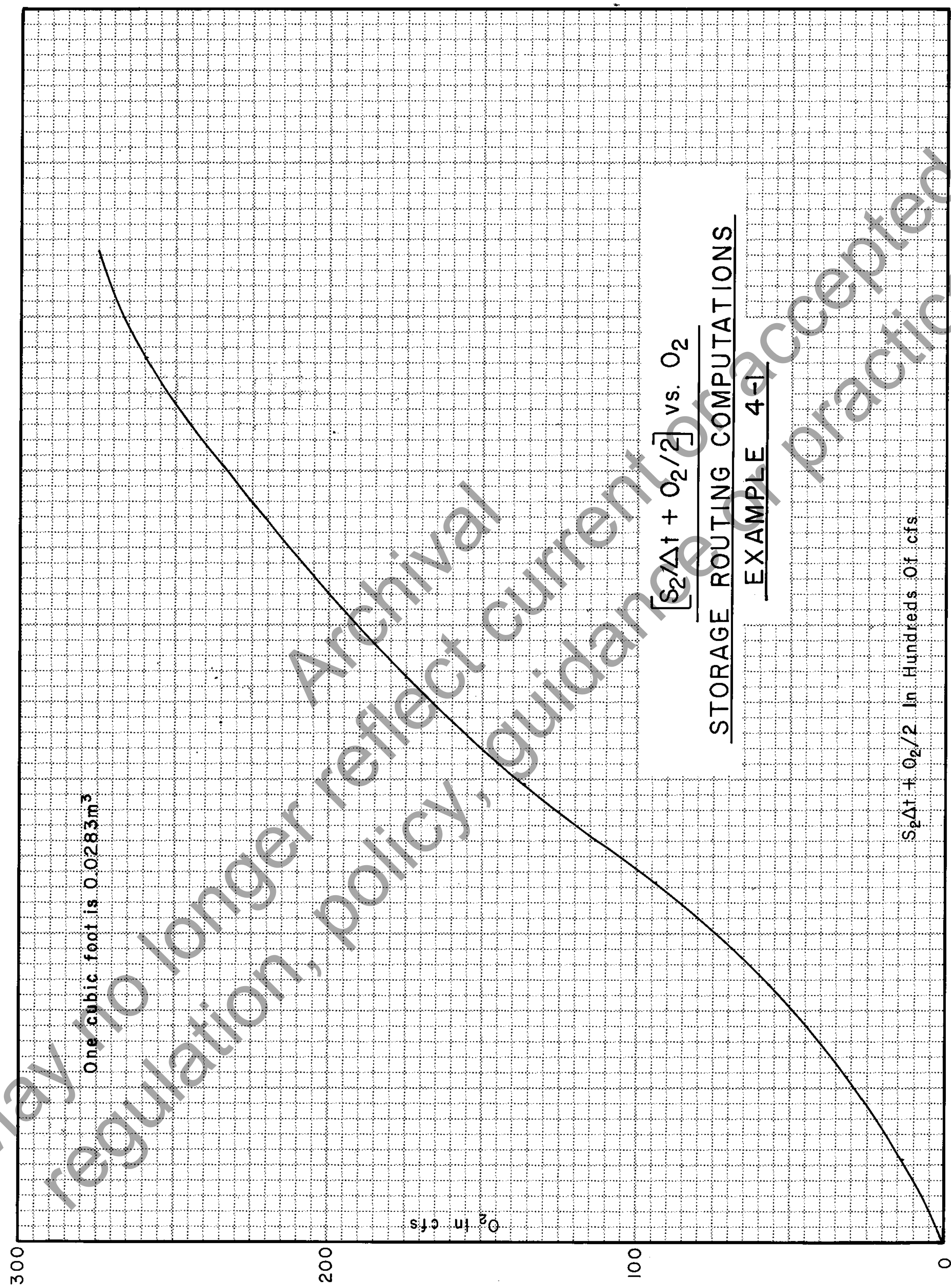
Column 7 - column 4 plus column 5 minus column 6.

Column 8 - enter the curve on Fig. 4-6 with the value in column 7 and read off the related value of O_2 which is inserted in column 8.

Note that Table 4-4 is a tabular way of solving the storage equation 4-21. The O_2 at the end of each routing interval becomes the O_1 at the beginning of the following interval. The value of $S_2/\ \Delta t + O_2/2$ at the end of each routing interval becomes the value of $S_1/\ \Delta t + O_1/2$ at the beginning of the next interval. And when O_1 (column 6) is subtracted from $S_1/\ \Delta t + O_1/2$ (column 5), it becomes $S_1/\ \Delta t - O_1/2$. This added to the average inflow (column 4) results in $S_2/\ \Delta t + O_2/2$ (column 7), all as given by equation 4-21.

Step 7: From the discharge-storage and depth-discharge curves of Fig. 4-5 the maximum storage required for the peak outflow rate of 209 cfs will be 466,000 cubic feet which occurs at a depth of 7.12 feet. This suggests that the emergency overflow spillway could be set at 8 feet.

Step 8: Estimate the probable maximum emergency spillway rate. Precipitation data are to be obtained from the most recent National Weather Service publication (Table 2-1 and Figs. 2-14, 2-15, 2-16) applicable to the area under study. The



[S2/Δt + O2/2] vs. O2
STORAGE ROUTING COMPUTATIONS
EXAMPLE 4-1
One cubic foot is 0.0283m3
O2 in cfs
S2Δt + O2/2 In Hundreds Of cfs
0
100
200
300
0
2
4
6
8
10
12
14

FIG. 4-6

1-hour 100-year rainfall will often be the desirable basis of design of the principal or emergency spillway, although where lives or high property values would be endangered by a breached detention basin, the probable maximum precipitation (PMP)(Table 2-1, Section C) should be used. The methods of runoff determination discussed in Chapter 3 may be used.

For this example the 60-minute 100-year precipitation at Denver of 2.25 inches will be used. A rational method C of 0.95 will be assumed. The peak 100-year 60-minute runoff will then be 0.95 x 2.25 x 210 = 449 cfs. Should it be desirable to assume the PMP, it would be 0.95 x 21.5 x 210 or 4300 cfs. The former of these would be reduced somewhat (perhaps 30%) by the assumed storage but the great size of the PMP assures complete flooding of the assumed storage with an outflow rate equal to the inflow rate. Actually, a specific design for so great an outflow would make it essential to carry out thorough detailed studies to have confidence that the spillway provided was satisfactory. The entire dam probably would become an overflow spillway and would need to be constructed accordingly.

Assuming the available storage would reduce the 100-year peak to about 315 cfs, the emergency spillway could then be designed as follows:

Using equation 4-14 with the assumption of C_{es} of 3.0 and H_p of 3.0, b is found to be 61 feet. If an H_p of 4.0 can be tolerated, the length of the weir could be shortened to 53 feet. Each foot of height of the dam increases its base width by 4 feet so it becomes a matter of the economic choice of broad-crested weir depth as opposed to critical velocity of flow through the weir and cost of dam fill. The 3-foot deep flow would have a critical velocity (assuming a turf n of 0.04) of 5.5 fps and a critical slope of 2.4%; the 4-foot deep flow would involve a critical velocity of 5.8 fps and 2.3% critical slope. While these velocities are a bit high for turf, the rare 1% frequency of their likelihood makes it feasible to decide upon a dem height of 12 feet assuming the sill of the overflow weir at 8-foot depth plus an overflow depth of 3 feet (and a related 61-foot length of weir along the axis of the dam or related thereto as topography best dictates) with a 1-foot freeboard.

For a thorough treatment of the design of emergency spillways for small dams refer to Refs. 4-11 and 4-12.

4.6.4.2 <u>Example 4-2:</u>

Given: A stretch of divided highway is symmetrical with 1.2% grades either side of a sump. Descending tangents each 820 feet in length and 225 feet in right-of-way width deliver runoff from 75% impervious areas (8 traffic lanes with shoulders and two 24-foot service roads). The total tributary area to the sump is 8.5 acres (two identical 4.25-acre watersheds). It is desired to determine the peak runoff into the sump from a 50-year frequency runoff and provide (a) storage sufficient to permit reasonable pumping rates to dispose of the runoff; or

(b) suitable storage to reduce the peak outflow to about 22 cfs, the capacity of the outlet channel.

General Procedure: Because routing through storage requires an inflow hydrograph, a 5-minute unit hydrograph will be developed and applied to the effective rainfall from a 60-minute 50-year rainfall to obtain an inflow hydrograph. The unit time of 5 minutes is chosen because of the small size of the tributary area (4.25 acres, duplicated due to symmetry). The 60-minute 50-year rainfall assures reasonable antecedent precipitation prior to the peak 5 minutes (the assumed concentration time of the 4.25 acres) and is consistent with the discussion in Chapter 3, as all procedures in this example are in conformity with the matters discussed in earlier chapters.

Five-Minute Unit Hydrograph: Using the empirical equations of Table 4-5 obtained from Ref. 4-14, the 10-minute unit hydrograph of Fig. 4-7 is drawn. Utilizing the principle of superposition, by off-setting the 10-minute unit hydrograph at 10-minute intervals, a 10-minute, S-curve can be tabulated as indicated in column 3 of Table 4-6 (with 5-minute ordinates read from the plotted work graph of the 10-minute S-curve). Ref. 4-15 gives especially clear detailed discussion of the S-curve and its use to develop unit hydrographs of longer or shorter unit rainfalls. The S-curve represents the runoffs resulting from a sequence of 1-inch effective rainfalls until the runoff rate becomes equivalent to the supply rate (the effective rainfall).

Again utilizing the superposition principle, the 10-minute unit hydrograph can be the basis for determining a 5-minute unit hydrograph as given in Table 4-6. The previously determined 10-minute S-curve values are entered at 5-minute intervals in column 3. Offset of lagged by 5 minutes, they are entered again in column 4. Column 5 then lists the difference between column 3 and column 4 which gives a hydrograph resulting from 1/2-inch of effective rainfall in 5 minutes. Under the unit hydrograph theory, column 6 which has ordinates twice those in column 5, then is the 5-minute unit hydrograph resulting from 1 inch of effective rainfall.

Determination of Effective Rainfall

As discussed in Chapter 3, to develop the inflow hydrograph, it is necessary to determine the effective or excess rainfall at 5-minute intervals. Table 4-7 computes the effective rainfall for an assumed 50-year 60-minute rainfall in the St. Louis metropolitan area. The values in column 2 are obtained from Table 2-2 converted to inches and interpolated where necessary. Column 4 is an arbitrary arrangement, placing the maximum 5 minutes at about the 40% point in the assumed 1-hour rain. The infiltration is assumed at an inch per hour uniform rate. Depression storage is assumed as 0.25-inch on the pervious areas and 0.10-inch on the impervious. For the assumption of 75% impervious area, column 12 gives the sequence of 5-minute

TEN-MINUTE UNIT HYDROGRAPH EQUATIONS

Equations	Total Explained Variation
$T_R = 3.1\ L^{0.23} S^{-0.25} I^{-0.18} \phi^{1.57}$	0.802
$Q = 31.62\times10^3 A^{0.96} T_R^{-1.07}$	0.936
$T_B = 125.98\times10^3 A\ Q^{-0.95}$	0.844
$W_{50} = 16.22\times10^3 A^{0.93} Q^{-0.92}$	0.943
$W_{75} = 3.24\times10^3 A^{0.79} A^{-0.78}$	0.834

L is the total distance (in feet) along the main channel from the point being considered to the upstream watershed boundary.

S is the main channel slope (in feet per foot) as defined by H/(0.8L), where L is the main channel length as described above and H is the difference in elevation between two points, A and B. A is a point on the channel bottom at a distance of 0.2L downstream from the upstream watershed boundary. B is a point on the channel bottom at the downstream point being considered.

I is the impervious area within the watershed (in percent).

ϕ is the dimensionless watershed conveyance factor as described previously in the text of Ref. 4-14.

A is the watershed drainage area (in square miles).

T_R is the time of rise of the unit hydrograph (in minutes).

Q is the peak flow of the unit hydrograph (in cfs).

T_B is the time base of the unit hydrograph (in minutes).

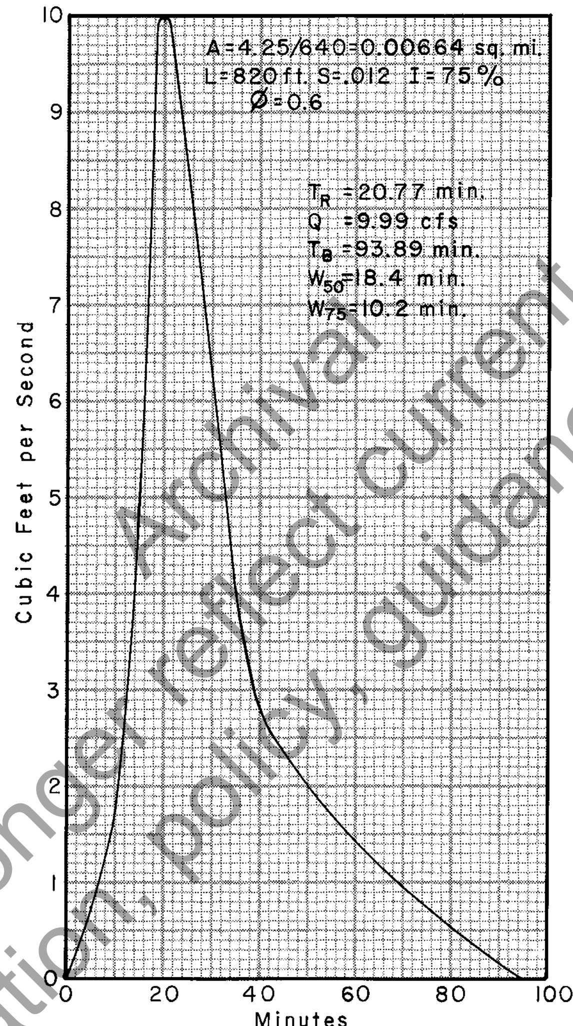
W_{50} is the width of the unit hydrograph at 50% of Q (in minutes).

W_{75} is the width of the unit hydrograph at 75% of Q (in minutes).

One foot is 0.3048m. One square mile is 2.59 km^2

TABLE 4-5
(From Ref. 4-14)

10 MINUTE UNIT HYDROGRAPH
EXAMPLE 4-2



One cubic foot is $0.0283 m^3$
One square mile is $2.59 km^2$

FIG. 4-7

DEVELOPMENT OF 5-MINUTE UNIT HYDROGRAPH FROM 10-MIN. UNIT HYDROGRAPH EXAMPLE 4-2

MIN. (1)	HOURS (2)	S-CURVE 10-MINUTE (3)	LAGGED S-CURVE (4)	(3)-(4) (5)	5-MIN. UNIT GRAPH (6)*
0	0	0		0	0
5	.08	0.25	0	0.25	.50
10	.17	1.70	0.25	1.45	2.90
15	.25	6.7	1.70	5.00	10.00
16.5	.27				11.00
20	.33	11.69	6.7	5.00	10.00
25	.42	15.6	11.69	3.99	7.98
30	.50	18.05	15.6	2.45	4.90
35	.58	19.6	18.05	1.55	3.30
40	.67	20.85	19.6	1.25	2.50
45	.75	22.0	20.85	1.15	2.03
50	.83	22.83	22.0	0.83	1.66
55	.92	23.6	22.83	0.77	1.54
60	1.00	24.25	23.6	0.65	1.20
65	1.08	24.85	24.25	0.60	0.98
70	1.17	25.20	24.85	0.35	0.77
75	1.25	25.5	25.20	0.30	0.60
80	1.33	25.73	25.5	0.23	0.40
85	1.42	25.8	25.73	0.07	0.25
90	1.50	25.87	25.8	0.07	0.10
93	1.55		25.87		0.0

*cfs. One cubic foot is $0.0283 m^3$

TABLE 4-6

DETERMINATION OF EFFECTIVE RAINFALL - EXAMPLE 4-2

	PRECIPITATION			PERVIOUS AREA 25%				IMPERVIOUS AREA 75%			
TIME (MIN)	50-YR TOTAL IN.	INCREMENTAL IN.	REARRANGED INCREMENTAL IN.	MAXIMUM INFILTRATION IN.	DEPRESSION STORAGE IN.	EFFECTIVE PRECIP. IN.	25% EFFECTIVE PRECIP. IN.	DEPRESSION STORAGE IN.	EFFECTIVE PRECIP. IN.	75% EFFECTIVE PRECIP. IN.	TOTAL AVG. EFFECTIVE PRECIP. IN.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
5	.76	.76	.12	.08	.04	-		.05	.07	.0525	.05
10	1.25	.49	.15	.08	.07	-		.05	.10	.0750	.08
15	1.59	.34	.23	.08	.14	.01	.0125		.23	.1725	.19
20	1.87	.28	.49	.08		.41	.1025		.49	.3670	.47
25	2.10	.23	.76	.08		.68	.1700		.76	.5700	.74
30	2.27	.17	.34	.08		.26	.0650		.34	.2550	.32
35	2.42	.15	.28	.08		.20	.0500		.28	.2100	.26
40	2.55	.13	.17	.08		.09	.0225		.17	.1275	.15
45	2.67	.12	.13	.08		.05	.0125		.13	.0975	.11
50	2.78	.11	.11	.08		.03	.0075		.11	.0825	.09
55	2.88	.10	.10	.08		.02	.0050		.10	.0750	.08
60	2.98	.10	.10	.08		.02	.0050		.10	.0750	.08
			2.98	0.96	0.25	1.77	.4525	.10	2.88	2.1595	2.62

One inch is 25.4mm

TABLE 4-7

amounts of effective precipitation.

Determination of Storm Hydrograph

Having the amounts of 5-minute effective precipitation in Table 4-7 and the 5-minute unit hydrograph from Table 4-6, the determination of the storm hydrograph of inflow is obtained from the computations in Table 4-8. Note that column 15 gives the hydrograph for 4.25 acres or one-half the total area. Therefor, because of the symmetry of the two tributary areas, the inflow hydrograph for the total 8.5 acres has ordinates just double those given in column 15.

Mass Curve of Inflow Hydrograph; Possible Pumping

The mass curve for the storm hydrograph is computed as shown in Table 4-9; it is plotted in Fig. 4-8. The peak rate of inflow of 33.0 cfs or 14,800 gpm can be reduced by pumping from storage or by sufficient storage together with outlet control to not overtax the capacity of the outlet facilities. The pumping can be at a constant rate, hence the outflow can be represented on Fig. 4-8 by a straight line and the maximum required storage by the maximum vertical intercept between the maximum mass curve and the sloping line representing the pumping rate. Two possible pumping rates, 10 cfs and 15 cfs are shown together with their associated storages of 35,800 cubic feet and 22,000 cubic feet respectively. Whether to pump or not and what storage and pumping capacity to provide are principally economic determinations.

Wherever gravity disposal is feasible within reasonable cost, drainage by pumping should be avoided. Fairly large expenditures can be justified for gravity drainage since pumping installations have high first and maintenance costs and the possibility of a power failure during a storm (or the costs associated with provision and maintenance of standby power). Long runs of pipe or continuing a depressed grade to a natural low area may be feasible alternates.

Determination of Storage Volume

To have approximately 20,000 cubic feet of storage volume at 4-foot depth, it is assumed that a rectangular basin with a 40-foot by 80-foot bottom and 2:1 side slopes will have satisfactory storage characteristics. Its depth-storage curve is given on Fig. 4-9. Actually, it has 17,000 cubic feet of volume at 4 feet of depth; this is satisfactory for purposes of this problem.

Determination of Depth-Discharge Curve

It is assumed that a pipe at about a 1% construction slope will serve as the outlet control for the earth embankment storage basin. Using references 4-5 or 4-6, the outflow capacities for about 22 cfs under 4 feet of head suggests an 18-inch pipe. Table 4-10 (columns 1 and 2) and Fig. 4-9 give the depth-discharge relationship used to solve problem 4-2.

DETERMINATION OF STORM HYDROGRAPH-EXAMPLE 4-2

TIME (MIN)	UNIT HYDROGRAPH (CFS)	EXCESS PRECIPITATION IN INCHES												HYDGR. (CFS)	2x COL. 15
		.05	.08	.19	.47	.74	.32	.26	.15	.11	.09	.08	.08		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
5	.50	.02	0											.02	.04
10	2.90	.15	.04	0										.19	.38
15	10.00	.50	.23	.10	0									.83	1.66
20	10.00	.50	.80	.55	.24	0								3.09	6.18
25	7.98	.40	.80	1.90	1.36	.37	0							4.83	9.66
30	4.90	.24	.64	1.90	4.70	2.14	.16	0						9.78	19.56
35	3.30	.17	.39	1.52	4.70	7.40	.93	.13	0					15.24	30.48
40	2.50	.12	.26	.93	3.74	7.40	3.20	.76	.07	0				16.48	32.96
45	2.03	.10	.20	.63	2.30	5.90	3.20	2.60	.44	.06	0			15.43	30.86
50	1.66	.08	.16	.47	1.55	3.62	2.55	2.60	1.50	.32	.05	0		12.90	25.80
55	1.54	.08	.13	.39	1.17	2.44	1.57	2.07	1.50	1.10	.26	.04	0	10.75	21.50
60	1.20	.06	.12	.31	.95	1.84	1.06	1.27	1.20	1.10	.90	.23	.04	9.08	18.16
65	0.98	.05	.10	.29	.78	1.50	.80	.86	.74	.88	.90	.80	.23	7.93	15.86
70	0.77	.04	.08	.23	.72	1.23	.65	.65	.49	.54	.72	.80	.80	6.95	13.90
75	0.60	.03	.06	.19	.56	1.14	.53	.53	.38	.36	.44	.64	.80	5.66	11.32
80	0.40	.02	.05	.15	.46	.89	.49	.43	.31	.28	.30	.39	.64	4.41	8.82
85	0.25	.01	.03	.11	.36	.72	.39	.40	.25	.22	.22	.26	.39	3.36	6.72
90	0.10	00	.02	.08	.28	.57	.31	.31	.23	.18	.18	.20	.26	2.62	5.24
95			.01	.05	.19	.44	.25	.25	.18	.17	.15	.16	.20	2.05	4.10
100				.02	.12	.30	.19	.20	.15	.13	.14	.13	.16	1.54	3.08
105					.05	.18	.13	.16	.12	.11	.11	.12	.13	1.11	2.22
110						.07	.08	.10	.09	.08	.09	.10	.12	.73	1.46
115							.03	.07	.06	.07	.07	.08	.10	.48	.96
120								.03	.04	.04	.05	.06	.08	.30	.60
125									.02	.03	.04	.05	.06	.20	.40
130										.01	.02	.03	.05	.11	.22
135											.01	.02	.03	.06	.12
140												.01	.02	.03	.06
145													.01	.01	.02

One inch is 25.4mm. One cubic foot is $0.0283m^3$

TABLE 4-8

MASS RUNOFF FROM STORM HYDROGRAPH
EXAMPLE 4-2

MIN. TIME	CFS	CF	ΣCF
0	0		
5	.04	6	6
10	.38	63	69
15	1.66	306	375
20	6.18	1176	1551
25	9.66	2376	3927
30	19.56	4383	8310
35	30.48	7506	15816
40	32.96	9516	25332
45	30.86	9576	34908
50	25.80	8499	43407
55	21.50	7095	50502
60	18.16	5949	56451
65	15.86	5103	61554
70	13.90	4464	66018
75	11.32	3783	69801
80	8.82	3021	72822
85	6.72	2331	75153
90	5.24	1794	76947
95	4.10	1301	78248
100	3.08	1077	79325
105	2.22	795	80120
110	1.46	552	80672
115	.96	363	81035
120	.60	234	81269
125	.40	150	81419
130	.22	93	81512
135	.12	51	81563
140	.06	27	81590
145	.02	12	81602
150	0	3	81605

One cubic foot is $0.0283 m^3$

TABLE 4-9

MASS RUNOFF INFLOW HYDROGRAPH
EXAMPLE 4-2

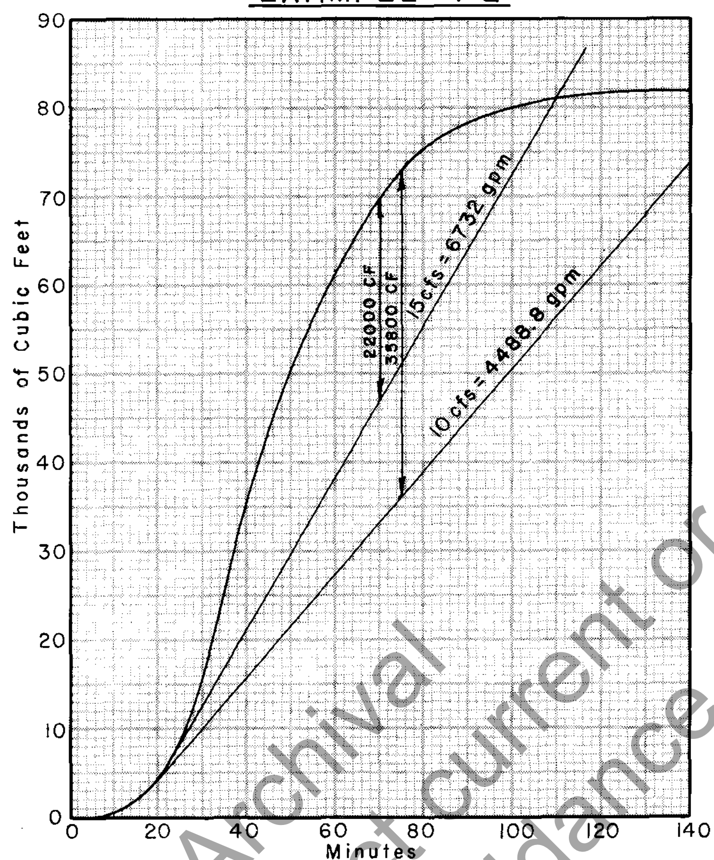


FIG. 4-8

One foot is 0.3048m One cubic foot is $0.0283m^3$

DEPTH-STORAGE & DEPTH-DISCHARGE
EXAMPLE 4-2

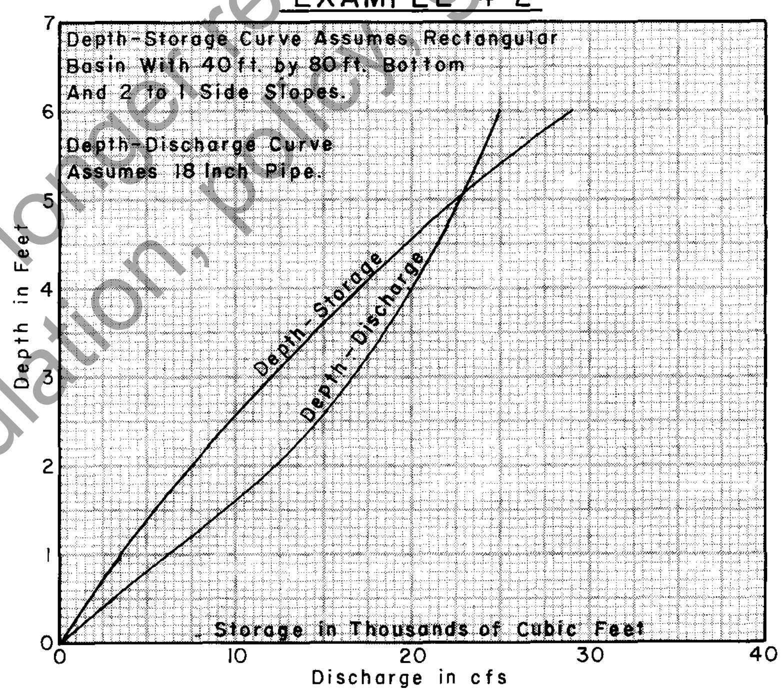


FIG. 4-9

STORAGE-INDICATION COMPILATION TABLE

Elevation (ft.)	Discharge O_2 (cfs)	Storage S_2 (cfs-min)	$\frac{O_2}{2}$ (cfs)	$\Delta t = 5$ min. $\frac{S_2}{\Delta t}$ (cfs)	$\frac{S_2}{\Delta t} + \frac{O_2}{2}$ (cfs)
(1)	(2)	(3)	(4)	(5)	(6)
1	6.1	58	3.05	11.6	14.65
2	12.5	124	6.25	24.8	31.05
3	16.7	199	8.35	39.8	48.15
4	19.8	283	9.90	56.6	66.50
5	22.6	378	11.30	75.6	86.90
6	25.0	483	12.50	96.6	109.10

One foot is 0.3048m. One cubic foot is $0.0283m^3$

EXAMPLE 4-2

TABLE 4-10

Routing Procedure

The same routing procedure discussed in Section 4.6.4 is used for Example 4-2. Table 4-10 for Example 4-2 is similar to Table 4-3 for Example 4-1. Columns 1, 2 and 3 are completed from the depth-discharge and depth-storage computations with the column 3 values the actual cubic feet of storage divided by 60. Columns 4, 5 and 6 are self-explanatory.

From Table 4-10 the curve of $S_2/\ \Delta t + O_2/2$ versus O_2 is plotted on Fig. 4-10.

The actual storage routing computations then proceed as shown in Table 4-11 which is a tabular solution of the storage equation. The following explains the table:

STORAGE ROUTING COMPUTATIONS

Routing Interval	Time Min.	Inflow cfs	Avg. Inflow cfs	$S_1/\Delta t+O_1/2$ cfs	O_1 cfs	$S_2/\Delta t+O_2/2$ cfs	O_2 cfs
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	0	0	0	0	0	0	0
2	5	.04	.02	0	0	.02	.01
3	10	.38	.20	.02	.01	.21	.01
4	15	1.66	1.02	.21	.01	1.22	.30
5	20	6.18	3.92	1.22	.30	4.84	2.00
6	25	9.66	7.92	4.84	2.00	10.76	4.50
7	30	19.56	14.61	10.76	4.50	20.87	8.80
8	35	30.48	25.02	20.87	8.80	37.09	14.10
9	40	32.96	31.72	37.09	14.10	54.71	17.30
10	45	30.86	31.91	54.71	17.30	69.32	20.10
11	50	25.80	28.33	69.32	20.10	77.55	21.40
12	55	21.50	23.65	77.55	21.40	79.80	21.70
13	60	18.16	19.83	79.80	21.70	77.93	21.50
14	65	15.86	17.01	77.93	21.50	73.44	20.90
15	70	13.90	14.88	73.44	20.90	67.42	19.90
16	75	11.32	12.61	67.42	19.90	60.13	18.90
17	80	8.82	10.07	60.13	18.90	51.30	17.20
18	85	6.72	7.77	51.30	17.20	41.87	15.10
19	90	5.24	5.98	41.87	15.10	32.75	13.00
20	95	4.10	4.67	32.75	13.00	24.42	10.10
21	100	3.08	3.59	24.42	10.10	17.91	7.50
22	105	2.22	2.65	17.91	7.50	13.06	5.50
23	110	1.46	1.84	13.06	5.50	9.40	3.90
24	115	.96	1.21	9.40	3.90	6.71	2.80
25	120	.60	.78	6.71	2.80	4.69	1.90
26	125	.40	.50	4.69	1.90	3.29	1.40
27	130	.22	.31	3.29	1.40	2.20	0.90
28	135	.12	.17	2.20	0.90	1.47	0.60
29	140	.06	.09	1.47	0.60	0.96	0.40
30	145	.02	.04	0.96	0.40	0.60	0.20
31	150	0	.01	0.60	0.20	0.41	0.10
32	160		0	0.41	0.10	0.31	0.08
	165			0.31	0.08	0.23	0.05
							0

One cubic foot is $0.0283 m^3$

EXAMPLE 4-2

TABLE 4-11

Column 1 - routing sequence for ease of reference.

Column 2 - cumulative time in 5-minute intervals.

Column 3 - from inflow hydrograph, Table 4-8, column 16.

Column 4 - average inflow in each 5-minute interval.

Column 5 - start with zero (0) in routing interval No. 1. Each subsequent figure in this column is the same as that in column 7 on the line immediately preceding.

Column 6 - start with zero (0) in routing interval No. 1. Each subsequent figure in this column is the same as that in the preceding line of column 8.

Column 7 - column 4 plus column 5 minus column 6.

Column 8 - enter the curve on Fig. 4-10 with the value in column 7 and read the related value of O_2 which is inserted in column 8.

Note from the foregoing that O_2 at the end of each routing interval becomes the O_1 at the beginning of the following interval. The value of $S_2/\Delta t + O_2/2$ at the end of each routing interval becomes the value of $S_1/\Delta t + O_1/2$ at the beginning of the next interval. And when O_1 (column 6) is subtracted from $S_1/\Delta t + O_1/2$ (column 5), it becomes $S_1/\Delta t - O_1/2$. This added to the average inflow (column 4) results in $S_2/\Delta t + O_2/2$ (column 7), all as given by equation 4-21.

Fig. 4-11 shows both the inflow and outflow hydrographs for Example 4-2. The curves of Fig. 4-9 indicate the assumed storage would reach a maximum depth of 4.65 feet and utilize a maximum storage of 20,600 cubic feet. The emergency spillway could be set at 5.5 feet.

Emergency Spillway, Example 4-2

Precipitation data can be obtained from the most recent National Weather Service publication (Table 2-1 and Figs. 2-14, 2-15, 2-16) applicable to the area under study. The 1-hour 100-year rainfall will often be the desirable basis of design of the principal or emergency spillway although where lives or high property values would be endangered by a breached detention basin, the probable maximum precipitation (PMP)(Table 2-1, Section C) should be used. The method of runoff determination discussed in Chapter 3 may be used.

For this example (4-2) the 60-minute 100-year precipitation in the St. Louis metropolitan area of 3.30 inches will be used. A rational method C of 0.95 will be assumed. The peak 100-year 60-minute runoff will then be 0.95 x 3.30 x 4.25 x 2 = 26.65 cfs. Since the unit

$[S_2/\Delta t + O_2/2]$ VS. O_2 EXAMPLE 4-2

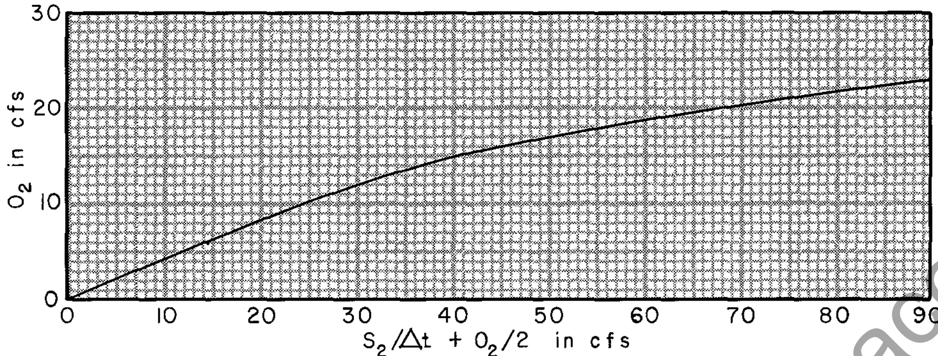


FIG. 4-10

One cubic foot is $0.0283 m^3$

INFLOW & OUTFLOW HYDROGRAPHS STORAGE ROUTING COMPUTATIONS EXAMPLE 4-2

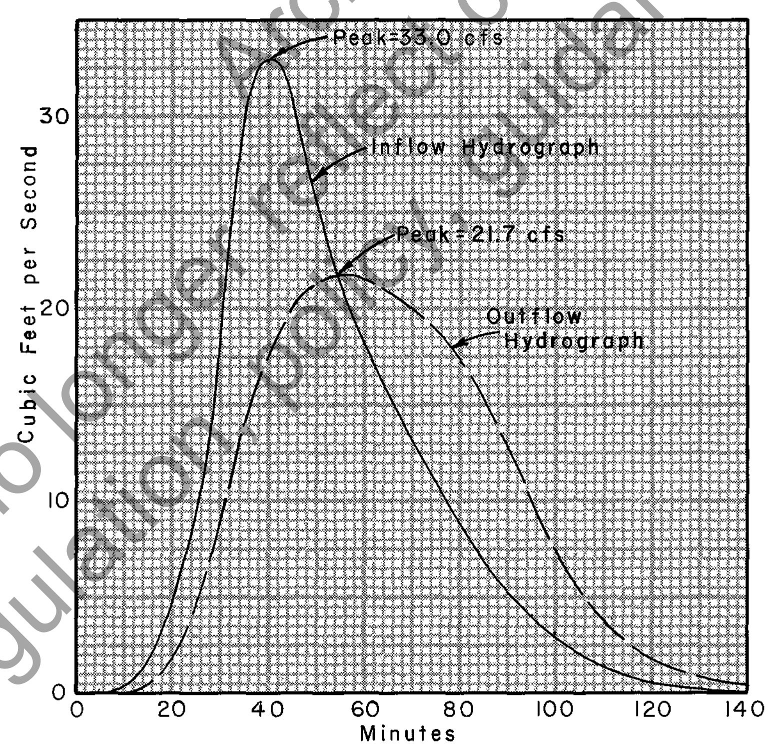


FIG. 4-11

hydrograph determination of the 5-minute 50-year maximum runoff gave 33 cfs, the 100-year 60-minute peak will be increased by 40% resulting in a probable 5-minute 100-year peak of 37.5 cfs. Should it be desirable to assume the PMP, it would be at least 0.95 x 27 x 8.5 or about 220 cfs. The former of these (37.5 cfs) would be reduced by about 35% by the assumed storage but the great magnitude of the PMP virtually assures complete flooding of the assumed storage with an outflow rate practically equal to the inflow rate. Actually, a specific design for so great an outflow would make it essential to carry out thorough detailed studies to be confident that the spillway provided was satisfactory. The entire dam would probably become an overflow spillway and would need to be constructed accordingly.

Assuming the available storage would reduce the 100-year 5-minute peak to about 25 cfs, Table 4-1 indicates that a 10-foot bottom width (2:1 side slopes) earth spillway could discharge a peak of 26 cfs under a head of 1 foot and a critical velocity of 4.1 feet per second (assuming an n of .040 for a grass-lined spillway). A 7-foot embankment would give 1.35 feet of freeboard above the 100-year 5-minute maximum pool level.

For a thorough treatment of the design of emergency spillways for small dams, see Refs. 4-11 and 4-12.

4.7 Summary of Significant Design Information in Chapter 4

1. Techniques and formulas are presented for the determination of usable depth-discharge relationships for practical outlets of detention basins: (a) a culvert-like pipe through the embankment; (b) a vertical riser connected with an elbow or tee to a flat-sloped pipe through the embankment; (c) an emergency spillway through the embankment.

2. For outlet (b) both a perforated and unperforated vertical riser are evaluated. Also, the flat-sloped pipe connected to the vertical riser is examined as to its capacity relative to that of the riser.

3. The storage equation, which states that for short time periods inflow minus outflow equals change in storage, is discussed and applied to two examples.

4. The steps in the examples of routing through storage include: (a) development of a design storm with a chosen temporal pattern; (b) determination of the net or effective rainfall; (c) determination of the unit hydrograph for the particular watershed; (d) utilization of the dimensionless unit hydrograph; (e) development of the depth-storage relationship for an assumed detention basin; (f) development of the depth-discharge relationship of a selected outlet facility; (g) routing the design hydrograph (determined from (b), (c) and (d) through the assumed detention basin to achieve the outlet hydrograph.

5. The estimation of the probable maximum emergency spillway rate is discussed and illustrated.

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ROADWAY DRAINAGE

5.1 General

The term "roadway drainage" includes the collection and removal of waters from the roadway of urban highways and arterial streets in the most expeditious manner. In some instances, it can involve the use of detention storage after runoff collection and removal from the roadway before ultimate disposal. Included are: (a) surface waters originating within the right-of-way; (b) surface waters originating outside the right-of-way and not confined to channels that would reach the travelled way if not intercepted; (c) surface waters entering the roadway from crossroads or streets.

Highway agency standards and criteria for selecting the design frequency of rainfall should be based on traffic service requirements, compatability with the local community drainage system, the presence or absence of shoulders or parking lanes to convey runoff and the function the proposed drainage facilities will serve in the total storm drainage needs of the immediate area. Where drainage is totally dependent on the storm drain facilities or where damage to other properties could be incurred because of inadequate highway drainage facilities, a larger or rarer rainfall event must logically be considered for design. "Consideration should be given in design to maintenance operations and possible traffic hazards due to sediment deposit on pavement and in the underground system. Special arrangements may be needed for collection and removal without interruption to traffic flow and extra inlets should be installed near low points of sag vertical curves to take overflow from clogged inlets." (Ref. 5-1)

5.2 General Requirements

Certain considerations are applicable to all roadway drainage.

(a) The design rainfall frequency to be used for the runoff determinations must be chosen.

(b) The maximum allowable extent of flooding or spread on the running pavement must be set.

(c) Concentration of sheet flow across pavements should be avoided, e.g. flow across gores or from gutters and shoulders near superelevation reversals. As a general guideline, runoff should be intercepted upstream of these locations in order to minimize, to the extent practicable, the occurrence of concentrated sheet flow across the pavement.

(d) Flows in excess of design frequency will generally overflow from overtaxed structures such as inlets and find their way overland to the nearest natural drainage course or body of water. This latter, in turn, may be out-of-banks or overcharging its outlet. Good design practice requires that such overflow paths be examined sufficiently by the designer to ensure that such excess paths will not damage the roadway and that runoff from the highway will not cause damage to other properties.

5.3 Roadway Drainage Systems

For the purposes of this document roadway drainage systems are collector structures and underground conduits which conduct flows to a single point of discharge. Often, critical problems are encountered where the surrounding local drainage is inadequate. A cooperative project with local participation may be the best solution.

5.4 Flow in Gutters

5.4.1 Factors Governing Capacity

The capacity of a gutter depends upon its cross-section, grade and roughness. The gutter cross-section generally has a right triangular shape with the curb forming the vertical leg of the triangle. The hypotenuse may be part of a straight slope from the pavement crown or it may be composed of two straight lines or on older pavements by a curved line and a straight slope in the gutter.

The effect of the gutter cross-section on capacity can be shown by comparing two gutters both on a 1% longitudinal grade and with a usual n of 0.016 (Table 5-1). One gutter has a straight slope of 3/16 inches per foot (15.63mm per m) from the roadway crown to the curb. The second gutter has the same pavement cross slope but has a 2-foot (0.610m) gutter section with a steeper cross slope of 1 inch per foot (83.33mm per m). If the flow is confined to a 2-foot (0.610m) width from the curb, the straight slope gutter will carry 0.02 cfs (0.00057 m^3/s) and the 2-foot (0.610m) gutter section will carry 0.35 cfs (0.0099 m^3/s). If the water can be allowed a 6-foot (1.829m) spread from the curb onto the pavement, the straight cross slope channel will carry 0.40 cfs (0.0113 m^3/s) as compared with 0.96 cfs (0.0272 m^3/s) for the 2-foot (0.610m) gutter section channel. For a 10-foot (3.048m) spread of water from the curb, the straight cross slope channel will carry 1.59 cfs (0.0450m^3/s) as compared with 2.28 cfs (0.0646m^3/s) for the 2-foot (0.610m) gutter section channel. The 2-foot (0.610m) gutter section has the additional advantage of greater depth of flow at the curb line which increases the capacity of an inlet on a continuous grade. The flow computations are explained in the following paragraphs.

5.4.2 Capacity of Gutters

The Manning equation cannot be used without modification to compute flow in triangular gutter sections because the hydraulic radius does not adequately describe the gutter cross-section, particularly when the top width of water surface (Zd) may be more than 40 times the depth (d) at the curb. To compute gutter flow the Manning equation for an increment of width is integrated across the width Zd (Ref. 5-2) and the resulting formula is:

$$Q = K(Z/n)\ S_o^{1/2} d^{8/3} \qquad \text{.............(5-1)}$$

or solving for d:

$$d = \left[\frac{nQ}{KZ\ S_o^{1/2}}\right]^{3/8} \qquad \text{.............(5-2)}$$

Where Q = rate of discharge in cubic feet per second (cubic metres per second)

Z = reciprocal of the cross slope $\frac{T}{d}$

n = Manning's coefficient of channel roughness

S_o = longitudinal slope in feet per foot (metres per metre)

T = top width of water surface in feet (metres)

d = depth of channel at deepest point, in feet (metres)

K = 0.56 for English units; equals 0.375 for metric units.

The designer is interested in both the depth of flow at the curb (d) and the spread of the water (T) on the pavement at the design discharge and sometimes at other discharges.

The spread of flow on the pavement is often a criterion for spacing inlets. Fig. 5-1 is a nomograph for solving equation 5-1 or 5-2. Instructions for use appear on Fig. 5-1 and examples are given herewith. The chart can also be used for flow computations of shallow V-shaped channels having side slopes flatter than about 10:1. Values of Manning's n are given in Table 5-1.

5.4.3 Gutters With Straight Cross-Section

The use of Fig. 5-1 to compute the depth of flow (d) at the curb and the spread (T) of water on the pavement is illustrated in the following example.

5.4.4 Example 5-1: Straight Cross-Section

Given: Q = 1.0 cfs; concrete pavement and gutter, float finish; cross slope 1/4-inch per foot; longitudinal slope 1%.

Find: depth of flow at curb and spread of flow on pavement.

Solution:

1. From Table 5-1, n = 0.014
2. Z = 48.00 and Z/n = 3429.
3. On Fig. 5-1, lay a straight edge on Z/n = 3429 and channel slope 0.01. Mark intersection of straight edge on turning line.
4. Lay straight edge on point marked in step 3 and the discharge 1.0 cfs. Read depth of flow at the curb, 0.14 feet.
5. The spread on the pavement is Zd or 48(0.14) = 6.72 feet.

NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS

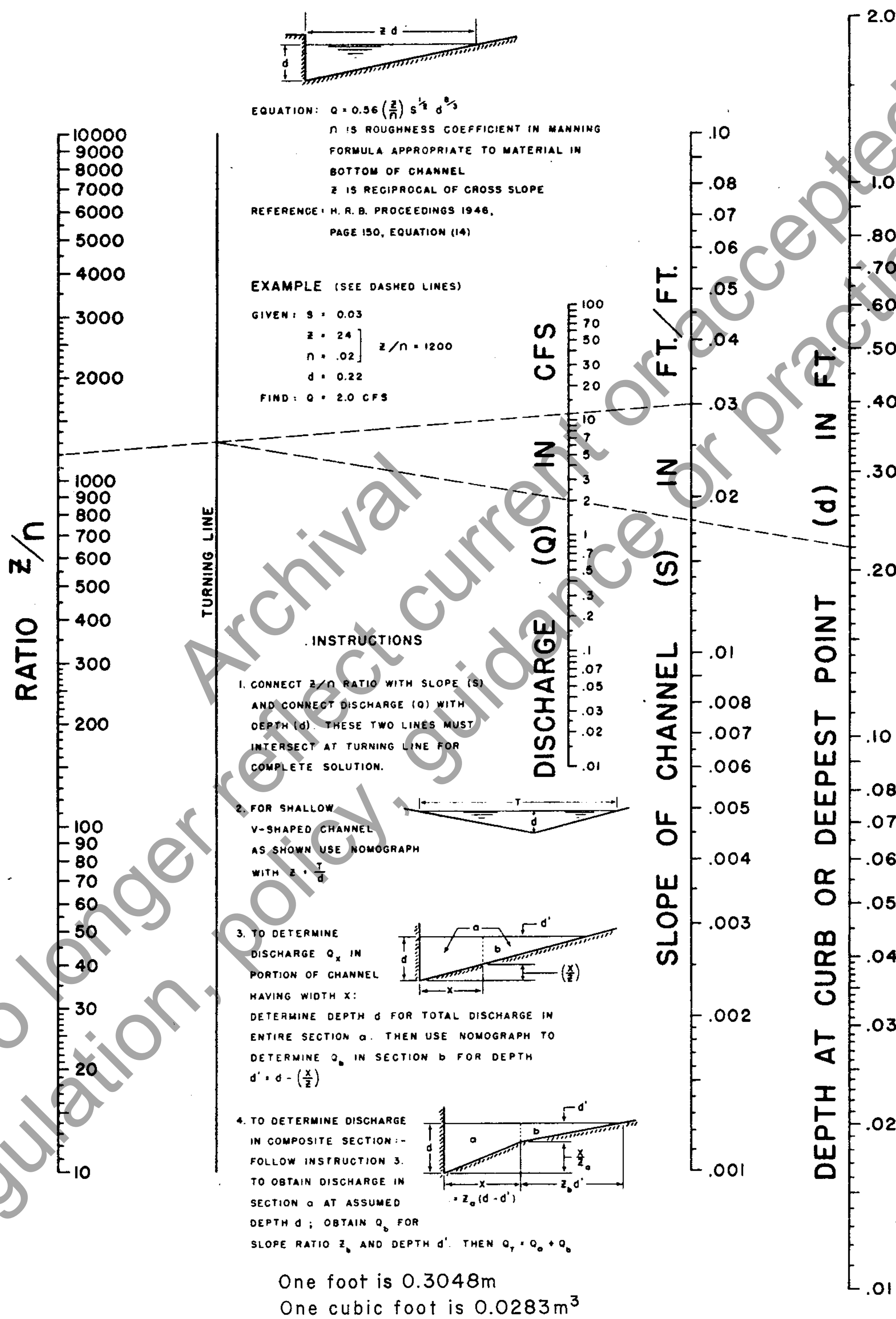


FIG. 5-1 *(After FHWA)*

ROUGHNESS COEFFICIENTS
(MANNING'S "n")

Highway Channels and Swales with Maintained Vegetation				
	Manning's "n"			
	Depth 0.7 ft.		Depth 0.7 - 1.5 ft.	
Velocity in fps	2	6	2	6
Type of Grass				
a. Kentucky Bluegrass Bermuda, Buffalo				
(1) Mowed to 2"	0.07	0.045	0.05	0.035
(2) Length 4" - 6"	0.09	0.05	0.06	0.04
b. Good stand, any grass				
(1) Length 12" ±	0.18	0.09	0.12	0.07
(2) Length 24" ±	0.30	0.15	0.20	0.10
c. Fair stand, any grass				
(1) Length 12" ±	0.14	0.08	0.10	0.06
(2) Length 24" ±	0.25	0.13	0.17	0.09

One inch is 25.4mm One foot is 0.3048m

Street and Expressway Gutters	
a. Concrete gutter troweled finish	0.012
b. Asphalt pavement	
(1) Smooth texture	0.013
(2) Rough texture	0.016
c. Concrete gutter with asphalt pavement	
(1) Smooth	0.013
(2) Rough	0.015
d. Concrete pavement	
(1) Float finish	0.014
(2) Broom finish	0.016
e. Brick	0.016
For gutters with small slope where sediment may accumulate, increase all above values of "n" by 0.002.	

TABLE 5-1

It is sometimes desirable to know the discharge in a part of the gutter channel. This is needed in determining the capacity of grate inlets. The procedure to be followed with a sketch is given in instruction 3 of Fig. 5-1. This procedure is illustrated by the following.

5.4.5 Example 5-2: Flow in Part of Gutter Channel

Given: Problem as in paragraph 5.4.4

Find: Discharge in first 2 feet from curb ($X = 2$).

Solution:

1. From step 4, example in 5.4.4, $d = 0.14$ foot.
2. Depth (d') at $X = 2$ is $d - X/Z$ or $0.14 - \frac{(2.0)}{48.0} = 0.14 - 0.04 = 0.10$ feet.
3. From Fig. 5-1 for $d' = 0.10$, $Z/n = 3429$ and $S = 0.01$, $Q_b = 0.4$ cfs.

The chart solution of Q_b is to lay a straight edge from Z/n to S and from the intersection of the straight edge with the turning line to $d' = 0.10$. Q_b is read on the discharge scale.

4. Q_x in the 2-foot width is the total Q (1.0 cfs) minus Q_b (0.4 cfs) from step 3 or $1.0 - 0.4 = 0.6$ cfs.

5.4.6 Gutters With Composite Sections

Fig. 5-1 can also be used for composite channel sections (two or more cross slopes) as might occur with a gutter section on a steeper cross slope than the cross slope of the pavement section. The procedures to be followed, with a sketch, are given in instruction 4 of the nomograph. The trial and error procedure consists of assuming a depth at the curb and comparing the capacity of the composite channel with the design Q. If these do not agree, a new assumption of d is made and the procedure repeated. An example illustrates the method for two cross slopes using the same symbols as the sketch in instruction 4. The method illustrated can be extended to a section with more than two slopes by treating each additional slope as a new section b. Sufficient need to work with a specific composite section will justify making up a design chart.

5.4.7 Example 5-3: Composite Section

Given: Rough texture asphalt pavement; cross slope 1/4-inch per foot; 2-foot concrete gutter section, cross slope 1 inch per foot; longitudinal slope 2%; Q = 2.0 cfs.

Find: Depth of flow at curb and spread on pavement.

Solution:

1. Assume n = 0.015 for both gutter and pavement.
2. For gutter section Z_a = 12.00; Z_a/n = 800; cross slope = 0.0833. For pavement section Z_b = 48.00; Z_b/n = 3205; cross slope = 0.0208.
3. Assume a depth at the curb. As a guide, use Fig. 5-1 for a straight slope equal to the gutter slope (Z_a/n = 800) and find d = 0.27 foot. This d must be decreased slightly to allow for the greater spread on the flatter pavement or in this case, assumed d = 0.27 - 0.01 = 0.26 foot.
4. Compute flow in gutter width (X = 2.0 feet) following instruction 3 of Fig. 5-1. Calculate X/Z_a = 2/12 = 0.17 foot which is the depth at pavement edge of the gutter. The total flow in a channel at the assumed curb depth, 0.26 foot with a continuous slope of Z_a = 12.00 from instruction 1, is 1.7 cfs. The flow beyond the gutter width, on the assumption of a continuous slope of Z_a = 12.00, is computed as for the total flow using d'= 0.26 - 0.17 = 0.09. From Fig. 5-1 this is 0.1 cfs. The flow in the gutter width is then 1.7 - 0.1 = 1.6 cfs at the assumed depth of 0.26 foot.
5. If the assumed depth is correct, the difference in design Q (2.0 cfs) and that carried in the gutter width (1.6 cfs) must be carried in the overflow section on the pavement. This flow is computed on Fig. 5-1 using d' = 0.09 foot (step 4) and the Z_b/n of the pavement section (3205). The Q in the pavement section is 0.4 cfs and the total Q = 1.6 + 0.4 = 2.0 cfs which checks the design Q and also the assumed value of d' = 0.26 foot. Failure of the total Q to equal the design Q would require a new assumption of d and a recomputation of steps 4 and 5.
6. The spread on the pavement = $Z_b d'$ or 48 (0.09) = 4.3 feet. The total width of flow measured from the curb is 2.0 + 4.3 = 6.3 feet.

5.4.8 Gutters With Curved Cross-Sections

Older arterial city streets and some older highways have curved paved cross-sections, often parabolic. For these the gutter flow capacity is computed by the original Manning formula, Equation 5-16, as shown in Table 5-2. A separate table is required for each crown height. The flow is computed for segments of widths of the cross-section; in column 1 of the table, this segment width is indicated as 2 feet.

Column 2 lists the depths which in the table are the parabolic offsets.

Column 3 is the width of each section.

Column 4 is the mean depth (hydraulic radius) of each section. This neglects the friction on the vertical face of the curb.

COMPUTATION OF DISCHARGES IN PARABOLIC PAVEMENT CROSS-SECTION

(FOR DEPTH AT CURB OF 0.48 FOOT; 24 FT. HALF PAVEMENT WIDTH; n=0.015)

Distance From Curb Ft.	Depth of Flow Ft.	Width of Section Ft.	Mean Depth R Ft.	Area Section Sq.ft.	$R^{2/3}$	$\frac{1.486}{n}$	K * Conveyance Factor
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0	0.4800						
		2.0	.44166	.8833	.579	99.07	50.67
2	0.4033						
		2.0	.36833	.7366	.513	99.07	37.44
4	0.3333						
		2.0	.30166	.6033	.450	99.07	26.90
6	0.2700						
		2.0	.24166	.4833	.388	99.07	18.58
8	0.2133						
		2.0	.18833	.3766	.329	99.07	12.27
10	0.1633						
		2.0	.14166	.2833	.271	99.07	7.61
12	0.1200						
		2.0	.10166	.2033	.217	99.07	4.37
14	0.0833						
		2.0	.06333	.1266	.159	99.07	2.00
16	0.0533						
		2.0	.04166	.0833	.120	99.07	0.99
18	0.0300						
		2.0	.02166	.0433	.078	99.07	0.33
20	0.0133						
		2.0	.008166	.0163	.041	99.07	0.07
22	0.0030						
		2.0		0			
24	0						
		Σ=24.0					Σ=161.23

One foot is 0.3048m

One square foot is $0.0929 m^2$

$* K = (1.486/n)AR^{2/3}; \ Q = KS^{1/2}$

TABLE 5-2

Column 5 is the area (column 3 times column 4)of each section.

Column 6 is self-explanatory.

Column 7: n assumed as .015; this might vary if the gutter n differs from the pavement n.

Column 8: the conveyance factor of the Manning formula, namely K/n $AR^{2/3}$. Q then, in cfs, is this conveyance factor multiplied by the square root of the longitudinal slope of the gutter. The coefficient K is 1.486 for the English system and one (1) for the metric system.

Fig. 5-2 shows the parabolic half-section of a 48-foot (14.63m) pavement and depth-discharge curves for various longitudinal gutter slopes. From these latter curves there can be determined for a known flow quantity and gutter slope, the depth at the curb and T, the spread of the flow (or width from curb). An example on the figure illustrates this.

While there are no experimental data on operation of curb opening inlets on parabolic sections, an equivalent straight section can be calculated which closely approximates the parabolic section, having the same discharge and same depth at curb. With curves similar to those of Fig. 5-2 the designer can determine T from a given Q and longitudinal slope, or can determine Q from a given T and slope.

The cross slope S_x of the equivalent straight section can be obtained from the equation for flow in triangular channels:

$$S_x = \frac{Kd^{8/3}S^{1/2}}{Qn} \quad \text{............(5-3)}$$

Values of Q, n, d and S can be obtained for the parabolic section and substituted to obtain S_x. The equivalent straight section is then used for computing curb opening inlet design. The coefficient K is 0.56 for English units, 0.375 for metric units.

5.4.9 Desirable Gutter Sections

When gutters are on a continuous grade, the depth of flow at the curb affects the capacity of curb opening inlets and the discharge within the width of a grate inlet determines its capacity. Thus, the ideal gutter section for hydraulic efficiency will carry the design discharge concentrated near the curb with flow at the greatest practical depth. Such a section is not compatible with flat pavement cross slopes. One solution used on many urban highways and most city streets is to add, outside the travelled way, a gutter section from 1 to 3 feet (0.305 to 0.914m) wide, sloping about 1 inch per foot (83.33mm per m).

On divided highways with a narrow median the choice must be made between crowning each pavement to drain in both directions or sloping each

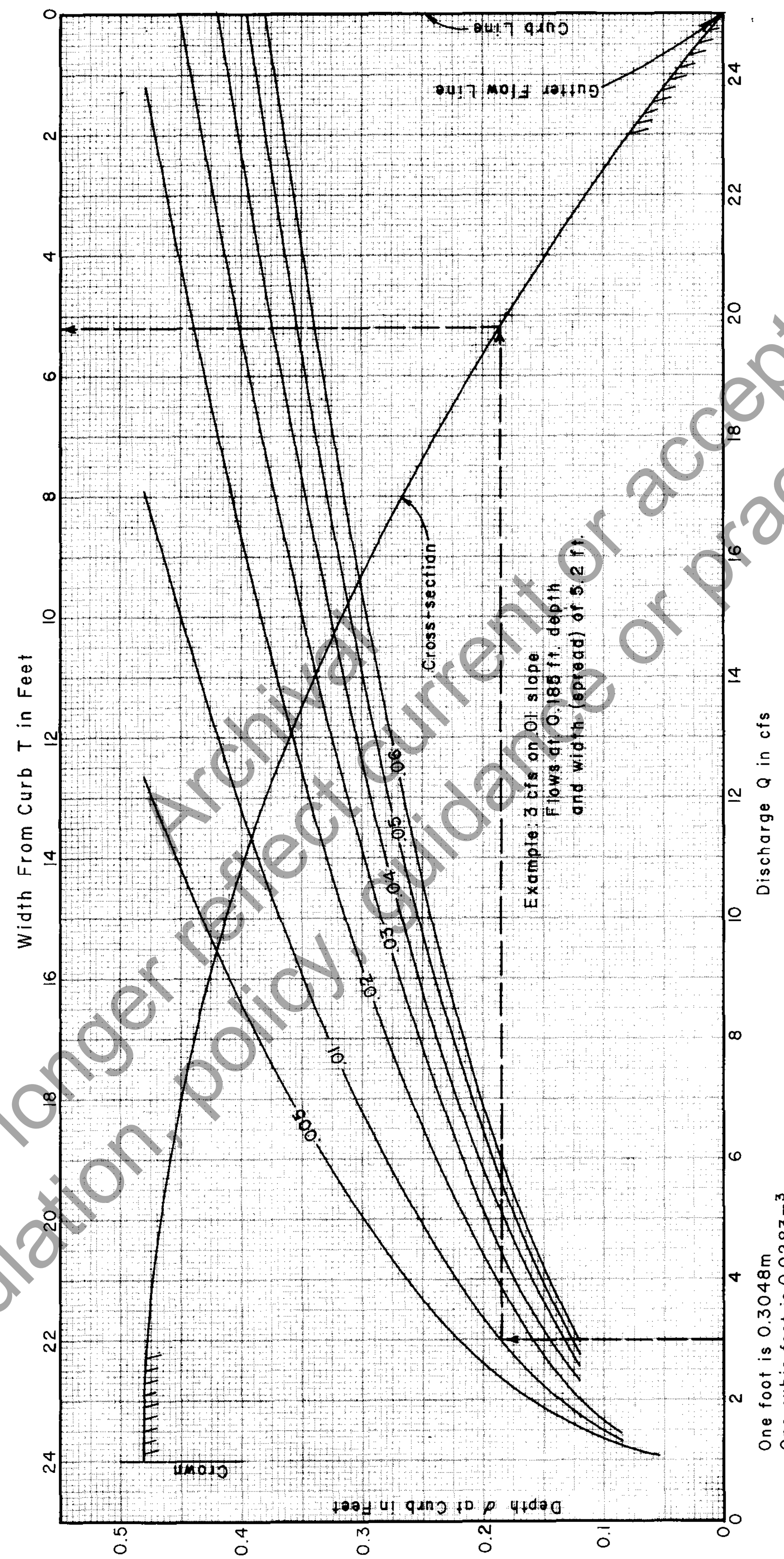


FIG. 5-2 Depth discharge curves for parabolic pavement cross-section.

pavement to drain in one direction. Minimization of the potential hazards of "hydroplaning" can be helped by keeping the depth of sheet flow as shallow as practicable; this suggests crowning pavements to drain in both directions. In northern climates it is preferable to prevent snow-melt water from running onto or across the pavement and becoming a hazard by freezing. This requires gutters on both sides of curbed pavements with inlets at close intervals.

5.5 Gutter Inlets

5.5.1 General

The hydraulic capacity of a gutter inlet depends upon its geometry and upon the characteristics of the gutter flow. The inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drain system. Many costly storm drains flow at less than design capacity because the storm runoff cannot get into the drains. Inadequate inlet capacity or poor inlet location may cause flooding on the travelled way which creates a hazard or at times, interrupts traffic.

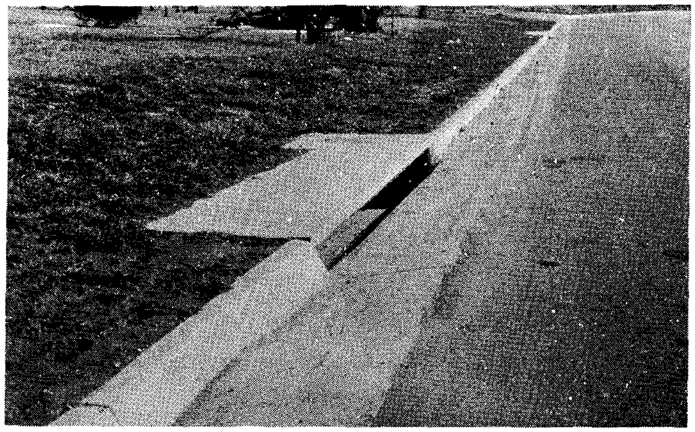
Water-borne debris and trash may be deposited on an inlet causing complete or partial clogging. Often freedom from clogging and non-interference with traffic requires an inlet of a specific type rather than the most efficient inlet from an hydraulic point of view. For example, a curb opening inlet might be used where a grate inlet would be more efficient.

5.5.2 Types of Inlets

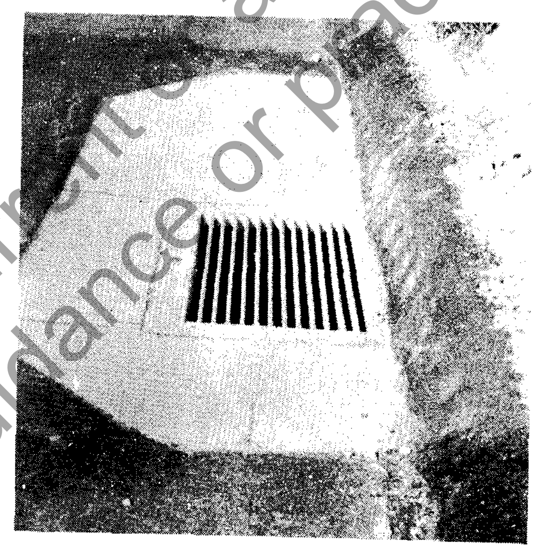
Gutter inlets (Fig. 5-3) can be divided into three major classes each with many variations. These classes are (1) curb opening inlets; (2) grate inlets; and (3) combination inlets. Each type of inlet may be installed with or without a depression of the gutter and may be a single or multiple inlet (two or more closely spaced inlets acting as a unit). Two identical units placed end to end are called double inlets. Additionally, there are occasional inlets in which the intake opening is normal to the flow; and slotted drain inlets with slots flush with the pavement.

A curb opening inlet generally requires a larger structure than a grate inlet of equal capacity but the curb opening is located back of the curb line and offers little interference with roadway traffic.

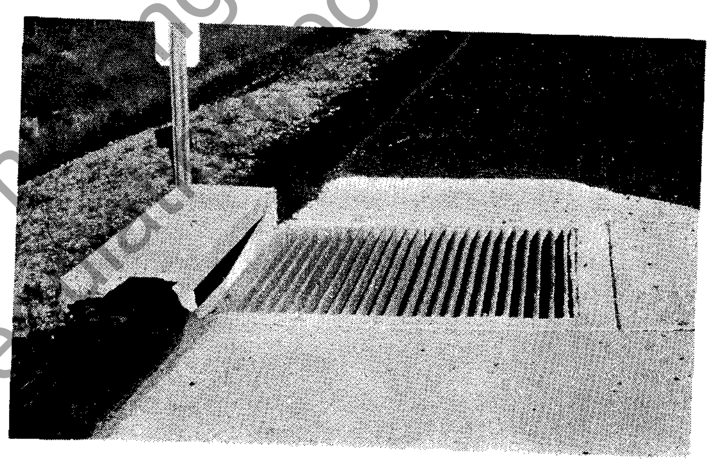
An undepressed inlet has less capacity than a depressed inlet. Curb opening inlets lose capacity rapidly with increase in longitudinal grade. Grate inlets generally lose capacity with increase in grade but to a lesser degree. A combination inlet without depression has little greater capacity than the grate inlet alone. Changes in cross slope affect the capacity of a curb opening inlet much more than the capacity of a grate inlet.



A-Curb-opening Inlet.



B-Grate Inlet.



C-Combination Inlet.

FIG. 5-3 Types of Gutter Inlets. *(After HEC-12)*

Choice of inlet cannot always be made upon capacity alone. Debris carried by the gutter flow and interference with vehicular traffic must also be considered. Curb opening inlets are relatively free of debris clogging while grate inlets have a tendency to clog and might clog completely where debris is a problem. Combination inlets are better than grate inlets alone where debris is prevalent.

Depressions may be objectionable to high speed traffic. Curb opening inlets with vertical openings greater than about 6 inches (152.4mm) are a hazard to children. Bicycle safety has become a major consideration in inlet design.

5.5.3 Characteristics and Uses of Inlets

1. The curb opening is most effective in sags and with flows carrying floating trash. As the gutter grade steepens, it's intercepting capacity decreases. Consequently, it is commonly used on sags and grades flatter than 3%.

Curb opening inlets are used on urban highways; with opening 6 inches (152.4mm) or more in height, a 3/4-inch (19.05mm) plain round bar is often placed horizontally across the opening for safety of small children.

2. Grate inlets, as a class, perform satisfactorily over a wide range of gutter grades. Their principal disadvantage is that they are easily clogged by floating trash. They warrant preference over the curb opening type on grades of 3% or more. Grate inlets are also used in locations where a gutter depression is not permissible or desirable. Preference shall be given to grate inlets in locations where out-of-control vehicles might be involved.

Rectangular grates can be used either inside or outside the roadbed. Typical uses within the roadbed include: a valley gutter location; the gutter of a driveway; within the shoulder against a dike; against the vertical face of a bridge abutment; street intersections upstream from cross-walks.

3. Combination inlets provide both a curb opening and a grate. These are high capacity inlets which may offer many advantages of both kinds of openings. Those combination inlets with the curb opening directly opposite the grate are typically used in a sag location either in a curb and gutter installation or within a shoulder fringed by a dike.

What may be termed a "sweeper" inlet has a curb opening preceding a grate. It is particularly useful as a trash interceptor during the initial phases of a storm. Used in a grade sag, the sweeper inlet can be modified by providing a curb opening on both sides of the grate.

4. Pipe drop inlets are made of a vertical commercial pipe section of concrete or corrugated metal with a removable grate flush with the drained surface. As a class they develop a high capacity and are

generally an economical type. These inlets are designed for use only outside the roadbed.

The grate pipe drop inlet intercepts water from any direction. Being round, it is most effective for flows that are deepest at the center.

5. Slotted drains are made of pipe with a continuous slot on top or of pipe with a flat top and transverse slots. These inlets can be used in flush, all-paved medians with superelevated sections to prevent sheet flow from crossing the centerline of the highway. Short sections of slotted drains may be used as an alternate solution to a grate catch basin in the median or edge of the shoulder.

5.5.4 Location and Spacing of Inlets

1. Governing factors in the location and spacing of inlets are: the amount of runoff; the grade profile; the location and geometries of interchanges, driveways and street intersections; width of flow limitations; the inlet capacity; accessibility for maintenance and inspection; volume and movements of vehicles and pedestrians; and amount of debris.

2. The aim in the location of inlets should be the most effective and economical installation. In urban areas, the volume of vehicular and pedestrian traffic constitute an important control. For street or road crossings, the usual inlet location is at the intersection at the upstream end of the curb or pavement return and clear of the pedestrian crosswalk. Safety of location for maintenance purposes is an important consideration.

3. The distance between inlets should be determined by a rational analysis of the governing factors as discussed later.

4. Inlets in series should have a minimum spacing to allow bypass flows to return to the curb face. Frequently, lengthening or widening a grate inlet is a desirable alternate.

5.5.5 Factors in Inlet Capacity

The discussion in this section is restricted to inlets on a grade.

The term "inlet capacity" is used to mean the hydraulic catch of the inlet under a given set of conditions rather than the maximum water that can be intercepted by the inlet if the discharge is increased without limit. The efficiency of an inlet is the discharge intercepted by the inlet (Q_i) divided by the flow in the gutter (Q). The discharge that bypasses the inlet (Q_c) is termed "carry-over".

A major factor in the capacity of a curb opening inlet is the depth of water in the gutter immediately adjacent to the opening. The capacity of a grate inlet depends principally upon the quantity of water flowing in the section formed by projecting the grate width upstream. An increase in transverse (cross) slope increases inlet capacity. Increase in length of a curb opening inlet and increases in width of a grate opening increase the capacity of the inlet. For grate inlets, the efficiency of the grate opening is an important factor in inlet capacity.

For a curb opening inlet, depressing the gutter increases the capacity of the inlet. The amount of the depression has more effect on the capacity than the arrangement of the depressed area with respect to the inlet. St. Louis experiments show that on a 1% grade, a 6-inch deep (152.4mm) depression has twice the capacity of a 4-inch (101.6mm) depression and six times the capacity of a 2-inch (50.8mm) depression (Ref. 5-3). Colorado State tests (Ref. 5-4) showed that for a 2-inch (50.8mm) depression, 2 feet (0.610m) wide, a transition beginning 2 feet (0.610m) upstream from a curb opening and ending 2 feet (0.610m) downstream from the inlet was an efficient arrangement and that where the efficiency of the curb opening inlet was greater than 75%, the difference in efficiency between the various transitions tested was less than 5%. The Johns Hopkins tests (Ref. 5-5) found that with a grade of 1% and cross slope of 0.056, a depression of 2.5 inches (63.5mm) increased inlet capacity ten times or more than that of an undepressed inlet. The effect was less at steeper grades. Their tests also showed that extending the depression upstream from the curb opening a short distance increased flow; but if the distance was increased beyond an optimum value, depending upon the longitudinal slope and the cross slope, the inlet capacity decreased almost to its original value. Johns Hopkins tested triangular-shaped depressions and found that a triangular depression with the base upstream and with the apex at the lower end of the curb opening had 65% greater capacity than a constant width depression of the same length and depth. When the upstream length was increased to its optimum length, the capacity of the triangular depression was 80% greater than that of the corresponding rectangular depression.

Most of the investigators (Refs. 5-6, 5-7) have pointed out that the capacity of an inlet is increased by allowing a small percentage of the flow to bypass the inlet. The carry-over is created by increasing the discharge in the gutter and while the catch (capacity) of the inlet increases with increased total flow, the efficiency of the inlet (percent of total flow) decreases. This loss of inlet efficiency is not a valid argument against deliberately designing the inlet for a carry-over. Perhaps a better way of showing the merits of designing for carry-over discharge is by examining the economics of the inlet. For a given gutter discharge, the catch of each additional increment of width (grate inlets) or length (curb opening inlets) becomes rapidly less. Thus, the cost of catching the small amount of flow near the thin edge of the triangular flow channel approaches the cost of catching the greater amount flowing nearer to the curb. For example, with a constant cross slope, a grate 50% of the width of flow will intercept 84% of the flow and to intercept the remaining 16% of flow, the width of the grate would have to be doubled.

5.6 Curb Opening Inlets

These inlets are used in many locations because they offer little interference with traffic and are relatively free from clogging by debris.

The best hydraulic type of curb opening inlet has a cantilevered top slab without supports in the opening and a depression of the gutter flow line of at least 2 inches. The length of the opening can be varied with the amount of water to be intercepted. If a support for the top slab is used in the design, it should be round in horizontal cross-section and recessed several inches back from the curb line. Supports to the top slab placed flush with the curb line reduce the effectiveness of the opening downstream from the support by as much as 50%. If drift catches on the support, the interception of the downstream portion of the opening may approach zero.

5.6.1 Standard Curb Opening Inlet

The standard curb opening inlet discussed herein is illustrated in Fig. 5-4. It has a depression beginning w feet out from the curb and dropping 1 inch per foot below the plane of the pavement. Transitions at the two ends extend w feet from the ends of the opening. The height of the opening must consider probable debris but generally, need not be more than 4 inches (101.6mm) since the water surface draws down as it accelerates on the depression apron; it should not exceed 6 inches (152.4mm) unless provided with a horizontal bar in order to prevent a child from being washed into the opening. The equations given in Table 5-3 apply only if the cross-section of the street has a uniform slope to the face of the curb. However, subsequent discussion considers how to take care of deviations therefrom.

5.6.2 Operation of Inlet

The operation of a curb opening inlet on a grade is usually described in terms of the ratio of the flow intercepted, Q_i, to the approach flow Q, which extends a distance, T, from the curb face. Q_i/Q can be defined in a dimensionless plot against $L_i/(F_W T)$ (Fig. 5-5), where L_i is the length of the inlet opening and F_W is the Froude Number related to the depth of the approach flow at a distance w from the curb. This is along a line at the outer edge of the inlet depression. The Froude Number is a measure of the gravity force acting on the flow in the gutter.

In Fig. 5-5, which is drawn for a cross slope $S_x = 0.015$ and w = 2 feet (0.610m), note that $Q_i/Q = L_i/L_1$ up to the point where the parameter $L_i/F_W T = 0.4$. Beyond that point, the relationship changes abruptly to a curved line for which $Q_i/Q = (L_i/L_3)^{0.4}$. L_1 is the value of L_i where the straight line intersects $Q_i/Q = 1.0$, while L_3 is the value of L_i where the curved line intersects $Q_i/Q = 1.0$. L_2 is the value of L_i at

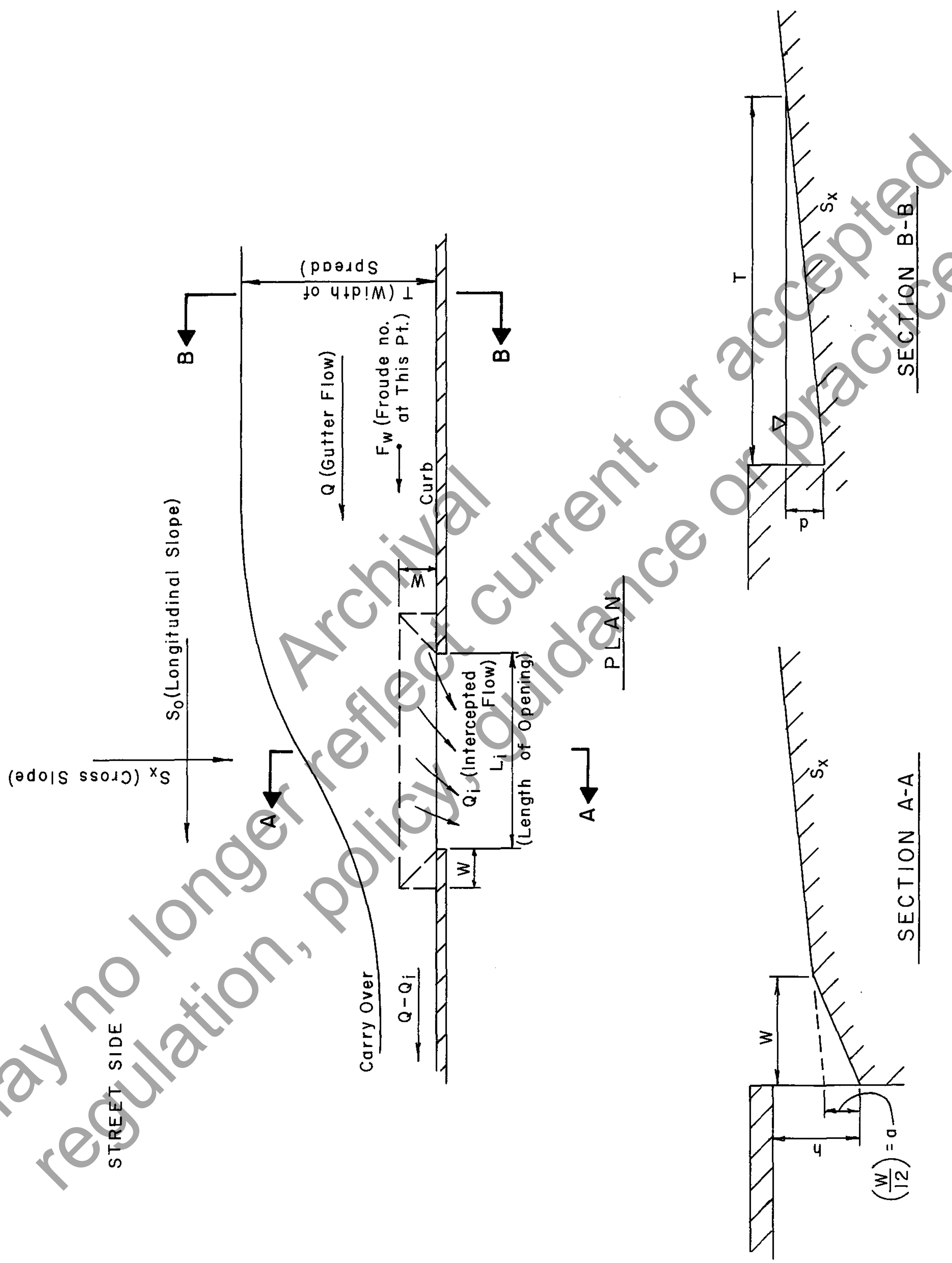


FIG. 5-4 Graphical definition of symbols—curb-opening inlet.

COMPUTATIONS: CURB-OPENING INLETS

STANDARD SECTION

$W = 2$ ft.
$n = 0.016$
$S_x = 0.02$

(1) $L_1 = 2.49\, S_x^{0.3}\, F_w T$

(2) $L_2 = 3.27\, S_x^{0.5}\, F_w T$

(3) $L_3 = 1.65\, F_w T$

(4) $F_w = 16.4 \left[(T-2)S_x\right]^{1/6} S_0^{1/2}$

(5) $Q = 35 S_x^{5/3}\, T^{8/3}\, S^{1/2} = .0515\, T^{8/3}\, S_0^{1/2}$

(6) $T = \left[\frac{Q}{35 S^{1/2}}\right]^{3/8} S_x^{-5/8} = 3.04 \left(\frac{Q}{S_0^{1/2}}\right)^{3/8}$

(7) $\frac{Q_i}{Q} = \frac{L_i}{L_1}$

(8) $\frac{Q_i}{Q} = \left(\frac{L_i}{L_3}\right)^{0.4}$

INLET NO.	S	$\frac{Q}{S^{1/2}}$ (cfs)	Q (cfs)	T (ft.)	$\frac{F_w}{S^{1/2}}$	F_w	$F_w T$ (ft.)	L_1 .770 (ft.)	L_2 .462 (ft.)	L_3 1.65 (ft.)	$\frac{Q_i}{Q}$	Q_i (cfs)	$\frac{Q_2}{Q}$ .600 (cfs)	L_i (ft.) $Q_i<Q_2$ $L_i<L_2$	$Q_i>Q_2$ $L_i>L_2$	USE L_i (ft.)	ACT. Q_i (cfs)	$Q-Q_i$ $=Q_c$ (cfs)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	.01	23.9	2.39	10.0	12.1	1.21	12.1	9.3	5.6	20.0	1.0	2.39	1.43		20.0	20.0	2.39	0
1	.01	23.9	2.39	10.0	12.1	1.21	12.1	9.3	5.6	20.0	0.80	1.91	1.43		11.4	12.0	1.95	0.44
2			2.83	10.7	12.2	1.22	12.2	9.4	5.6	20.1	0.81	2.26	1.70		11.9	12.0	2.30	0.53
3			2.92	10.8	12.3	1.23	12.3	9.5	5.7	20.3	0.81	2.37	1.75		11.9	12.0	2.37	0.55
3 alternate											0.99					20.0	2.90	0.02
4	.03	23.9	4.14	10.0	12.9	2.24	22.4	17.2	10.3	37.0	0.58	2.40	2.48	10.0		10.0	2.40	1.74
4 alternate 1											0.80	3.31			21.1	22.0	3.36	0.78
4 alternate 2			0.65	5.0		2.05	10.25			16.9	0.80	0.52	0.39		10.0		0.53	0.12

(See text for sequence of computations)

One foot is 0.3048m One cubic foot is 0.0283m^3

Required criteria

TABLE 5-3

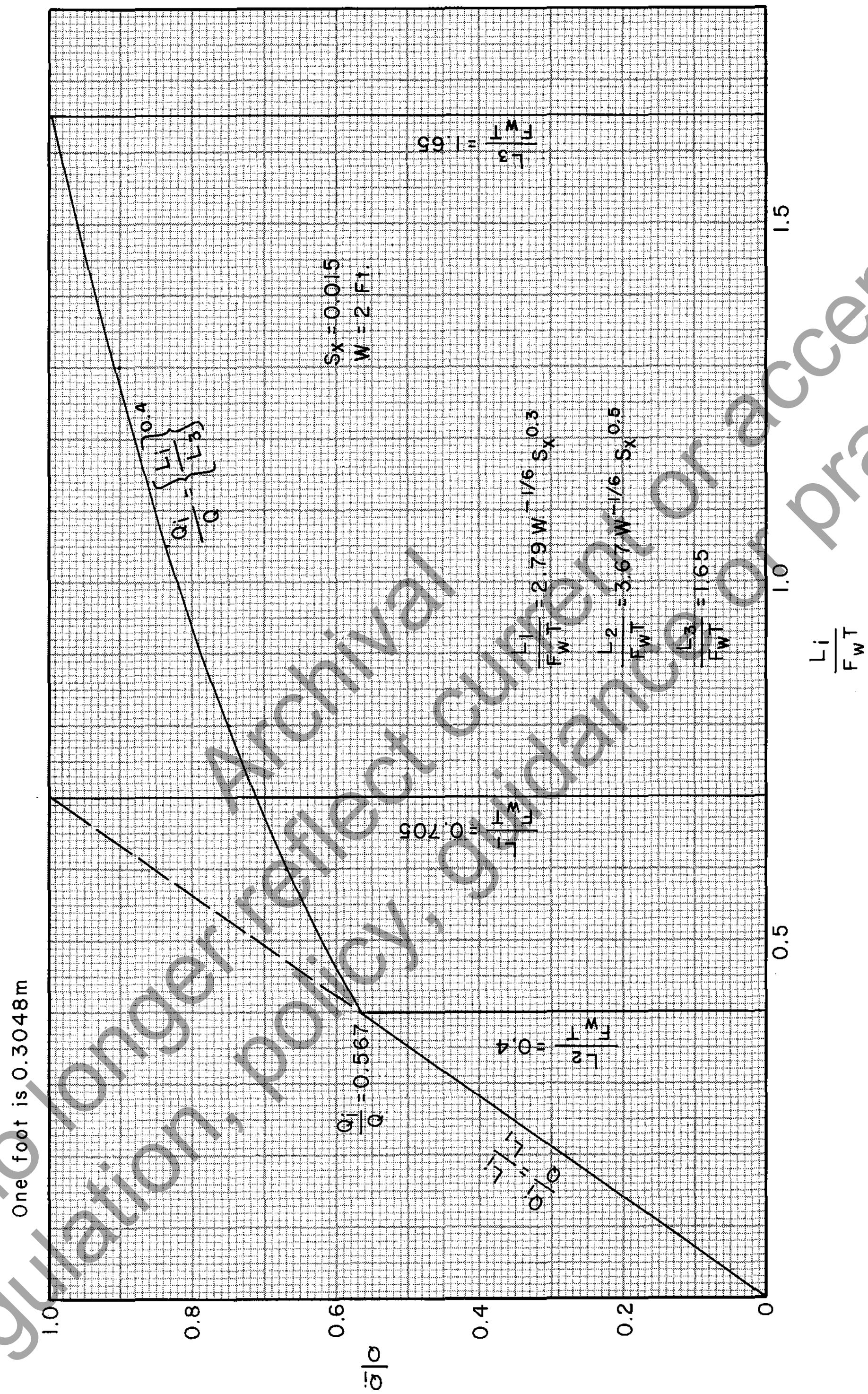


FIG. 5–5 Dimensionless Graph of Q_i/Q vs. L_i/F_wT Showing Characteristic Lengths.

the breakpoint between the straight and curved lines.

From the diagram it will be apparent that if we know the value of $F_w T$ and L_i, the value of Q_i/Q can be read from the ordinate scale, remembering that this diagram is for specific values of S_x and w. The position of the S_x line varies with these variables in accordance with the equations on the figure. The position of the curved line remains fixed. Solutions for Q_i/Q may be read from Fig. 5-5 for w = 2 feet (0.610m) or may be computed using the equations as tabulated in the examples in Table 5-3. That table also gives the equations for F_w and Q in terms of the cross-section variables.

The understanding of the dimensionless curves of Fig. 5-5 is improved by knowledge of their physical significance. With the product $F_w T$ assumed constant for a given flow situation, the abscissa is the length of inlet divided by that constant. If desired, the scale could be recalibrated to read directly in feet of inlet length. For short inlets up to the length L_2, where the curve breaks, the inlet acts as a weir. In fact, the flow intercepted is practically the same as would be intercepted by the same inlet at a sump, using the modified weir equation for that case. The major part of the flow is intercepted (60% or more depending on S_x), up to the length L_2. For greater lengths of inlet, the remainder of the flow moves in gradually as indicated by the lesser increments of Q_i as length increases.

5.6.3 Curb Opening Inlet Design Chart

Izzard (Ref. 5-8), from whom is taken this discussion of the hydraulics of curb opening inlets, has developed Fig. 5-6 as a graphical solution for standard curb opening inlet design. His work is based upon original experimental data for full-scale inlets reported in 1961 by Karaki and Haynie (Ref. 5-4) which was analyzed by Bauer and Woo (Ref. 5-9). The graphical solution presented here has the advantage of being applicable to any grade (S_o), cross slope (S_x), roughness coefficient (n), and flow spread (T), while giving a direct reading from a single chart. Fig. 5-6 is based upon w = 2 feet (0.610m); a = 2 inches (50.8mm) and h = 6 inches (152.4mm). The achievement of an h substantially equal to 6 inches (152.4mm) with a depression of 2 inches (50.8mm) and a 6-inch (152.4mm) curb height can be accomplished as illustrated by a standard curb inlet of the Virginia Department of Highways and Transportation, (Fig. 5-7).

The use of the chart (Fig. 5-6) is illustrated by an example in dotted lines and described as follows:

1. The starting point is in the street section at a point w (2 feet (0.610m) from the curb face), where the depth of flow is d_w.

2. The example assumes S_x = 0.02 feet per foot (0.02mm per m); T = 10 feet (3.048m); S_o = 0.03 feet per foot (0.03mm per m) and n = .016. It requires the determination of inlet lengths to accept

STANDARD CURB-OPENING INLET CHART

W = 2 ft.
a = 2 in.
h = 6 in.

One inch is 25.4mm
One foot is 0.3048m

EXAMPLE

Given — S_x = 0.02 ft./ft.; T = 10 ft.; S_o = 0.03 ft./ft.

Find — L_i = 11.8 ft., Q_i/Q = 0.65; L_i = 34 ft., Q_i/Q = 1.0

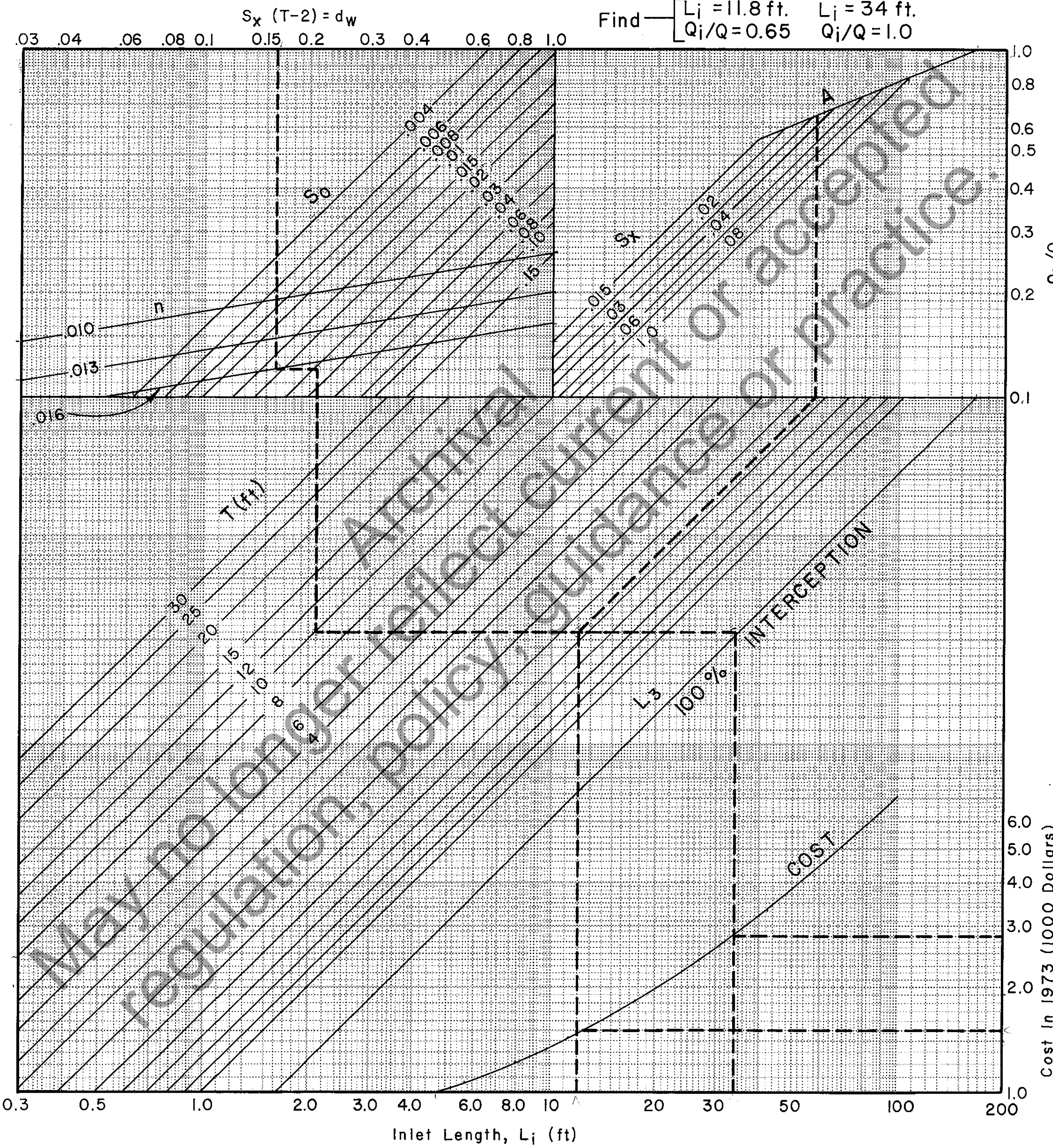


FIG. 5-6

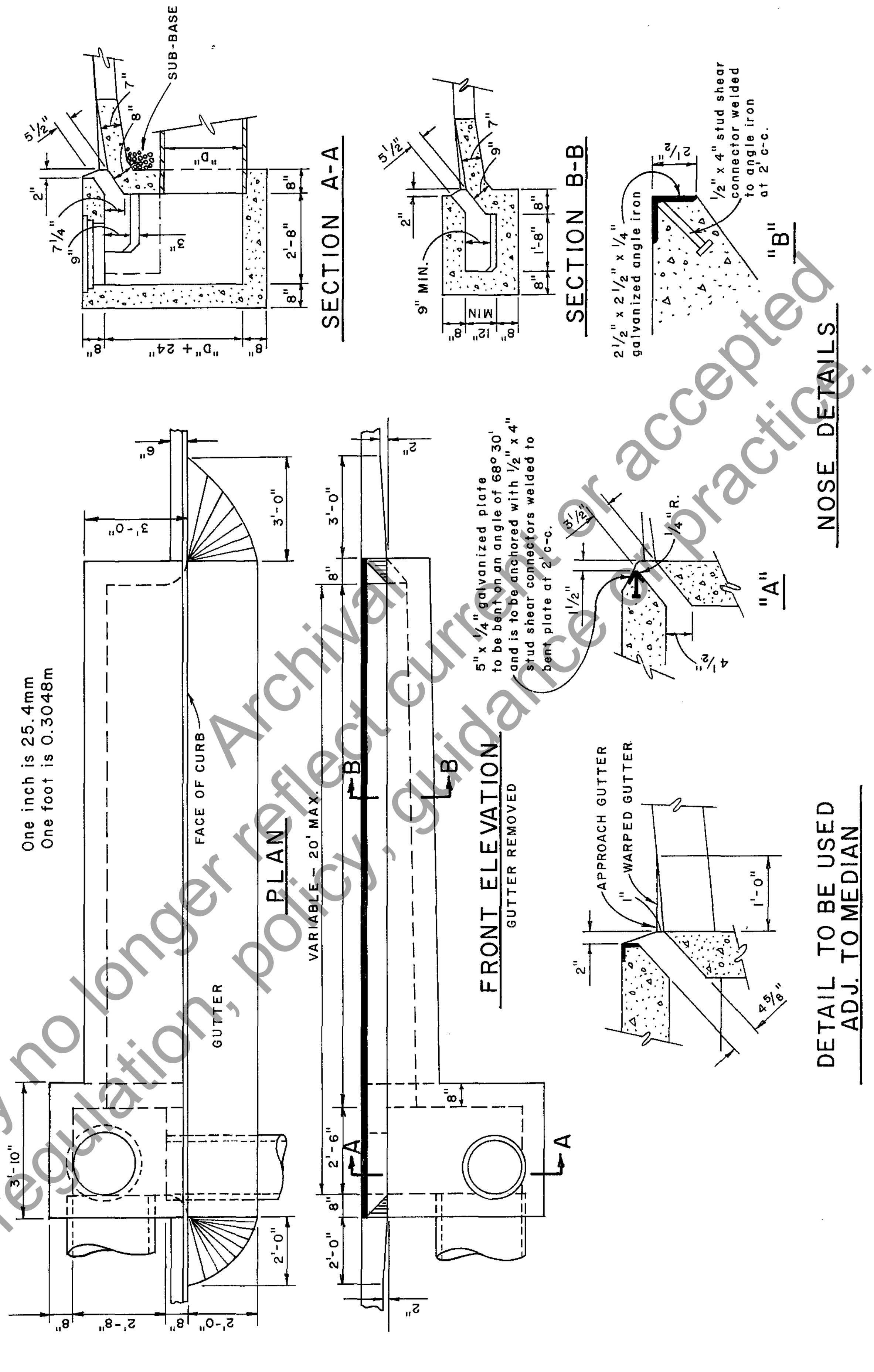


FIG. 5-7 Standard Curb Drop Inlet. *(After Fairfax County, Virginia)*

Q_i/Q ratios of 0.65 and 1.0.

3. Enter at top left-hand edge of chart the value of $S_x(T-2)$ which for the example is .02 (10-2) or 0.16.

4. Follow vertically down to the line representing Manning's n of 0.016.

5. Move horizontally across to longitudinal slope S_o of 0.03.

6. Follow vertically down to flow spread T of 10 feet (3.048m). This establishes a horizontal line for the example.

7. With the given Q_i/Q of 0.65, enter the upper right of the chart, follow horizontally across to line A or line for assumed S_x whichever is intersected first.

8. Move vertically down to the lower margin of the upper right quadrant where Q_i/Q is 0.1 and then, diagonally to intersection with the horizontal line in step 6.

9. Follow vertically down to find the required inlet length L_i; for the example, 11.8 feet (3.597m).

10. The horizontal line in step 6 can be continued to the right until it intersects the sloping line L_3, to find the needed curb opening to achieve 100% interception. From intersection with line L_3 move vertically down to the 100% inlet length. For the example, this is 34 feet (10.363m).

11. If the length of inlet is given, enter with that length, move up to the horizontal line established in step 6, diagonally to Q_i/Q = 0.1, then vertically to S_x (or line A) and across to Q_i/Q.

The cost curve in the lower right corner of Fig. 5-6 shows how inlet costs may be estimated. It is based upon 1973 contract prices for Virginia State Highway Department curb opening inlets. It can be useful in consideration of alternate criteria for T and S_x.

The maximum interception per foot (metre) of inlet occurs in the straight portion of the function in Fig. 5-5. Since cost is related to length, the least cost per cfs (m^3/s) intercepted, occurs in this range.

As illustrated in the example, the length of inlet decreases markedly when Q_i/Q is assumed as less than 1.0. If a slight increase in spread T is tolerable for successive inlets, the carry-over flow added to the runoff from the intervening watershed increases the interception ratio. Consequently, by the third inlet, all the intervening flow is intercepted. Cost savings can be substantial even when the last inlet is sized to pick up the total flow.

5.6.4 Steeper Gutter Section

It is quite common practice to build gutters with steeper cross slopes than the pavement. This increases the depth at the curb and the discharge for a given spread.

There have been no experimental tests on inlets placed where the gutter has a steeper cross slope than the pavement. A method of estimating the increase in interception capacity due to the increased flow in such compound or composite sections is suggested here. When the gutter slopes more steeply than the pavement, an increase in gutter flow results. If the gutter has the same width as the inlet depression, it is practical (although not conservative) to assume that the increased increment in gutter flow will be intercepted by the inlet. Using the method outlined on Fig. 5-1, Fig. 5-8 has been computed and drawn to give the relative increase in total flow for various cross slopes and values of T, based upon a commonly encountered gutter 2 feet (0.610m) wide with a cross slope of 1:12.

To use Fig. 5-8, first estimate inlet interception flow for the given inlet using the method previously described. Knowing S_x and T read $\Delta (Q_i/Q)$ on the ordinate scale and multiply it by the previously estimated Q to obtain the increase in the interception to be added to the original Q_i.

5.6.5 Parabolic Roadway Section

Experimental data on operation of curb opening inlets on parabolic sections is lacking. However, an equivalent straight section can be calculated as discussed in 5.4.8. This closely approximates the parabolic section, having the same depth at the curb and the same street flow Q. Using the equivalent cross slope and flow spread, the curb opening inlet design can proceed using Fig. 5-6.

5.6.6 Tabular Design of Curb Opening Inlets

Table 5-3 illustrates the sequence of steps. As a rule, the designer will be working with a standard inlet and cross-section for which S_x, n and w are fixed. In the heading, equations (1)*, (2)* and (3)*, taken from those in Fig. 5-5, reduce to the numerical coefficients in the heading of columns (9), (10) and (11).

On the first line, inlet 1, the designer is to find the inlet length required for 100% interception on a 1% grade with T = 10 feet (3.048m). The encircled numbers represent the required criteria. Column 3 is used if there are a succession of grades for which Q is computed by equation (5)*. Similarly, column 7 is for F_w computed by equation (4)*. Multiplying F_wT in column 8 by the coefficients in the headings of columns 9, 10, and 11, gives characteristic lengths L_1, L_2 and L_3. As stated, $Q_i/Q = 1$, so Q_i in column 13 equals Q in column 4. Q_2/Q for the standard conditions is simply 0.462/0.770 = 0.600 as recorded in the heading for column 14. Either columns 15 or 16 are used to record L_i depending on whether $Q_i < Q_2$ or $> Q_2$ (or in the case where L_i is given, $L_i < L_2$ or $> L_2$). In this case, $Q_i > Q_2$, so L_i is computed by equation (8)*. Since $Q_i/Q = 1$, $L_i/L_3 = 1$ and L_i = 20 feet (6.096m) as

*Equations given at top of Table 5-3.

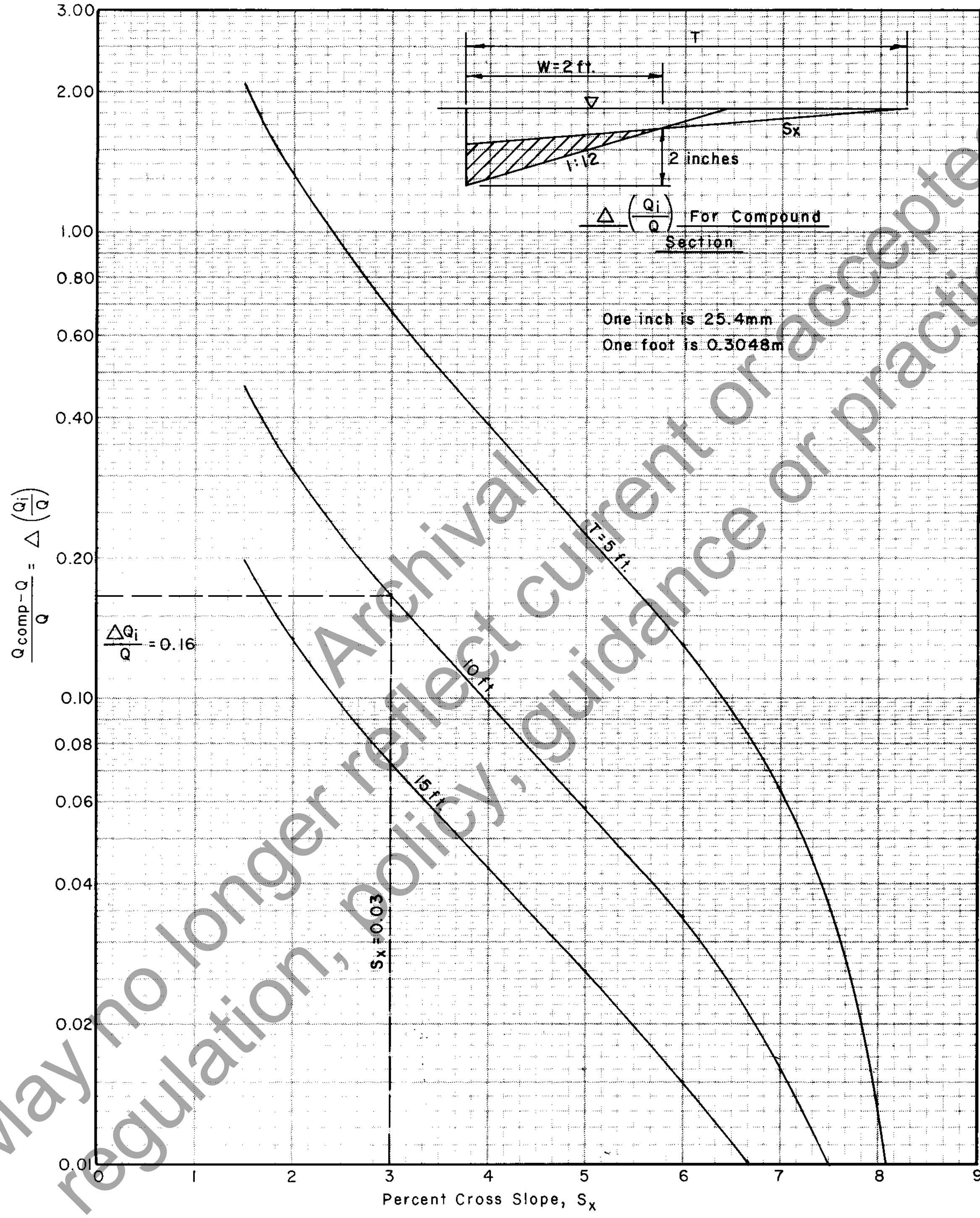


FIG. 5-8 Graph to give increment of discharge $\Delta\left(\frac{Q_i}{Q}\right)$ for composite section.

taken from column 11. Actually, for 100% interception one may go directly from column 11 to column 16. Column 17 records the selected length L_i, usually as a multiple of 2 feet depending on design standards. In the next example, the computed length 11.4 becomes 12 feet (3.658m). If desired, Q_i can then be recomputed by equation (7)* or (8)* in column 18 and subtracted from Q to give the carry-over discharge Q_c.

In the next three examples the independent variables are the same as the first example except that $Q_i/Q = 0.8$. For inlet 1, the required length reduces to 12 feet (3.658m) with $Q_c = 0.44$ cfs ($0.0125 m^3/s$). Assuming the increment in runoff for the next subwatershed is the same as for inlet 1, the second inlet will then have Q = 2.39 + 0.44 = 2.83 cfs ($0.0801 m^3/s$). This requires a recomputation of T; this can be done simply as $(2.83/2.39)^{3/8}$ 10 = 10.7 feet (3.261m) in accordance with equation (6)*. L_3 changes slightly to 20.1 feet (6.126m). It is now assumed that the same size of inlet will be used again, so Q_i/Q is computed as $(12/20.1)^{0.4} = 0.81$ making Q = 0.81(2.83) = 2.30 cfs ($0.0651 m^3/s$) and $Q_c = 0.53$ cfs ($0.0150 m^3/s$). For inlet 3, again using 12 feet (3.658m), the adjusted value of T becomes 10.8 feet (3.292m) and $L_3 = 20.3$ feet (6.187m) which leaves $Q_i/Q = 0.81$ and $Q_i = 2.37$ cfs ($0.0671 m^3/s$). The flow intercepted has now become substantially equal to the increment in runoff for the intervening watershed.

Supposing that inlet 3 is just above an intersection making carry-over flow undesirable, the third inlet may be increased to 20 feet to intercept practically all the flow.

The cost savings (1973 dollars) generated by using $Q_i/Q = 0.80$ can be computed using the cost curve in Fig. 5-6.

Cost of inlets only ($Q_i/Q = 1$) 3 x 1970 = 5910

Cost of inlets only ($Q_i/Q = 0.8$) 3 x 1470 = 4410

Alternate: 2 at 12-foot (3.658m) length, 1 at 20 feet (6.096m):

2 x 1470 = 2940
1 x 1970 = 1970
$4,910

If 0.55 cfs ($0.0156 m^3/s$) can be allowed to get by inlet 3, then the cost saving with three 12-foot (3.658m) inlets is $1500 or 25% by assuming $Q_i/Q = 0.80$. If no carry-over flow is allowed, the saving reduces to 17%. These calculations omit consideration of the cost of pipe since the pipe size probably would not change for the several alternates.

In inlet 4, it is assumed that a 10-foot (3.048m) inlet is to be used and Q_i/Q is to be found. In this case, $L_i < L_2$ so equation (7)* is used.

*Equations given at top of Table 5-3.

For alternate 1, the length for $Q_i/Q = 0.8$ is to be computed.

For inlet 4, alternate 2, the problem is to find T which would enable the 10-foot (3.048m) inlet to intercept 80% of the flow. This would tell how far upstream the inlet would have to be moved to reduce Q to that amount. In this case, $(10/L_3)^{0.4} = 0.80$ so $L_3 = (1/0.80)^{2.5}\ 10 =$ 17.5 feet (5.334m) which must equal 1.65 F_wT. Therefore, $F_wT =$ 17.5/1.65 = 10.6. As a first trial assume $F_w = 2.2$ making T = 10.6/2.2 = 4.8 feet (1.463m). Substituting this T in equation (4)(Table 5-3) we find $F_w = 2.03$. A second trial with $F_w = 2.1$ yields T = 5 feet (1.524m) and computed $F_w = 2.05$ which is close enough. Taking T = 5 feet by equation (5)*, $Q = 0.0515(5)^{8/3}\ 0.03^{1/2} = 0.65$ cfs ($0.0184 m^3/s$). This is an absurdly small discharge demonstrating that a 10-foot (3.048m) inlet isn't worth much on a 3% grade. Rather than moving the inlet that far upgrade, it would obviously be more economical to go to a 22-foot (6.706m) inlet, at the original location, and save the extra length of pipe.

5.6.7 Significance of Cross Slopes

In Table 5-3 computations are limited to one cross slope $S_x = 0.02$. Substantial economies can be achieved by selecting a steeper cross slope as demonstrated in Fig. 5-9. The ordinate is cost/cfs intercepted by a single inlet ($Q_i/Q = 1$) on a 4% grade, plotted against T for several cross slopes. Note that the cost is roughly cut in half by adopting a criterion of 0.03 instead of 0.02 for the cross slope. Note also that the cost rises sharply as the criterion for spread is reduced. Costs per cfs (m^3/s) increase appreciably with spreads limited to less than 10 feet (3.048m); T of 8 feet (2.438m) involves inlet costs approximately one and one-half times those for T equal to 10 feet (3.048m). Costs would be in about the same proportion for $Q_i/Q = 0.80$. Inlet costs are taken from the cost curve in Fig. 5-6.

5.6.8 Checking for Greater Storms

In checking inlets for performance with storms greater than the design storm, note that the spread on the pavement increases as $Q^{3/8}$, other variables remaining constant. Thus, if runoff is doubled, spread increases only $2^{3/8} = 1.3$ times. Assuming the inlet to have been designed for $Q_i/Q = 0.80$, this would reduce to about 0.7 but Q_i would increase about 90.7/0.8)2 = 1.75 times. One would then have to check the pipe capacity and particularly the head loss entering the pipe to see if the greater Q_i could be accepted. Depending on consequences of street flooding, consideration might be given to increasing the pipe capacity.

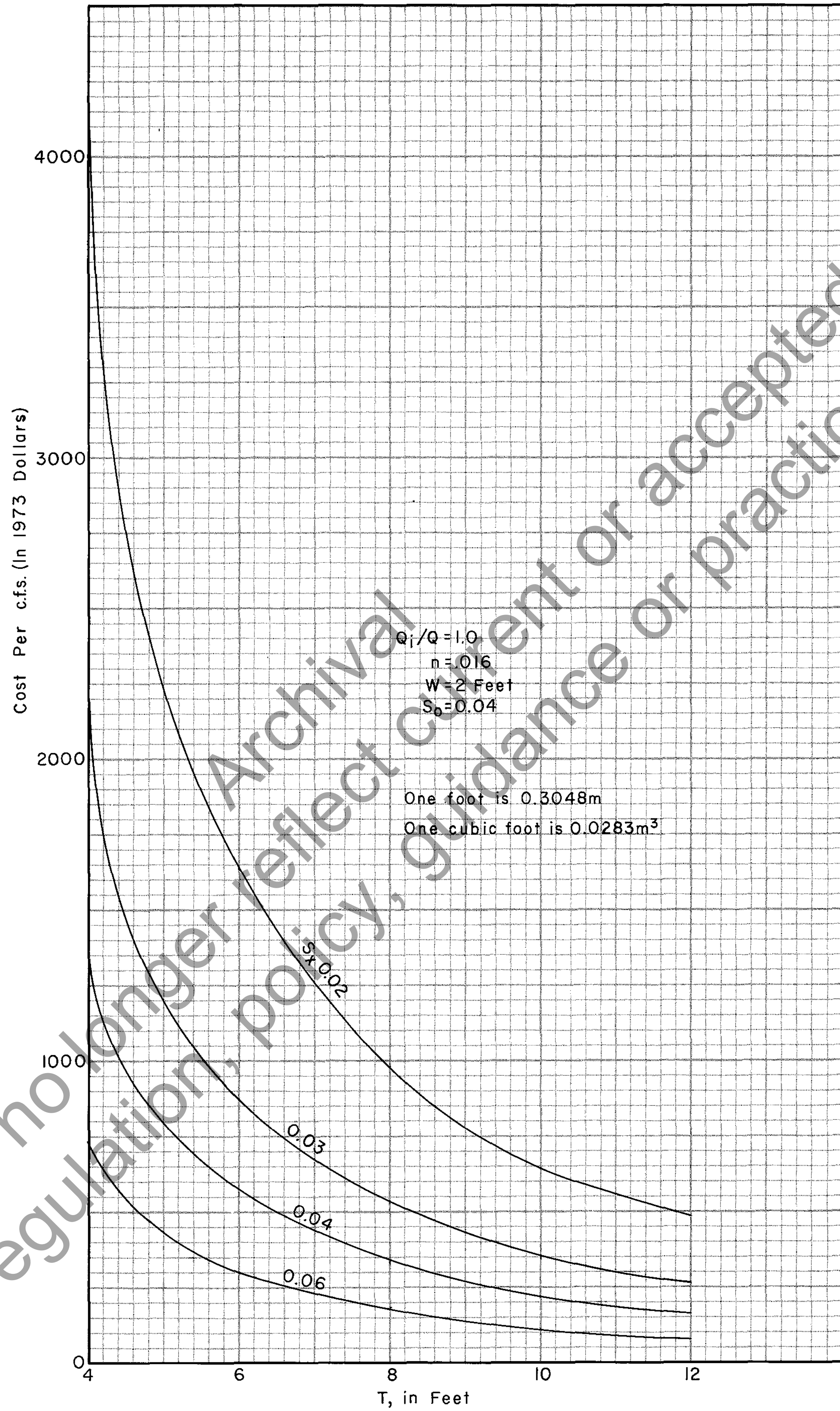


FIG. 5-9 Curb-opening inlets cost per c.f.s. for single inlet.

5.6.9 Capacity of a Curb Opening Inlet in a Sag

The capacity of curb opening inlets in a sag depends upon the depth of water at the inlet and the inlet geometry. The inlet operates as a weir until the water submerges the entrance. When the water depth exceeds about 1.4 times the height (h) of the curb opening entrance, the inlet operates as an orifice. Between weir-type operation and orifice-type operation, the capacity is indeterminate. Fig. 5-10 gives the minimum height (h_m) of opening required for weir-type operation. If the opening height (h) equals or exceeds h_m, Figs. 5-11 through 5-13 will give the depth of ponding measured at the curb, just above the depressed area. The use of these figures is explained in the following example.

Figs. 5-11 through 5-13 are based on experiments made at Colorado State University and apply to depressed curb opening inlets with a height of opening equal or exceeding the appropriate h_m from Fig. 5-10. When the inlet is not depressed, the approximate capacity can be computed by the weir equation:

$$Q_i = 3.0KL_i d_i^{1.5} \quad \text{............(5-4)}$$

where

Q_i = capacity of the inlet in cubic feet per second (cubic metres per second)

d_i = depth of water above inlet lip in feet (metres)

L_i = length of clear opening in feet (metres)

K = 1 (one) for English units; 0.5521 for metric units.

When the depth at the opening exceeds 1.4 h the capacity may be computed by the equation:

$$Q_i = KA\left[2g(d_i - \frac{h}{2})\right]^{1/2} \quad \text{...........(5-5)}$$

where

A = area of opening in square feet (square metres) (hL_i)

h = height of opening in feet (metres)

Q_i, d_i and L_i are the same as in equation 5-4

g = 32.16 ft/sec/sec (9.8024 metres/sec/sec)

K = 0.67 for English units or metric units.

5.6.10 Example 5-4: Curb Opening Inlet in a Sag

Given: a curb opening inlet in a sag; pavement cross slope 0.03; concrete broom finish (n = 0.016); depression, width = 1 foot, amount 1 inch; height of inlet opening = 0.75 foot, design discharge from both sides of the inlet, Q_1 = 2 cfs, Q_2 = 8 cfs; total Q = 10 cfs.

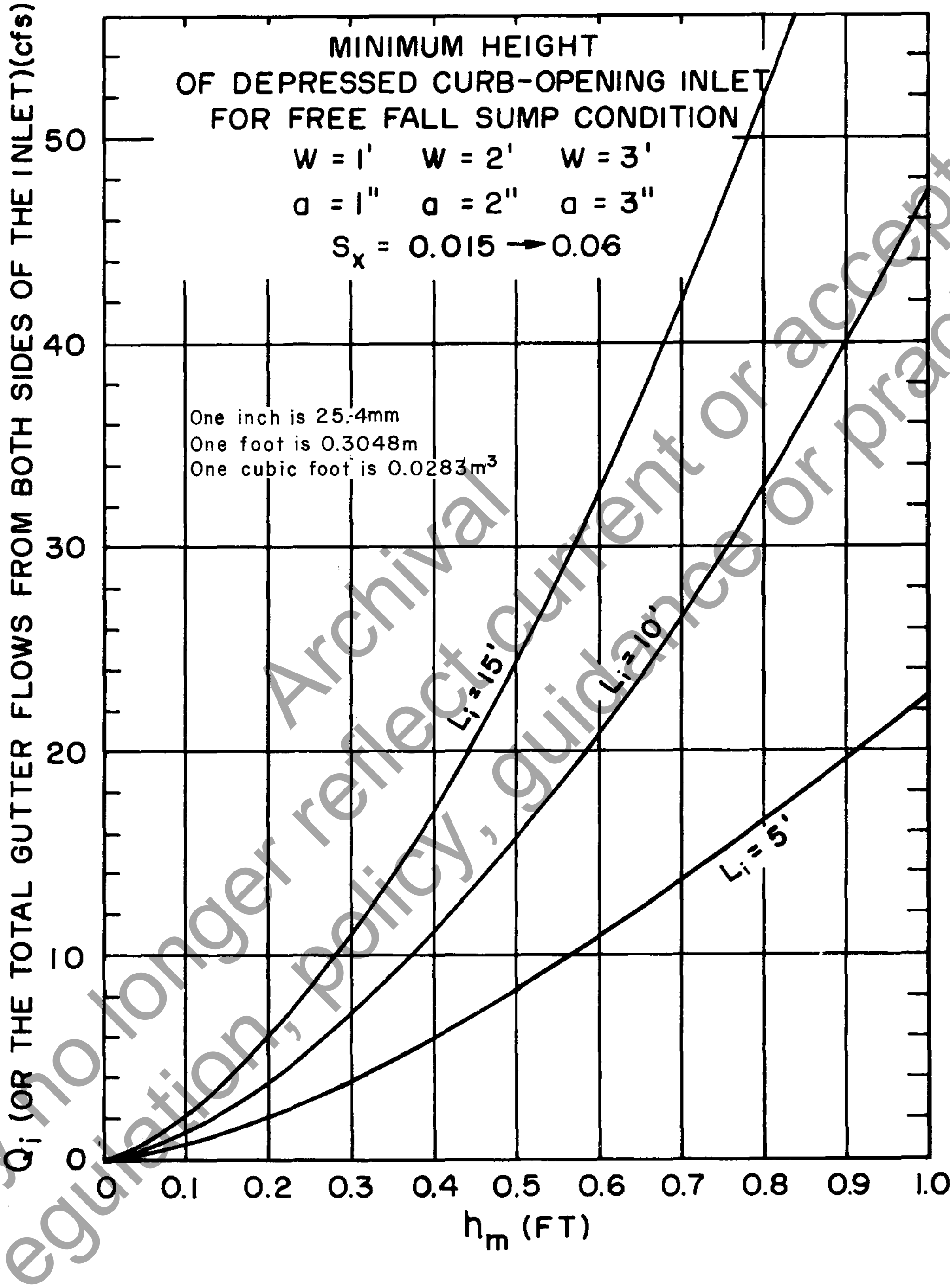


FIG. 5-10 *(After HEC-12)*

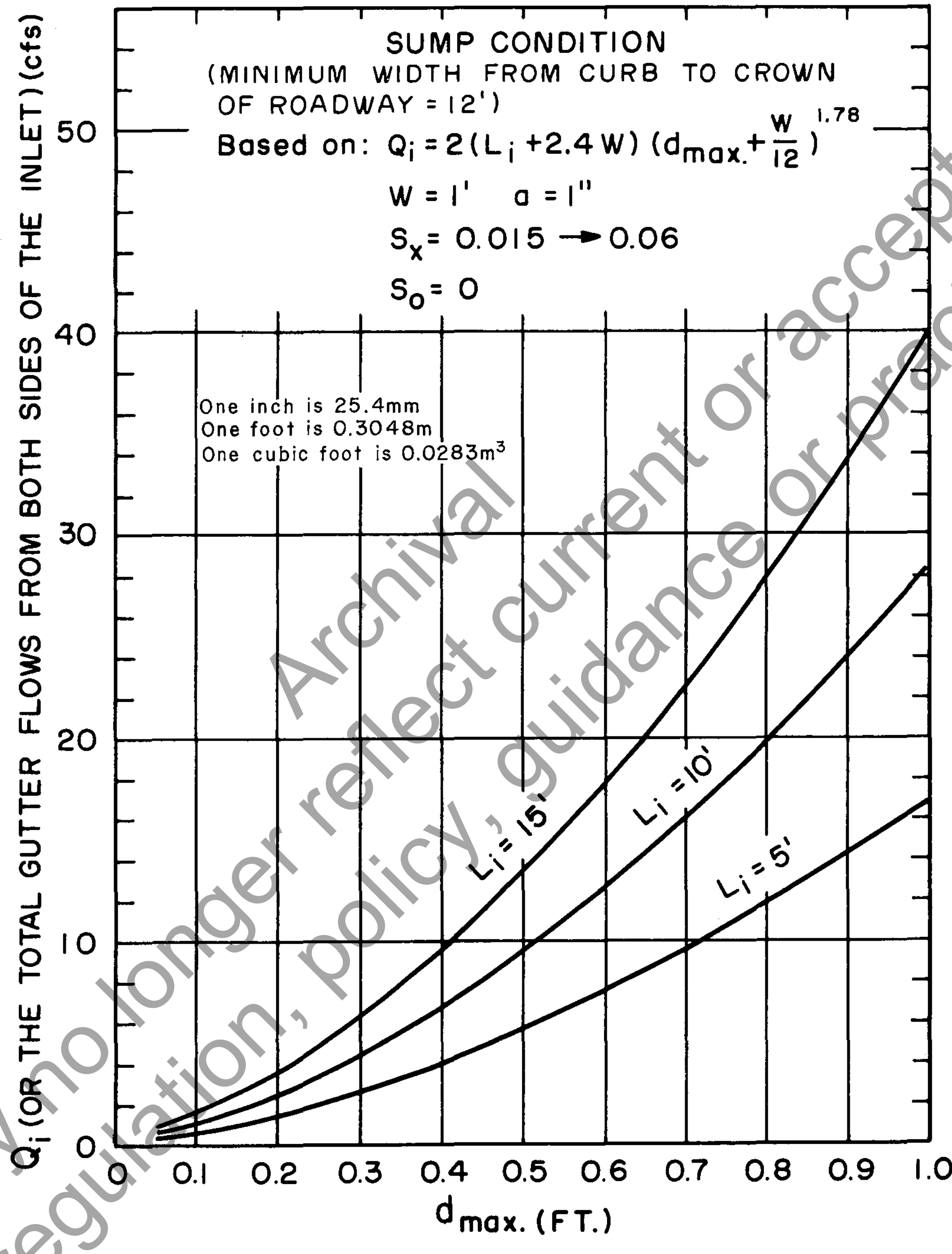


FIG. 5-11 *(After HEC-12)*

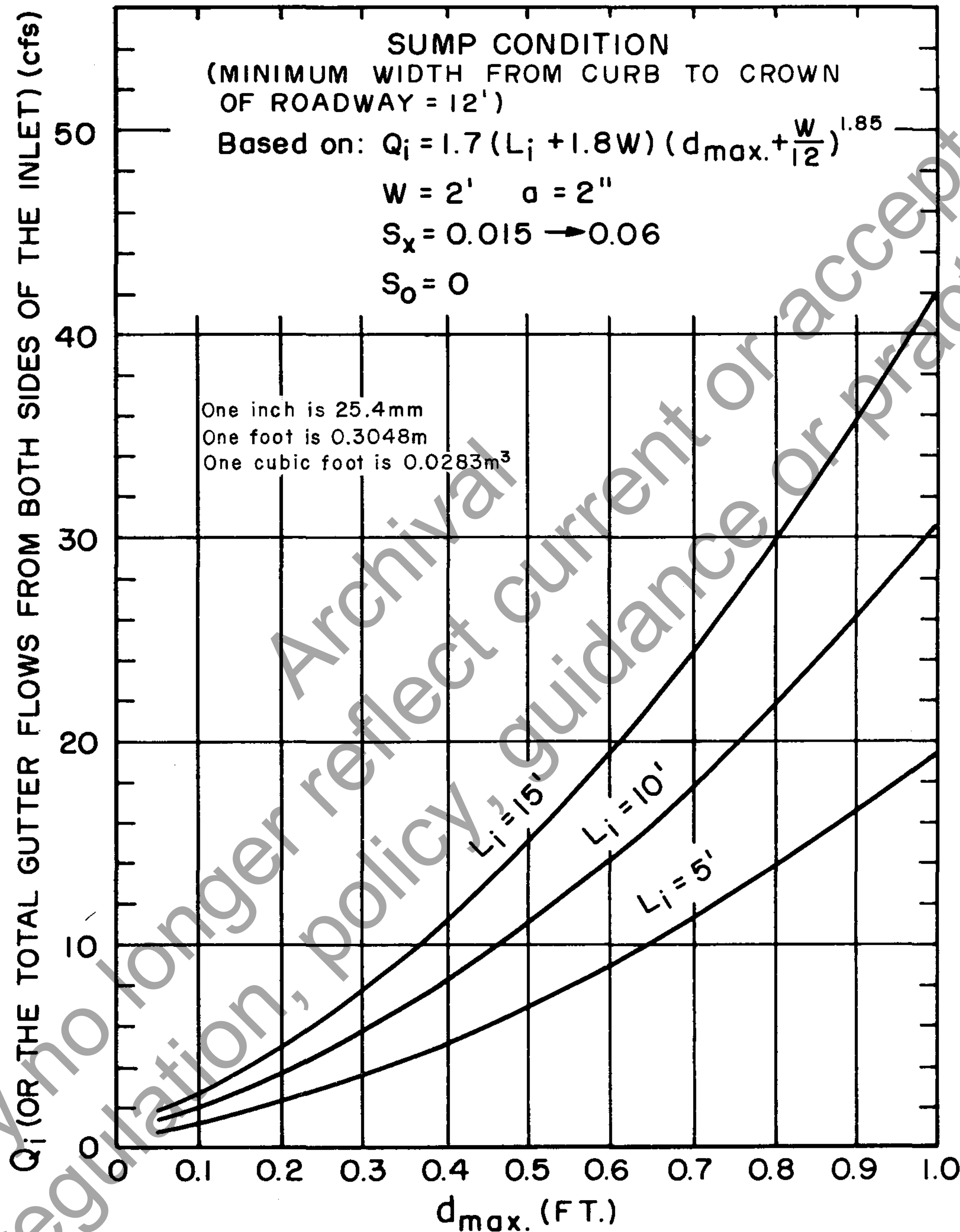


FIG. 5-12 *(After HEC-12)*

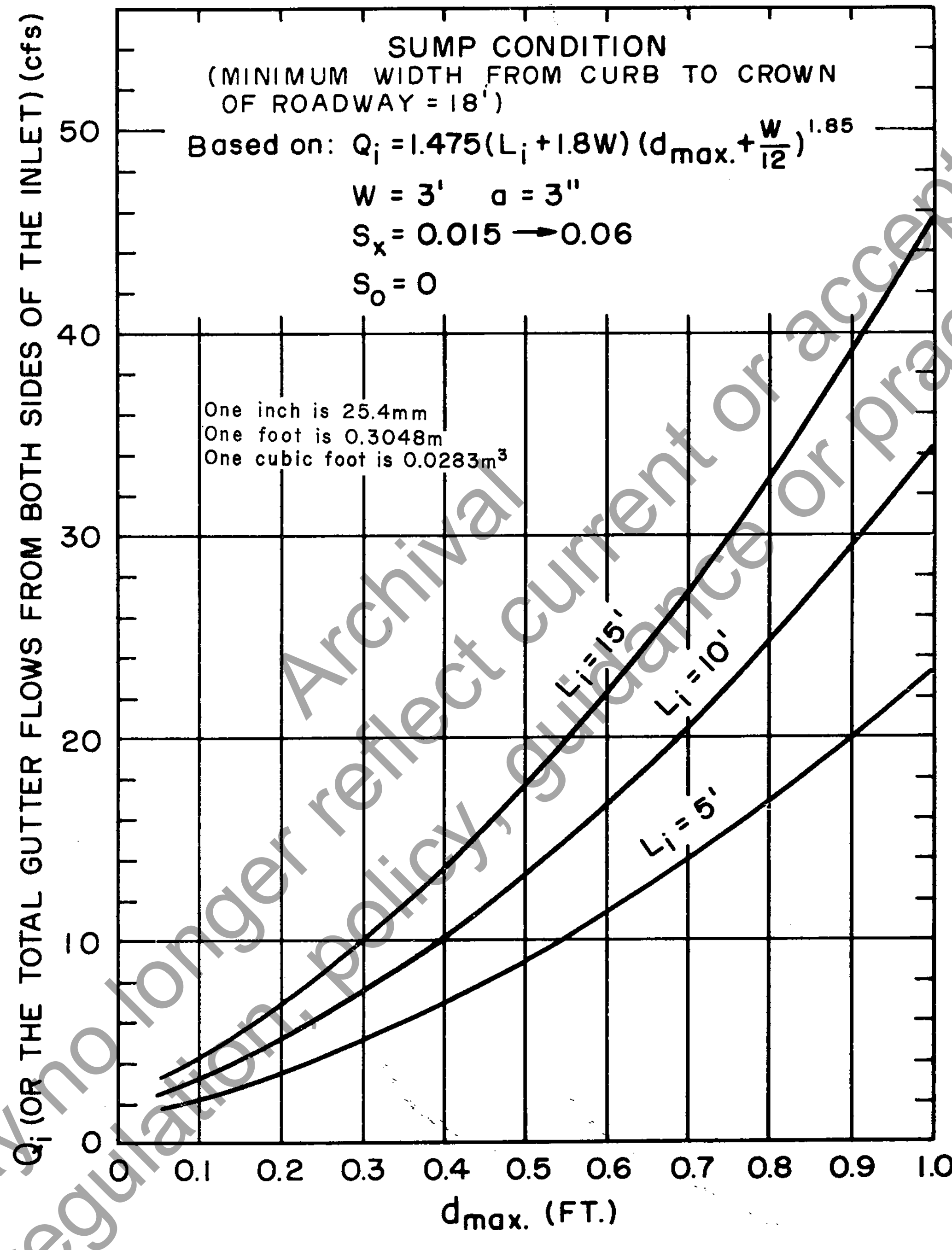


FIG. 5-13 *(After HEC-12)*

Find: maximum depth of ponding (d_{max}) for L_i = 5 feet; 10 feet; and 15 feet.

Solution:

1. Use Fig. 5-10 to check adequacy of the opening height to maintain free fall in the inlet. For Q = 10 cfs, the requirements are: L_i = 15 feet, h_m = 0.28 foot; L_i = 10 feet, h_m = 0.38 foot; L_i = 5 feet, h_m = 0.56 foot. The opening height, 0.75 foot exceeds the requirement for free fall for the three opening lengths and Fig. 5-11 can be used to determine depth of ponding.

2. From Fig. 5-11 the maximum ponding is:

L_i	15 feet	10 feet	5 feet
d_{max}	0.41 foot	0.52 foot	0.72 foot
T	13.7 feet	17.3 feet	24.0 feet

3. The maximum depth of ponding at the curb opening may be exceeded in the approach gutter, particularly on low flows. The depth of ponding in the gutter can be checked at the point where the gutter slope is 0.002 by using Fig. 5-14.

For L_i = 15 feet

Q_1 = 2 cfs

d_{max} = 0.41 foot (step 2), d = 0.3 foot (Fig. 5-14).

The gutter depth for Q_1 is less than the ponding depth at the inlet and water will back up in the gutter channel.

Q_2 = 8 cfs

d_{max} = 0.41 foot (step 2), d = 0.5 foot (Fig. 5-14).

The gutter depth for Q_2 is greater than the ponding depth at the inlet and the water profile tends to draw down on approaching the inlet.

For L_i = 10 and 5 feet

d_{max} = 0.52 or 0.72 foot (step 2), d = 0.3 or 0.5 foot

The gutter depth for both Q_1 and Q_2 is less than the ponding depth for both 5- and 10-foot inlets and water will back up in the gutter on both sides of the inlet.

In addition to illustrating the use of the sag curves in Figs. 5-10 to 5-14 this example shows the desirability of picking up most of the gutter flow before it reaches the low point of the sag vertical curve. Spreads on the pavement (T) and depths at curb (d_{max}) noted in step 2 should not be tolerated on a high-speed highway. The more common application

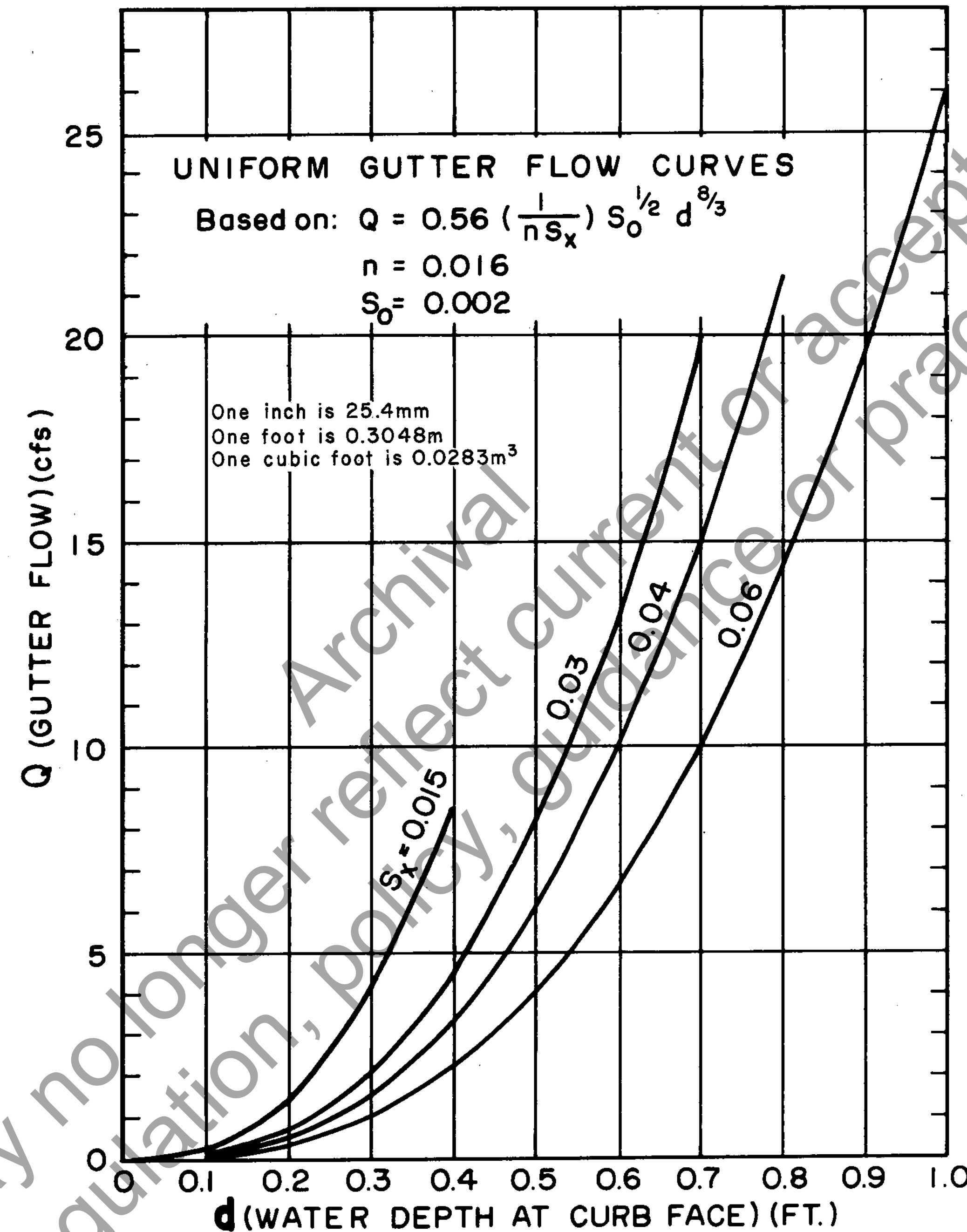


FIG. 5-14 *(After HEC-12)*

of the sag curves would be in designing curb opening inlets or their spacing to keep the depth of ponding and spread on the pavement within tolerable limits.

5.6.11 Spacing of Inlets in a Sag

It is desirable that three inlets be placed in a sag vertical curve: one at the low point and one on each side of this point where the grade elevation is approximately 0.2 foot (61.0mm) higher than that at the low point.

As a result, the inlets in the sag of a highway must at times be designed to remove the stormwater resulting from a large storm over the contributing area minus the flow intercepted by the inlets on the grade which are designed to limit the spread of water to a tolerable limit. The inlets on the grade will intercept a greater quantity of water during the larger storm than the quantity used to determine their spacing but the spread of water on the pavement will exceed the spread designated as the tolerable limit for design.

Because of the various combinations in which sag inlets are used, examples cannot be given to fit all problems encountered by the designer. The example given will illustrate the spacing of inlets in a sag which is designed for a 50-year frequency when the inlets on the grade are spaced for a 10-year frequency. The problem of sag inlets designed for some other frequency can be solved with a slight modification of the procedure used in the example. The procedure can be used for other type inlets whose capacities are known.

5.6.12 Example 5-5: Design of Curb Opening Inlets in a Sag

Given: high-type pavement ($n = .016$) with two 12-foot lanes draining into a 2-foot wide gutter with vertical curb; grades -3% and +3%, each 2170 feet long intersecting at Station 50. A 600-foot vertical curve connects the tangents. Pavement cross slope $S_x = 0.03$. Permissible spread on pavement, T = one-half traffic lane or 6 feet (total spread is 8 feet including 2-foot wide gutter); design frequency is 10 years; time of concentration is 5 minutes. The inlets on the grades are 10-foot long curb opening inlets designed to limit the spread on the pavement to 6 feet at the 10-year frequency. The gutter is 2 feet wide and the depression is 2 feet wide and 2 inches deep.

Find: size and location of the three curb opening inlets in the sag. The inlets will be designed to limit the spread on the pavement for a 50-year frequency storm to 6 feet.

Solution:

1. The grades are symmetrical about the P.I. of the vertical curve and only a half section need be considered. The first inlet is located 830 feet from the crest and successive inlets are spaced at 520-foot intervals. The computations for peak flow arriving at the sag inlet at

Station 50 are given in Table 5-4.

2. The 10-foot curb openings are spaced (column 3) for a 10-year frequency storm. An inlet of width opening to be determined, is placed at the P.V.I. Station 50+00. The inlet at Station 49+40 is placed where the grade elevation is about 0.2-foot higher than the grade elevation at the P.V.I. A 10-foot curb opening is tentatively placed here and the computations shown in Table 5-4 are made to determine the width of opening required in the sag. If the sag inlet opening is excessive, wider openings can be used at the 0.2-foot higher elevation point. The spacing and width of opening on the grades might require adjustment in some instances.

3. The runoff between inlets (column 10) is computed by the rational method based on the 50-year rainfall intensity (column 9) during the accumulated time of concentration (column 8). Column 11 is the Q arriving at the inlet and consists of the Q_c (column 16) bypassing the last inlet plus the Q (column 10) from the area between inlets. On the grade, the spread on the pavements, T (column 13) does not exceed the allowable spread, 8 feet which was based on a 10-year rainfall intensity.

4. The discharge arriving at the sag inlet from both sides is 0.94 cfs (column 11). From Fig. 5-10 this Q would require the following height of opening; L_i = 15 feet, h_m = 0.05 foot; L_i = 10 feet, h_m = 0.07 foot; L_i = 5 feet, h_m = 0.10 foot.

5. The depth at the curb for an allowable spread of 6 feet on the travelled way is d = T/Z = 8/33.33 = 0.24 foot. On Fig. 5-12 for w = 2 feet, a = 2 inches and Q = 0.94 cfs; a 10-foot opening will carry the flow with a depth of ponding in the gutter (d_{max}) = 0.01 foot and a 5-foot opening will carry the flow with a depth of ponding 0.06 foot. The ponding with the 10-foot opening is less than the allowable (0.24 foot) and the 5-foot opening with a clear height at least (0.16 foot) (step 4) is satisfactory.

5.6.13 Conclusions

In designing a drainage system with curb and gutter, the criteria established for cross slope of pavement must consider the effect of cross slope on inlet efficiency. A composite section concentrates more flow near the curb and probably increases the inlet efficiency as discussed earlier. Inlet lengths can be reduced greatly if Q_i/Q is 0.80 or less and carry-over flow can be permitted. This is especially effective when inlets are in series. The criterion for spread should not be less than 10 feet (3.048m) unless cross slope is very steep, as the cost per cfs (m^3/s) intercepted rises sharply as spread is reduced.

COMPUTATIONS FOR SAG INLETS
(EXAMPLE IN PARAGRAPH 5.6.12)

Inlet Station	Elev. (ft.)	Dist. (ft.)	Grade %	A (acres)	C	CA	Tc (min.)	i* 50-yr.	Q (cfs)	Q (cfs)	d (ft.)	T (ft.)	** Qi/Q	Qi (cfs)	Qc (cfs)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
36 + 60	140.20	830	3.0	0.50	0.8	0.40	5	9.1	3.64	3.64	0.22	7.39	0.67	2.44	1.20
41 + 80	124.60	520	3.0	0.31	0.8	0.25	5	9.1	2.28	3.48	0.22	7.21	0.69	2.40	1.08
P.V.C. 47 + 00	109.00	520	3.0	0.31	0.8	0.25	5	9.1	2.28	3.36	0.22	7.25	0.69	2.32	1.04
49 + 40	104.69	240	0.6	0.14	0.8	0.11	5	9.1	1.00	2.04	0.24	8.04	0.90	1.84	0.20
P.V.I. 50 + 00	104.50	60	0.0	0.04	0.8	0.03	5	9.1	0.27	0.47					
							Total at Sta. 50 + 00			0.94	0.06Ɣ	2.03Ɣ			

One foot is 0.3048m. One cubic foot is 0.028m^3

Assumed or given: $n = 0.016$
$S_x = 0.03$; $Z = 33.33$
$S_o = \pm 0.03$
Max $T = 8$ ft.
$T_c = 5$ min.
$L_i = 5$ ft.

* From Fig. 2-7 (p.2-13)
** From Fig. 5-6 (p.5-27)
Col. 10 = Col. 7 x Col. 9
Col. 11 = Col. 10 + Col. 16
Col. 12 = T/Z
Col. 16 = Col. 11 - Col. 15
Col. 13 from Equation (6), Table 5-3, p. 5-24
Ɣ From Equation on Fig. 5-12, p. 5-38

TABLE 5-4

5.7 Inlet Grate Design Procedure

Initially, hydraulic, structural and debris handling characteristics of seven bicycle-safe grate inlets and one standard parallel bar grate inlet were evaluated by the Bureau of Reclamation's Engineering and Research Center for the Federal Highway Administration (Ref. 5-10). The tests were conducted at cross slopes of 1 to 48, 1 to 24 and 1 to 16; and longitudinal slopes of 0.5, 1, 2, 4, 6, 9 and 13 percent with gutter flows up to 5.6 cfs (0.1589m^3/s). The grates were 2 feet (0.610m) wide by 2 feet (0.610m) long and 2 feet (0.610m) wide by 4 feet (1.219m) long.

Subsequent tests were made on selected grates in several other sizes: 1.25 feet (0.381m) wide by 2.0 feet (0.610m) long; 1.25 feet (0.381m) wide by 2.67 feet (0.813m) long; 3 feet (0.914m) wide by 2 feet (0.610m) long; and 3 feet (0.914m) wide by 4 feet (1.219m) long. These tests were made for the same range of cross slopes and longitudinal slopes as the original set of tests. The configuration and dimensions of the grates which were tested are given on Figs. 5-15 to 5-20 inclusive. The grates were placed flush with the pavements in all instances.

5.7.1 Hydraulic Characteristics

For purposes of hydraulic analysis, it is convenient to consider the flow intercepted by an inlet grate as consisting of two parts: (1) frontal flow or that portion of the intercepted flow which passes over the upstream front edge of the grate, and (2) side flow or that portion of the intercepted flow which passes over the edge of the grate parallel to and away from the curb.

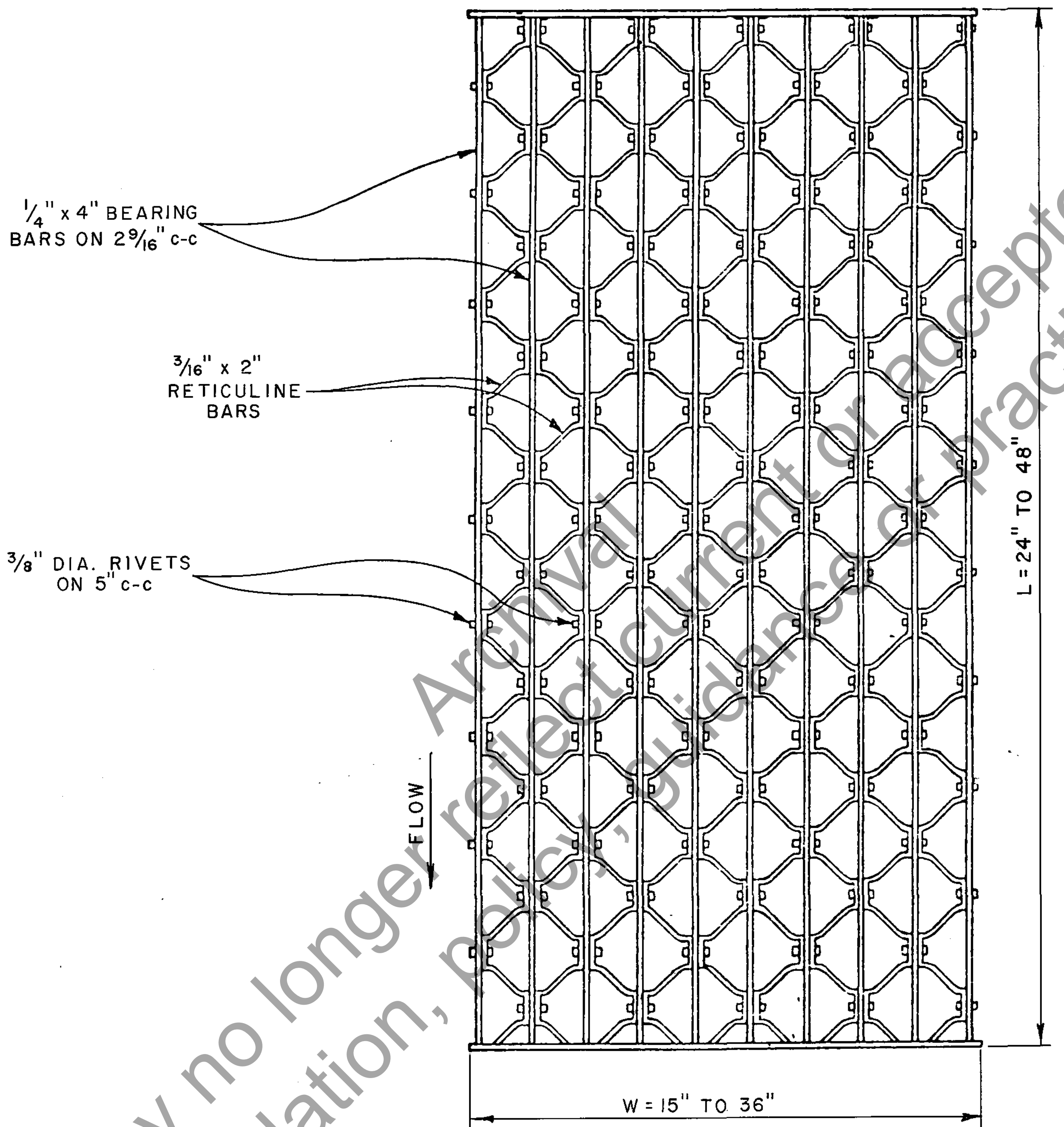
The hydraulic efficiency, E, of a grate is defined as the ratio of the total flow intercepted, Q_i in cfs or m^3/s to the total gutter flow, Q_T also in cfs or m^3/s.

$$E = Q_i/Q_T \qquad \text{...........(5-6)}$$

The percent of frontal flow intercepted depends mainly on bar configuration, grate length and velocity of flow. On mild slopes normally 100% of the frontal flow will be intercepted. On steep slopes the higher velocity flow may cause the water to splash over the grate. When splash-over occurs, only a portion of the frontal flow is intercepted.

The amount of side flow intercepted increases as the length of the grate increases and it decreases as the velocity of flow increases.

For grates on a continuous grade, the quantity of flow intercepted increases as the spread increases and for this reason, economy of design usually requires that a percentage of the approach gutter flow be allowed to flow around the inlet and be subsequently picked up by downstream inlets or at the sump. The spacing of grate inlets on continuous grades is therefore determined by the allowable width of water on the pavement and the efficiency of the inlets.



One inch is 25.4mm

FIG. 5 – 15 *Steel fabricated reticuline grate.*

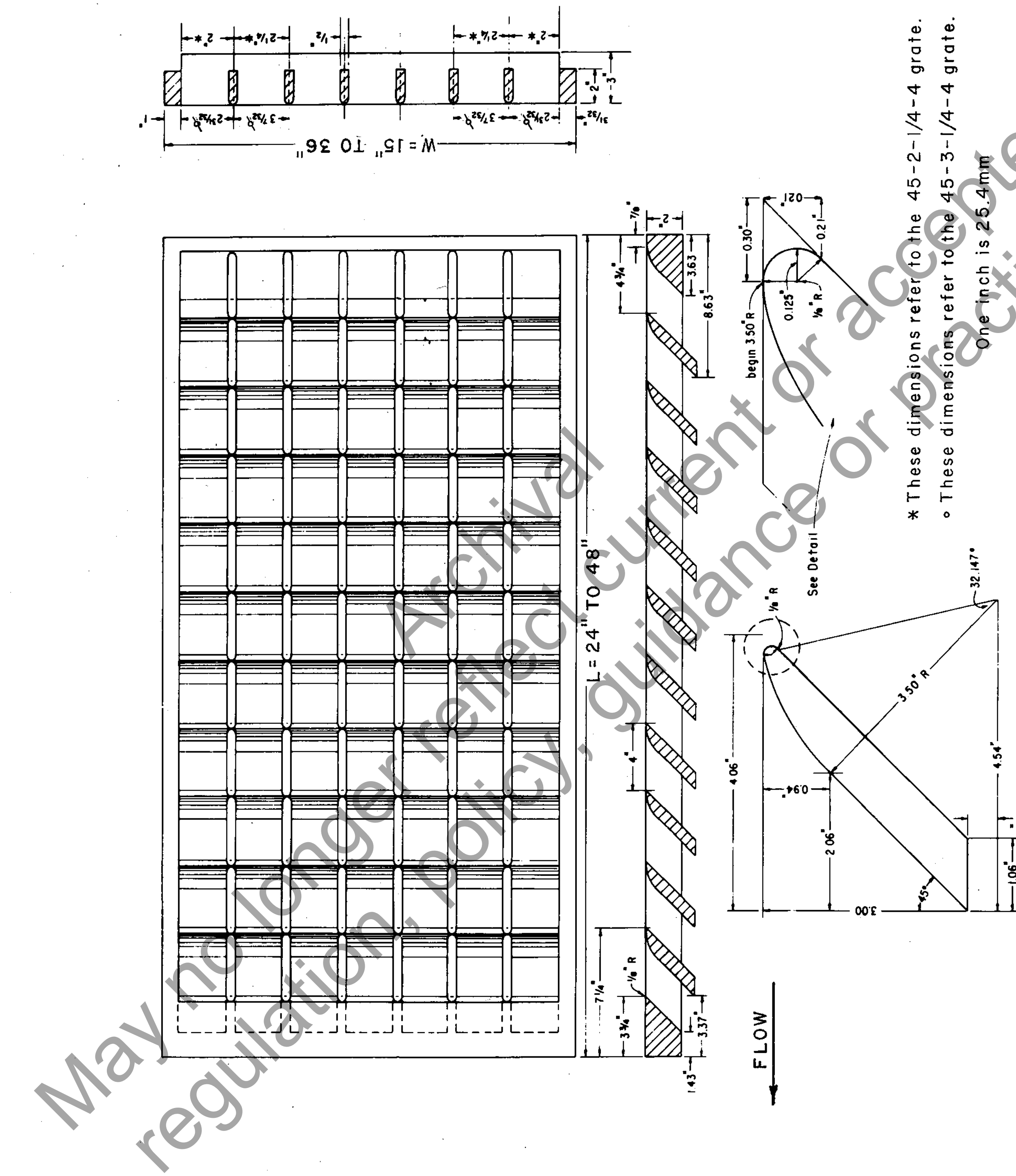


FIG. 5-16 *Cast 45° tilt grate.*

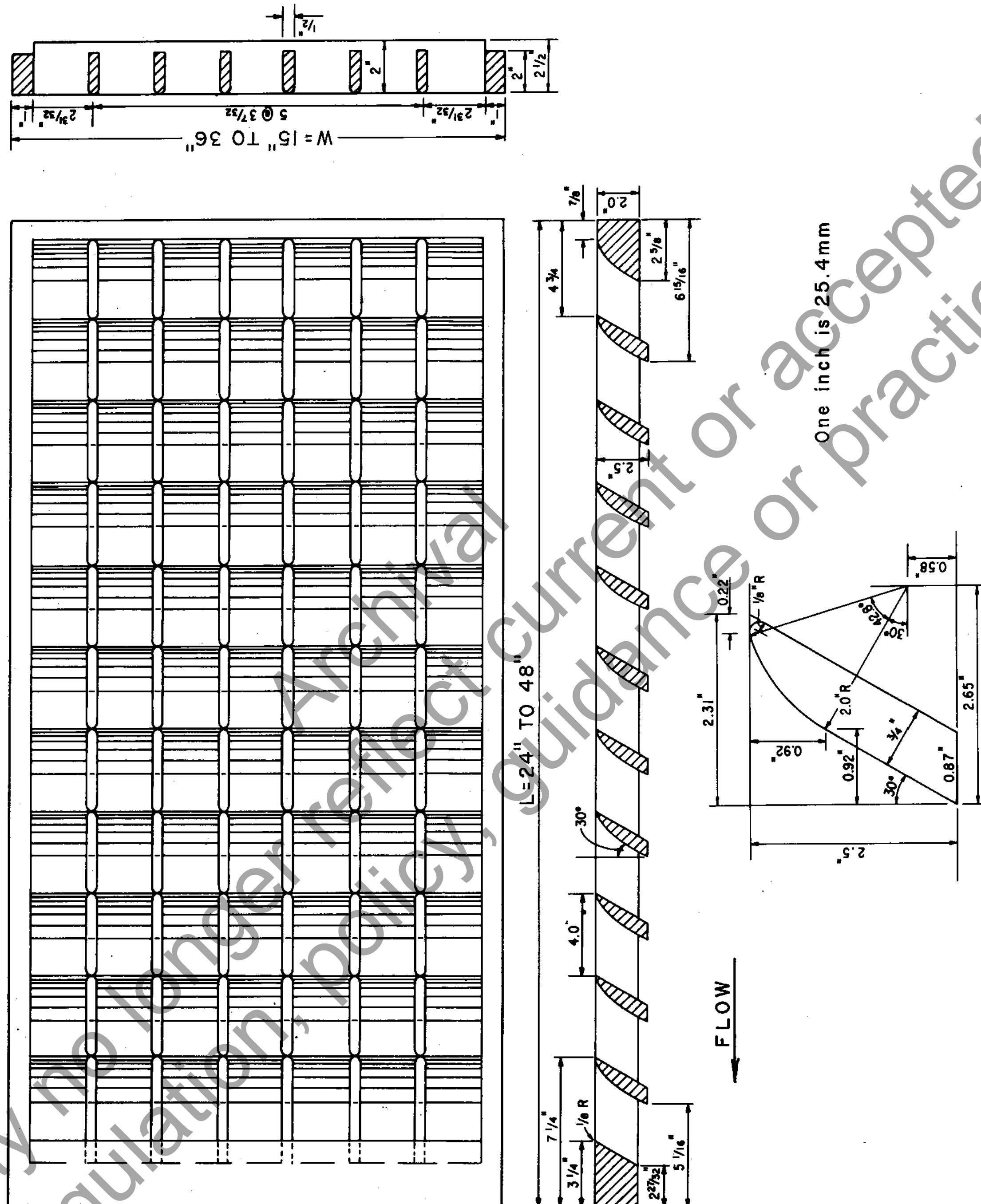


FIG. 5-17 *Cast 30° tilt bar grate.*

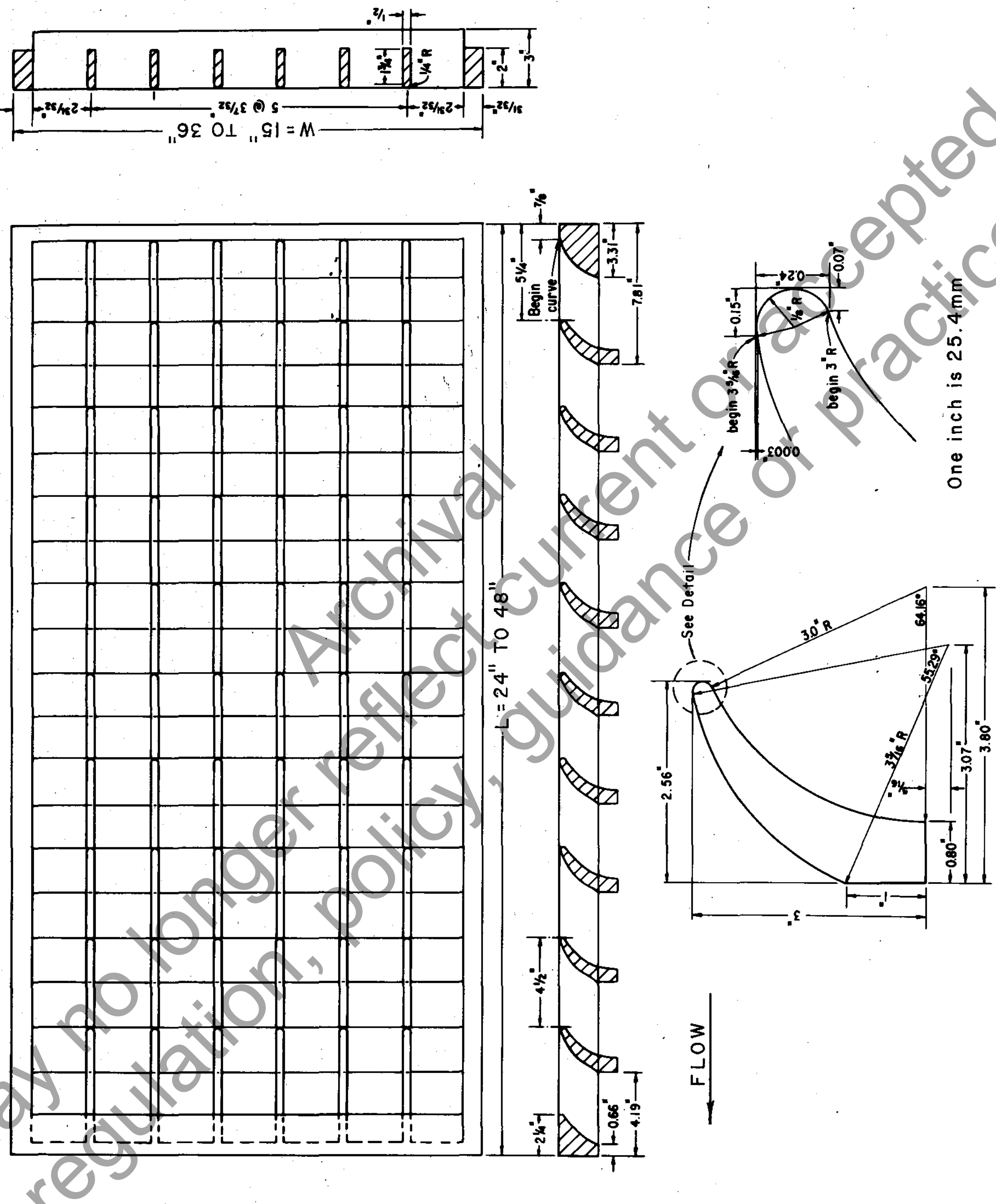


FIG. 5-18 *Curved vane grate.*

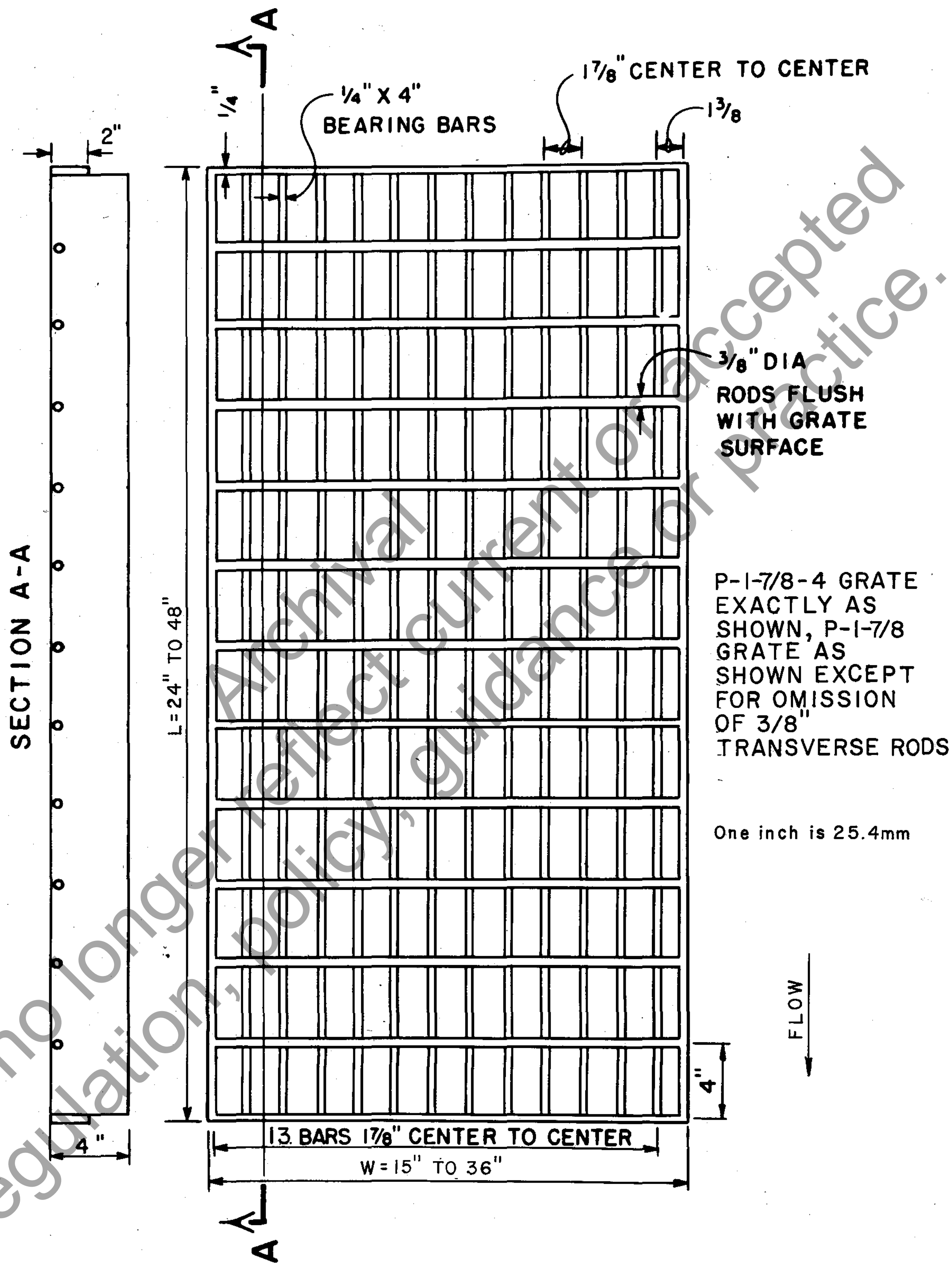


FIG. 5-19 *Steel fabricated P-1-7/8-4 & P-1-7/8 grates.*

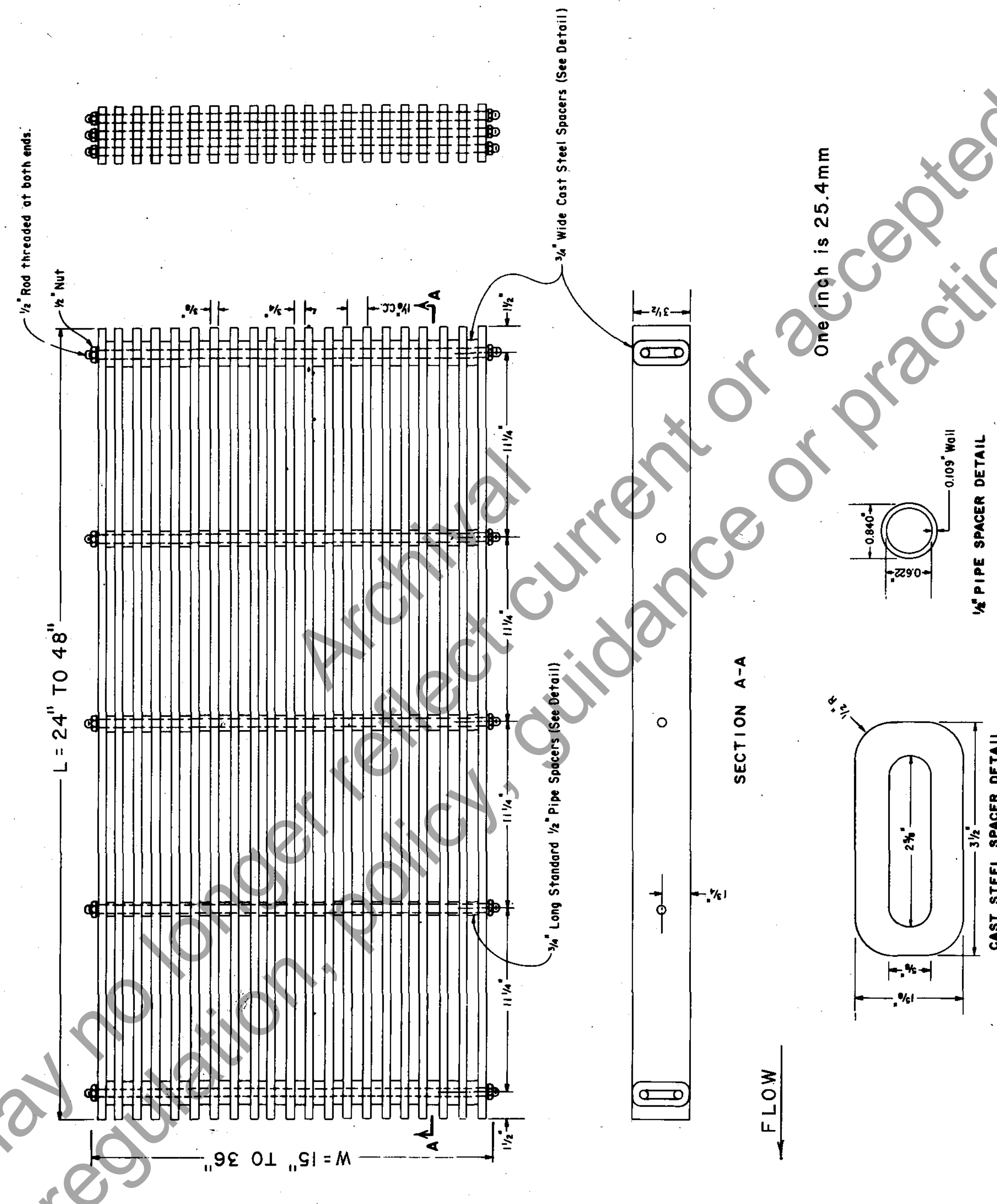


FIG. 5-20 *Steel fabricated P-1-1/8 grate.*

From the modified Manning equation for gutter flow it can be derived that the ratio of approach frontal flow, Q_F in cfs or m^3/s, to total gutter flow, Q_T in cfs or m^3/s, is:

$$\frac{Q_F}{Q_T} = 1 - \left[1 - \frac{W}{T}\right]^{8/3} \qquad \text{.........(5-7)}$$

where W is the width of the grate in feet or metres and T is the width of water or spread (in feet or metres) on the pavement. This ratio is equal to the theoretical efficiency of a grate inlet assuming 100% frontal flow interception and no side flow interception.

Side flow may be considered in the above equation by substituting an effective width W_E for W. The effective width W_E in feet or metres is equal to W plus ΔW where ΔW is the extra grate width (in feet or metres) which would be necessary for the inlet to have the same efficiency without side flow interception. ΔW is a constant for any given longitudinal slope, cross slope, grate size and bar configuration. The equation for estimating grate inlet efficiency, E_o, without splash-over is therefore:

$$E_o = 1 - \left[1 - \frac{W_E}{T}\right]^{8/3} \qquad \text{.........(5-8)}$$

This equation may be solved graphically using Fig. 5-21.

Values of ΔW for the eight grate configurations tested may be obtained from Figs. 5-22a through 5-29a.

The efficiency of the inlet under splash conditions depends on frontal velocity and is computed by multiplying equation 5-8 by a reduction factor R. Frontal flow velocity is the average flow velocity of that portion of the intercepted flow which passes over the upstream front edge of the grate. Figs. 5-22b through 5-29b give R as a function of V_F the average frontal flow velocity in feet or metres per second. This latter can be obtained by multiplying the average flow velocity in the gutter V_{av} (in feet or metres per second) by the coefficient K as given in Fig. 5-30. The equation for computing frontal flow velocity is

$$V_F = KV_{av} = K(2Q_TZ/T^2) \qquad \text{..........(5-9)}$$

in which Q_T is the total gutter flow in cfs or m^3s; Z is the reciprocal of the cross slope; T is the spread in feet or metres.

5.7.2 Factor of Safety

Grate inlets should be designed to allow for unpredicted hydraulic conditions or partial plugging. The latter may considerably reduce inlet efficiency. Grate lengths longer than necessary for 100% frontal flow interception will allow for some debris accumulation. The grate length necessary to intercept 100% of the frontal flow is given in Fig. 5-31 as a function of frontal flow velocity. In this figure L' is the effective or unclogged grate length, which is assumed in the design.

GRATE INLET DESIGN CURVES

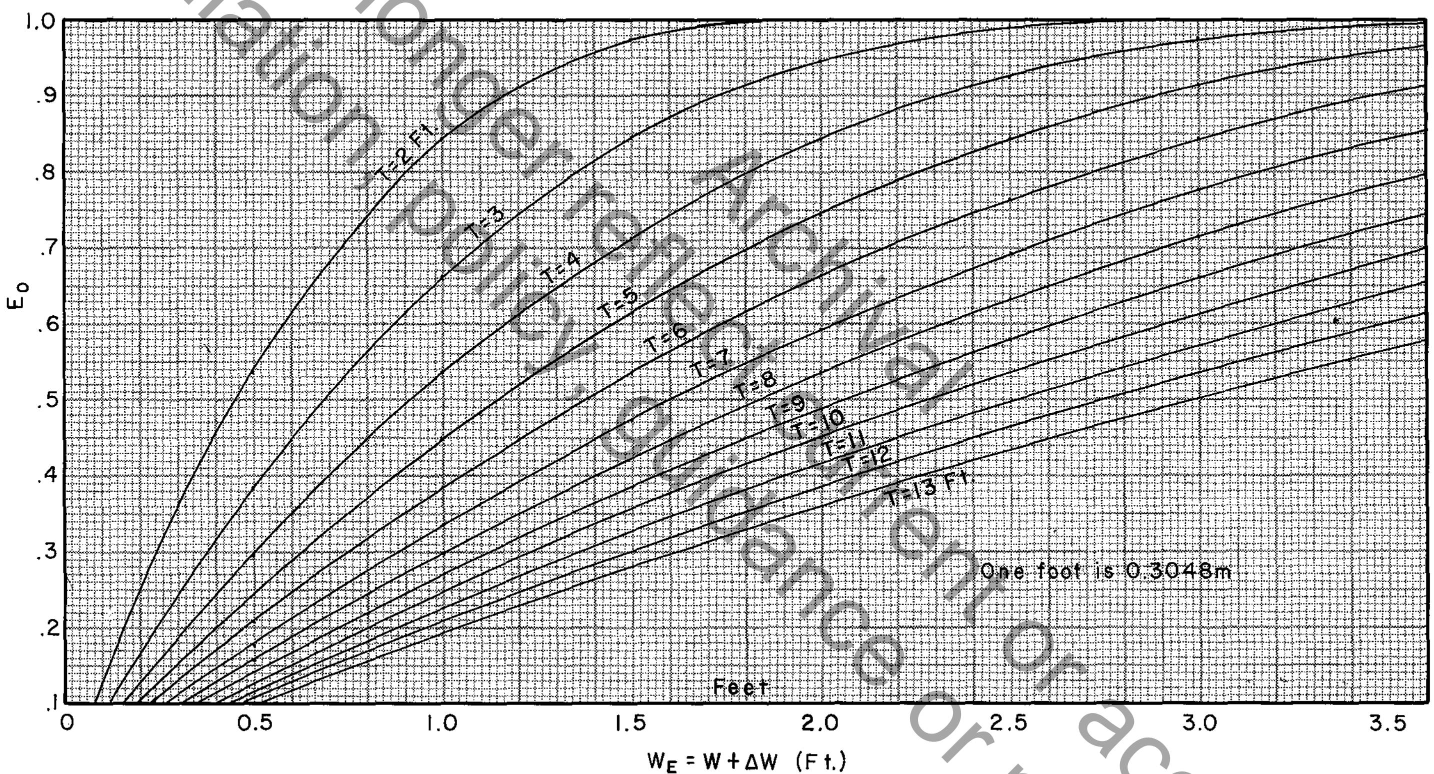
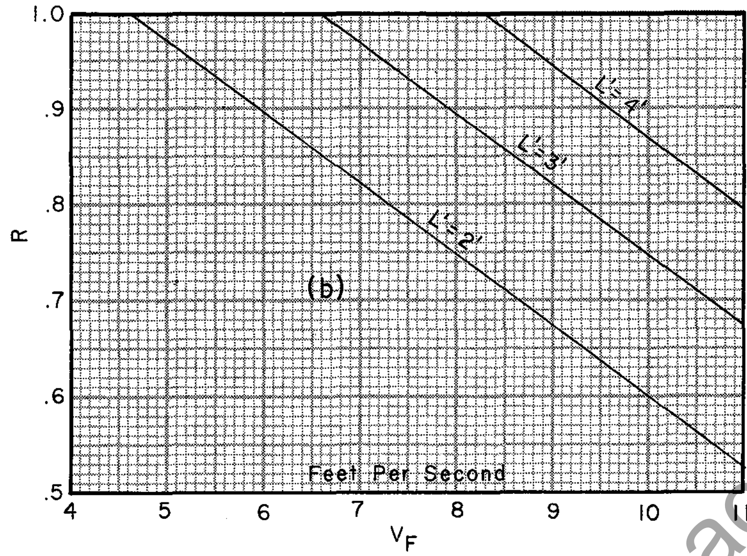


FIG. 5-21

GRATE INLET DESIGN CURVES



One foot is 0.3048m

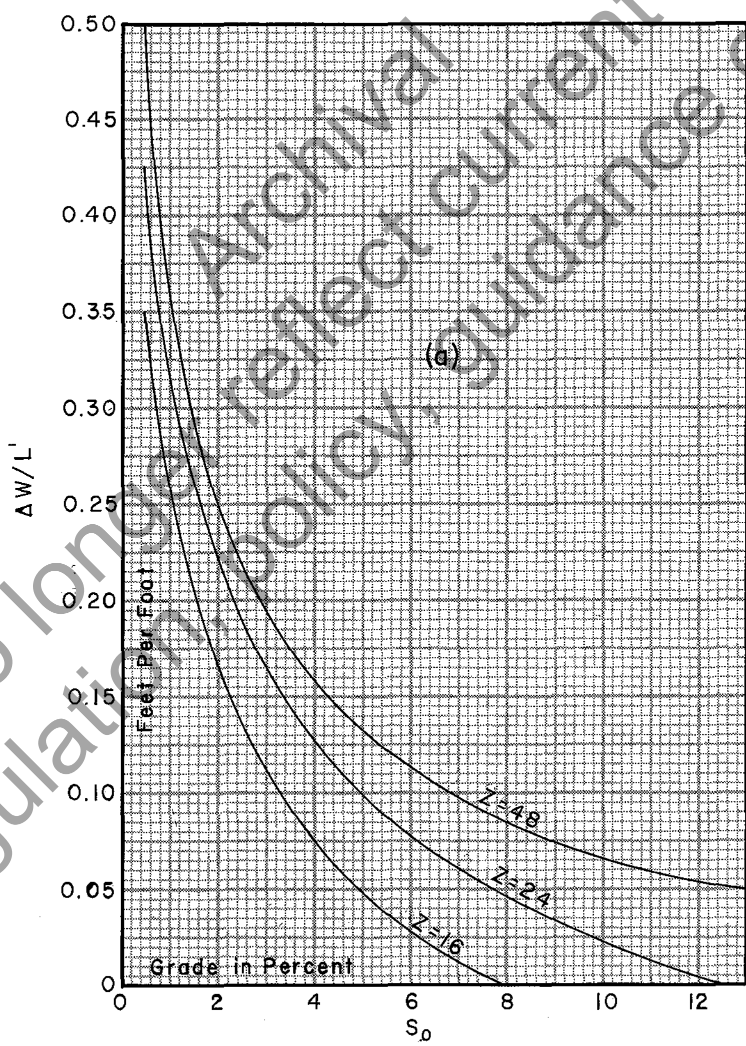
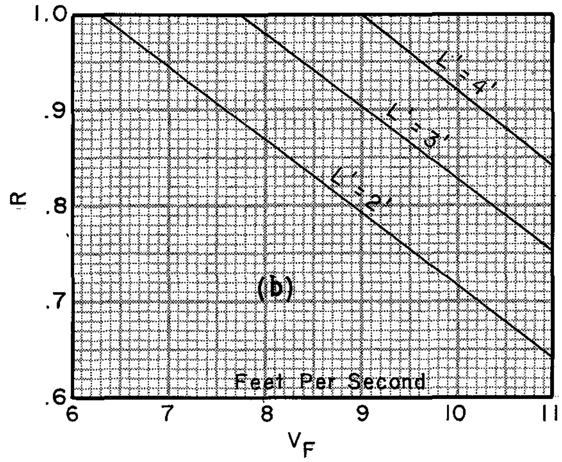


FIG. 5-22 (a) & (b) Reticuline Grate.

GRATE INLET DESIGN CURVES



One foot is 0.3048m

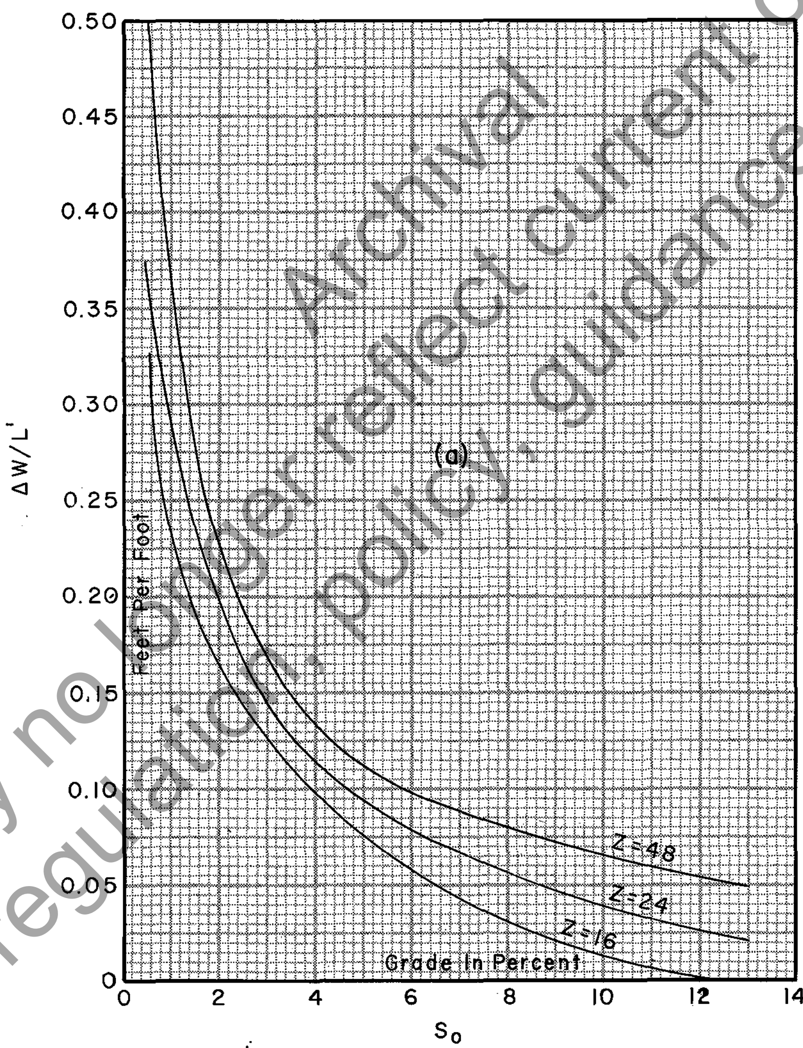
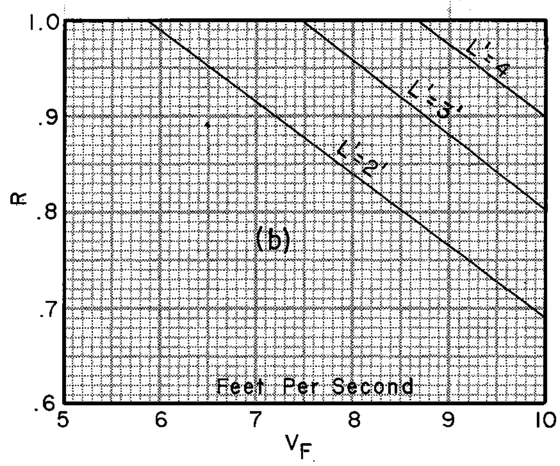


FIG. 5-23(a) & (b) 45° Tilt Bar Grate (2 1/4" cc)

GRATE INLET DESIGN CURVES



One foot is 0.3048m

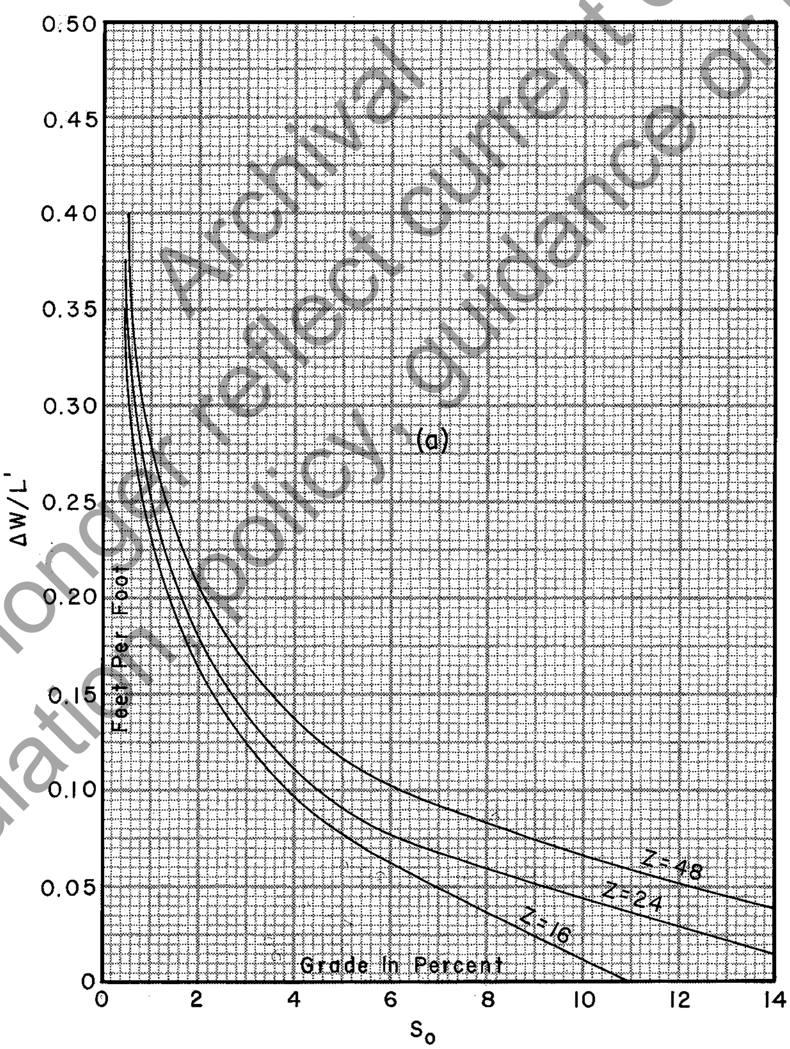
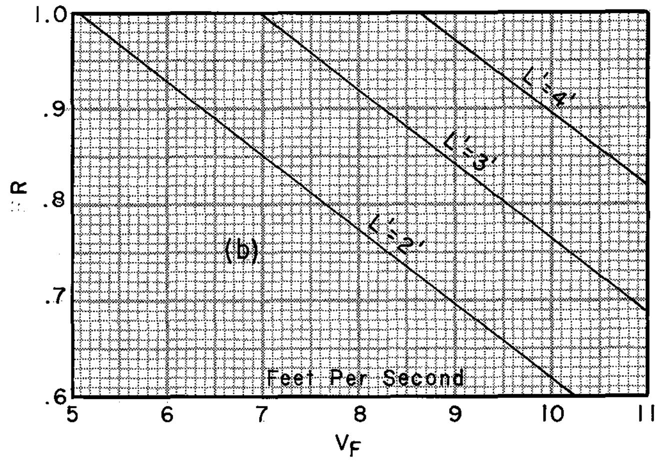


FIG. 5-24(a) & (b) 45°, 3 1/4" – 4" Grate

GRATE INLET DESIGN CURVES



One foot is 0.3048m

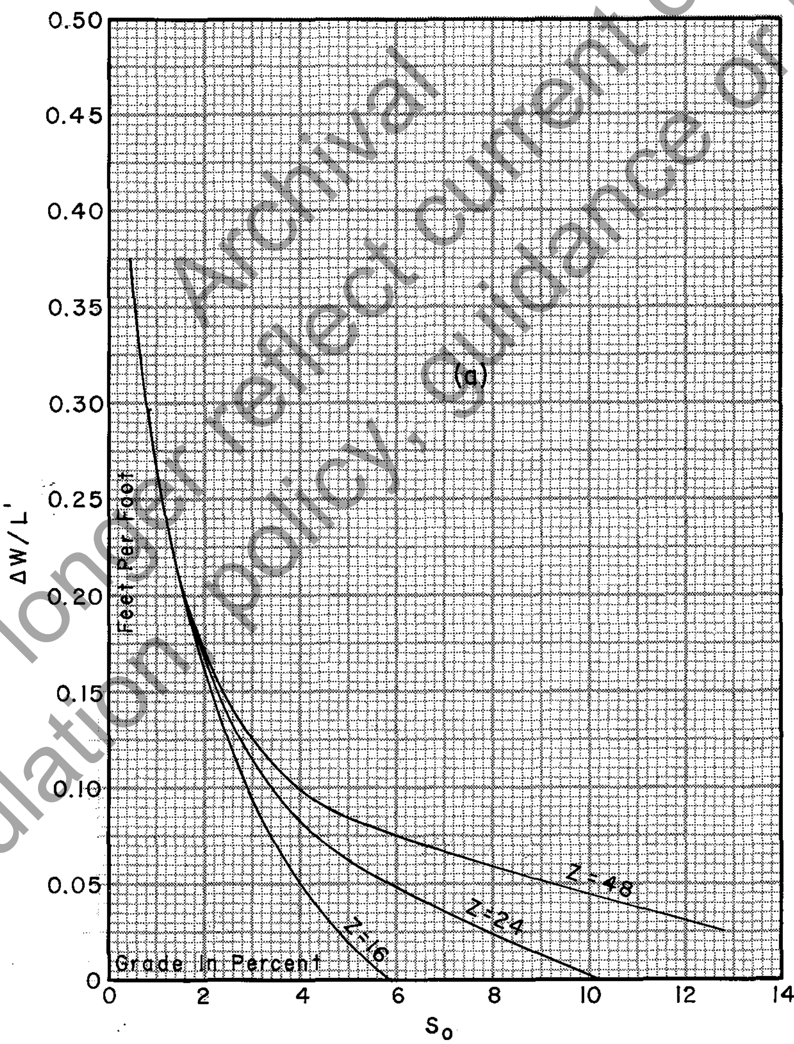
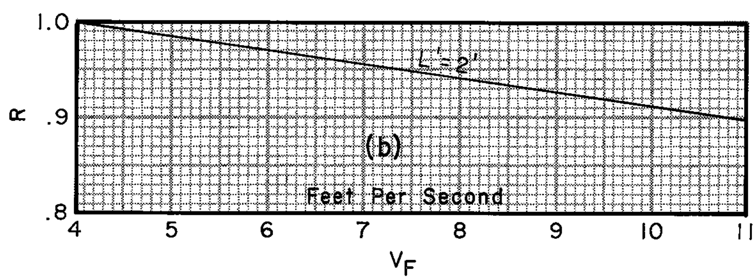


FIG. 5-25 (a) & (b) 30° Tilt Bar Grate

GRATE INLET DESIGN CURVES



One foot is 0.3048m

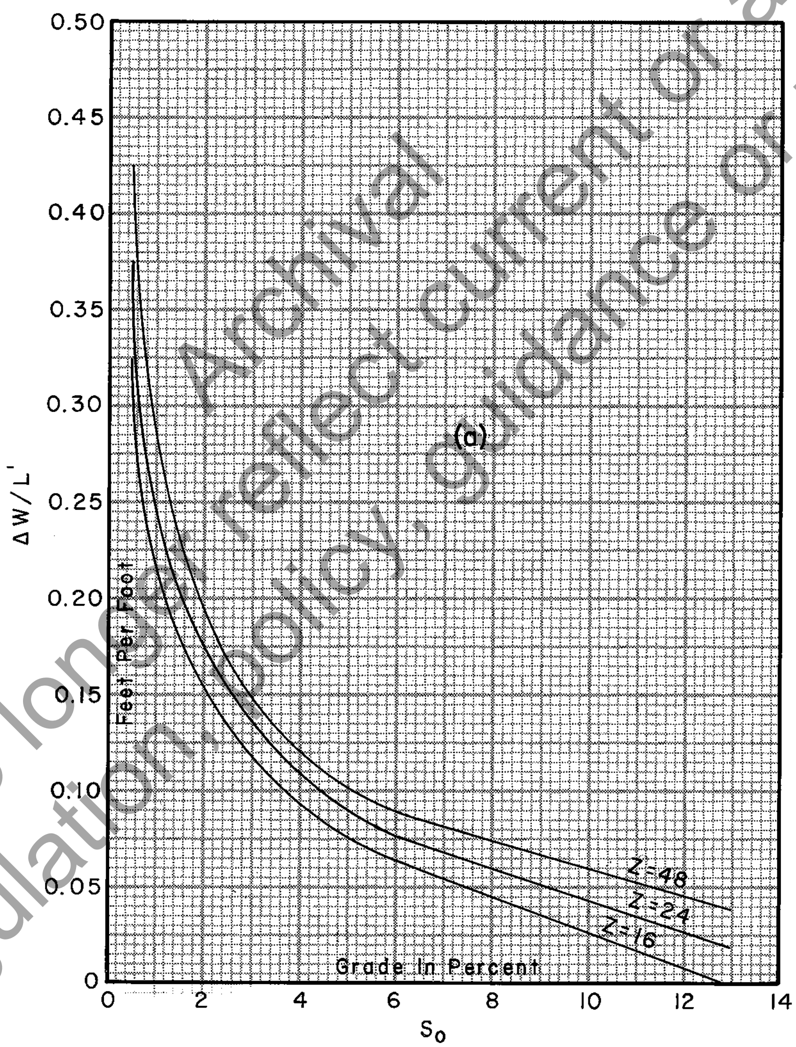
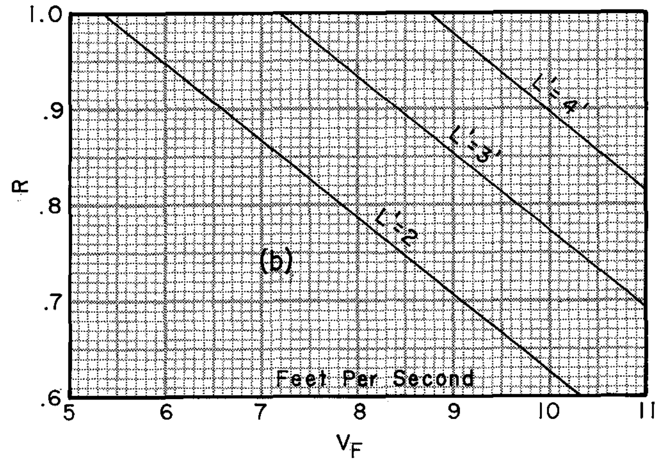


FIG. 5-26(a) & (b) Curved Vane Grate

GRATE INLET DESIGN CURVES



One foot is 0.3048m

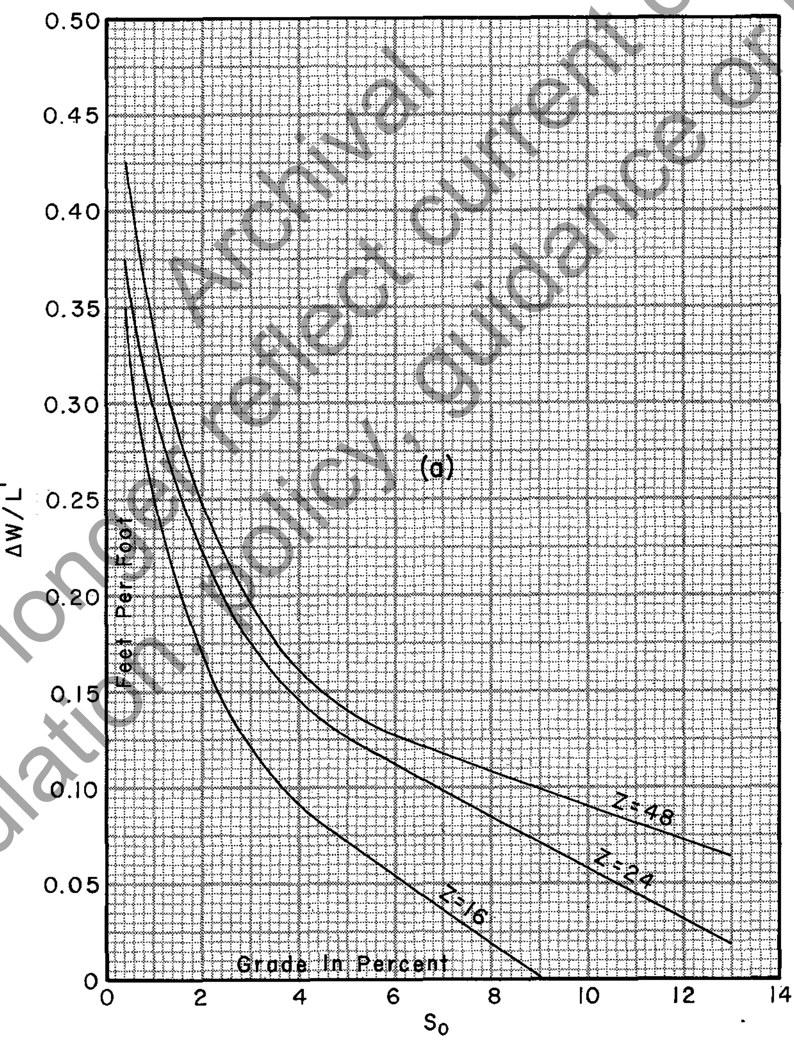
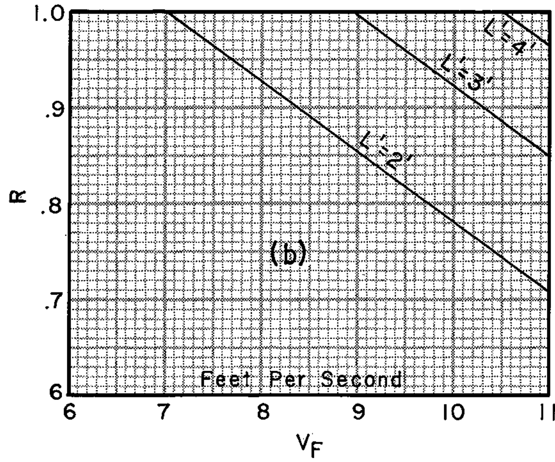


FIG. 5-27(a) & (b) P-1 7/8"-4" Grate

GRATE INLET DESIGN CURVES



One foot is 0.3048m

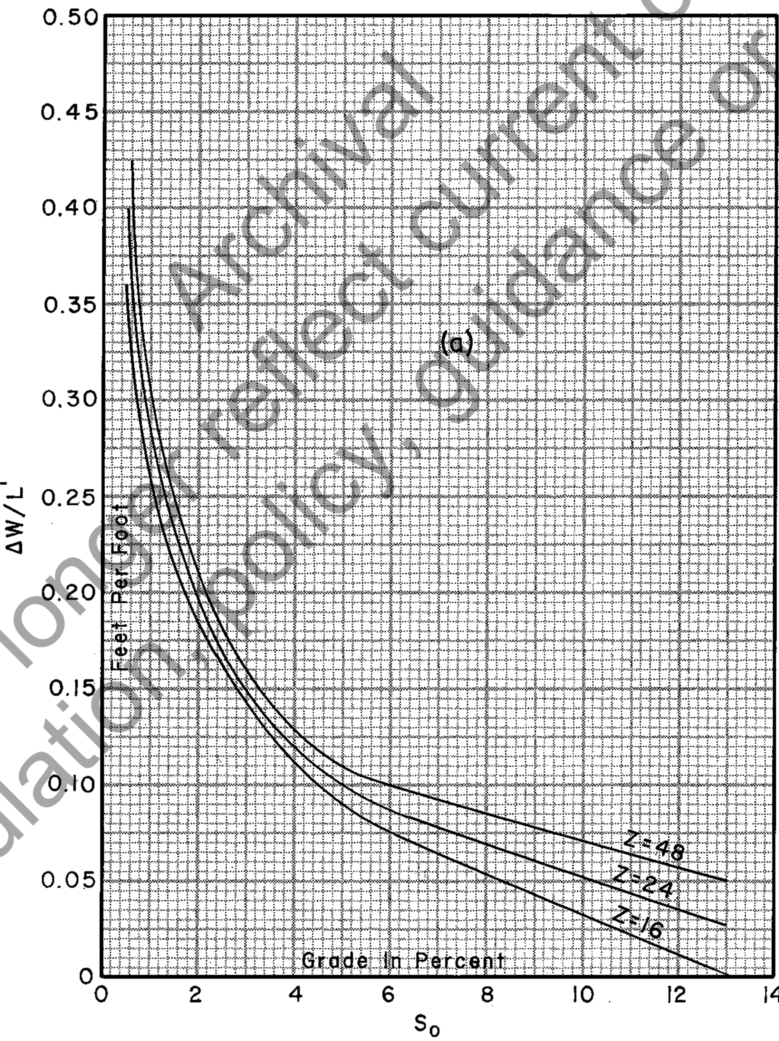


FIG. 5-28(a) & (b) P-1 1/8" Grate

GRATE INLET DESIGN CURVES

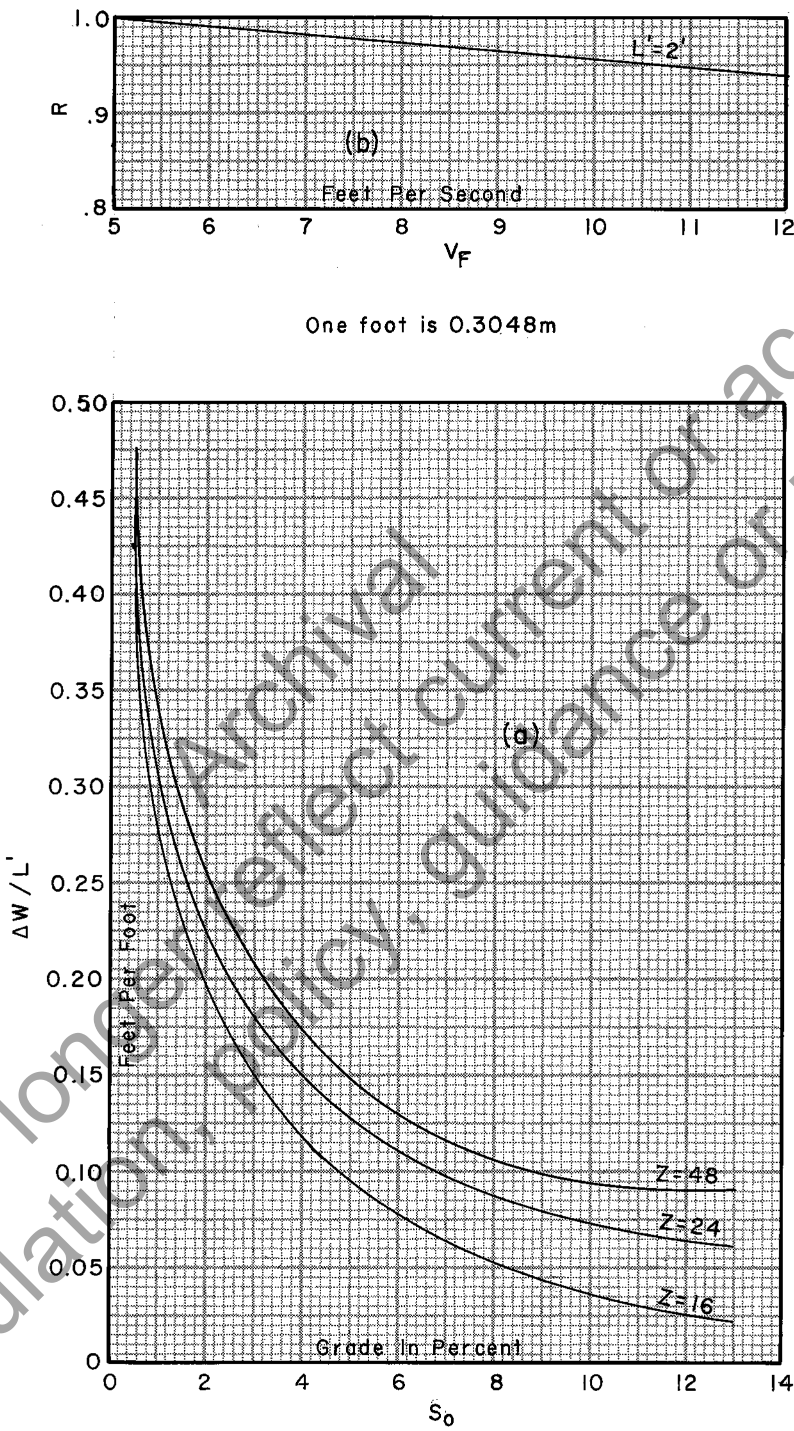


FIG. 5-29(a) & (b) P-1 7/8" Parallel Bar Grate

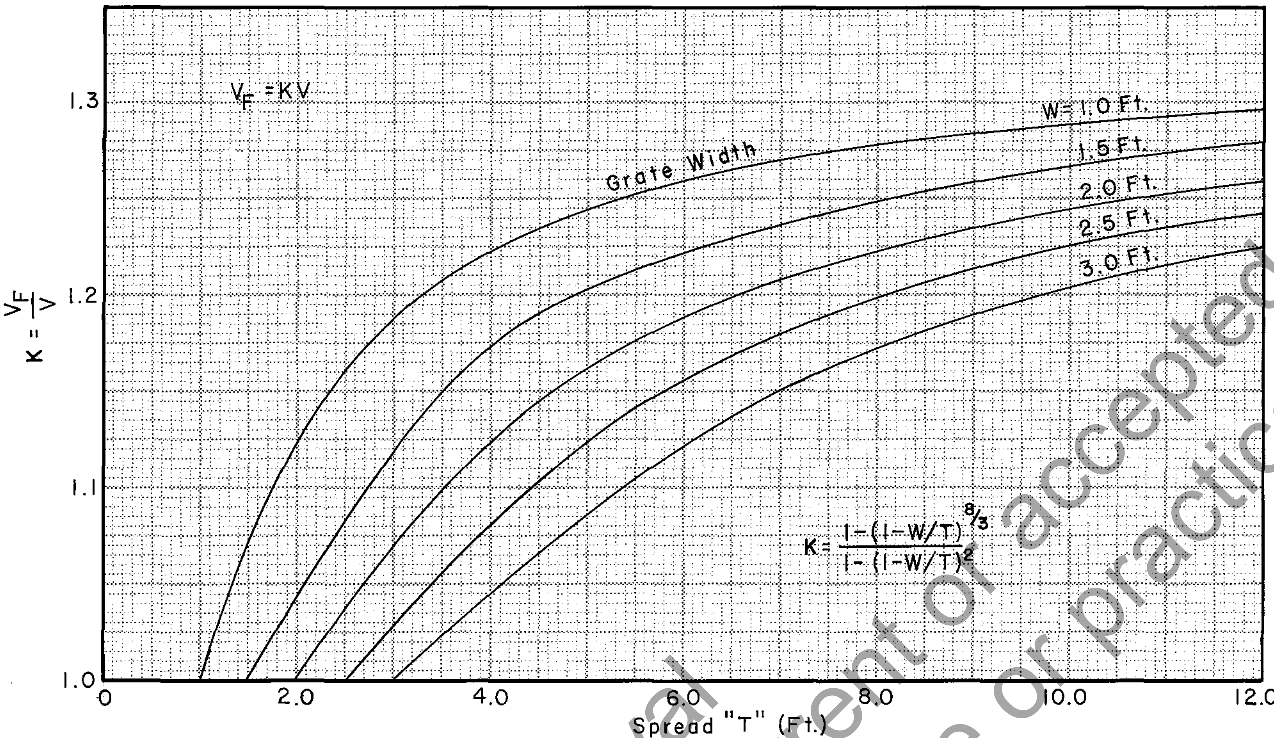


FIG. 5-30 Spread vs. K (Frontal Flow Velocity Coefficients).

One foot is 0.3048m

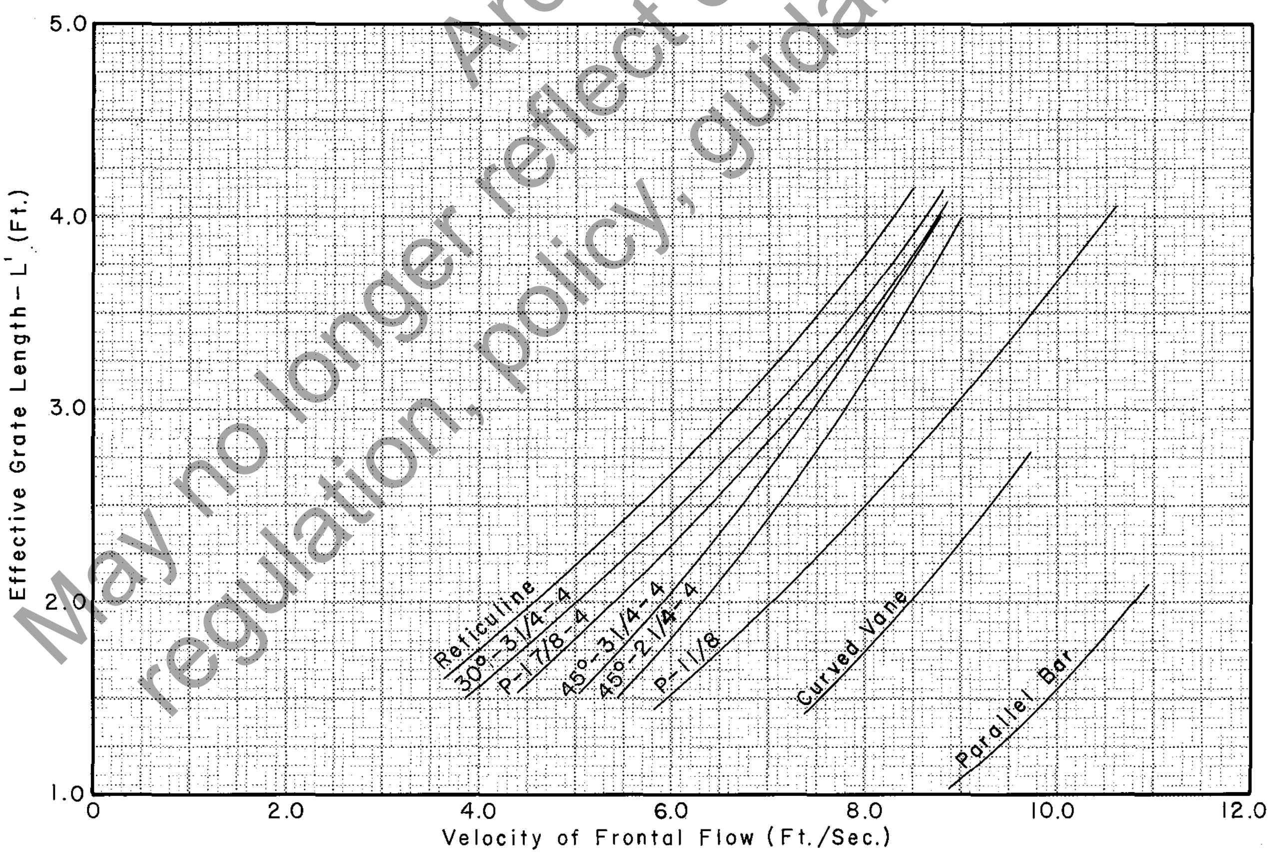


FIG. 5-31 Minimum Effective Grate Length (Without Splashover).

The extra grate length needed will depend on site conditions, grate type, frequency of maintenance, etc. It is recommended however, that the design allows a factor of safety of 1.5 or more with respect to grate length.

5.7.3 Selection of Grate Type

Grate type selection should consider such factors as hydraulic efficiency, pedestrian and bicycle safety, debris handling characteristics and fabrication costs.

Fig. 5-31 compares the relative hydraulic efficiencies of the various grate types. The parallel bar grate (P 1-7/8) is hydraulically superior to all others but is not considered bicycle-safe. The curved vane and the P 1-1/8 grates have good hydraulic characteristics with high velocity flows. The other grates tested are hydraulically effective at lower velocities.

Debris-handling capabilities as determined in the research studies are reflected in Table 5-5. As stated in the record report "The table shows a clear difference in efficiency between the grates with the 3-1/4 inch (83mm) longitudinal bar spacing and those with smaller spacings. In general, the increased flow velocity at the 4% slope results in a higher debris-handling efficiency. The efficiencies shown in the table are suitable for comparisons between the grate designs tested. Since the debris testing procedure used in the laboratory was a qualitative attempt to simulate field conditions, the individual efficiencies noted are no indication of actual field performance. However, the grates which performed best in the laboratory tests would be expected to perform best under field conditions also".

Table 5-6 also from the referenced research study (Ref. 5-10) ranks the grates according to relative bicycle and pedestrian safety. Whereas bicycle safety gratings were based on a test program, evaluation of pedestrian safety was arrived at subjectively.

Since no single grate type of those tested ranks highest in every category, some trade-offs must be made in selecting from the various grate types.

5.7.4 Example 5-6:

Given: $Q_T = 3.50$ cfs; $S_o = 4.5\%$; $Z = 24$; $n = 0.016$

24" x 48" grated inlet (30^o - 3-1/4 - 4) Fig. 5-17

Find: Intercepted flow, Q_i assuming a) no clogging and b) only 60% of the grate length is effective due to clogging.

Solution:

1. Compute T and V_F

$$T = \left[\frac{Q\ n\ Z^{5/3}}{0.56\ S_o^{\ 1/2}}\right]^{3/8} = \left[\frac{3.5(0.016)\,(24)^{5/3}}{0.56(0.045)^{1/2}}\right]^{3/8} = 5.50 \text{ ft.}$$

AVERAGE DEBRIS HANDLING EFFICIENCIES FOR TEST GRATES

Rank	Grate Style	Longitudinal .5%	Slope 4%
1	CV - 3-1/4 - 4-1/4	46	61
2	30 - 3-1/4 - 4	44	55
3	45 - 3-1/4 - 4	43	48
4	P - 1-7/8	32	32
5	P - 1-7/8 - 4	18	28
6	45 - 2-1/4 - 4	16	23
7	Reticuline	12	16
8	P - 1-1/8	9	20

TABLE 5-5

RANKING OF TEST GRATES WITH RESPECT TO BICYCLE AND PEDESTRIAN SAFETY

Rank	Grate Style
1	P - 1-7/8 - 4
2	Reticuline
3	P - 1-1/8*
4	45^{o} - 3-1/4 - 4
5	45^{o} - 2-1/4 - 4
6	CV - 3-1/4 - 4-1/4
7	30^{o} - 3-1/4 - 4

*Los Angeles County Flood Control District, "Evaluation of Three Types of Catch Basin Grates for Streets With Bicycle Traffic" Systems and Standards Group, Design Division

TABLE 5-6

$K = 1.18$ Fig. 5-30

$$V_F = \frac{2\ K\ Q\ Z}{T^2} = \frac{2(1.18)(3.50)(24)}{(5.50)^2} = 6.55 \text{ ft/sec.}$$

2. Determine Q_i without clogging:

   $L' = 4.0'$

   $\Delta W/L' = 0.07$ (Fig. 5-25a)

   $W_E = W + \Delta W = 2.0 + 0.07(4) = 2.28$ ft.

   $E_o = 0.76$ (Fig. 5-21)

   $R = 1.0$ (Fig. 5-25b)

   $E = E_oR = 0.76(1.0) = 0.76$

   $Q_i = E\ Q_T = 0.76(3.50) = 2.66$ cfs

3. Determine Q_i with 60% of the length effective:

   $L' = 4(0.60) = 2.4$ ft.

   $\Delta W/L' = 0.07$ (Fig. 5-25a)

   $W_E = W + \Delta W = 2.0 + 0.07(2.4) = 2.17$ ft.

   $E_o = 0.74$ (Fig. 5-21)

   $R = 0.95$ (Fig. 5-25b)

   $E = E_oR = (0.74)(0.95) = 0.70$

   $Q_i = E\ Q_T = 0.70(3.50) = 2.45$ cfs

Since the 4-foot grate length is about 45% longer than the minimum grate length without splash-over (see Fig. 5-31); and side flow interception is small, only a slight reduction in efficiency results from the reduced effective length.

5.7.5 <u>Example 5-7:</u>

Given: $Q_T = 4.0$ cfs; $S_o = 3.75\%$; $Z = 16$; $n = 0.016$; P - 1-1/8" grated inlet, Fig. 5-20

Find: Grate size required to intercept 70% of the gutter flow assuming 60% of the grate length is effective.

Solution:

1. Compute T and V_F

$$T = \left[\frac{Q\ n\ Z^{5/3}}{0.56\ S_o^{\ 1/2}}\right]^{3/8} = \left[\frac{4.0(0.016)(16)^{5/3}}{0.56(0.0375)^{1/2}}\right]^{3/8} = 4.64 \text{ ft.}$$

$$K = 1.15 \text{ (Fig. 5-30)}$$

$$V_F = \frac{2\ K\ Q\ Z}{T^2} = \frac{2(1.15)(4.0)(16)}{(4.64)^2} = 6.84 \text{ ft/sec.}$$

2. Determine W_E required

   $E_o = 0.70$
   $W_E = 1.70$ ft. (Fig. 5-21)

3. Determine the required grate length from Fig. 5-28b with

   $V_F = 6.84$ ft/sec.
   $L_i = 1.9$ ft. Therefore, the required grate length is

   1.9/.60 = 3.17 ft. or approximately 38".

4. Determine the grate width, W, based on L'

   From Fig. 5-28a with $S_o = 3.75\%$ and $Z = 16$, $\Delta W/L' = 0.12$;

   therefore, $\Delta W = 0.12(1.9) = 0.23'$

   $W = 1.70 - 0.23 = 1.47$ (say 18")

   A P-1-1/8 grate, 18" wide and 38" long is required.

5.8 Hydraulic Design of Conduit Systems

Closed conduits should be designed for the full condition. They may be designed to operate under pressure so long as the hydraulic gradient is below the intake lip of any inlet which may be affected. As a rule of thumb, 0.75 feet is an acceptable allowance. Provision should be made in accordance with the recommendations herein, for energy losses at bends, manholes or junctions and at transitions.

In the design and analysis of closed conduit storm drainage systems, it should be recognized that the hydraulics involve two basic types of flow depending on whether the conduits flow full or part full. At design discharges, sewer systems with full flow operate under pressure.

Some parts of some storm drain systems flow part-full even at design discharges; velocity in such instances, is usually greater than critical velocity. Segments of the system function as an open channel with rapid or shooting flow and the analysis should be made using the principles of open-channel hydraulics.

Chapter 2 of Ref. 5-11 clearly describes the principles of flow in open channels with especially good discussions of energy of flow, uniform, non-uniform and critical flow. The illustrations and discussion are quite clear concerning the non-uniform flow conditions in a conduit where subcritical (slow velocity-full) flow on a mild slope changes to supercritical (fast velocity part-full) flow on a steep slope. This set of circumstances is one of the very few under which an open channel condition can arise within a drainage system which otherwise operates with full conduits or pipes.

In comparison with the computations required for exact analysis of the hydraulics of open channels, it is relatively easy to analyze conduits or pipelines flowing full. The mean velocity for the latter is always the discharge divided by the area of the pipe. Friction slope, velocity transformations, curve losses and heads at junction chambers can be determined as discussed elsewhere herein, with more accuracy than is justified by the precision of the present knowledge of the rates of runoff. Most methods of design or analysis involve the computation of the elevation of the energy line or the hydraulic elevation (water surface). The energy line is one velocity head above the hydraulic elevation or pressure line and the only advantage one method might have over the other depends upon which elevation is most useful in design and checking. It is recommended that the most useful elevation at inlets, manholes and junctions is the actual water level or hydraulic elevation. In the few instances where knowledge of the energy gradient is desirable, it can be found by adding the velocity head to the hydraulic elevation. If the design criteria specify a maximum velocity to be used in design, most cases requiring knowledge of the energy grade can be taken care of by the amount of freeboard or hydraulic depth specified.

5.8.1 Downstream and Upstream Control

For open-channel portions of urban highway drainage systems, the designer should know that the depth in a given channel may be influenced by conditions either upstream or downstream, depending upon whether the slope is steep (supercritical) or mild (subcritical). Fig. 5-32 sketches the definitions of the hydraulic terms. Critical depth, d_c, is the depth of flow at minimum specific energy content (Fig. 5-32B) and it can readily be determined for the commonly used channel sections. It depends only on the discharge and shape of the channel and is independent of the slope or channel roughness.

Critical slope is that channel slope, for a particular channel and discharge, at which the normal depth for uniform flow will be the same as the critical depth.

Points on the left of the low point of the specific head curve (Fig. 5-32B) are for channel slopes steeper than critical (supercritical or steep slopes) and indicate relatively shallow depths and high

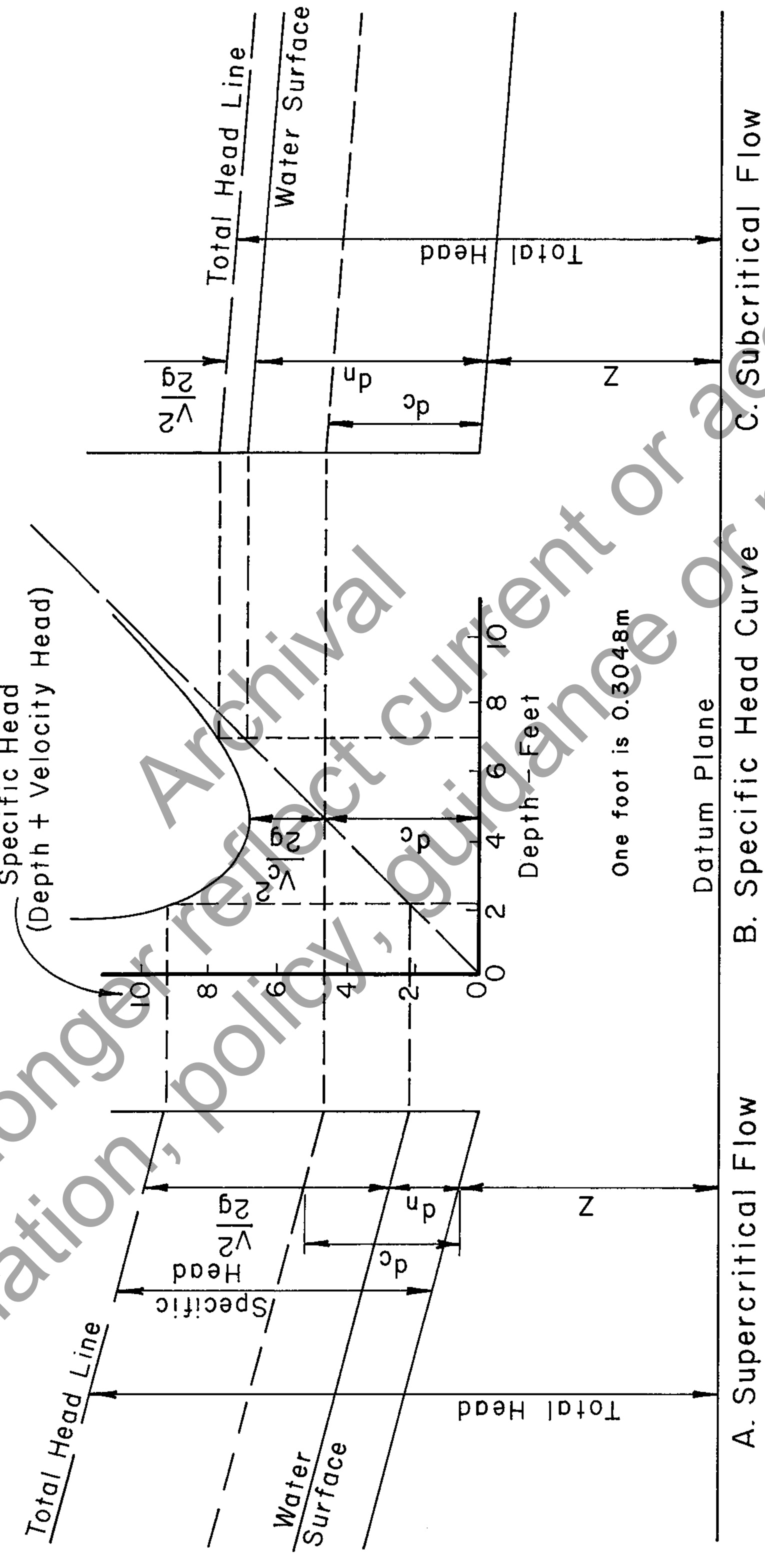


FIG. 5-32 Definition sketch of hydraulic terms. *(After FHWA HDS No. 3)*

velocities (Fig. 5-32A). Such flow is called supercritical flow and the depth of flow at any point is influenced by a control upstream, usually critical depth. A change in channel shape, slope or roughness cannot be reflected upstream except for very short distances. However, the depth of flow at downstream points may be affected. Hence the flow is said to be under upstream control.

Points on the right of the low point of the specific head (Fig. 5-32B) are for slopes flatter than critical (subcritical or mild slopes) and indicate relatively large depths with low velocities (Fig. 5-32C). Such flow is termed subcritical; the depth at any point is influenced by a downstream control which may be either critical depth or the water surface elevation in a pond or larger downstream channel.

Critical depth is an important value in hydraulic analyses because it is a control in reaches of non-uniform flow whenever the flow changes from subcritical to supercritical. Typical occurrences of critical depth are: (1) entrance to a restrictive channel, such as a culvert or flume, on a steep slope; (2) at the crest of an overflow dam or weir; (3) and at the outlet of a culvert or flume discharging with a free fall or into a relatively wide channel or a pond in which the depth is not enough to submerge critical depth in the culvert or flume. Flow that varies in depth and velocity along the channel is called non-uniform.

5.8.2 Velocity Head

Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water. Consider a stream of water flowing with a discharge of Q cubic feet per second (cfs) at a velocity of V feet per second (fps), weighing w pounds per cubic foot. Its kinetic energy (KE) per unit of time Δt, is:

$$(1/2)Q(w/g)\,\Delta t\, V^2 \qquad \text{..........(5-10)}$$

The potential energy (PE) of a flow can be expressed by:

$$PE = Qwh\,\Delta t \qquad \text{..........(5-11)}$$

where h is the height or potential in feet.

Combining these equations:

$$(1/2)Q(w/g)\,\Delta t\, V^2 = Qwh\,\Delta t \qquad \text{.........(5-12)}$$

from which

$$h = V^2/2g \qquad \text{.........(5-13)}$$

In the metric system these equations become:

$$KE = 1/2\ Q\rho\Delta t\ V^2 \qquad \text{.........(5-14)}$$

$$PE = Q \rho g \Delta t\ h \qquad \text{..........(5-15)}$$

in which ρ is the density of the water.

Velocity head is the height through which water would have to fall freely to attain the velocity V; conversely, it is the distance it would rise due to its own momentum.

5.8.3 Pressure Head

Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

5.8.4 Manning's Formula

As with all movement, the free flow of water cannot occur without friction. To move water in conduits, the force of gravity is used to overcome friction by the simple expedient of building the drain on a grade; the water then moves down the grade. The velocity at which water will travel through a sloping conduit or open channel is given with practical accuracy by Manning's formula:

$$v = (K/n) R^{2/3} S_o^{1/2} \qquad \text{.............(5-16)}$$

in which

- v = velocity in feet per second or metres per second
- K = constant of proportionality; 1.486 for English units, 1 (one) for metric units.
- n = friction coefficient depending upon material and construction of drains.
- R = hydraulic radius or area of conduit divided by wetted perimeter; for full circular pipe: R = D/4
- S_o = slope of pipe in feet per foot or metres per metre.

5.9 Guidance for Roadway Drain Pipe Location

5.9.1 Location and Alignment

Longitudinal drains for the collection and disposal of roadway drainage should not be placed under the travelled way. Whenever a location under the shoulder is necessary, manholes should be located outside the shoulder.

5.9.2 Manholes

(a) General Notes: A manhole consists of a chamber at the bottom large enough for a man to work in and a shaft which provides access directly from the surface; limited usually to the intersection of small pipes.

(b) Location: Common locations for manholes are: where two or more drains join; at intermediate points on long tangent pipe runs; where the conduit changes in size; at sharp curves or angle points in excess of 10 degrees; points where an abrupt change in grade occurs and on the smaller conduits at the downstream end of a sharp curve.

Manholes are not required if the conduit is large enough to accomodate a man, unless access or ventilation criteria govern. Manholes should not be placed within the travelled way. Exceptions are frontage roads and city streets, but intersection locations should be avoided.

(c) Spacing: In general, the larger the conduit, the greater the manhole spacing. For pipe diameters of 48 inches (1.219m) or more or other shapes of equal cross-sectional area, the manhole spacing ranges from 700 to 1200 feet (213 to 366 metres). For diameters of less than 48 inches (1.219m) the spacing may vary from 300 to 700 feet (91 to 213 m). In the case of small conduits where self-cleaning velocities (usually at least 3 fps (0.914m/s) flowing full) are unobtainable, the 300-foot (91m) spacing should be used. With self-cleaning velocities and alignment without sharp curves, the distance between manholes should be in the upper range of the above limits.

(d) Access Shaft: For drains less than 48 inches (1.219m) in diameter, the access shaft should be centered over the axis of the drain. When the drain diameter exceeds the shaft diameter, the shaft should be offset and made tangent to one side of the pipe for better location of the manhole steps. For drains 48 inches (1.219m) or more in diameter, where laterals enter from both sides of the manhole, the offset should be toward the side of the smaller lateral.

Commercial precast pipe shaft manholes are effective and more economical than cast-in-place shafts. Brick manholes may be used in reconstructing or relocating existing facilities.

(e) Arrangement of Laterals: To avoid unnecessary head losses, the flow from laterals which discharge opposite each other should converge at an angle in the direction of flow. If conservation of head is critical, a training wall should be provided.

5.9.3 Junction Structures

A junction structure is an underground chamber used to join two or more large conduits but does not necessarily provide direct access from the surface. It is designed to prevent turbulence in the flow by providing a smooth transition. This type of structure is usually needed only where the trunk drain is 42 inches (1.067m) or more in diameter. Where access is required by spacing criteria, a manhole should be used.

5.9.4 Pipe Diameter

Unless specified in the standards of the particular highway department involved, any pipe wholly or partly under a roadbed should be a minimum diameter of 18 inches (0.46m). Elsewhere, trunk laterals and inlet connections should be a minimum of 15 inches (0.38m) in diameter.

5.10 Hydraulic Losses in Storm Drainage Design

5.10.1 Losses or Pressure Changes at Storm Drain Junctions

Hydraulic grade line computations must account for all pressure changes required to convey the stormwater to the disposal locations. In addition to the principal energy involved in overcoming the friction in the full closed pipes, a not inconsiderable amount of energy is required to take care of the so-called minor losses which occur at changes in direction of flow and turbulence due to introduction of additional flows at inlets, manholes or other junction structures. Data concerning the performance of manholes and junction structures has been extremely meager in the past and designs have therefore been based on rather arbitrary procedures. In the literature, the sole extensive study has been one sponsored in the late 1950's by the Missouri State Highway Department, the Federal Bureau of Public Roads (now FHWA) and the University of Missouri Engineering Experiment Station (Ref. 5-12).

For large pipes or conduits (too large to be brought together in the usual 48-inch (1.219m) diameter manhole), hydraulic analysis of the junction requires the evaluation of pressures and momentums at various locations in the junction. Los Angeles (Ref. 5-13) has evolved a mathematical derivation which has simplified these calculations. The head loss, h_j (feet or metres) at a junction is computed as follows:

$$h_j = \Delta y + h_{v1} - h_{v2} \qquad \text{.........(5-17)}$$

with

h_{v1} = upstream velocity head, feet or metres

h_{v2} = downstream velocity head, feet or metres

Δy = change in hydraulic grade line or water surface through the junction in feet or metres.

The general formula for Δy is as follows:

$$\Delta y = \frac{Q_2V_2 - Q_1V_1 - Q_3V_3 \cos\theta}{(1/2)(A_1 + A_2)g} \qquad \text{.....(5-18)}$$

in which

Q_1, Q_2 and Q_3 are the discharges in cubic feet per second (or cubic metres per second) at the upper end, the outlet and the lateral of the junction chamber. V_1, V_2 and V_3 are the velocities in feet per second (or metres per second) respectively at the upper end, the outlet and the lateral. A_1 and A_2 are the cross-sectional areas of flow in square feet (or square metres) at the upper and lower ends of the junction chamber. And g is the gravitational constant, 32.16 feet per second per second (or 9.8024 metres/sec/sec). The angle θ is that between the axes of the outfall and the lateral.

Fig. 5-33 shows in plan and profile, sketches of junctions for a rectangular open channel, a circular open channel, both without transitions; and for the same two types of cross-section under pressure; also similar sketches incorporating transitions at each end of the junction.

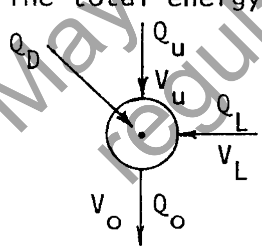
The Los Angeles analysis has been confirmed as to its accuracy by "numerous model tests conducted over a period of several years at the Experimental Hydraulic Research Laboratory of the Bureau of Engineering" of the City of Los Angeles. The general formula shows that "regardless of the shape of the conduit, the summation of all pressures acting at the junction, ignoring friction, is equal to the average cross-sectional area through the junction, multiplied by the change in the hydraulic gradient through the junction" (Ref. 5-13).

The original reference (Ref. 5-13) gives "sample problems and their solutions, illustrates the use of the general formula in determining the hydraulic changes at a junction". And it "includes (1) the derivation of the general formula for both rectangular and circular conduits under open flow and pressure flow conditions, (2) the determinations of the control points for subcritical and supercritical flow in open channels, and (3) the solution for the hydraulic grade of the lateral under pressure flow conditions".

5.10.2 Losses at Junctions of Several Flows in a Manhole

The computation of the losses in a manhole with several entering flows utilizes the principle of the conservation of energy, involving both position energy (elevation of water surface) and momentum energy (mass times velocity head). Thus, for a manhole with several entering flows, the energy content of the inflows is equal to the energy content of the outflows plus the additional energy required by the collision and turbulence of the flows passing through the junction manhole. In some circumstances, some of the entering velocity head is converted to a position head and a recovered head is noted.

The total energy at the sketched intersection is as follows:



$$Q_o(H_o+V_o^2/2g)=Q_L(H_L+V_L^2/2g)+Q_u(H_u+V_u^2/2g)-H\ Q_L+H_DQ_D \quad (5\text{-}19)$$

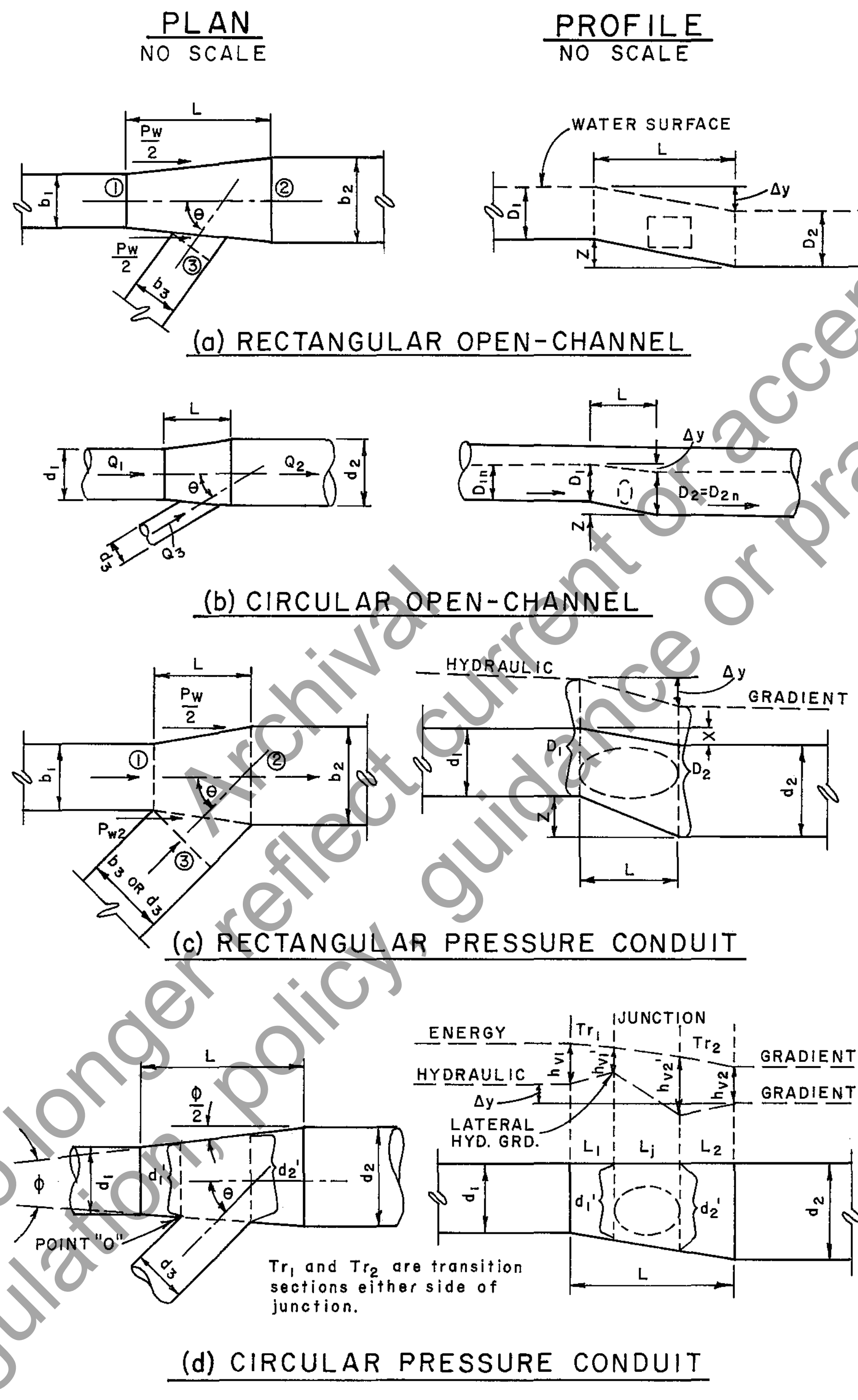


FIG. 5-33 Hydraulic analysis of junctions. *(After Los Angeles.)*

Assume no horizontal velocity for water dropping directly into manhole.
Assume water surface in manhole to be level: $H_L=H_u=H_o$
Assume $H = 0.7\ V_L^2/2g$ for 90^o change in direction.

In the foregoing equation, the symbols have the following meanings:

Q_o, Q_L, Q_u and Q_D are discharges in cfs (or metres per second) in the outlet pipe, the lateral inflow pipe (at 90^o with the outflow pipe), the in-line upstream inflow pipe and the vertical dropped-in flow from an inlet. V_o, V_L, V_u and V_D are the horizontal velocities of the foregoing flows, respectively, in feet per second (or metres per second); V_D is assumed to be zero. H_o, H_L and H_u are the water elevation at the manhole ends of the outlet, lateral and in-line flow pipes; for these computations these are all assumed to be the same. H is the loss in head chargeable to turning the lateral inflow through 90^o and imparting the requisite outlet velocity to it.

$$Q_o(H_o+V_o^2/2g)=H_L(Q_L+Q_u+Q_D)+Q_LV_L^2/2g+Q_uV_u^2/2g-0.7\ Q_LV_L^2/2g \quad .....(5\text{-}20)$$

divide through by $Q_o=Q_L+Q_u+Q_D$ .....(5-21)

$$H_o+V_o^2/2g=H_L+(Q_L/Q_o)(0.3\ V_L^2/2g)+(Q_u/Q_o)(V_u^2/2g) \quad .....(5\text{-}22)$$

$$H_L-H_o=V_o^2/2g-(Q_L/Q_o)(0.3\ V_L^2/2g)-(Q_u/Q_o)(V_u^2/2g) \quad .....(5\text{-}23)$$

which is the change in the hydraulic grade at the manhole (or loss due to the change of direction of flow, expansion and contraction, collision of flows, etc.).

In determining loss at a manhole, assume that no velocity head of an incoming line is greater than the velocity head of the outgoing line.

Also assume losses for changes in direction of less than 90^o to be as follows:

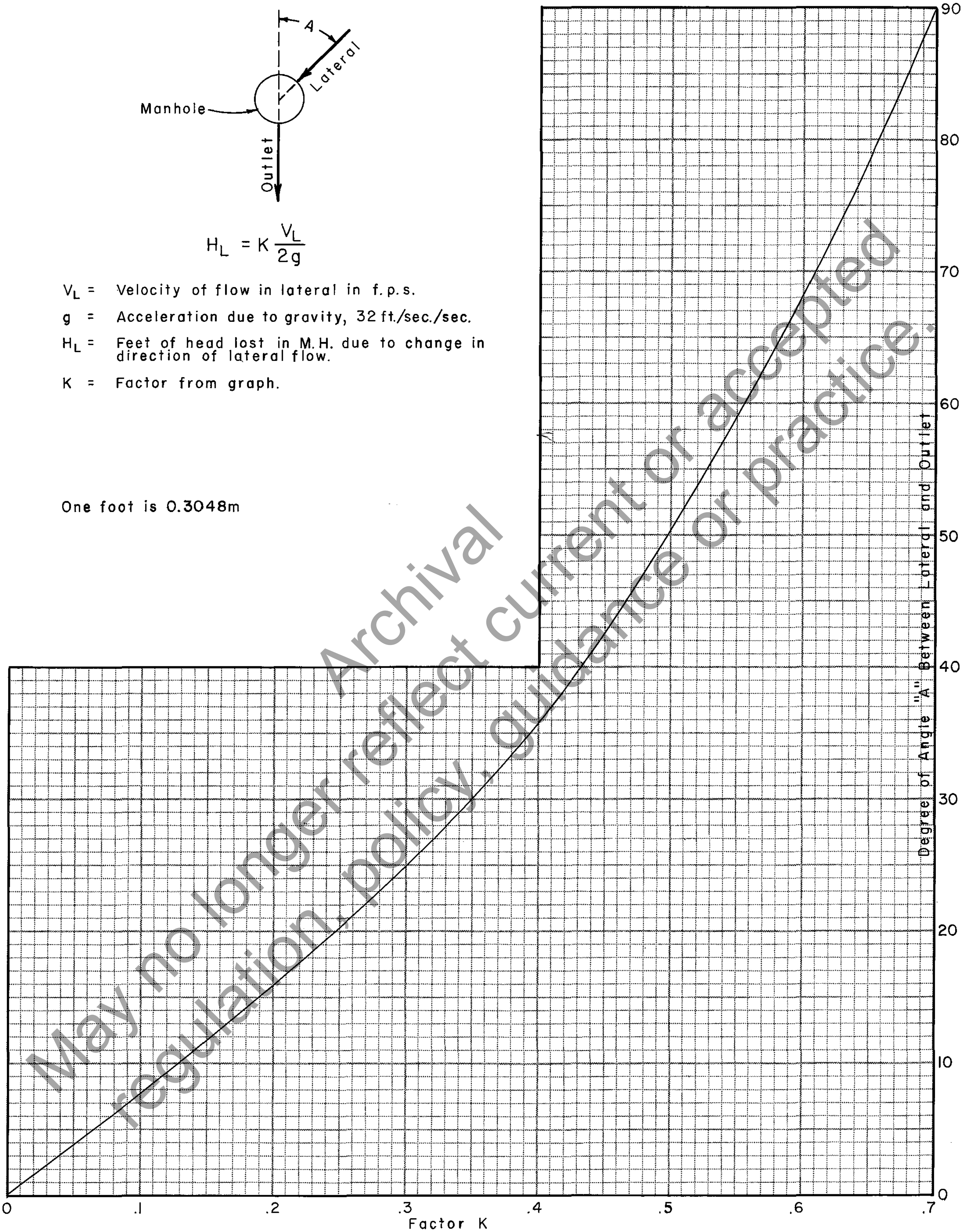
90^o $0.7\ V^2/2g$ of velocity of water being turned

45^o $2/3$ of $0.7\ V^2/2g$

30^o $1/2$ of $0.7\ V^2/2g$

Fig. 5-34 plots these values making it easy to select the appropriate coefficient for other angles.

When losses are computed for any manhole condition for the same or a lesser number of inflows, the above equation will be used with zero



$$H_L = K \frac{V_L}{2g}$$

V_L = Velocity of flow in lateral in f.p.s.

g = Acceleration due to gravity, 32 ft./sec./sec.

H_L = Feet of head lost in M.H. due to change in direction of lateral flow.

K = Factor from graph.

One foot is 0.3048m

FIG. 5-34 Loss in manhole due to change in direction of flow in lateral.

quantities used for those conditions not present.

If more directions or quantities are at the manhole, additional terms will be inserted with consideration given to the relative magnitudes of flow and the coefficient of velocity head for directions other than straight through.

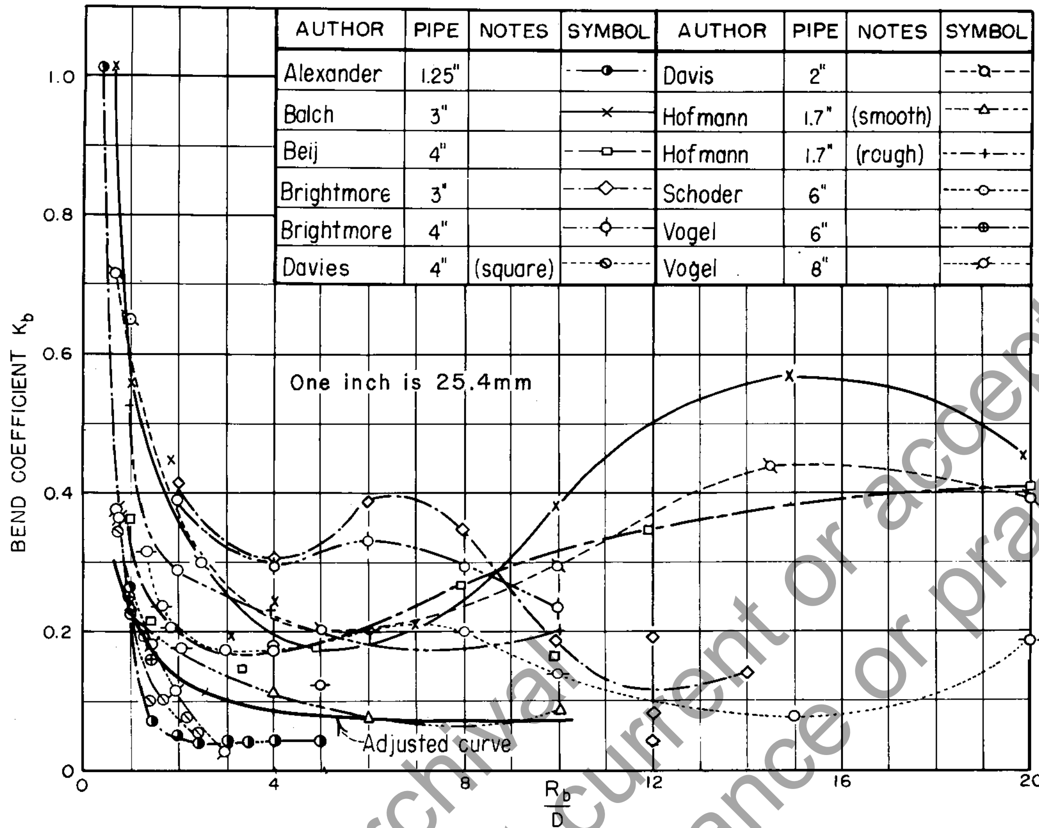
The only condition of flow to which the equation fails to apply is for two almost equal and opposing flows and no others, meeting head on with the outlet direction perpendicular to both incoming directions. In this latter case, the head loss is considered as the total velocity head of the outgoing flow.

Turn losses in a manhole or junction chamber for combining large flows can be minimized by setting flowline elevations so that pipe centerlines in the manhole will be approximately in the same plane, thereby reducing spiralling of the combined flows and partially balancing opposing moments.

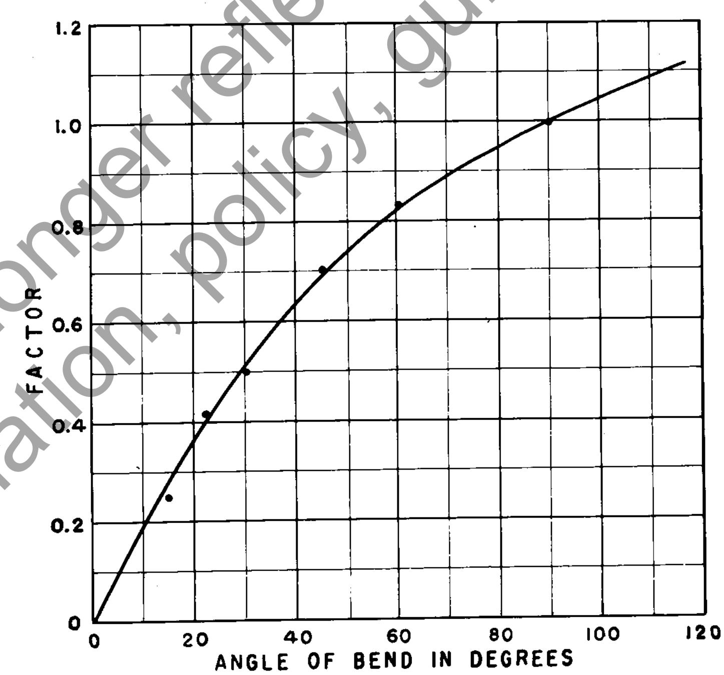
Turn losses may be minimized by reducing the angle between an inflow line and its outflow line or by so inclining two inflow lines with respect to their outflow line that, in a momentum or vector diagram of flow times velocity for each line, the direction of the resultant will be parallel to the direction of the outflow line and the longer vector will make the least possible angle with the resultant. This possibility should be examined during the preliminary location of lines and consideration given to it if possible or practical within the limits set by other governing requirements.

5.10.3 Head Loss Due to Curves

Curve loss in pipe flow is the additional head required to maintain the required flow because of curved alignment and is in addition to the friction loss of an equal length of straight alignment. Such additional head required is a function of the bend or curve radius (R_b in feet or metres), the pipe diameter (D in feet or metres) and the angle through which the bend turns. There are meager experimental data on bend losses in large pipes but the Bureau of Reclamation (Ref. 5-14) has plotted as Fig. 5-35, the coefficients found by various investigators for 90° bends of small diameter pipe for various ratios of radius of bend to diameter of pipe; there has been added an "adjusted curve" assumed to be suitable for large pipes. As part (B) of Fig. 5-35, there are suggested factors by which the 90° coefficient should be multiplied to give the corrected coefficient for an angle of bend other than 90°. The curve or bend loss is obtained by multiplying the velocity head, $V^2/2g$, of the flow in the curve by the coefficient taken from Fig. 5-35. Note that for R_b/D of about 6 or greater, the 90° coefficient is 0.07. The studies at the Bureau of Standards (Ref. 5-18) by Beij (referring to several of the same investigators reflected on Fig. 5-35) for 90° bends indicate a progressively smaller coefficient K_b as R_b/D increases; and a significant influence of the pipe roughness on the coefficient. This latter is supported on Fig. 5-35 by the two curves reporting



(A) VARIATION OF BEND COEFFICIENT WITH RELATIVE RADIUS FOR 90° BENDS OF CIRCULAR CROSS SECTION, AS MEASURED BY VARIOUS INVESTIGATORS



(B) FACTORS FOR OTHER THAN 90° BENDS

FIG. 5-35 Bend Loss Coefficients *(After Bureau of Reclamation)*

Hofmann's work; the smooth pipe values are roughly one-half those for the rough pipe. The Bureau of Reclamation's "adjusted curve" appears to represent smooth pipe for its range of R_b/D.

5.10.4 Transitions

In small sewers, transitions may be confined within the manhole. Special structures may be required for larger conduits. The head loss, h_t, at transitions for pressure flow is computed as shown on Fig. 5-36. If the top and floor slabs expand or contract at a rate different from that at which the side walls expand or contract, the head loss is based on the condition which produces the greater loss. If the rate of contraction or expansion is not symmetrical on both side walls, the head loss is based on one-half the total expansion. Where an obstruction is to be cleared and the conduit is to be transitioned and then returns to a normal section, a transition loss should be attributed to both ends, upstream and downstream. Transitions at each end of a junction chamber should include the transition head loss with the junction head loss.

For transitions involving open-channel flow, the formulas differ depending on whether the flow is subcritical or supercritical. For the former, Hind's equations (Ref. 5-15) based on his experiments are:

$$\text{Contraction:} \quad h_t = 0.10 \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \qquad \ldots\ldots\ldots\ldots (5\text{-}24)$$

$$\text{Expansion:} \quad h_t = 0.20 \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \qquad \ldots\ldots\ldots\ldots (5\text{-}25)$$

The head loss at transitions for open-channel supercritical flow is computed by equations based on Gibson's experiments on enlargers as follows:

$$\text{Contraction:} \quad h_t = \frac{0.10\,(V_1 - V_2)^2}{2g} \qquad \ldots\ldots\ldots\ldots (5\text{-}26)$$

$$\text{Expansion:} \quad h_t = \frac{0.20\,(V_2 - V_1)^2}{2g} \qquad \ldots\ldots\ldots\ldots (5\text{-}27)$$

In these equations, h_t is the transition head loss in feet or metres; V_1 is the velocity of flow in feet or metres per second in the larger cross-section; V_2 is the velocity of flow in feet or metres per second in the smaller cross-section; and g is the gravitational constant 32.16 ft/sec/sec (9.8024 m/sec/sec).

A summary of transitions is shown on Fig. 5-37.

The design of high velocity open-channel transitions is different for expansion than it is for contraction. An expansion transition is designed to retain flow against the side walls to prevent cavitation. A contraction transition is designed to minimize wave disturbances. For design of high velocity transitions the reader is referred to Ref. 5-13.

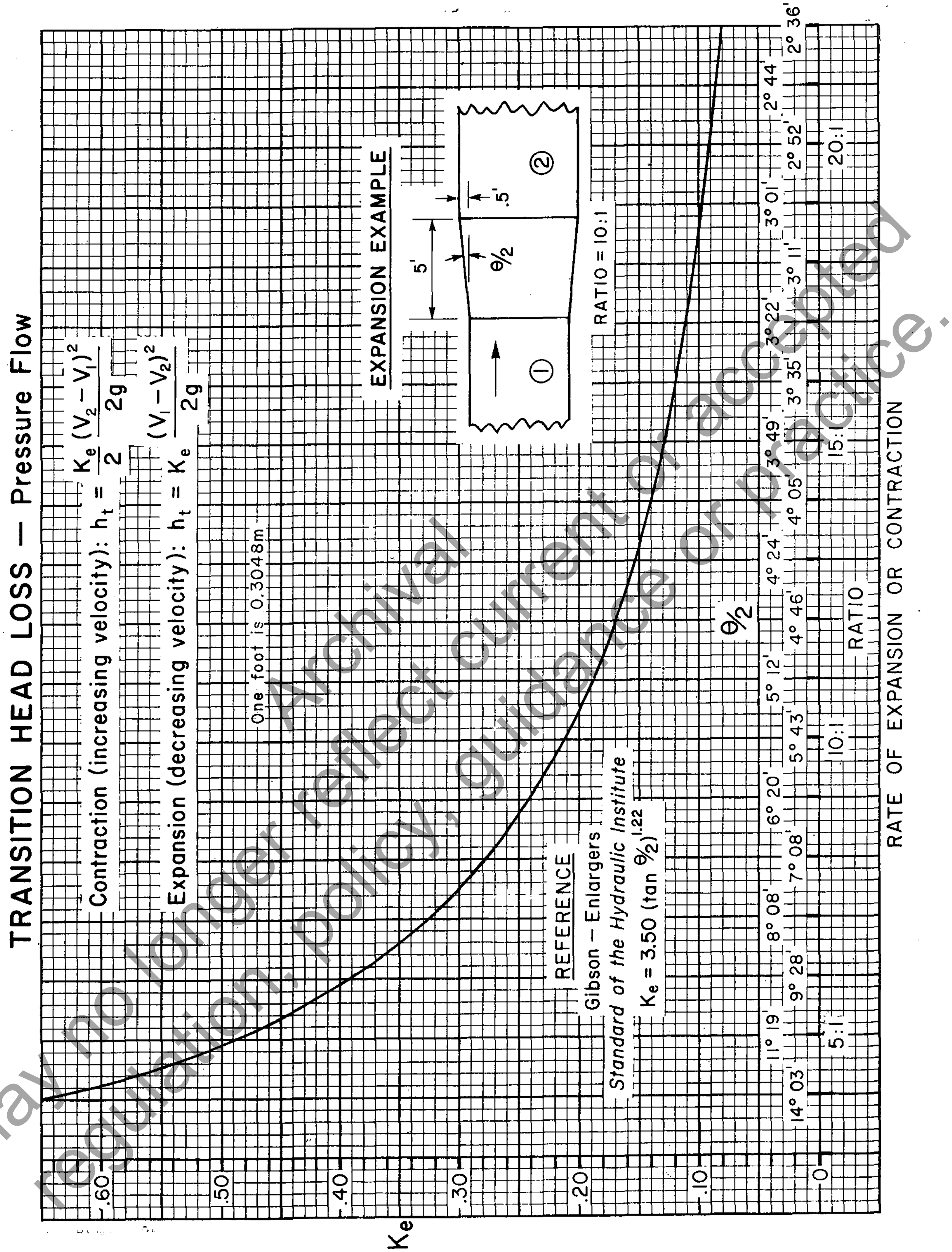


FIG. 5-36 *(From Los Angeles Standards)*

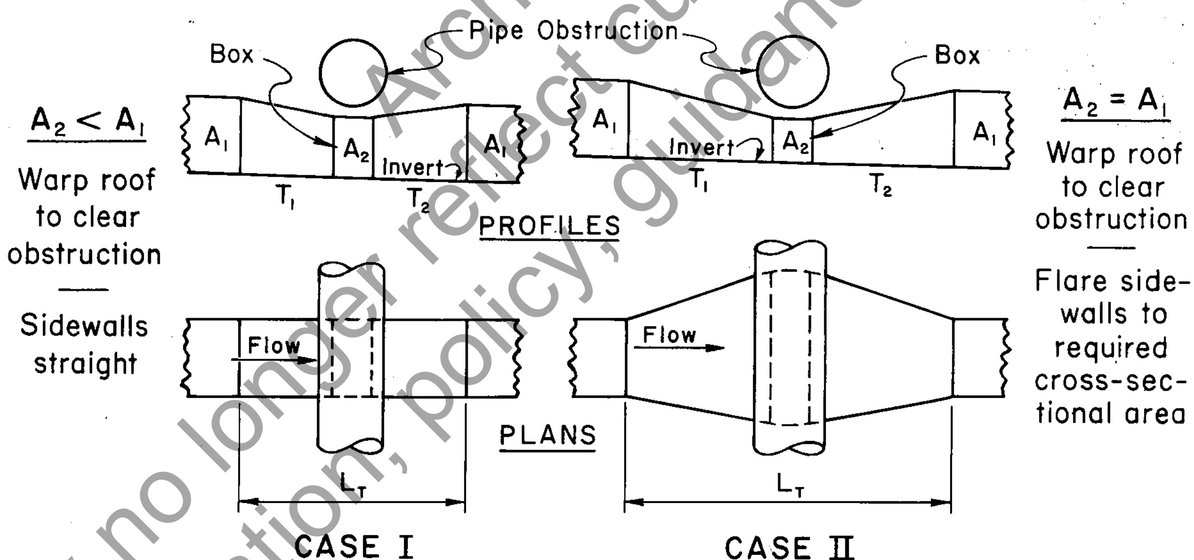
SUMMARY OF TRANSITION STRUCTURES

TYPE	FULL CLOSED CONDUIT FLOW		FREE WATER SURFACE FLOW	
	$V < 20$	$V \geq 20$	$V < 20$	$V \geq 20$
Expansion	Straight walls Ratio – 5:1 to 10:1	Straight walls Ratio – 10:1 to 20:1	Straight walls Ratio – 5:1 to 10:1	Curved walls
Contraction	Straight walls Ratio – 5:1 to 10:1	Straight walls Ratio – 10:1 to 20:1	Straight walls Ratio – 5:1 to 10:1	Straight walls
Obstruction	See Cases I and II below			

For head losses at transitions, see Figure 5-36 and Subsection 5.10.4

TRANSITIONS TO AVOID OBSTRUCTION

Subcritical Flow



NOTE:

Use Case I whenever practical. Design transition length and width to maximum allowable head loss. See Subsection 5.10.4 for head loss determination when sidewalls and top slab both expand or contract.

FIG. 5-37 *(From Los Angeles Standards)*

5.10.5 Hydraulic Grade Line Computations

The foregoing methods of calculating pressure changes or "losses" in storm drainage design are recommended and considered to be satisfactory. It is of prime importance to recognize that such losses do occur and allowances for them should be made in accordance with best engineering judgement.

The hydraulic grade line is a line coinciding with (1) the level of flowing water at any point along an open channel, or (2) the level to which water would rise in a vertical tube connected at any point along a pipe or closed conduit flowing under pressure. For a proper design, the hydraulic grade line should not rise above the limiting line determined by the required hydraulic depth below the design reference line; and, in the interest of economy, neither should it fall too far below it. Under this condition, hydraulic losses affecting the hydraulic grade line cannot be ignored but should be evaluated with as much care and judgement as possible. This requirement becomes of increasing importance as the required hydraulic depth is reduced and the hydraulic grade line is permitted to approach the ground surface, since flooding can be expected at more frequent intervals for smaller than for larger depths.

The hydraulic grade line should be computed to show its elevation at inlets, manholes and junction points of flow in pipes, conduits and open channels, and should provide for the losses and differences in elevations as required herein. Since it is based on design flow in a given size of pipe, conduit or channel, it is of importance in determining minimum sizes of pipes within narrow limits. Sizes larger than the required minimum provide extra capacity which will be available only to the extent that losses have not been disregarded.

The hydraulic grade line is affected by friction loss and velocity head transformations and losses.

Friction loss is the head required to maintain the required flow in a straight alignment against frictional resistance because of pipe or channel roughness. It is determined by the equation

$$h_f = l \times s$$

h_f = difference in surface elevation or head in feet or metres in length l

l = length in feet or metres of pipe or channel

s = hydraulic slope required for a pipe of given diameter or channel of given cross-section and for a given roughness "n" expressed as feet or metres of slope per foot or metre of length.

From Manning's formula:

$$s = \left[\frac{vn}{K\ R^{2/3}} \right]^2 \qquad ....(5\text{-}28)$$

s is the hydarulic slope and not necessarily equal to the flowline slope except under certain conditions.

K = 1.486 for English units, one (1) for metric units

R = hydraulic radius of pipe, conduit or channel

v = velocity of flow in feet per second or metres per second

n = Manning's value for coefficient of roughness

Use n = .013 for pipes of concrete or vitrified clay

n = .012 for formed monolithic concrete

n = 0.15 for concrete lining in ditch or channel inverts

n = .016 for concrete or grouted riprap lining on ditch or channel side slopes.

n = .024 for corrugated metal pipe, 2-2/3" x 1/2" corrugations

n = .027 for corrugated metal pipe, 3" x 1" corrugations

n = .031 for corrugated metal pipe, 6" x 2" corrugations

n = .025 for trimmed earth side slopes in channels with lined inverts.

n = .030 for straightened, unlined channels.

"n" will have a weighted value for partially lined channels.

5.11 Example 5-8:

5.11.1 Description

Route 340 in suburban St. Louis County, Missouri, crosses a small valley at Station 205+95 as shown on Fig. 5-38a, with -1.30% and +1.83% grades resulting in a sump at the center of a 200-foot vertical curve. As Fig. 5-38 shows, grated inlets catch the runoff in curbed gutters at Station 204+00 and at the gutter sumps at Station 205+78.1. In addition, sodded ditches intercept the runoff from the drainage area to the south of the highway and these ditch runoffs are collected by grated inlets in the ditch at 204+00 and at the low sag at 205+95. The runoff from the inlets on the south edge of the highway is then conveyed under the highway where the north side inlets are picked up and the accumulated runoff discharged into a small natural watercourse.

The drainage areas as to size and distribution between impervious and pervious are listed in the computations of Table 5-7. The traversed area is suburban in character and its zoning indicates residential apartments assumed to result in 70% imperviousness. It is found most convenient for tabular computations to list the pervious, impervious and total tributary areas separately.

DESIGN COMPUTATIONS FOR STORM DRAINAGE

PROJECT: 6-S-340-9
DISTRICT: No. 6
SUBDISTRICT: St. Louis County

STORM FREQ: 10 YR. PLACE: St. Louis Co., Mo.
MANNING "n": .013
REFERENCE PLAN: Fig. 5-13a
HYDRAULIC PROFILE: Fig. 5-13b

SHEET 1 OF 1
COMPUTED BY: SWJ DATE: 12/21/77
CHECKED BY: CBD DATE: 1/5/78

	LINE		LENGTH	STRUCT.	TRIBUTARY ACREAGE				UNIT RUNOFF CFS/ACRE		TOTAL RUNOFF			PIPE SIZE	FRICT. HEAD	V	$\frac{V^2}{2g}$	TIME IN MINUTES		CONST. SLOPE	REQD. HYDR. SLOPE	HEAD LOSS	HYDRA. ELEV.		F.L. ELEV.		UPPER STREET ELEV.	REMARKS
					PERVIOUS		IMPERV.																					
	UPP.	LOW.			ADD.	TOT.	ADD.	TOT.	PERV.	IMPV.	PERV.	IMPV.	TOTAL					t	Σ t				UPPER	LOWER	UPPER	LOWER		
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	30% (9)	70% (10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)
1	1	2	20.5	Gr. Inl.	0.78	0.78	1.81	1.81	2.16	6.84	1.68	12.38	14.06	18"	0.37	7.96	0.98	.04	5.00	.018	.0179	1.47	548.70	548.33	547.20	546.83	552.30	
2	2	3	178	Gr. Inl.	0.14	0.92	0.34	2.15	2.16	6.84	1.99	14.71	16.70	21"	1.98	6.95	0.75	.43	5.04	.013	.0111	0.50	547.83	545.85	546.08	543.76	551.44	
3	3	4	17	Gr. Inl.	0.10	1.02	0.44	2.59	2.13	6.75	2.17	17.48	19.65	21"	0.26	8.17	1.04	.03	5.47	.020	.0155	0.57	545.28	544.74	543.53	543.19	549.66	Part of trib. area is east of sump.
4	5	4	17.5	Gr. Inl.	0.95	0.95	2.21	2.21	2.16	6.84	2.05	15.11	17.16	18"	0.42	9.71	1.47	.03	5.0	.020	.0242	2.21	545.57	545.15	544.00	543.65	550.00	
5	4	6	92	M.H.		1.97		4.80	2.13	6.75	4.20	32.40	36.60	27"	1.28	9.22	1.32	.17	5.50	.007	.0140	1.83	540.89	539.61	538.00	537.36	550.18	
6	7	6	167	Gr. Inl.			0.50	0.50	2.16	6.84		3.42	3.42	15"	0.47	2.78	0.12	1.0	5.0	.009	.0028	0.18	Pt. Full	544.25	544.50	543.0	550.52	
7	8	6	11	Gr. Inl.			0.46	0.46	2.16	6.84		3.15	3.15	15"	0.03	2.56	0.10	0.07	5.0	.020	.0024	0.15	Pt. Full	545.88	544.74	544.63	548.74	Part of trib. area is east of sump.
8	6	OUTL	31	M.H.		1.97		5.76	2.10	6.62	4.12	38.13	42.24	30"	0.33	8.60	1.14	0.06	5.67	.005	.0106	0.52	535.66	535.2	533.16	533.00	550.16	
9																												
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TABLE 5-7

May no longer reflect current or accepted regulation, policy, guidance or practice.

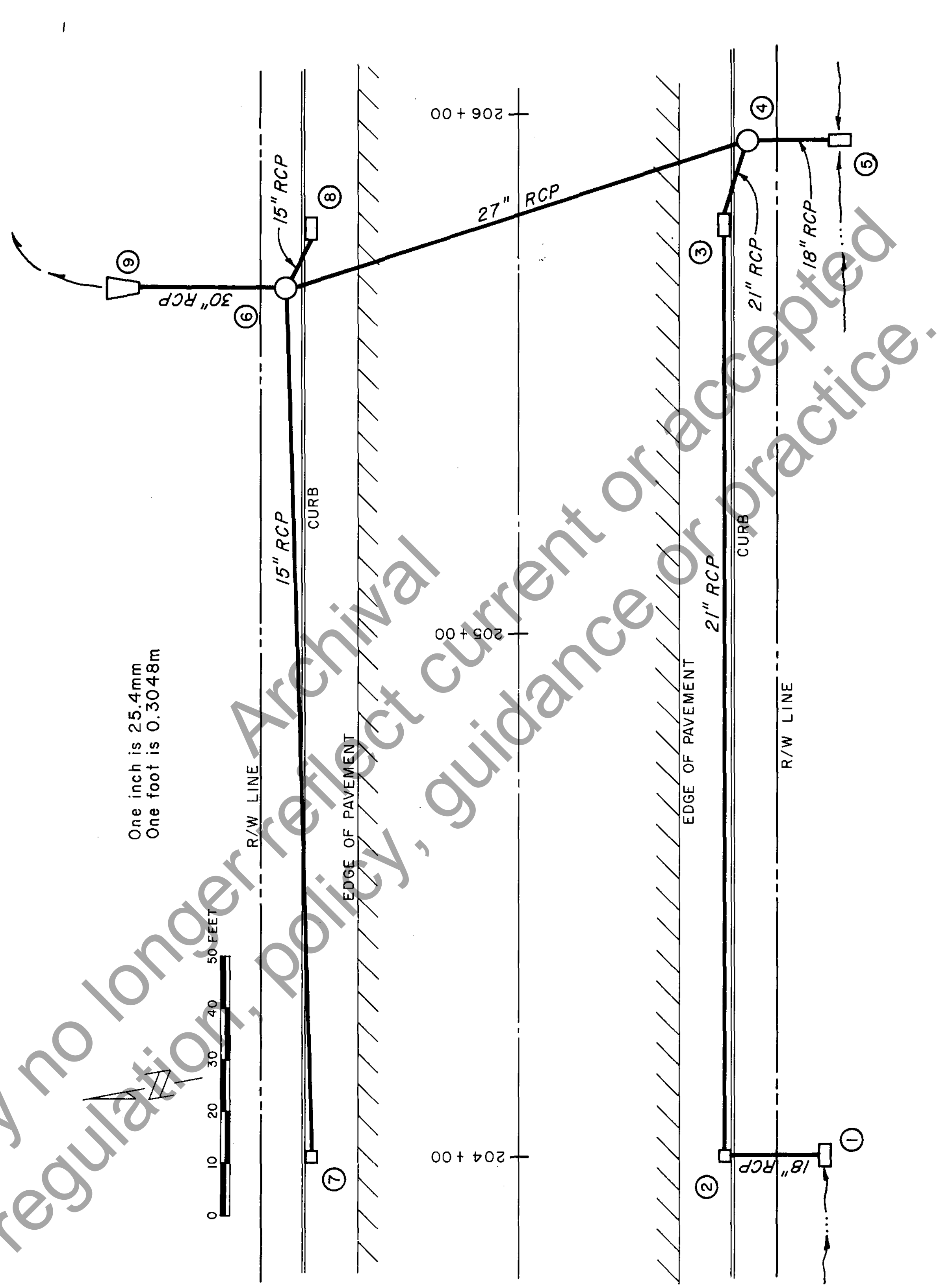


FIG. 5-38 (a) Plan of Storm Drainage System.

The rational method is to be used and the assumptions are made that the coefficients of runoff will be 0.30 for pervious areas and 0.95 for impervious areas.

The frequency of rainfall to be used in the rational formula is assumed to be that with a once in 10-year recurrence interval and an initial concentration time of 5 minutes. The rainfall duration-intensity-frequency curves for the St. Louis area are shown on Fig. 2-4; the 5-minute 10-year rainfall rate is 7.2 inches per hour (or cubic feet per second per acre).

In addition to the plan view of the design problem, profiles of the proposed collecting drains, Fig. 5-38b, are necessary. These give essential information concerning among other things, the probable practical construction slopes of the pipe reaches. In general, construction slopes almost parallel to the general surface grades will prove to be practical. Short connections can be constructed to any practical grade. Initial choice of pipe size for each reach can be guided by the available surface grades, recognizing that if the full velocity of a chosen pipe is high, the head allowances or losses in manholes, inlets or other junctions will be correspondingly high.

5.11.2 Design Computation Table

Having located the proposed storm drainage facilities and prepared profiles of the collecting system, it is next desirable to fill in a table of design computations such as shown in Table 5-7. In preparation for this table, the design points in a drainage system must be numbered in some systematic fashion. Since the rational method will be the most frequent used, it is suggested the numbering start at the uppermost inlet, manhole or junction chamber and progress sequentially downstream from design point to design point (Fig. 5-39). At each junction point the sequence picks up the branch or branches before proceeding down the main trunk drain. It has been found helpful to place the design point number in a small circle close to the design point; such encircled numbers are easily seen on plan or profile.

The table 5-7 can be filled out as follows:

Columns 1, 2 and 3: from the plan and profiles

Column 4: indicates what type of structure the upper design number refers to; i.e. manhole (MH), grated inlet(G), curb opening inlet (C), junction chamber (J.C.), etc.

Columns 5, 6, 7, 8: the acreage tributary at each design point broken down into pervious and impervious and given as "added" or "total".

Columns 9,10: the unit runoffs in cfs per acre for the pervious and impervious total areas.

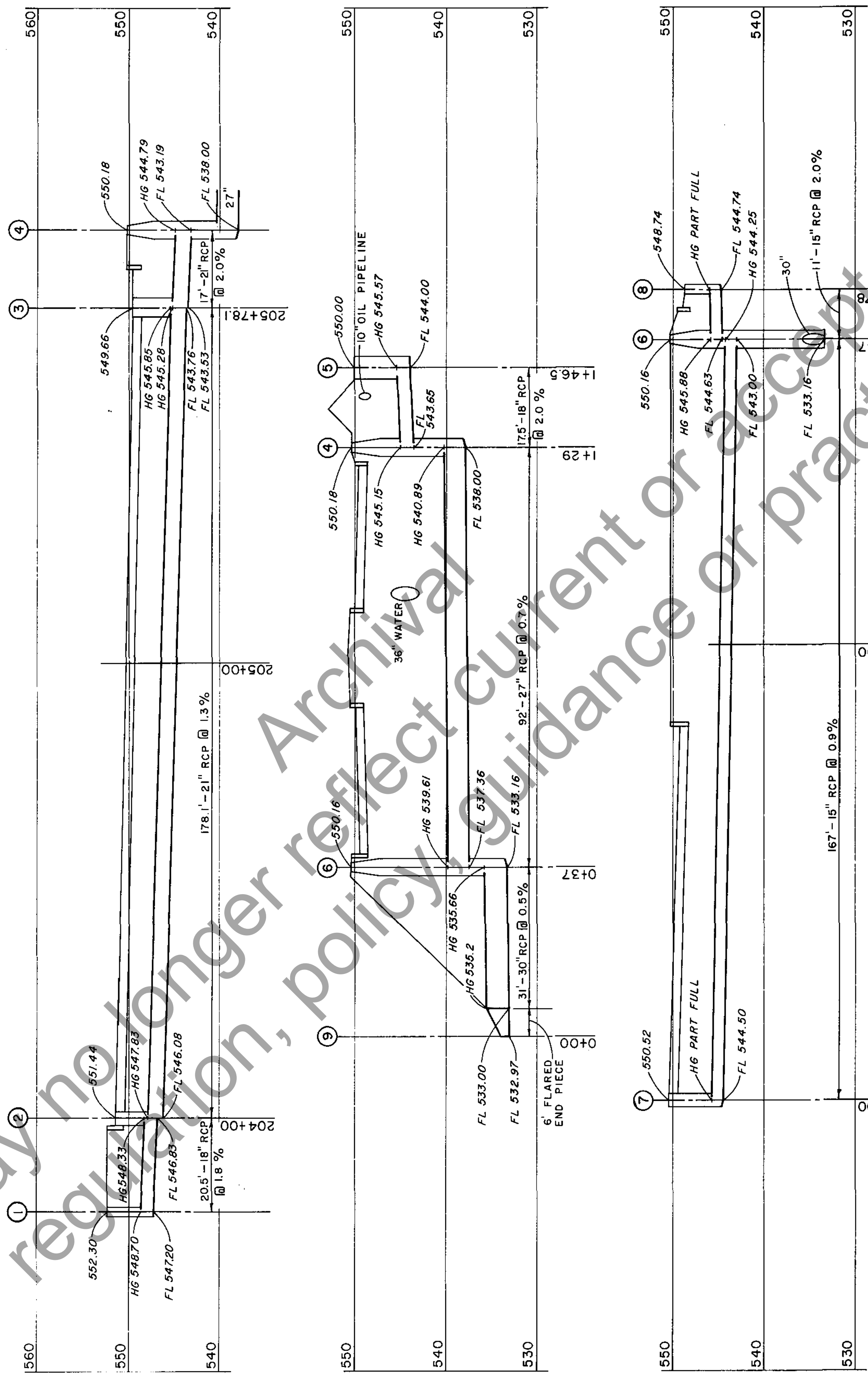


FIG.5-38(b) Profiles of Storm Drains.

SUGGESTED NUMBERING FOR
ENCLOSED (PIPE) STORM DRAINAGE SYSTEM

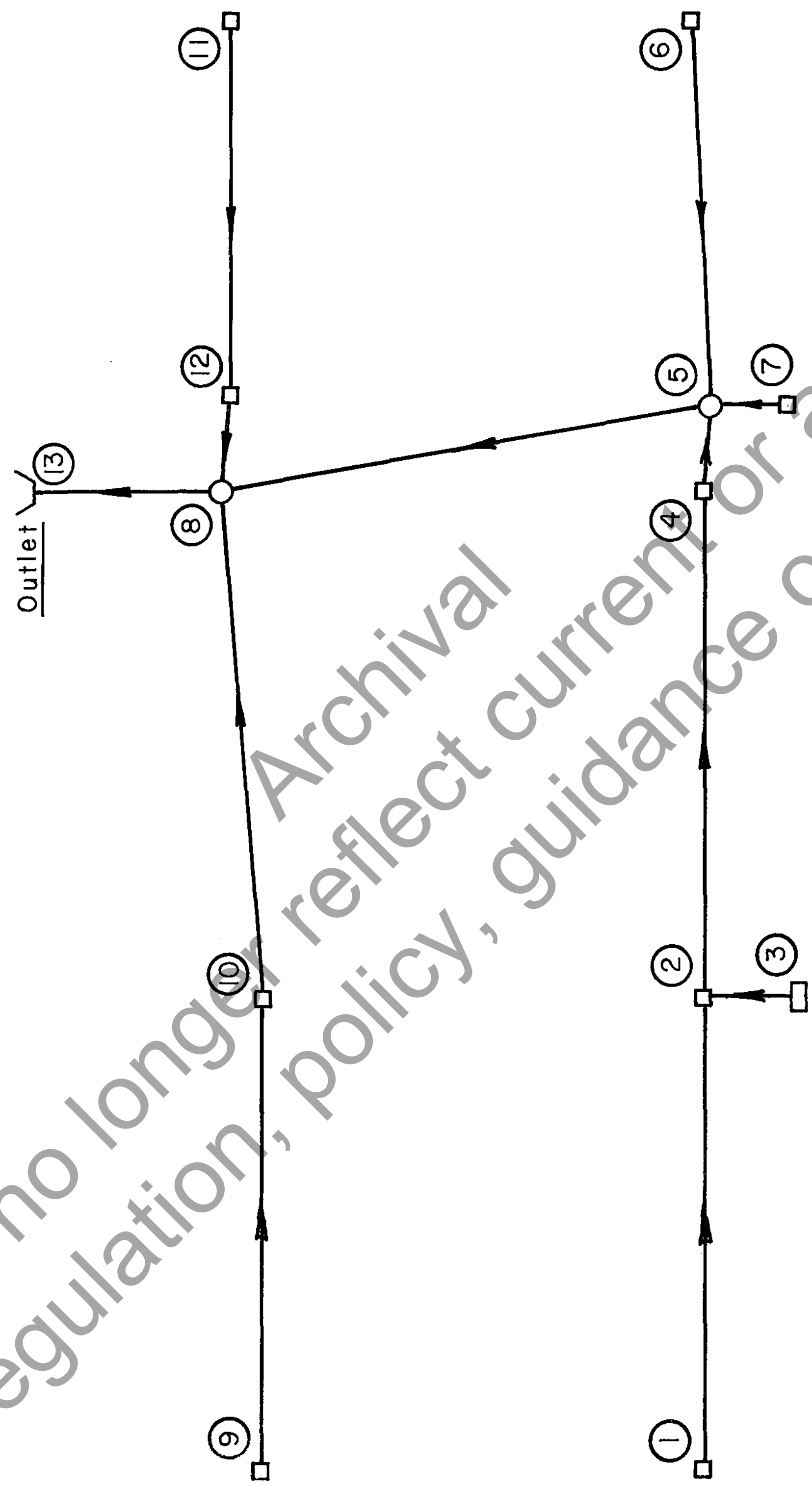


FIG. 5-39

At the top of columns 9 and 10 should be noted the percentages assumed for pervious and impervious areas respectively. The values placed in these columns represent the product of the appropriate "C" value and the rainfall rate in inches per hour (considered equivalent to cubic feet per second per acre) for the time of concentration in column 19 and the assumed design frequency.

Columns 11,12 and 13: the total runoffs as determined by multiplying the figures in columns 6 and 8 by their appropriate counterparts in columns 9 and 10. Column 13 is the sum of values in columns 11 and 12.

Column 14: insert first estimate of probable pipe size. Use existing surface grade and desirable velocity range in guiding judgement. Assume an n value of 0.013.

The principal reason for this initial estimate of pipe size is to determine probable velocity in the pipe reach and hence time of travel to develop the sequent time of concentration and hence design rainfall rate. The flatter the area, the more desirable it is to keep pipe velocities as low as practical to ensure minimum head losses in structures.

Column 15: the friction head required for the reach is computed by multiplying the length of reach (col. 3) by the required hydraulic slope (col. 21).

Columns 16, 17: using the total runoff of column 13 and the estimated pipe size of column 14, determine the probable velocity and velocity head, $V^2/2g$ and place in columns 16 and 17.

Columns 18, 19: with the length of reach in column 3 and the velocity in column 16, determine the time of travel in the reach in minutes (col. 18) and the accumulated time in minutes (col. 19).

Column 20: the construction slope will be firmly established late in the actual design procedure and will be entered in this column.

Column 21: the required hydraulic slope can be calculated from the Manning formula or obtained from a nomograph of the Manning formula.

Column 22: this column is reserved for the amount of head (or depth of water) required to take care of the energy required to transfer the flows through the manhole, inlet or junction chamber. It is calculated on separate computation sheets which will be described later.

Columns 23, 24: hydraulic elevations at the upper and lower ends of each reach are entered here and will be determined as discussed later.

Columns 25, 26: flowline or invert elevations are determined after satisfactory hydraulic grades have been established.

Generally, these construction elevations should have the crown (uppermost elevation inside the pipe) at or below the hydraulic grade line. Where the crown is above the hydraulic gradient, the pipe will not flow full and computations for part-full pipe flow will have to be made with consequent adjustments to the hydraulic grade.

Column 27: the finished grade elevation at the upper end of the reach should be entered here. This will be the top of manhole, top of inlet grate, flowline of gutter or other pertinent finished grade.

Column 28: any pertinent, very brief remarks may be entered here.

5.11.3 Curb and Gutter Hydraulics

Concomitantly with the development of the foregoing table, there should be a study of the proposed inlets to determine their type(s), capacities and the curb and gutter characteristics. The example has 6-inch high vertical curb and gutter with a cross slope of 1/4-inch per foot with a longitudinal slope from the west of 1.30%. The following details the calculations of gutter flow depth and spread toward crown for inlets 2 and 7. The nomograph, Fig. 5-1, facilitates these determinations. Fig. 5-14 is also helpful. For grate inlets in sumps, Fig. 5-40 is useful; for curb opening inlets in sumps, Figs. 5-10 through 5-13 inclusive should be used.

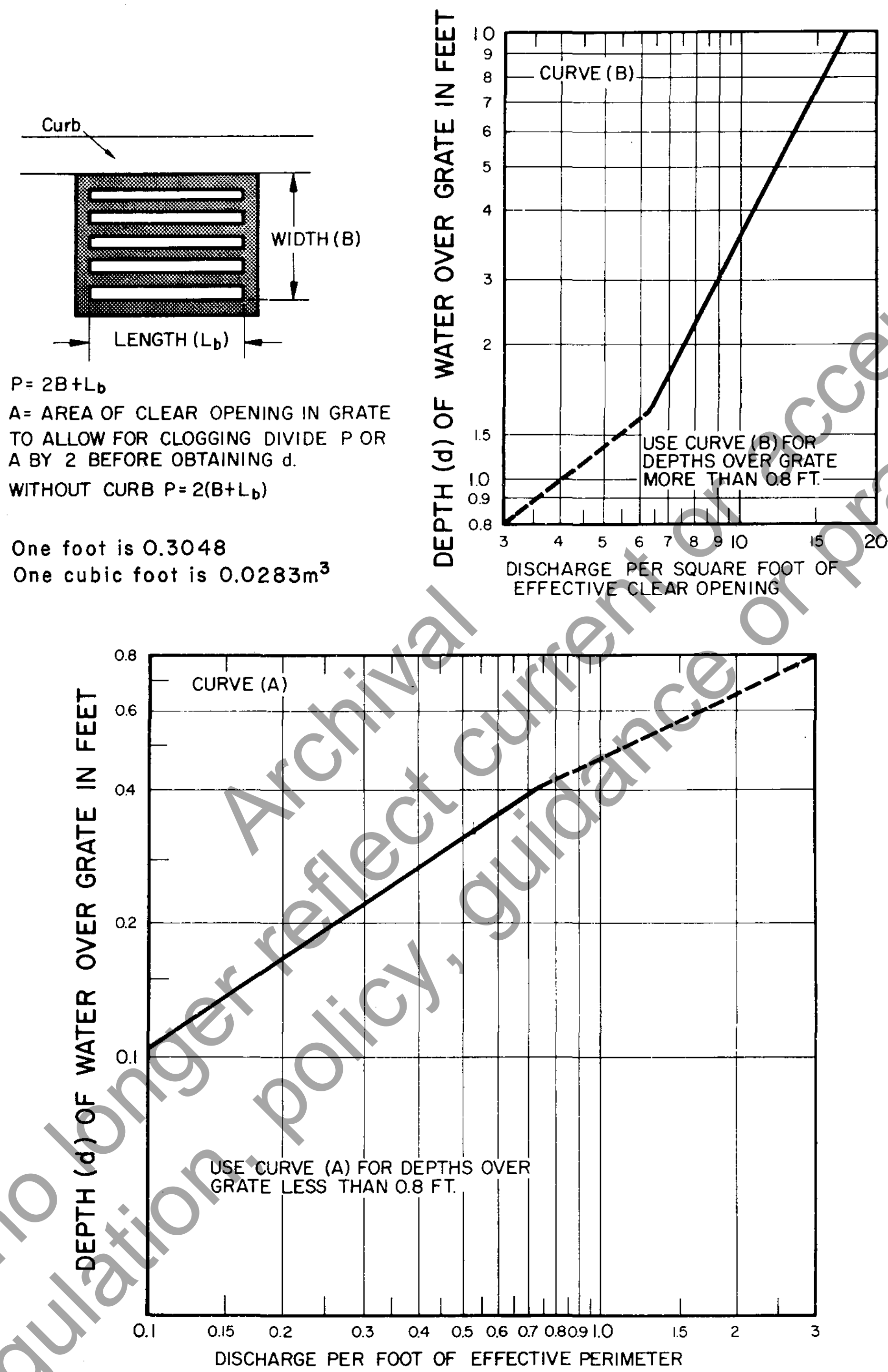
5.11.3.1 Gutter Flow Depths and Spreads

Inlet 2, Grate in Pavement at Curb

Cross slope: 1/4-inch per foot; $n = 0.016$; $Z = 48$; $Z/n = 3000$;
$S_x = 0.0208$
From table 5-7: $Q = 2.64$ cfs
Gutter slope $= S_o = 0.013$
From Fig. 5-1: $d = 0.20$ ft.; $x = 2$ ft.
Spread $= T = Zd = 48 \times 0.20 = 9.60$ ft.

Inlet 7, Grate in Pavement at Curb

Cross slope: 1/4-inch per foot; $n = 0.016$; $Z = 48$; $Z/n = 3000$;
$S_x = 0.0208$.
From table 5-7: $Q = 3.42$ cfs
Gutter slope $= S_o = 0.013$
From Fig. 5-1: $d = 0.23$ ft.
Spread $= T = 48 \times 0.23 = 11.04$ ft.



BUREAU OF PUBLIC ROADS
REV. AUG. 1968

HYDRAULIC CAPACITY OF GRATE INLET IN SUMP

FIG. 5-40 *(After HEC-12)*

5.11.3.2 Hydraulics of Grates 2 and 7

Assuming the use of one of the grates discussed in subsection 5.7, the hydraulics of the proposed grates at inlets 2 and 7 are checked as follows:

Inlet 2

Given: $Q = 2.64$ cfs; $S_x = 0.0208$; $Z = 48$, $Z/n = 3000$; $S_o = 0.013$; $n = 0.016$.

Find Q: assume 2 ft x 2 ft 45° tilt-bar grate.

From Fig. 5-1, $d = 0.20$ ft.; $T = 48 \times 0.20 = 9.60$ ft.

From Fig. 5-23a, $\Delta W = 0.62$ ft.

$$E_o \text{ (Fig. 5-21)} = 1 - \left[1 - \frac{W_E}{T}\right]^{8/3} = 1 - \left[1 - \frac{2.62}{9.60}\right]^{8/3} = 0.57$$

From Fig. 5-30, $K = 1.24$

$$V_F = \frac{2\ K\ Q\ Z}{T^2} = \frac{2(1.24)(2.64)(48)}{(9.60)^2} = 3.41 \text{ fps}$$

From Fig. 5-23b, $R = 1.0$

$E = RE_o = 0.57$

$Q_i = EQ_2 = (0.57)(2.64) = 1.50$ cfs

Q_c = carry-over = $Q - Q_i = 2.64 - 1.50 = 1.14$ cfs

Check safety factor (SF) against clogging.

$\bar{d} = (T - W/2)\ S_x = (9.60 - 1)(0.0208) = 0.179$ ft.

$$V_F = \frac{QE_o}{w\bar{d}} = \frac{(2.64)(0.57)}{(2)(0.179)} = 4.20 \text{ fps}$$

From Fig. 23b, 2 feet of the grate is used at $V_F = 6.3$ fps.

$$\left(\frac{6.3}{4.2} = 1.5\right)$$

therefore: SF 1.5 against clogging.

Inlet 7

Given: $Q = 3.42$ cfs; $S_x = 0.0208$; $Z = 48$; $Z/n = 3000$; $S_o = 0.013$; $n = 0.016$

Find Q_i: assume 2 ft x 2 ft 45° tilt-bar grate.

From Fig. 5-1, $d = 0.23$ ft.; $T = 48 \times 0.23 = 11.04$ ft.

From Fig. 5-23a, $\Delta W = 0.62$ ft.

E_o (Fig. 5-21) = 0.51

From Fig. 5-30, K = 1.26

$$V_F = \frac{2(1.26)(3.42)(48)}{(11.04)^2} = 3.39 \text{ fps}$$

From Fig. 5-23b, R = 1.0

$E = RE_o = 0.51$

$Q_i = EQ = (0.51)(3.42) = 1.74$ cfs

Q_c = carry-over = $Q - Q_i = 3.42 - 1.74 = 1.68$ cfs

Check safety factor (SF) against clogging.

$\bar{d} = (T - W/2)\ S_x = (11.04 - 1)(0.0208) = 0.209$ ft.

$$V_F = \frac{QE_o}{w\bar{d}} = \frac{(3.42)(0.51)}{(2)(0.209)} = 4.17 \text{ fps}$$

From Fig. 23b, 2 feet of the grate is used at V_F = 6.3 fps

$$\frac{6.3}{4.17} = 1.51$$

therefore: SF 1.5 against clogging.

5.11.3.3 Probable Depths at Sumps

Inlet 1 is in a pocket at the lower end of a swale. Inlets 3 and 8 are at the pavement sag. Inlet 5 is at the sag but in the intercepting ditch on the south side of the roadway. The following are the computations of the probable depths of design flow at each of these sumps.

Inlet 1, Grate in Swale Pocket at Lower End

Given: From table 5-7, Q = 14.06 cfs
Swale slope = 0.013
assume 2 ft x 4 ft grate, 1/4-inch bars at 1-7/8-inch c.c. and 3/8-inch Ø cross bars at 4 inches

Perimeter: 2 x 24.125 + 48 = 96.25 = 8.02 ft.

Q_{perim} = 14.06/8.02 = 1.75 cfs/ft.

From Fig. 5-40: d = 0.61 ft depth of water above grate for for Q_{perim} = 1.75 cfs/ft.

This is satisfactory since grate is pocketed at end of swale.

Inlet 3, Grate in Pavement at Curb (Sump)

Gutter slopes: 1.30% from W; 1.83% from E

From table 5-7: $Q = A_{perv}\ q_{perv} + A_{imp}\ q_{imp} = (0.10)(2.13)+(0.44)(6.75)$
$= 3.18$ cfs direct

Q_c from previous calculations for Inlet 2: $\underline{+1.14 \text{ cfs}}$
4.32 cfs

assume 2 ft x 4 ft grate; 1/4-inch bars at 1-7/8-inch c.c. and 3/8-inch Ø at 4-inch c.c. transverse.

Perimeter: (2)(24) + 48 = 9.6 inches = 8.0 ft.
Clear opening = 6.41 square feet

$Q_{perim} = 4.32/8.0 = 0.540$ cfs/ft.

for 1/2 perimeter, Q = 1.08 cfs/ft.

From Fig. 5-40: d = 0.50 ft. = depth of water above grate for $Q_{perim} = 1.08$ cfs/ft.

Inlet 5, Grate in Swale at Low Point (Sump)

Swale slopes 1.30% from W; 1.83% from E

Swale from W: 2 ft bottom, 2:1 side slopes, 180 feet long; sodded

Swale from E: 2 ft bottom, 2:1 side slopes, 155 feet long, sodded

Tributary area: 3.16 acres - 70% impervious.

From table 5-7: Q = 17.16 cfs

assume 2 ft x 4 ft grate; perimeter = 8.00 feet

Clear opening: 6.41 square feet

To allow for clogging divide perimeter by 2.

$Q_{perim} = 17.16/8.0 = 2.15$ cfs/ft

for 1/2 perimeter, Q = 4.29 cfs/ft.

$$\frac{Q}{1/2 \text{ clear opening}} = \frac{17.16}{6.41/2} = 5.35$$

From Fig. 5-40, curve B: d = 1.5 ft.; depth of water is 1.5 feet above grate for Q = 5.35 cfs per square foot of effective opening.

This is satisfactory since grate is pocketed at end of swale.

Inlet 8, Grate in Pavement at Curb (Sump)

Gutter slopes 1.30% from W; 1.83% from E

From table 5-7: $Q = A_{imp}\, q_{imp} = 0.46 \times 6.84 = 3.15$ cfs

Q_c from previous calculation for Inlet 7: $\underline{+1.68}$ cfs
4.83 cfs

assume 2 ft x 4 ft grate; 1/4-inch bars at 1-7/8-inch c.c. and 3/8-inch Ø at 4-inch c.c. transverse.

Perimeter: 8.00 feet

Clear opening = 6.41 square feet

$Q_{perim} = 4.83/8.00 = 0.604$ cfs/ft

for 1/2 perimeter, Q = 1.21 cfs/ft

From Fig. 5-40 D = 0.50 feet; depth of water above grate for $Q_{perim} = 1.21$ cfs/ft.

5.11.4 Hydraulic Design Procedure

Hydraulic design of a piped drainage system is a trial and error procedure. For example, the first choice of pipe sizes may require modification in one or more reaches after a trial hydraulic gradient is computed. The estimated hydraulic losses in some structure may be excessive. This most likely will involve too large a velocity head in one or more of the pipes entering or leaving a manhole or junction. An increase in pipe size may be the most satisfactory way to correct this problem. If a pipe size is increased, the time of flow in the reach (column 18 of table 5-7) will increase because of the slower velocity; and consequently, the time of concentration (column 19) and the unit runoffs (columns 9 and 10) will change. This, in turn, affects the runoffs in columns 11 and 12.

If a reach of pipe size is changed, the required hydraulic slope (column 21) is modified with a consequent change in the required friction head for the reach (column 15).

5.11.4.1 Losses in Manholes and Inlets

It becomes necessary to evaluate the hydraulic losses or pressure changes in or at each manhole or inlet. In general, this will be estimated in accordance with subsection 5.10.2. Where applicable, the designer may, at his option, estimate pressure changes using the material discussed in Ref. 5-12. There follow the detailed calculations of estimated losses as tabulated in column 22 of table 5-7.

Inlet 1

Q_o = 14.06 cfs; D_o = 18 inches; V_o = 7.96 fps, $V_o^2/2g$ = 0.985 ft.

Entire velocity to be generated: H_v = 0.98 ft.
Entrance head loss: o.5H_v 0.49 ft.
1.47 ft.

Inlet 2

Q_L = 14.06 cfs; Q_o = 16.70 cfs; Q_G = 2.64 cfs; V_L = 7.96 fps;

$V_L/2g$ = 0.985 ft.; V_o = 6.95 fps; $V_o^2/2g$ = 0.75 ft.; D_o = 21 inches

$$H_L - H_o = V_o^2/2g - Q_L/Q_o \left[0.3\ V_L^2/2g\right]$$

$H_L - H_o$ = 0.75 - 14.06/16.70(0.3 x 0.98) = 0.50 ft. loss at inlet

No velocity assumed for grate flow, Q_G, dropping into inlet.

Inlet 3 Sump

Q_u = 16.70 cfs; Q_o = 19.65 cfs; Q_G = 2.95 cfs; V_u = 6.95 fps;

V_u = 6.95 fps; V_o = 8.17 fps

$V_u^2/2g$ = 0.75 ft.; $V_o^2/2g$ = 1.043 ft.

$D_u = D_o$ = 21 inches

$H_u - H_o = V_o^2/2g - Q_u/Q_o(V_u^2/2g)$ = 1.043 - (16.70/19.65)0.75=0.41 ft.

Turn loss 17 degrees (Fig. 5-34): 0.215 x $V_u^2/2g$ = .16 ft.

Total loss at inlet 3: .16 + .41 = .57 ft.

Manhole 4

Q_u = 17.15 cfs; Q_L = 19.65 cfs; Q_o = 36.60 cfs; V_u = 9.71 fps;

V_L = 8.17 fps; V_o = 9.22 fps; $V_u^2/2g$ = 1.47 ft.; $V_L^2/2g$ = 1.04 ft.;

$V_o^2/2g$ = 1.33 ft.; $H_L - H_o = V_o^2/2g - (Q_L/Q_o)(.3\ V_L^2/2g) - (Q_u/Q_o)(V_u^2/2g)$

= 1.33 - (19.65/36.60)(.3 x 1.04)-(17.16/36.60)(1.47) = 1.33 - 0.17 -

0.69 = 0.47 ft.

Turn loss of Q_L: assume full velocity head: 1.04 ft.

Turn loss of Q_u: assume 0.22 x $V_u^2/2g$: 0.32 ft.

Total loss at MH4: 0.47 + 1.04 + 0.32 = 1.83 ft.

Inlet 5

$Q_o = Q_G = 17.15$ cfs; $V_o = V_G = 9.71$ fps

$V_o^2/2g = V_G^2/2g = 1.47$ ft.

Entire velocity to be generated: $H_v = 1.47$ ft.

Entrance head loss: $0.5H_v = 0.74$ ft.

Total head loss at inlet 5 = 2.21 ft.

Manhole 6

$Q_u = 36.60$ cfs; $Q_L = 3.42$ cfs; $Q_R = 3.15$ cfs

$Q_o = 42.24$ cfs; $D_u = 27$-inch; $D_L = 15$-inch;

$D_R = 15$-inch; $D_o = 30$-inch

$V_u = 9.21$ fps; $V_L = 2.79$ fps; $V_R = 2.57$ fps

$V_o = 8.61$ fps; $V_u^2/2g = 1.33$ ft.; $V_L^2/2g = 0.12$ ft.

$V_R^2/2g = 0.10$ ft.; $V_o^2/2g = 1.16$ ft.

$H_u - H_o = V_o^2/2g - (Q_L/Q_o)(0.3\ V_L^2/2g) - (Q_u/Q_o)(V_u^2/2g) - (Q_R/Q_o)(.3\ V_R^2/2g)$

$= 1.15 - 0.0029 - 1.15 - .0022 = 0.0$

Turn loss for $Q_u = 0.23 \times 1.33 = 0.30$ ft.

Turn losses: $V_R^2/2g + V_L^2/2g = \underline{0.22}$ ft.
Total turn loss 0.52 ft.

Inlet 7

$Q_o = Q_G = 3.42$ cfs; $V_o = 2.79$ fps

$V_o^2/2g = 0.12$ ft.

Entire velocity to be generated: $H_v = 0.12$ ft.

Entrance head loss: $0.5H_v =$ $\underline{0.06}$ ft.
Total head loss: 0.18 ft.

Inlet 8

$Q_o = Q_G = 3.15$ cfs; $V_o = 2.57$ fps; $V_o^2/2g = 0.10$ ft.

Entire velocity to be generated: $H_o = 0.10$ ft.

Entrance head loss, $0.5H_o$ $\underline{0.05}$ ft.
Total head loss 0.15 ft.

Outlet 9 H.G.

The critical depth of 42.2 cfs in a 30-inch pipe is 2.2 feet (Chart 56, HDS No. 3) and a draw-down curve brings the hydraulic gradient to the inside top of the 30-inch pipe (with a .003 construction grade; a velocity of 4.6 fps flowing full) at a distance of about 30 feet upstream from the outlet. At the critical depth of 2.2 feet the velocity of the design flow of 42.2 cfs is 9.2 fps, slightly more than the 8.5 fps flowing full. Step backwater computations indicate that the hydraulic grade elevation at the upper end of the outlet pipe will be within about 0.1 foot of full pipe. For practical design purposes, it will be assumed that the outflowing hydraulic grade at manhole 6 is at the soffit of the 30-inch pipe.

5.11.4.2 System Hydraulic Gradient

With the friction head for each reach (column 15) and the head or pressure loss in each structure (column 22), it is possible to calculate the elevations of the hydraulic gradient in the system. These computations must always start at the outlet for the system. The last paragraph of the previous subsection discusses the determination of this starting grade for the example design. If the discharge is into a pond or perennial stream, the stage for the frequency of the system design should be used if the data are available. Rarely are such data available for frequency analysis. In such instances, the storm drainage designer must exercise his best judgement, usually evaluating the outlet conditions hydraulically. This may require some field information such as alignment, cross-sections and elevations for some practical length downstream; note should be taken of any hydraulic control structures such as culverts, small bridges or other constrictions.

For the example problem, the 30-inch outlet pipe is placed at a 0.5% construction grade to ensure that the many part-full flows responsive to rainfalls more frequent than the once in 10-year recurrence interval, will travel through the pipe and exit therefrom at relatively non-erosive velocities. This flat construction gradient results in the upper end of the 30-inch pipe being 17 feet below the top of manhole. To shallow up the outlet pipe at MH 6 would save some excavation but would result in virtually all flows discharging from the 30-inch pipe at higher velocities, most of them quite erosive.

Proceeding upstream, the 27-inch pipe of reach 6 to 4 would customarily be placed to be virtually continuous with the outlet pipe. The head loss at manhole 6 has been computed as 0.52 ft. (column 23) and if the design was to continue unbroken from the hydraulic grade from downstream, the hydraulic elevation of the 27-inch pipe at manhole 6 would be 535.66 plus 0.52 or 536.18. This would place the flowline or invert of the 27-inch at 536.18 less 2.25 or 533.93. However, this would place the 27-inch pipe deeper than necessary, or it would require a steep construction grade between manholes 6 and 4 with consequent high velocities. The decision is made to place the 27-inch on a flat construction grade at a depth with adequate clearance under the 36-inch water pipe (see profiles, Fig. 5-38b). The arbitrarily chosen invert elevation of 537.36 gives the hydraulic elevation as 539.61 at the lower end of the 27-inch pipe. To this is added the friction loss of 1.28 feet (column 15) for an upper end hydraulic elevation of 540.89.

At manhole 4, 21- and 18-inch pipes bring in flows from inlets 3 and 5. Since neither of these lateral lines need be deep (to clear existing buried utilities or for hydraulic reasons), they will be connected to manhole 4 at drop-in levels above the downstream hydraulic grade elevations. The downstream hydraulic grade in each instance will be assumed to be at the critical depth at the incoming lateral pipe or at the crown of the pipe whichever is the lower.

For the 21-inch pipe of reach 4 to 3, the critical depth for a Q of 19.65 cfs is 1.6 feet (Chart 56 of FHWA HDS No. 3). A draw-down computation indicates that at the upper end of the 21-inch pipe at inlet 3, the hydraulic elevation is within a few hundredths of a foot of full. For practical design it will be assumed full. For the 18-inch pipe of reach 4 to 5, the critical depth for a Q of 17.16 cfs is over 18 inches so the hydraulic grade will be assumed at the crown of the pipe.

The beginning of reach 3 to 2 will then have an hydraulic gradient of 543.53 (F.L. of upper end of 4 to 3 reach plus 1.75 ft. plus 0.57 from column 22) or 545.85. The flowline or invert of the 21-inch from inlet 2 will be set arbitrarily at elevation 543.76 or about 3 inches higher than the outgoing flowline to assure good flow rates through the inlet. To the incoming hydraulic grade of 545.85 is then added the friction loss of 1.98 feet to result in an outgoing hydraulic grade elevation of 547.83 at inlet 2. Deducting 1.75 gives the outgoing flowline elevation of 546.08.

Reach 2 to 1 has an incoming hydraulic grade of 548.33 at inlet 2 determined by adding 0.50 foot (column 22) the loss in inlet 2 to the outgoing hydraulic elevation of 547.83. The invert elevation or flowline of the 18-inch pipe at inlet 2 is then 548.33 less 1.50 or 546.83. The hydraulic elevation at inlet 1 is the incoming hydraulic elevation of 548.33 at inlet 2 plus 0.37 foot (column 15) of friction head or 548.70. To this latter elevation should be added 1.47 feet (column 22) pressure head required to introduce the grate inflow into the pipe outlet including some actual entry head as well as generating the velocity head in pipe reach 1 to 2.

The flow from inlet 5 can be dropped into manhole 4 so an arbitrary depth of 6 feet for inlet 5 establishes its flowline as 544.00. Assuming a 2% construction slope results in the 18-inch flowline in manhole 4 as 543.65. Since critical depth for a Q of 17.2 cfs in an 18-inch pipe is virtually at full depth, it will be assumed that the hydraulic elevation of reach 5 to 4 at manhole 4 will be 543.65 plus 1.50 or 545.15. From column 15 of table 5-7, the required friction head is 0.42 foot so the design hydraulic elevation at inlet 5 is 545.15 plus 0.42 or 545.57. To this should be added 1.83 feet (column 22) to develop the velocity head in reach 5 to 4 and allow for entrance losses from the inlet box into the pipe.

The flows from inlets 7 and 8 into manhole 6 can be permitted to drop into the manhole so the entering flowlines are arbitrarily chosen as 543.00 and 544.63 with hydraulic grade elevations at the outlets of these reaches assumed at intrados. Since the construction grade of each of these reaches is greater than the required hydraulic gradient, the pipes will flow part-full, which is satisfactory.

To be checked at each inlet is whether the design hydraulic grade elevation is sufficiently below the gutter line or top of grate. A minimum freeboard of 0.75 feet is recommended as a design target. In the example problem the analysis disclosed the following:

Inlet	Maximum H.G.	Top of Grate
1	550.17	552.30
2	548.33	551.44
3*	546.42	549.66
5X	547.40	550.00
7	545.75	550.52
8*	546.00	548.74

* inlet in pavement sag
X inlet in ditch sag

The two grate inlets in the pavement sag will have very short time ponding depths above the grates of 0.50 foot for each of inlets 3 and 8; these depths assume the grates half clogged. Completely clear, unclogged grates would change these depths to 0.34 foot for each pavement grate. The very high rainfall intensity for the 5-minute 10-year rainfall makes these pondages appear worse than they are in reality, since the volumes involved are correspondingly low.

Grate inlet number 5 is in a ditch sump and will pond about 1.5 foot of depth over the grate with half the grate clogged; or about 0.72 foot depth if the grate is clean. This pondage is confined within the ditch banks.

5.12 "Major" Drainage System

As discussed very briefly in Chapter 1, there is a "major drainage system" for each urban drainage area. Whether that major system is planned or not, it comes into operation whenever the runoff from a storm is in excess of that for which the "minor" or "convenience" system was designed. The curbs, gutters, inlets, pipes, swales and channels constituting the convenience system collect and transport all the flows they can. However, under some rainfall intensities and durations, all of the runoff cannot be accomodated and the excess must find its way overland to streets and to graded swales, artificial and natural channels to a point or points of suitable disposal.

Except for the man-made changes incident to urban development, namely, the provision of streets and grading as it affects overland flow and provides swales and artificial channels, or closed storm sewers, the routing of runoff from a major storm follows the minor and major valleys of the design area. Excepting in very flat terrain, the flow paths of the natural valleys can be readily determined on topographic maps. It is of critical importance that the major storm system flow paths are such in location and hydraulic character that the accumulated excess runoff can find its way to a suitable outlet such as a major valley, lake or the ocean. Major storm flow paths should not direct flows against houses or other structures; should not fill up low areas which have no suitable outlet; should not result in scour and subsequent sedimentation; should not make it impossible for emergency vehicles to get through streets.

Wherever practicable, the swales and channels should have slow flow characteristics, be wide and shallow and natural in appearance.

Estimates of the runoff rates from a 100-year rainfall should be made for various reaches of the probable flow path. The probable hydraulic behavior of the critical reaches should be examined.

The possibility of the flooding of property, streets and highways should be examined. Can practical, economic modifications be incorporated in the major drainage system to minimize or eliminate undesirable problems?

If the topography permits consideration of alternate major drainage flow routings, they should be carefully evaluated through field checking. Social impacts on neighborhoods and general environmental design constraints should be determined. Ability of the major drainage system to serve the total tributary basin when a 100-year rain occurs, should be determined.

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