Manual on

DRAINAGE DESIGN FOR HIGHWAYS

METRIC/ENGLISH

REVISION DATE: APRIL 1998

NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION

CONTENTS

FIGURES

FIGURE		PAGE

ACKNOWLEDGMENT

In preparation of this manual, material was obtained from manuals furnished by other States, Federal Highway Administration, U. S. Geological Survey, U. S. Weather Bureau and many others. The contribution of material from all sources is gratefully acknowledged.

INTRODUCTION

THE MANUAL

The purpose of this Drainage Design Manual is to establish uniform guidelines and procedures with regard to the Department of Transportation's practices. This manual is not intended to be a legal standard for these functions. Projects shall be completed in conformance with the outlines contained in the manual. Exceptions are permissible where existing conditions indicate another acceptable solution; all exceptions should be based on the results of thorough engineering study. Changes must be approved by the Design Chief prior to execution. This manual is published solely for the information and guidance of the officers and employees of the Department of Transportation.

The scope of the manual is neither that of a textbook nor is it a substitute for engineering knowledge, experience or judgment. It includes techniques as well as graphs and tables not ordinarily found in textbooks. These are intended as aids in the solution of routine field and office problems.

This revision is necessary to provide updated guidelines and procedures and convert all numeric criteria and other values to a metric format. As more information and basic data become available through continuing hydraulic and hydrologic studies, this manual will be revised as necessary.

CONVENTIONAL SYMBOLS AND SIGNS

A,a	Area.
A_d	Drainage area.
b	Bottom width of a channel; width of constriction of bridge.
C	A coefficient; coefficient of discharge.
D	Diameter.
d	Depth of flow
d_c	Critical depth
d_n	Normal depth
E	Energy.
F_N	Froude number.
f	Friction loss coefficient.
g	Acceleration due to the force of gravity.
H,h	Head loss; head.
i	Rainfall intensity.
K,k	A constant; conveyance factor in Manning's formula.
L	Length.
L/s,L	Discharge in liters per second.
M	Momentum; bridge opening ratio.
m	Mass.
n	Manning's roughness coefficient.
p,P	Wetted perimeter; pressure
Q	Discharge
q	Discharge per unit width of channel .
R,r	Hydraulic radius; radius.
R_n	Reynolds number.
s	Slope in percent or meters per meter (feet per foot).
s_c	Critical slope.
s_f	Friction slope.
s_o	Flowline slope.
TW	Tailwater
T_c	Time of concentration.
V,v	Velocity.
V_n	Normal velocity.
V_c	Critical velocity.
Vol.	Volume.
W	Width; width of water surface.
w	Unit weight.
Z	Ratio used to determine capacity of shallow medians; a height above or below some datum elevation.
$\propto$	A coefficient applied to velocity head to account for nonuniform velocity distribution over a flow section.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	millimeters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 it shall be shown in m^3				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
			(or "metric ton")	(or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celsius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	Newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	millimeters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg	megagrams	1.103	short tons (2000 lb)	T
(or "t")	(or "metric ton")			
		TEMPERATURE (exact)		
°C	Celsius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	Newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

A. DRAINAGE DESIGN INSTRUCTIONS - GENERAL

1. Drainage structures will be designed according to the following design frequency tables:

Structure	Interstate Projects	Federal Aid Primary & Secondary and Major State Aid & Urban Highways	Minor State Aid Highways and Betterments
Bridges	Flood of record or 50-year storm whichever is greater	Flood of record or 50-year storm whichever is greater	50 years
Culverts	50 years	50 years	25 years
Storm sewers for depressed sections	Evaluate for 50-year effects	Evaluate for 50-year effects	Evaluate for 25-year effects
Curbed Roadway & Roadside Ditches	10 years	10 years	10 years
Storm sewers	10 years	10 years	10 years

2. The appropriate runoff estimation method will be selected based on the following usage listing:

Estimation Method	Usage Criteria
a) SCS - TR 55	For use on watersheds up to 810 hectares (2000 Acres) in area and to determine runoff volume from the watershed
b) Rational Method	For use on watersheds up to 80 hectares (200 Acres) in area should be limited to closed system design and small URBAN watersheds and to determine peak discharge rate
c) "Potters Small Watershed"	For use on RURAL watersheds from 0.4 to 405 hectares (1.0 to 1000 Acres). Used when detailed topographic in unavailable.
d) New England Hill & Lowland and Andirondack, White Mt. & Maine Woods Methods	For use on watersheds from 2.59 sq. kilometers to 2590 sq. kilometers (1 to 1000 sq. miles).

When estimating runoff, it is recommended that a second technique be utilized as a check to the first.

Actual runoff measurements and predicted peak flows for selected New Hampshire watersheds ≥ 7.5 square kilometers (2.9 sq. miles) are available from the Hydraulic Section. Since this is measured data every effort should be made to utilize it when estimating peak runoff.

3. Underground utilities must be considered when setting location and grade of drainage structures.

4. The outlet velocity of all drainage structures will be determined and proper precautions taken against predictable downstream erosion and sediment deposition.

5. The design of drainage structures and ditches will include an analysis of any adverse effects they may have on upstream and downstream public and private lands or facilities including but not limited to the following examples:

    a) Contamination of public and private water supplies, ponds, pools, or wells;
    b) Increased flows in existing drainage channels;
    c) New drainage outlets on public or private property;
    d) Disruption of existing public or private surface or subsurface drainage systems;
    e) Temporary or permanent ponding on public or private property; and,
    f) Erosion of or sediment deposition on public or private property.

6. Field inspections should be made to check design and to verify design assumptions.

STORM DRAINAGE DESIGN

1. Storm sewers should be designed on the assumption that each inlet intercepts all runoff that contributes to it, providing the inlet capacity is equal to or greater than the design runoff.

2. The compatibility of grate capacity, pipe capacity and design flow must be considered in closed system design.

3. The minimum grade of closed system pipes will be 0.4% or able to maintain a velocity of 0.60 m/s (2 ft/s) while flowing one-third full.

4. Catch basin outlet pipes should be at least 75 mm (3 in.) lower than the lowest inlet pipe. Increases in pipe size in a catch basin should be accomplished by keeping the crowns of the pipes equal.

5. Manholes shall be placed wherever a change in grade or alignment of a storm drain occurs but, in any case, a storm drain shall normally not have a manhole, catch basin or drop inlet more than 90 m (300 ft.) apart.

6. In general use catch basins rather than drop inlets. Call for drop inlets only where no pipe inlet occurs and where soils and other debris are not liable to wash in, on slope drainage, in embankments, and in culverts.

7. Minimum pipe sizes for closed systems under main roadways are as follows:

   a) Under pavement – 375 mm (15 in.)
   b) Parallel to roadway – 300 mm (12 in.)
   c) Slope drainage – 300 mm (12 in.)

   NOTE: PIPE SIZE, DOWNSTREAM FROM A 375 mm (15 in.) PIPE, WILL NOT BE REDUCED.

8. Minimum cover for closed system pipes will be as follows:

   a) Under pavement – 1.2 m (4 ft.)
   b) Other – 0.60 m (2 ft.)

Where the minimum cover requirements cannot be met the strength of the pipe should be checked.

9. The maximum pipe size entering and/or outletting from the standard 1.2 m (4 ft.) DI, CB, or manhole will be a 750 mm (30 in.) CMP or a 600 mm (24 in.) RCP. Larger pipes will require special size structures according to the following schedule.

Structure Diameter	Maximum Pipe Size	
	RCP	CMP
1.5 m (5 ft.)	900 mm (36 in.)	1050 mm (42 in.)
1.8 m (6 ft.)	1050 mm (42 in.)	1200 mm (48 in.)
2.1 m (7 ft.)	1200 mm (48 in.)	1350 mm (54 in.)
2.4 m (8 ft.)	1350 mm (54 in.)	1500 mm (60 in.)
Greater than 2.4 m (8 ft.)	Special Design	

10. The type of grate chosen will be based on the following usage restrictions:

Grate Std.	Where Utilized
A	On limited access roadways, ramps, and medians where bicycle traffic is not anticipated.
B	In roadways, ditches, and medians where bicycle and pedestrian traffic is anticipated. Used for low, multi-directional flow on grades $\leq$ 3%.
C	To be used in berm ditches and at locations inaccessible to vehicular traffic.
E	Where high, one-directional flow is expected and bicycle or pedestrian traffic is not anticipated.
F	Where high grate capacity is required on slopes >3% and bicycle/pedestrian traffic is anticipated.

11. Curb noses roll-overs shall have a Type A grate 9 m (30 ft.) up grade from them to prevent concentrated flows from crossing the pavement.

12. Placement of CBs in curbed roadway sags shall conform to the following:

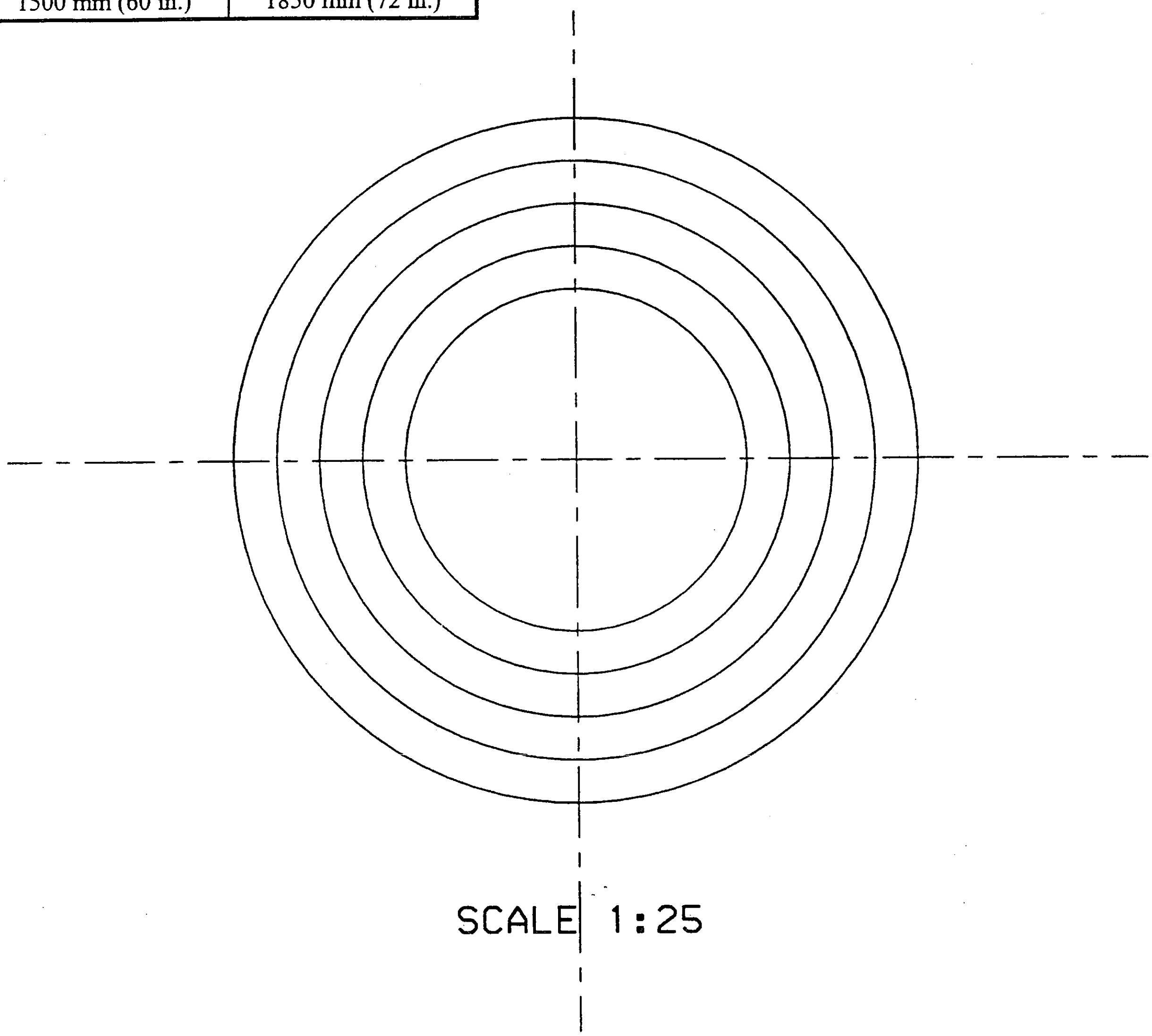
At least one CB will be located at the bottom of a sag. Depending on roadway classification and design considerations, an additional CB on either/both sides could be necessary. The spacing between the three CBs will be such as to prevent ponding of ½ of the traveled way for roadway typicals involving no shoulders. Roadway typicals with shoulders will not allow ponding encroachment into the traveled way. Basins will also be placed where snow curb is anticipated.

CBs will be placed uphill of bridges to prevent freezing on the bridge structure.

Structures with multiple inlet pipes must be checked to assure that one of two controls is met: the horizontal distance between the outside walls of any two pipes is no less than 300 mm (12 in.), measured along the inside of the structure (see figure below), or the minimum vertical clearance between the outside wall of the lower pipe and the invert of the upper pipe is no less than 300 mm (12 in.).

Inside Diameter	Total Diameter
300 mm (12 in.)	400 mm (16 in.)
375 mm (15 in.)	500 mm (19.5 in.)
450 mm (18 in.)	575 mm (23 in.)
600 mm (24 in.)	750 mm (30 in.)
750 mm (30 in.)	950 mm (37 in.)
900 mm (36 in.)	1100 mm (44 in.)
1050 mm (42 in.)	1300 mm (51 in.)
1200 mm (48 in.)	1450 mm (58 in.)
1350 mm (54 in.)	1650 mm (65 in.)
1500 mm (60 in.)	1850 mm (72 in.)

CB Size	Wall Thickness	Base Thickness
1.2 m (4 ft.)	125 mm (5 in.)	150 mm (6 in.)
1.5 m (5 ft.)	150 mm (6 in.)	175 mm (7 in.)
1.8 m (6 ft.)	175 mm (7 in.)	200 mm (8 in.)
2.1 m (7 ft.)	200 mm (8 in.)	225 mm (9 in.)
2.4 m (8 ft.)	225 mm (9 in.)	250 mm (10 in.)



CULVERT DESIGN

1. All computations for culverts will be documented and design assumptions will be stated and explained.

2. All available records concerning rainfall and floods shall be used in the design of culverts and storm sewers.

3. Pipe culverts will be designed as open flow channels. They will be designed either by inlet or outlet control. The exact control can be found by following the procedure outlined in "Hydraulic Charts for the Selection of Highway Culverts", published by U.S. Department of Transportation, Federal Highway Administration as H.E.C. No. 5 and "Hydraulic Design of Highway Culverts," as H.D.S. No. 5.

4. Minimum pipe culvert sizes under main roadways are as follows:

   a) State-Aid Highways - 375 mm (15 in.)
   b) Primary and Secondary Highways - 450 mm (18 in.)
   c) Interstate Highways - 600 mm (24 in.)

5. Pipe culverts with a span of 3.048 meters (10 ft.) or more will be considered as bridges.

6. It is preferred that culverts be aligned to fit natural channels wherever possible.

7. The minimum grade of culverts shall be 0.4% or able to maintain a velocity of 0.60 m/s (2 ft/s) while flowing one-third full.

8. Anti-Seep collars will be designed for culverts whose grade exceeds 20%; erosion control should be considered on all culverts.

9. When the computed outlet velocity is in the range beyond normal design of 3 m/s (10 ft/s) additional outlet protection will be considered for erosion control/water quality purposes.*

10. In mountainous terrain and areas of flash runoff, culvert pipe sizes will be determined by special design considerations.

11. The maximum headwater depth of flow immediately upstream from a pipe culvert shall be controlled by the following:

    a) Damage to adjacent property
    b) Damage to the culvert and the roadway
    c) Encroachment on roadway structural material

* For a more in-depth procedure on design criteria for additional outlet protection refer to the H.E.C. No. 14 Manual: "Hydraulics Design of Energy, Dissipators for Culverts and Channels," published by U.S. Department of Transportation, Federal Highway Administration.

d) Traffic interruption
e) Hazard to human life
f) Damage to stream & floodplain environment.

As a guide, the following table may be used under "normal conditions":

PIPE SIZE	MAXIMUM ALLOWABLE HEADWATER
300 mm - 750 mm (12-30 in.)	2 times pipe diameter
900 mm - 1200 mm (36-48 in.)	1 1/2 times pipe diameter
1350 mm (54 in.) - up	1 times pipe diameter

12. Minimum cover for culverts will be as follows:

    a) Under pavement - 1.2 m (4 ft.)
    b) Under drives - 0.30 m (1 ft.)
    c) Under grassed area - 0.60 m (2 ft.)

Where the minimum cover requirements cannot be met the strength of the pipe should be checked.

13. Design pipe lengths in multiples of tenth-meters.

14. Omit headers and slope paving on all culverts up to and including 450 mm (18 in.), except where there is an active stream.

15. End sections should be used instead of headers or slope paving at pipe outlets less than 1200 mm (48 in.) where field conditions permit. End sections shall not be used at pipe inlets when operating under headwater conditions.

16. When pipe material of the Contractor's option is not a requirement, reinforced concrete, corrugated aluminum or smooth bore metal pipe and polyethylene plastic pipe shall be specified in accordance with accepted engineering principles, practices, and economy.

DITCHES

1. A maximum length of 120 m (400 ft) for a ditch to a catch basin or drop inlet is desirable. Local conditions may require variations.

2. In order to keep the ditch self-cleaning, a minimum grade of 0.5% will be employed where possible.

3. All ditches will be checked for possible erosion and subsequent siltation of streams. Acceptable methods of treatment include matting for erosion control, stone for erosion control, stone fill and riprap.

4. Ditches should be at the top of cut slopes only when excessive off site runoff could damage slopes and/or overtax on-site systems. The recommendation to construct such ditches will be made by the Bureau of Materials and Research.

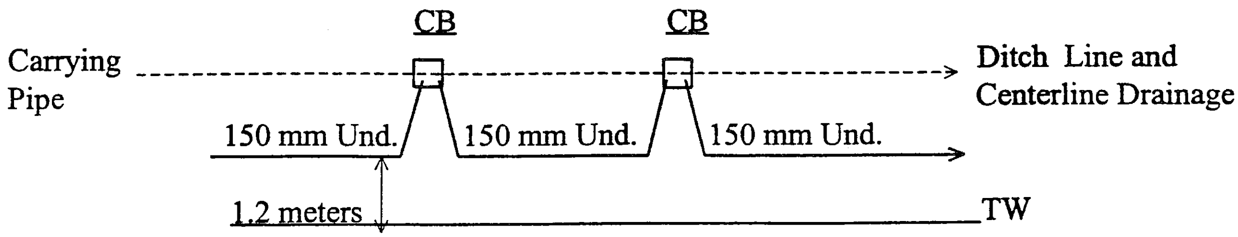
5. Ditches along aquifers and water supply areas will be lined in accordance with governing environmental agencies.

6. A typical Manning's Roughness value of n = 0.03 to 0.10 shall be used for channels and roadside ditches.

UNDERDRAIN

1. Underdrain shall be used as shown in typical sections.

EARTH OR LEDGE SECTION - UNDERDRAIN OUTLET



SLOPE DRAINAGE

1. Slope drainage will be shown on plans and cross sections.

2. Adequate outlet protection will be provided were necessary to prevent erosion.

3. Slope drainage will be collected in DIs with outlet pipes. Paved and metal sluices are considered sub-standard/outdated and are not recommended.

SECTION 2 - RUNOFF CALCULATION METHODS

The calculation of runoff is of prime importance to the overall scheme of drainage design. The Department uses four methods to compute runoff.

1. Soil Conservation Service - Urban Hydrology for Small Watersheds (SCS-TR55). This method is usually used on drainage areas up to 810 hectares (2000 acres) and where runoff volumes are needed to determine detention capacity.

2. The Rational Method - This method is usually used when the drainage area is less than 80 hectares (200 acres). It is also used extensively for on-site design.

3. The NEHL-AWM Method - This method is usually used when the drainage area is greater than 2.6 square kilometers to 2590 square kilometers (1 sq. mi. to 1000 sq. mi.).

4. The Potter Method - This method is usually used when the drainage area is between 0.4 to 405 hectares (1 acre to 1000 acres). This method should only be used when detailed topographic information is unavailable.

The methodology used in computing the runoff for the SCS-TR55, Rational Method, NEHL-AWM Method and Potter Method are enclosed in the following section.

A. SCS TR-55 METHODS

INTRODUCTION

[Note: The following is an outline of the SCS TR-55 Methodology procedure, for more in-depth explanation and description, please refer to the Soil Conservation Service: Technical Release 55 - Urban Hydrology for Small Watersheds (June 1986) Manual.]

TR-55 presents simplified procedures to calculate storm runoff volume, peak rate of discharge, partial hydrographs and storage volumes for water control structures. The procedures are applicable to small watersheds, especially urbanizing watersheds with time of concentration between 0.1 hours and 10.0 hours. TR-55 is an approximation of the more detailed TR-20 method and does not have the capability to flood route. The user should examine the sensitivity of the analysis being conducted to ensure that the degree of error is tolerable. TR-55 contains two methods, the Tabular Hydrograph method and the Graphical Peak Discharge method. The accuracy of both methods is comparable; they differ only in their output. Both methods are based on open and unconfined flow over land and in channels.

The TR-55 Tabular Method can develop partial composite flood hydrography at any point in a watershed by dividing the watershed into homogeneous subareas. By doing this, the method can estimate runoff from a larger nonhomogeneous watershed. The method is especially applicable for estimating the effects of land use change in a portion of a watershed. It can also be used to estimate the effects of proposed structures. The TR-55 Graphical Peak Discharge method calculates peak discharge using an assumed unit hydrograph and a thorough, but rapid, evaluation of the soils, slope, and surface cover characteristics of the watershed. This method is recommended for use in the design of all erosion and sediment control measures and simple stormwater management practices. When more detail and accuracy are required or when an accurate simulation of natural conditions is required, one of the other appropriate methods should be used. The TR-55 Graphical Peak Discharge method is the method that is discussed in this manual.

The peak discharge equation used in this method is:

$$q_p = q_u A_m Q F_p$$

where:

q_p is the peak discharge in cubic feet per second (cfs) [NOTE: 1.0 cfs = 0.028 m^3/s].
q_u is the unit peak discharge in cubic feet per second per square mile per inch of runoff (csm/in).
A_m is the drainage area in square miles.
Q is the runoff from the watershed in inches.
F_p is a pond and swamp adjustment factor that can be applied for ponds or swamps that are spread throughout the watershed and not in the time of concentration flow path.

ESTIMATING RUNOFF FACTORS

The major factors that must be taken into consideration for estimating runoff from a given watershed include soils, rainfall, and the land use characteristics.

Soils

Infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates. Soils are divided into four hydrologic soil groups: A, B, C, and D, according to their minimum infiltration rate:

Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission; greater than 7.6 mm/hr (0.3 in/hr). Soil textures in this group include: sands, loamy sands, or sandy loams.

Group B soils have moderate infiltration rates and consist chiefly of moderately well to well drained soils with moderately fine to moderately coarse textures. The soils have a moderate rate of water transmission; 3.8 - 7.6 mm/hr (0.15 - 0.30 in/hr). Soil textures in this group include: silt loams and loams.

Group C soils have low infiltration rates and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission; 1.3 - 3.8 mm/hr (0.05 - 0.15 in/hr). Soil textures in this group are the sandy clay loams.

Group D soils have high runoff potential. They have very low infiltration rates and consist 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 low rate of water transmission; 0 - 1.3 mm/hr (0.05 in/hr). Soil textures in this group include: clay loams, silty clay looms, sandy clays, silty clays and clays.

Hydrologic soil groups for various soils are identified in soil surveys published by the Soil Conservation Service which are available at the local County Conservation District office. Two lists, "Hydrologic Soil Groups for Determining Runoff in New Hampshire" by group type and by soil series, are located at the end of this section.

RAINFALL/NEW HAMPSHIRE RAINFALL MAPS

The highest peak discharges from small watersheds are usually caused by intense brief rainfalls that may occur as distinct events or a part of a longer storm. The intensity of the rainfall varies considerably during a storm as well as over geographic regions. SCS developed four synthetic 24-hour rainfall distributions for TR-55.

Figures A-1 through A-6 provide rainfall maps for New Hampshire for the 2-, 5-, 10-, 25-, 50-, and 100-year frequency 24-hour duration storm events and also indicates the rainfall distribution boundary found in New Hampshire for a Type II and Type III storms. Type II is the most intense short duration rainfall and Type III represents the area of New Hampshire where tropical storms bring large 24-hour rainfall amounts.

Rainfall Conversion

1 inch = 25.4 mm

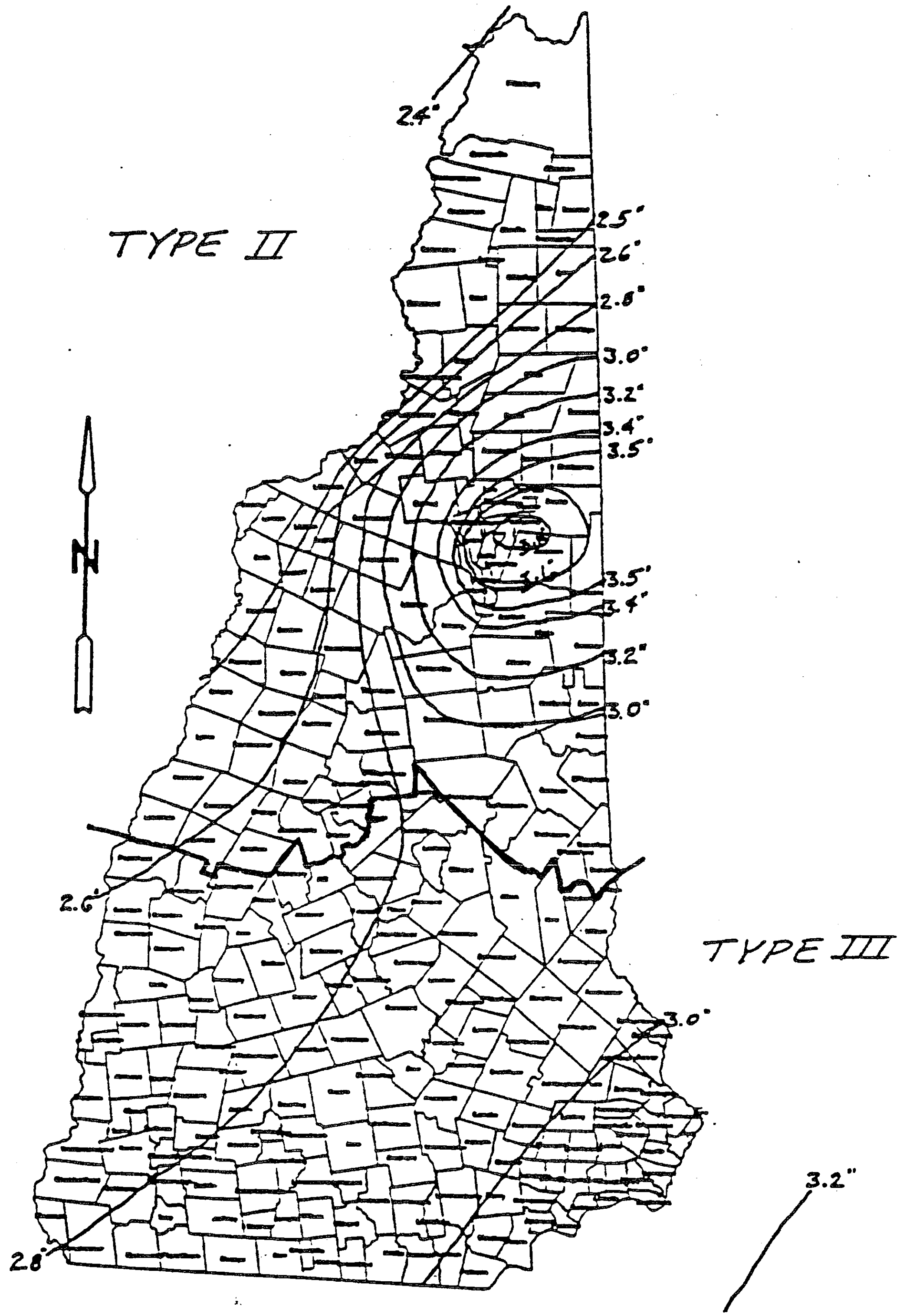


Figure A-1 -- 2-YEAR FREQUENCY 24-HOUR DURATION RAINFALL

Source: USDA Soil Conservation Service

Rainfall Conversion

1 inch = 25.4 mm

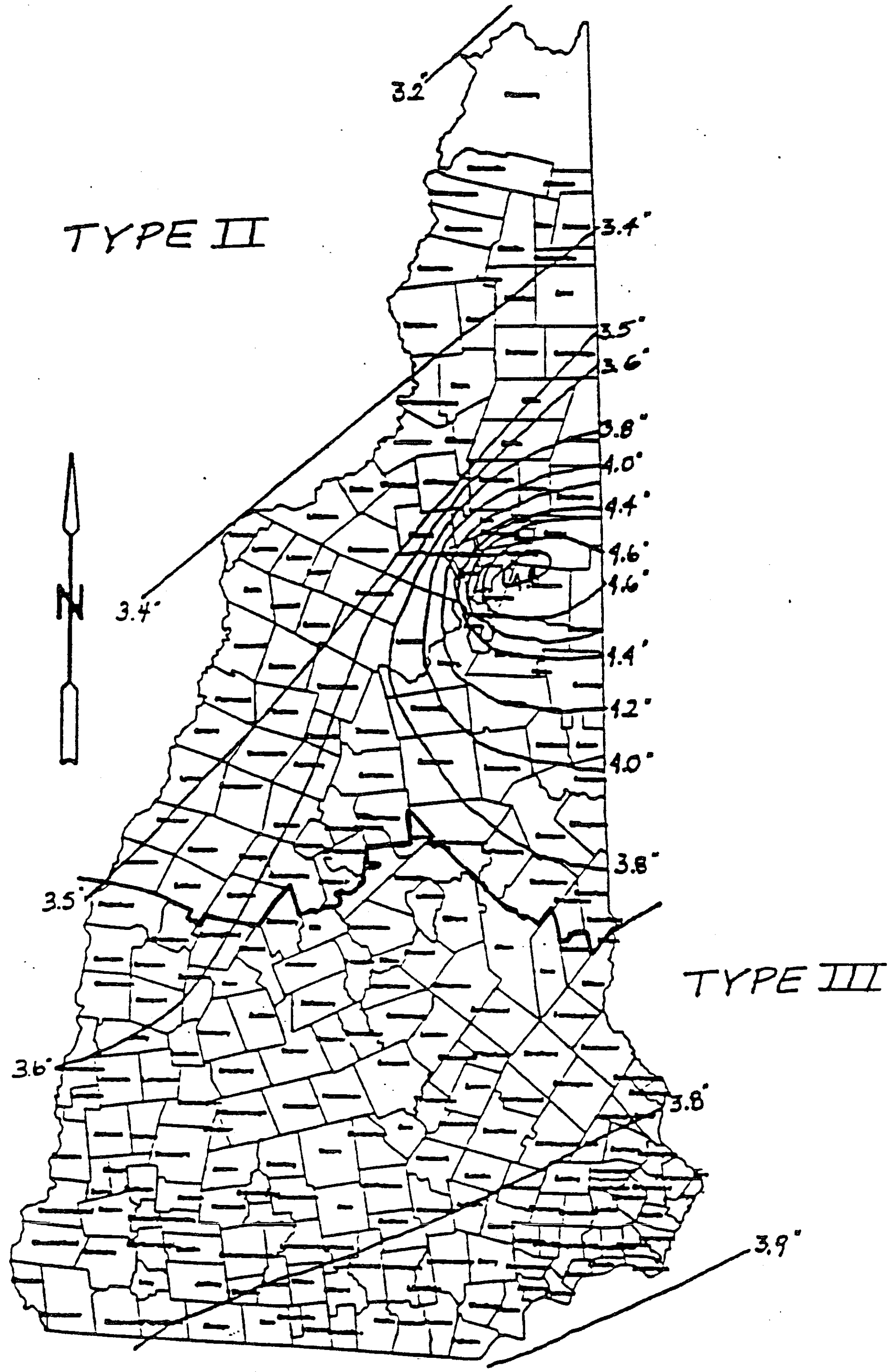


FIGURE A-2 -- 5-YEAR FREQUENCY 24-HOUR DURATION RAINFALL

Source: USDA Soil Conservation Service

Rainfall Conversion

1 inch = 25.4 mm

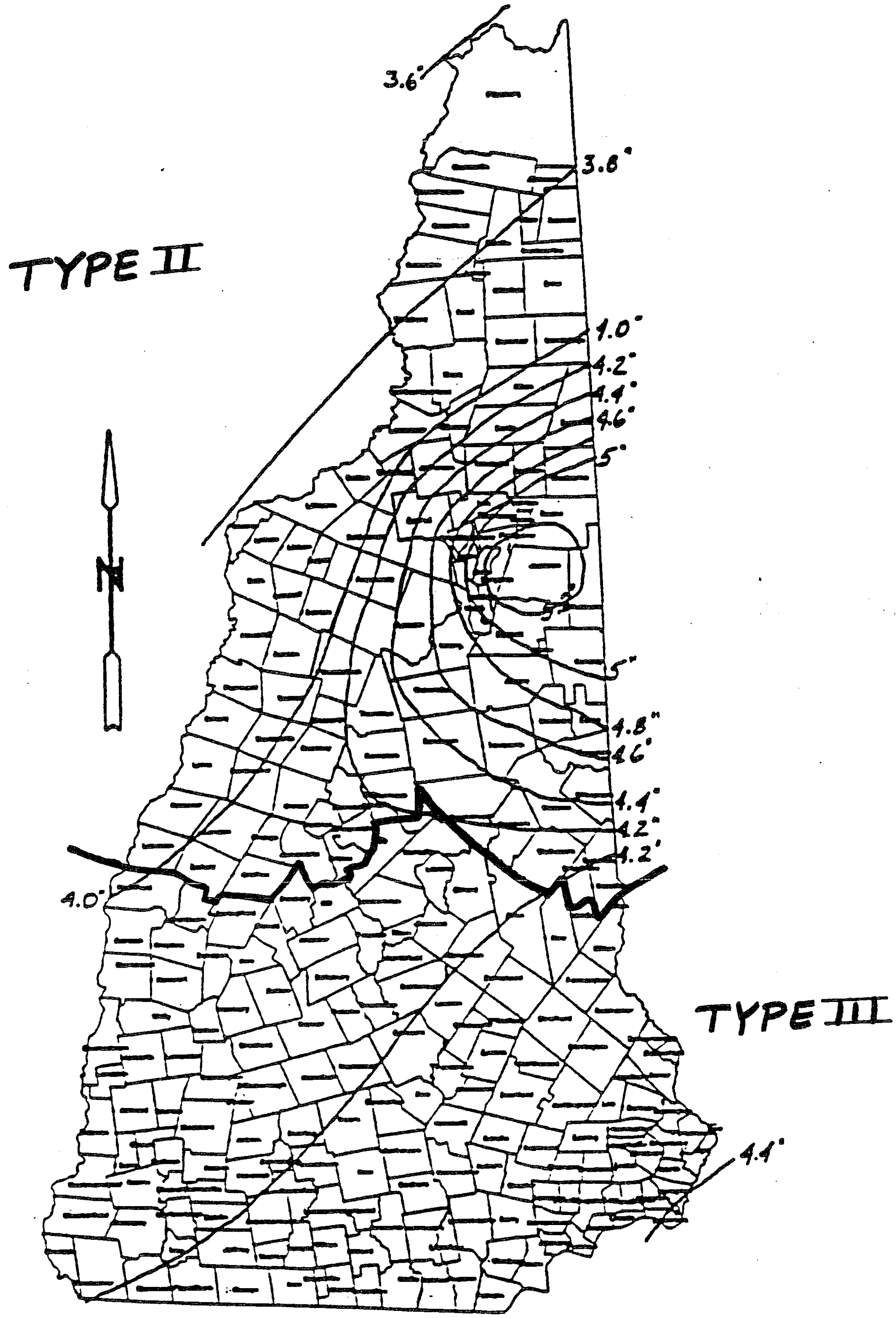


FIGURE A-3 -- 10-YEAR FREQUENCY 24-HOUR DURATION RAINFALL

Source: USDA Soil Conservation Service

Rainfall Conversion

1 inch = 25.4 mm

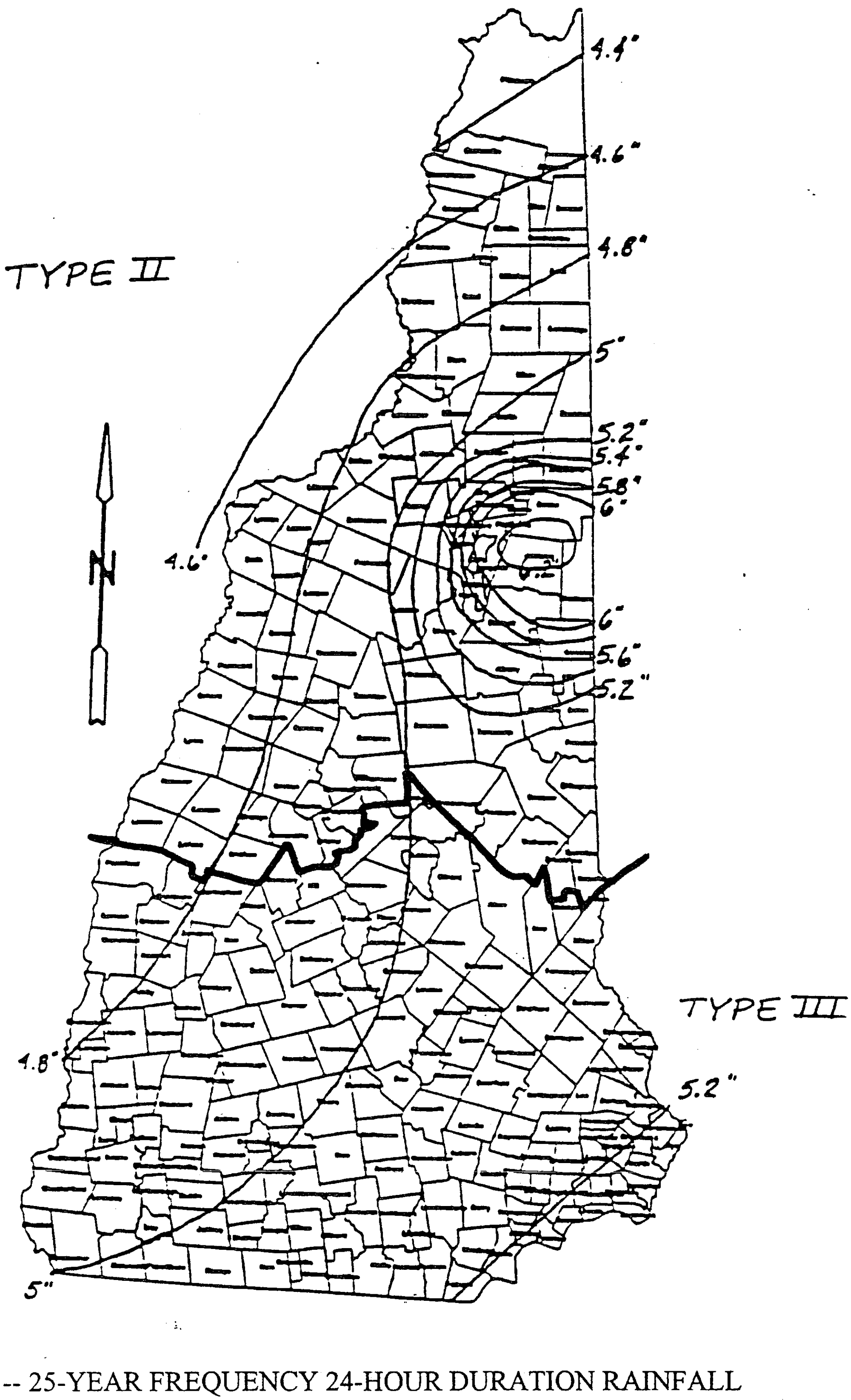


FIGURE A-4 -- 25-YEAR FREQUENCY 24-HOUR DURATION RAINFALL

Source: USDA Soil Conservation Service

Rainfall Conversion

1 inch = 25.4 mm

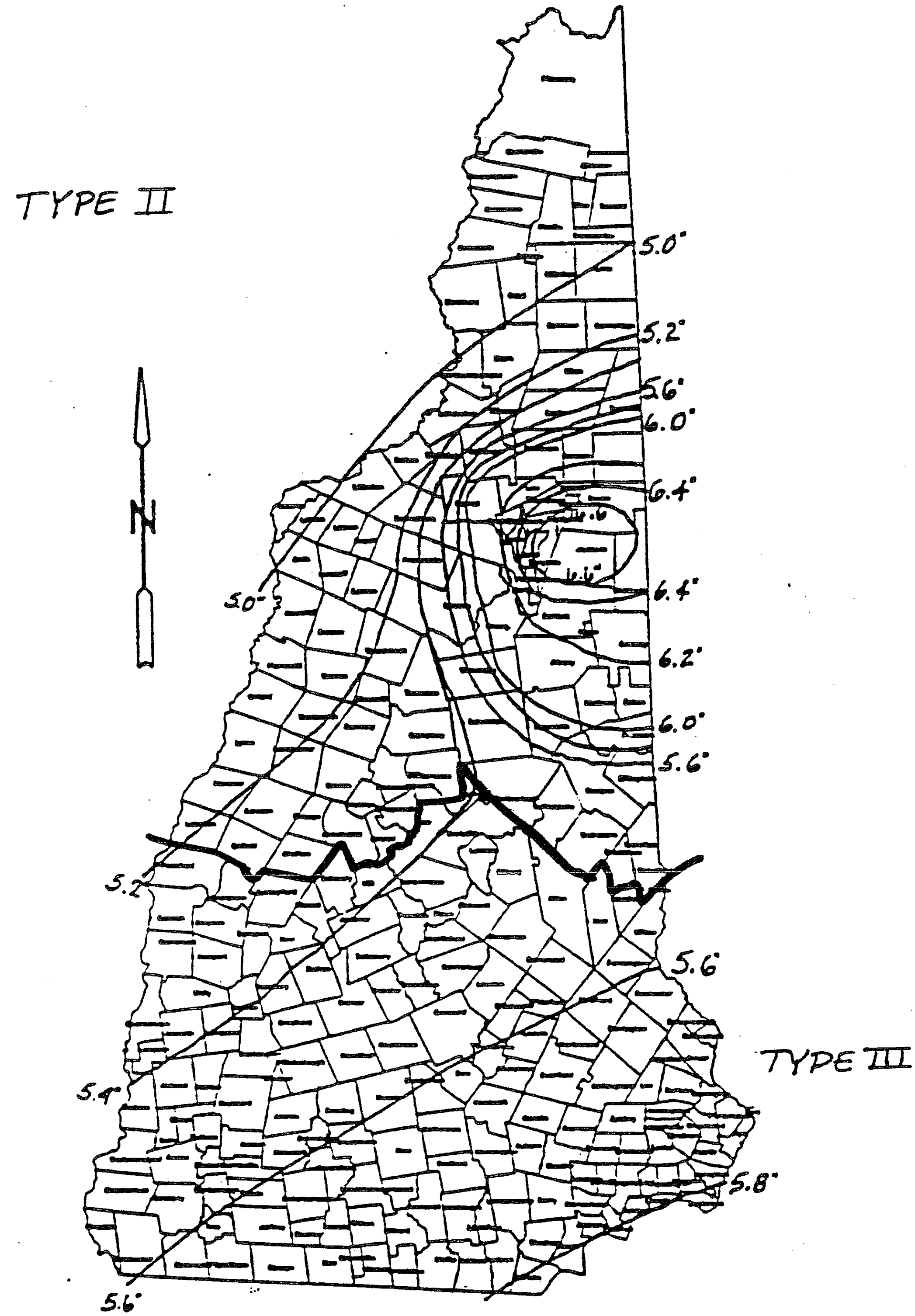


FIGURE A-5 -- 50-YEAR FREQUENCY 24-HOUR DURATION RAINFALL

Source: USDA Soil Conservation Service

Rainfall Conversion

1 inch = 25.4 mm

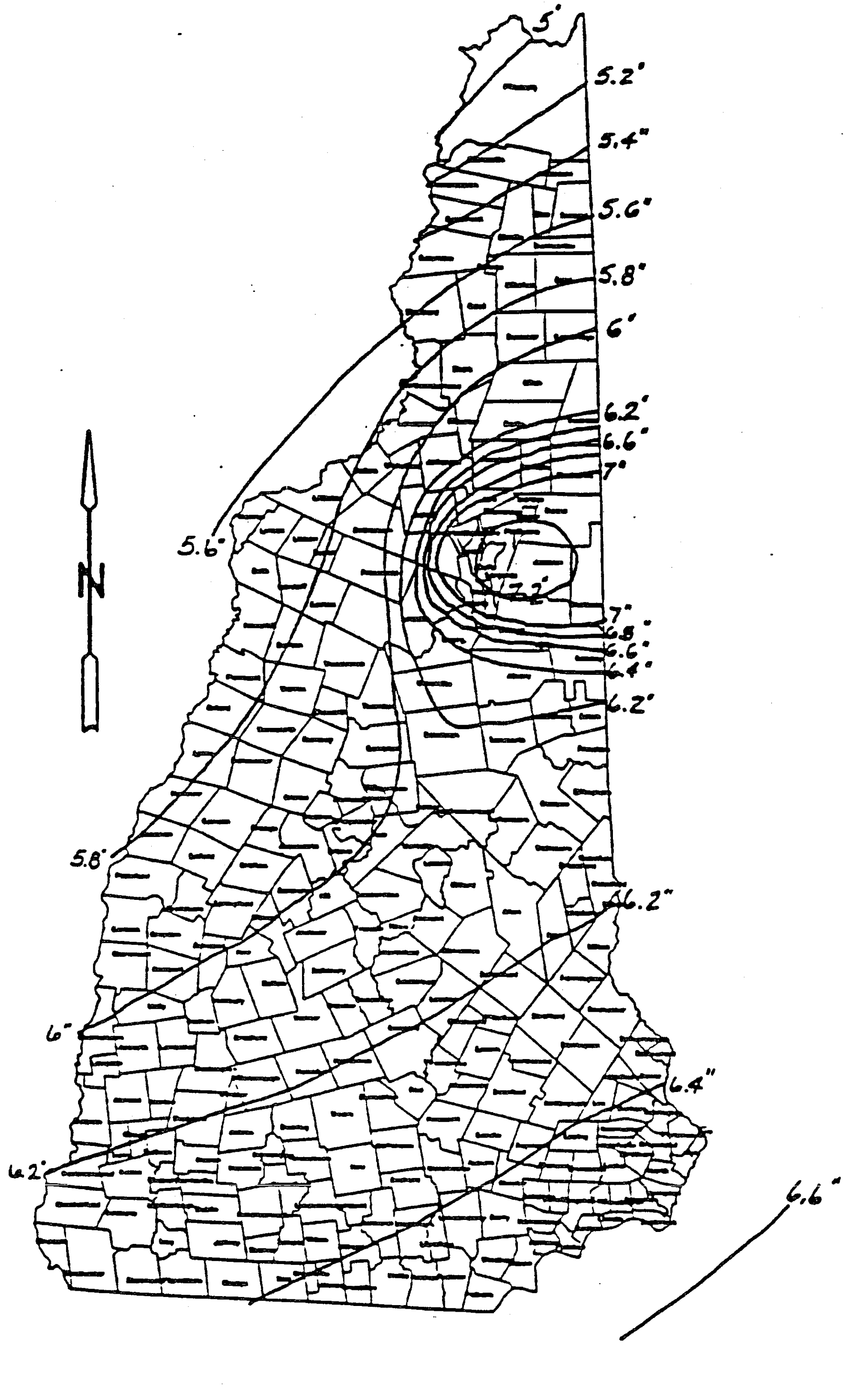


FIGURE A-6 -- 100-YEAR FREQUENCY 24-HOUR DURATION RAINFALL

Source: USDA Soil Conservation Service

RUNOFF CURVE NUMBER (RCN)

The runoff curve number is a factor that relates mass rainfall to mass runoff. It is based on soil characteristics, cover type, hydrologic condition, and land treatment. Tables B-1 through B-3 provide runoff curve numbers for urban areas, cultivated agricultural areas, and other agricultural areas for various hydrologic conditions.

Cover type relates to the kind of cover found on the ground such as vegetation, bare soil, and impervious surfaces such as parking areas, roofs, streets, and roads.

Hydrologic condition indicates the effects of cover type and treatment on infiltration and runoff rates. It is generally estimated from the density of plant and crop residue on the area. Good hydrologic condition indicates that the soil usually has low runoff potential for that specific hydrologic soil group, cover type and treatment. Some factors to consider in estimating the effect of cover on infiltration and runoff are: canopy or density of leaves, amount of year-round cover, amount of grass or close-seeded legumes in a rotation, percent of residue cover, and the degree of surface roughness.

Treatment is a cover type modifier used to describe the management of cultivated agricultural lands. It includes mechanical practices such as contouring and terracing, and management practices, such as crop rotations and reduced or no tillage.

Table B-1 - Runoff curve numbers for urban areas[1]

Cover description		Curve numbers for hydrologic soil group-			
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[4]		77	86	91	94
Idle lands (RCNs are determined using cover types similar to those in Table 2-2c).					

[1]Average runoff condition, and $I_a = 0.2S$.

[2]The average percent impervious area shown was used to develop the composite RCNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have an RCN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. RCNs for other combinations of conditions may be computed using figure 2-3 or 2-4.

[3]RCN's shown are equivalent to those of pasture. Composite RCNs may be computed for other combinations of open space cover type.

[4]Composite RCNs to use for the design of temporary measures during grading and construction should be computed using Figure 2-3 or 2-4, based on the degree of development (impervious area percentage) and the RCNs for the newly graded pervious areas.

Table B-2 - Runoff curve numbers for cultivated agricultural lands[1]

Cover description			Curve numbers for hydrologic soil group-			
Cover type	Treatment[2]	Hydrologic condition[3]	A	B	C	D
Fallow	Bare soil	–	77	86	91	94
	Crop residue cove (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR+CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C+CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

[1]Average runoff condition, and $I_a = 0.2S$.

[2]*crop residue cover* applies only if residue is on at least 5% of the surface throughout the year.

[3]Hydrologic condition is based on combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes in rotations, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Table B-3 - Runoff curve numbers for other agricultural lands[1]

Cover description		Curve numbers for hydrologic soil group-			
Cover type	Hydrologic condition[3]	A	B	C	D
Pasture, grassland, or range - continuous forage for grazing.[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow - continuous grass, protected from grazing and generally mowed for hay.	--	30	58	71	78
Brush - brush-weed-grass mixture with brush the major element.[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[4]30	48	65	73
Woods - grass combination (orchard or tree farm).[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods.[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads - buildings, lanes, driveways, and surrounding lots.	--	59	74	82	86

[1]Average runoff condition, and $I_a = 0.2S$.

[2]
Poor: < 50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

[3]
Poor: < 50% ground cover.
Fair: 50 to 75% ground cover.
Good: > 75% ground cover.

[4]Actual curve number is less than 30; use RCN = 30 for runoff computations.

[5]RCNs shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the RCNs for woods and pasture.

[6]
Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

URBAN IMPERVIOUS AREA MODIFICATIONS

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system should be considered in computing the RCN for urban areas. There are two situations that can occur with impervious areas in an urban development, either connected or unconnected.

Impervious areas are considered to be connected if runoff from them flows directly into the drainage system. They are also considered connected if the runoff from them flows as concentrated flow over a pervious area (not as sheet flow) and then into the drainage system. Figure C can be used to calculate a composite RCN when the percentage of connected impervious area is known and the RCN of the pervious area is known. Table B-1 gives "Average percent impervious areas" for particular situations listed in the table. Figure C should be used to calculate percent connected impervious area for situations not listed.

Impervious areas are considered to be unconnected if the runoff from the area is spread over a pervious area as sheet now. To determine the RCN when all or part of the impervious area is not directly connected to the drainage system and where the total impervious area is less than 30% see Figure D. If the impervious area is 30% or greater, then use Figure C because the absorptive capacity of the remaining pervious area will not significantly affect runoff.

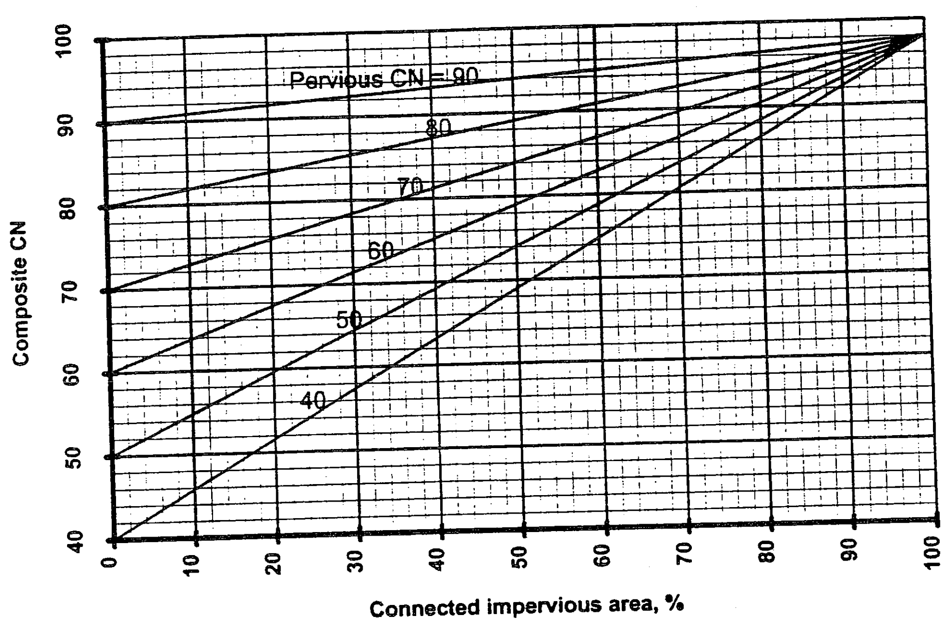


Figure C -- COMPOSITE RCN WITH CONNECTED IMPERVIOUS AREAS

Source: Adapted from USDA Soil Conservation Service

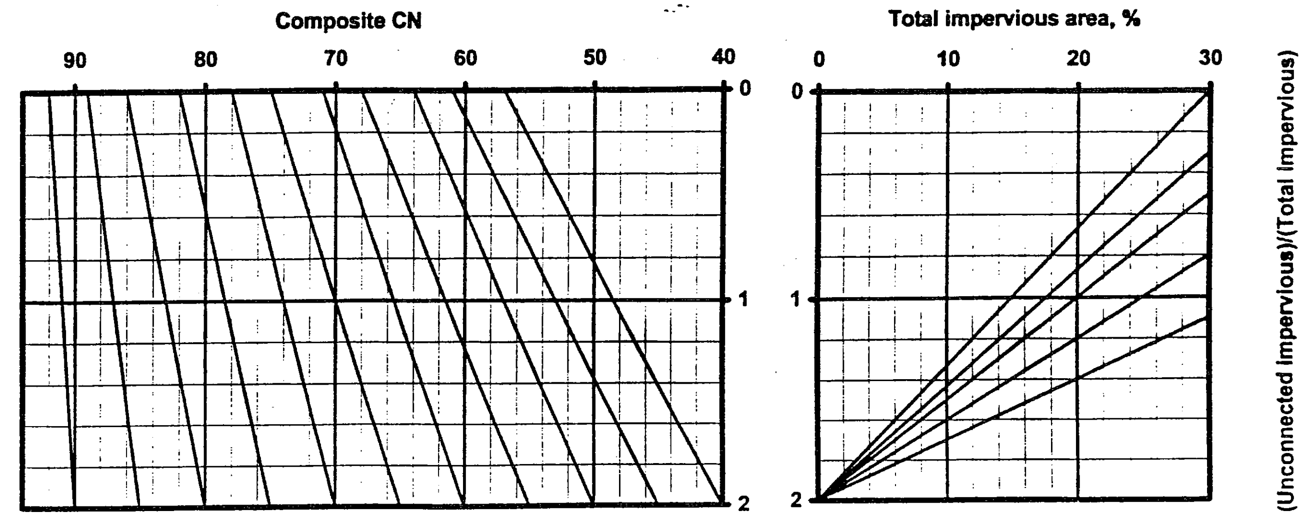


FIG. D -- COMPOSITE RCN WITH UNCONNECTED IMPERVIOUS AREAS AND TOTAL IMPERVIOUS AREA LESS THAN 30%

Source: USDA Soil Conservation Service

Table E-- RUNOFF DEPTH FOR SELECTED RCNs AND RAINFALLS

Rainfall in inches	Runoff depth for curve number of -											
	40	45	50	55	60	65	70	75	80	85	90	95	98
1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.17	0.32	0.56	0.79
1.2	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.07	0.15	0.27	0.46	0.74	0.99
1.4	0.00	0.00	0.00	0.00	0.00	0.02	0.06	0.13	0.24	0.39	0.61	0.92	1.18
1.6	0.00	0.00	0.00	0.00	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.00	0.00	0.00	0.00	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.00	0.00	0.00	0.02	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5	0.00	0.00	0.02	0.08	0.17	0.30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.00	0.02	0.09	0.19	0.33	0.51	0.71	0.96	1.25	1.59	1.98	2.45	2.77
3.5	0.02	0.08	0.20	0.35	0.53	0.75	1.01	1.30	1.64	2.02	2.45	2.94	3.27
4.0	0.06	0.18	0.33	0.53	0.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
4.5	0.14	0.30	0.50	0.74	1.02	1.33	1.67	2.05	2.46	2.91	3.40	3.92	4.26
5.0	0.24	0.44	0.69	0.98	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	0.50	0.80	1.14	1.52	1.92	2.35	2.81	3.28	3.78	4.30	4.85	5.41	5.76
7.0	0.84	1.24	1.68	2.12	2.60	3.10	3.62	4.15	4.69	5.25	5.82	6.41	6.76
8.0	1.25	1.74	2.25	2.78	3.33	3.89	4.46	5.04	5.63	6.21	6.81	7.40	7.76
9.0	1.71	2.29	2.88	3.49	4.10	4.72	5.33	5.95	6.57	7.18	7.79	8.40	8.76
10.0	2.23	2.89	3.56	4.23	4.90	5.56	6.22	6.88	7.52	8.16	8.78	9.40	9.76
11.0	2.78	3.52	4.26	5.00	5.72	6.43	7.13	7.81	8.48	9.13	9.77	10.39	10.76
12.0	3.38	4.19	5.00	5.79	6.56	7.32	8.05	8.76	9.45	10.11	10.76	11.39	11.76
13.0	4.00	4.89	5.76	6.61	7.42	8.21	8.98	9.71	10.42	11.10	11.76	12.39	12.76
14.0	4.65	5.62	6.55	7.44	8.30	9.12	9.91	10.67	11.39	12.08	12.75	13.39	13.76
15.0	5.33	6.36	7.33	8.29	9.19	10.04	10.85	11.63	12.37	13.07	13.74	14.39	14.76

[1] Interpolate the values shown to obtain runoff depths for CN's or rainfall amounts not shown.

Source: Adapted from USDA Soil Conservation Service

Metric Conversion: 1 inch = 25.4 mm

Once the runoff curve number has been calculated for a drainage area and the rainfall for a design frequency storm has been determined, then Table E can be used to find the runoff depth. The volume of runoff is calculated by multiplying the runoff depth times the drainage area. Runoff volumes are usually expressed in acre-feet.

TIME OF CONCENTRATION AND TRAVEL TIME

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. Travel time is a component of the time of concentration (T_C), which is the time it takes for runoff to travel from the hydraulically most distant point of the watershed to the design point. Time of concentration is computed by adding all of the travel times for the consecutive components of the drainage conveyance system.

Computation of Travel Time And Time of Concentration

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. It is best to use field inspection to determine the types of flow in the drainage system of a watershed.

Travel time is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600\ V}$$

where:

T_t = travel time in hours

L = flow length in feet

V = average velocity in the section in feet per second

3600 = conversion factor from seconds to hours.

Time of concentration is the sum of the travel times for the various consecutive flow segments:

$$T_c = T_t 1 + T_t 2 + T_t 3 + \ldots + T_t m$$

where:

T_c = time of concentration

m = number of flow segments.

Sheet Flow

Sheet flow is flow over a plane surface. It usually occurs in the upper reaches of a watershed. Sheet flow is for very shallow depths of about 30 mm (0.1 foot) or less. Sheet flow does not exceed 90 meters (300 feet) and this is usually on very flat slopes. For small watersheds, the sheet flow length will normally not exceed 30 meters (100 feet). Good judgment should be used in determining sheet flow length in a watershed. Manning's kinematic solution is used to compute travel time for sheet flow. Table F gives Manning's "n" values for sheet flow under various flow conditions. These "n" values should only be used in computing sheet flow.

Manning's kinematic solution is:

$$T_t = \frac{K\,(nL)^{0.8}}{(P_2)^{0.5}\,s^{0.4}}$$

where:

T_t = travel time in hours

n = Manning's roughness coefficient

L = flow length in meters (feet)

K = 0.09 Metric (0.007 English)

s = land slope in meter/meter (ft/ft).

P_2 = 2-year, 24 hour rainfall in mm (in.)

TABLE F -- ROUGHNESS COEFFICIENTS (MANNING'S n) FOR SHEET FLOW

Surface Description	n
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grasses	0.15
Dense grasses	0.24
Woods:	
Light underbrush	0.40
Dense underbrush	0.80

Note: When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Source: Adapted from USDA Soil Conservation Service

Shallow Concentrated Flow

Sheet flow usually becomes shallow concentrated flow. Shallow concentrated flow generally flows at depths from 30 to 150 mm (0.1 to 0.5 feet). The average velocity for this type of flow can be determined from Figure G, where the average velocity can be found depending on the slope and type of channel. The travel time equation on page 2-8 is then used to estimate travel time for the shallow concentrated flow segment.

Open Channel Flow

Open channels are assumed to begin where surveyed cross section information has been obtained. Usually these channels are visible on aerial photographs or appear as a blue line on USGS topographic maps. Manning's equation can be used to estimate average flow velocity in these sections. Average flow velocity is usually determined for bank full conditions.

Manning's equation is:

$$V = \frac{r^{2/3}S^{1/2}}{n} \text{ (Metric)} \qquad V = \frac{1.49r^{2/3}S^{1/2}}{n} \text{ (English)}$$

Where:

V = average velocity in meters per second (feet per second)

r = hydraulic radius which is the area of the channel divided by the wetted perimeter of the channel in meters (feet).

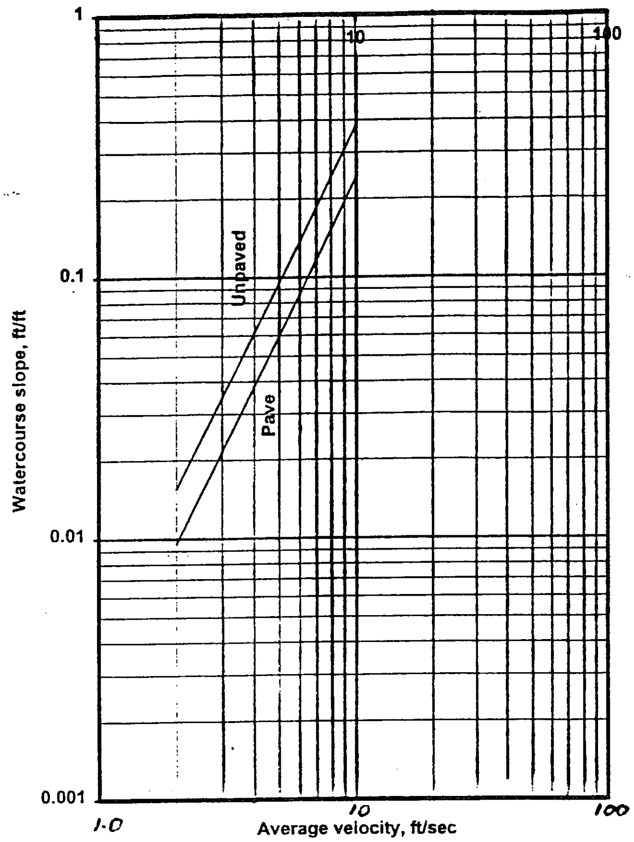
S = channel slope in meters per meter (feet per foot).

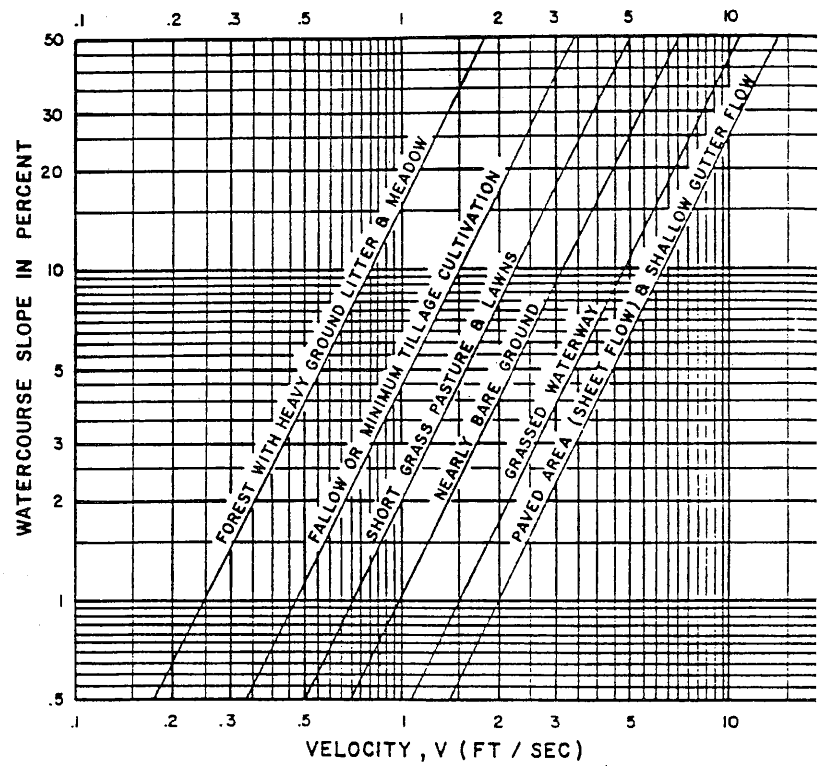
n = Manning's roughness coefficient for open channel flow.

Manning's "n" values for open channel flow can be found in various textbooks. A good range for typical open channel flow in New Hampshire is 0.03-0.10.

Reservoirs And Lakes

Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed to be zero for small bodies of water. However, if the watershed contains several ponds or a large pond, their effects need to be determined. Either compute the increase in the T_c or determine the detention storage effects for reducing the peak discharge.





Hydraulic Engineering Circular No. 19

Velocities for Upland Method of Estimating T_c

Metric conversion: 1 meter = 3.28 feet

FIGURE G -- AVERAGE VELOCITIES FOR ESTIMATING TRAVEL TIME FOR SHALLOW CONCENTRATED FLOW

Source: Adapted from USDA Soil Conservation Service

GRAPHICAL PEAK DISCHARGE METHOD

The Graphical Peak Discharge method was developed from hydrograph analysis using TR-20. The peak discharge formula was given earlier in the chapter. The input requirements for the Graphical method are: T_c (hr.), drainage area in square miles, appropriate rainfall distribution (II or III), the 24-hour rainfall in inches, and the RCN. If pond and swamp areas are spread throughout the watershed and are not considered in the T_c computation, an adjustment for pond and swamp areas is also needed.

Peak Discharge Computation

For a selected rainfall frequency, the 24 hour rainfall (P) is obtained from the New Hampshire precipitation maps in Figures A-1 through A-6. Using Worksheet 1 (pg. 2-19), the weighted runoff curve number (RCN) can be calculated. The RCN is then used to determine the initial abstraction (I_a) from Table H. Then the I_a/P ratio can be computed.

Once the I_a/P ratio is obtained and the appropriate rainfall distribution has been determined, then either Figure J-1 or Figure J-2 can be used to find the unit peak discharge (q_u). If the I_a/P ratio is outside the limits in the charts, then the limiting I_a/P ratio value should be used. If the I_a/P ratio is within the limits of the charts, then interpolation should be used.

Peak discharge per square mile per inch of runoff (q_u) is obtained from Figure J-1 or Figure J-2 by using the T_c , rainfall distribution type, and the I_a/P ratio. The pond and swamp adjustment factor is obtained from Table I. Worksheet 2 (pg. 2-20) can be used to compute time of concentration or travel time. Worksheet 3 (pg. 2-21) can be used to aid in computing the peak discharge using the Graphical Method.

There are several limitations to using the Graphical Method. This method only provides a determination of peak discharge. The Tabular Method or TR-20 should be used if a hydrograph is needed, watershed subdivision is required, or a higher degree of accuracy is necessary. Some other considerations that may limit the use of the Graphical Method are:

1. The watershed must be hydrologically homogeneous or describable by one RCN where land use, soils and type of cover are distributed uniformly throughout the watershed.
2. The watershed should have only one main stream. If not the streams should have nearly equal T_c values.
3. The method cannot perform valley or reservoir routing.
4. The F_p factor can only be applied for the ponds and swamps not in the Tc path.
5. T_c values may range only from 0.1 hours to 10 hours.

6. When this method is used to develop estimates of peak discharge for both present and developed conditions in a watershed, the same procedure for estimating T_c should be used.

TABLE H -- I_a VALUES FOR RUNOFF CURVE NUMBERS					
CURVE NUMBER	I_a (mm)	I_a (in)	CURVE NUMBER	I_a (mm)	I_a (in)
40	76.2	3.000	70	21.8	0.857
41	73.1	2.878	71	20.8	0.817
42	70.2	2.762	72	19.8	0.778
43	67.3	2.651	73	18.8	0.740
44	64.6	2.545	74	17.9	0.703
45	62.1	2.444	75	16.9	0.667
46	59.6	2.348	76	16.0	0.632
47	57.3	2.255	77	15.2	0.597
48	55.0	2.167	78	14.3	0.564
49	52.9	2.082	79	13.5	0.532
50	50.8	2.000	80	12.7	0.500
51	48.8	1.922	81	11.9	0.469
52	46.9	1.846	82	11.2	0.439
53	45.1	1.774	83	10.4	0.410
54	43.3	1.704	84	9.7	0.381
55	41.6	1.636	85	9.0	0.353
56	39.9	1.571	86	8.3	0.326
57	38.3	1.509	87	7.6	0.299
58	36.8	1.448	88	6.9	0.273
59	35.3	1.390	89	6.3	0.247
60	33.9	1.333	90	5.6	0.222
61	32.5	1.279	91	5.0	0.198
62	31.1	1.226	92	4.4	0.174
63	29.8	1.175	93	3.8	0.151
64	28.6	1.125	94	3.2	0.128
65	27.4	1.077	95	2.7	0.105
66	26.2	1.030	96	2.1	0.083
67	25.0	0.985	97	1.6	0.062
68	23.9	0.941	98	1.0	0.041
69	22.8	0.899			

Source: Adapted from USDA Soil Conservation Service

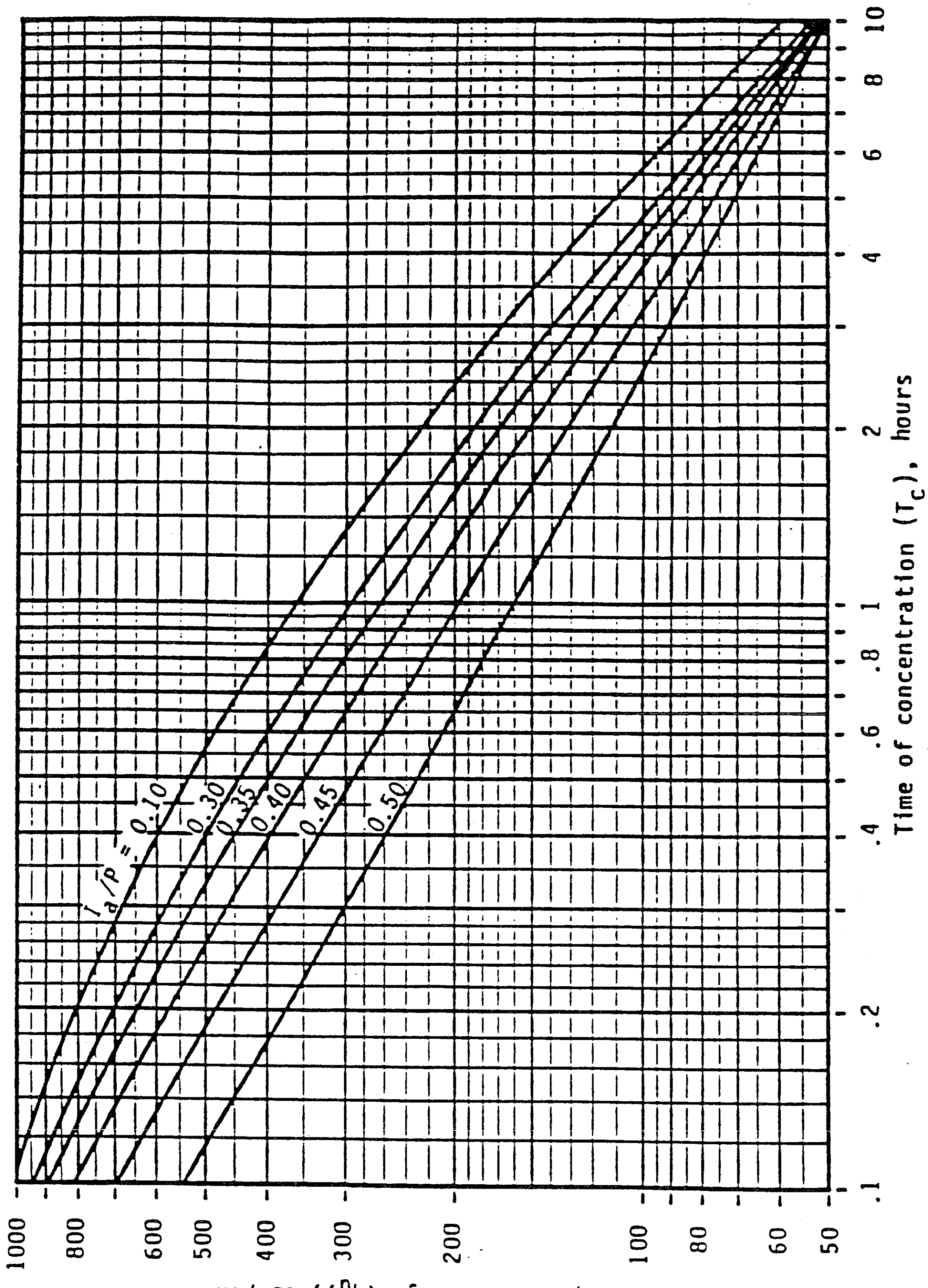
TABLE I -- ADJUSTMENT FACTORS FOR PONDED AND SWAMPY AREAS

Peak flow determinations assume that the topography is such that surface flow into ditches, drains, and streams is approximately uniform. On very flat areas and where ponding or swampy areas occur throughout the watershed, a considerable amount of the surface runoff may be retained in temporary storage. The peak rate of runoff should be reduced to reflect this condition. This table provides adjustment factors to determine this reduction based on the ratio of the ponding or swampy area to the total watershed area.

Ratio of Drainage Area to Ponding and Swampy Area	Percentage of Ponding and Swampy Area	Adjustment Factor (F_p)
500	0.2	0.97
200	0.5	0.92
100	1.0	0.87
50	2.0	0.82
40	2.5	0.78
30	3.3	0.75
20	5.0	0.72

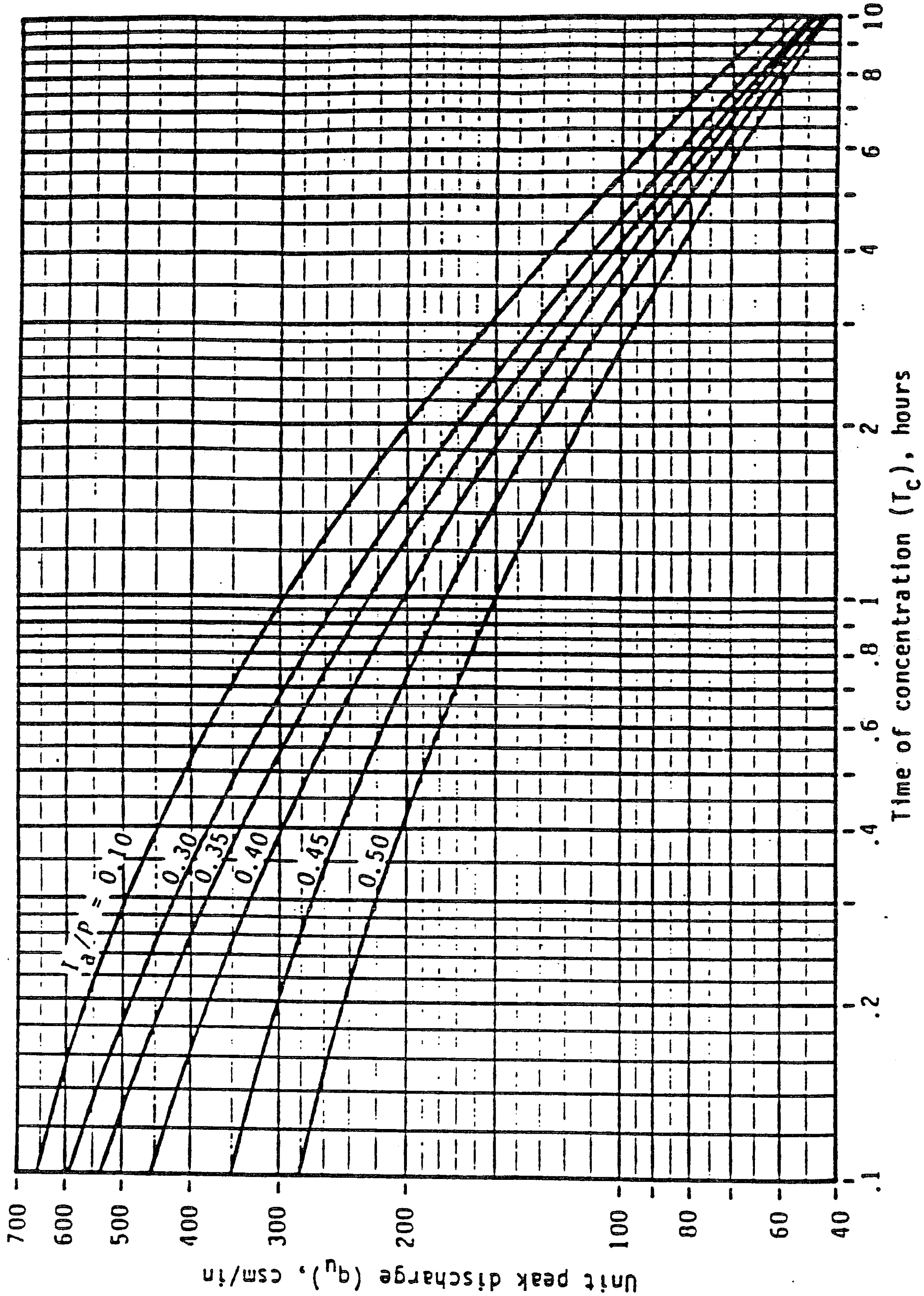
Source: USDA Soil Conservation Service

FIGURE J-1 -- UNIT PEAK DISCHARGE (q_u) FOR SCS TYPE II RAINFALL DISTRIBUTION



Metric Conversion: 1 inch = 25.4 mm
1.0 cfs = 0.028 m^3/s
1.0 mile = 1.61 km

FIGURE J-2 -- UNIT PEAK DISCHARGE (q_u) FOR SCS TYPE III RAINFALL DISTRIBUTION



Metric Conversion: 1 inch = 25.4 mm
1.0 cfs = 0.028 m^3/s
1.0 mile = 1.61 km

HYDROLOGIC SOIL GROUPS

By Group Type

The hydrologic grouping of soils is based upon infiltration rates as they affect runoff. The four groups are described as follows:

Group A—Soils having high infiltration rates even when thoroughly wetted. These consist chiefly of deep, well to excessively drained sands or gravel. These soils have a high rate of water transmission and would result in low runoff potential.

Adams	Hinckley	Quonset	Sunday
Ceasar	Hoosic	Redstone	Udipsamments
Colton	Made Land**	Ricker	Udorthents**
Dumps**	Masardis	Rubble Land	Warwick
Gloucester*	Merrimac	Success	Windsor
Hermon	Pits, gravel	Suncook	

Group B—Soils having moderate infiltration rates when thoroughly wetted. These consist 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.

Abenaki	Deerfield	Metallak	Pootatuck
Acton	Duane	Monadnock	Salmon
Agawam	Dutchess	Newfields	Salmon Variant
Au Gres	Fryeburg	Ninigret	Scio
Bangor	Groveton	Occum	Sheepscot
Belgrade	Hadley	Ondawa	Stetson
Berkshire	Hartland	Ondawa Variant	Sudbury
Canton	Haven	Pennichuck	Sunapee
Cardigan	Hitchcock	Pipestone	Sutton
Charlton	Kearsarge	Plaisted Variant	Unadilla
Chatfield	Lovewell	Podunk	Unadilla Variant
Croghan	Machias	Podunk Variant	Waumbek
Dartmouth	Madawaska	Poocham	Winooski

* See local SCS office for alternative interpretation as some areas have been updated or re-correlated.

** Evaluation of each site is required to determine hydrologic group.

Group C -- Soils having slow infiltration rates when thoroughly wetted. These consist chiefly of (1) soils with a layer that impedes the downward movement of water, or (2) soils with moderately fine to fine texture, or (3) soils with moderately high water tables. These soils have a slow rate of water transmission.

Acton Variant
Alluvial land
Becket
Bernardston
Bernardston Variant
Boxford
Buxton
Canaan*
Charles
Cohas
Dixmont
Eldridge
Elmridge
Elmwood
Glebe
Grange
Houghtonville
Howland
Kinsman
Leicester
Leicester Variant
Lim
Limerick
Limerick Variant
Lyme
Marlow
Melrose
Millis
Mixed Alluvial land
Montauk
Moosilauke
Mundal
Naumburg
Nicholville
Nicholville Variant
Paxton
Pemi
Peru
Pillsbury
Pittstown
Pittstown Variant
Plaisted
Raynham
Raynham Variant
Ridgebury
Rippowam
Riverwash**
Roundabout
Rumney
Saugatuck
Scitico
Scituate
Sisk
Skerry
Squamscot
Stissing
Stratton
Suffield
Surplus
Tunbridge
Walpole
Wareham
Windsor Variant
Winnecook
Woodbridge

Group D -- Soils having very slow infiltration rates when thoroughly wetted. These consist chiefly of very poorly drained material soils or organic soils. These soils have a very slow rate of water transmission.

Biddeford
Binghamville
Borofibrists
Borohemists*
Brockport
Chocorua
Fresh Water Marsh
Ipswich
Marsh
Matunuck
Maybid
Medomak
Medomak Variant
Monarda
Ossipee
Pawcatuck
Peachum
Quarries**
Rock Outcrop*
Saco
Saco Variant
Scantic*
Scarboro
Searsport
Sebago
Tidal Marsh
Urban land
Vassalboro
Westbrook
Whitman
Wilmington

* See local SCS office for alternative interpretation as some areas have been updated or re correlated.

** Evaluation of each site is required to determine hydrologic group.

Soils in dual hydrologic groups have infiltration rates dependent on depth to water table or bed-rock. Onsite analysis is desirable in order to develop a representative composite RCN.

Soil Name	Hydrologic Soil Group
Glover	C/D
Greenwood	A/D
Hollis	C/D
Lombard	C/D
Londonderry	C/D
Lyman	C/D
Muck & Peat	A/D
Saddleback	C/D
Shapleigh	C/D
Swanton*	C/D
Thorndike	C/D

* See local SCS office for alternative interpretation as some areas have been updated or re-correlated.

** Evaluation of each site is required to determine hydrologic group.

By Soil Series

The hydrologic grouping of soils is based upon infiltration rates as they affect runoff. The four groups are described as follows:

Group A—Soils having high infiltration rates even when thoroughly wetted. These consist chiefly of deep, well to excessively drained sands or gravel. These soils have a high rate of water transmission and would result in low runoff potential.

Group B—Soils having moderate infiltration rates when thoroughly wetted. These consist chielfy of moderately deep to deep, moderately well to well drained soils to moderately fine to moderately coarse textures. These soils have a moderate rate of water transmisson.

Group C—Soils having slow infiltration rates when thoroughly wetted. These consist chiefly of (1) soils with a layer that impedes the downward movement of water, or (2) soils with moderately fine to fine texture, or (3) soils with moderately high water tables. These soils have a slow rate of water transmission.

Group D—Soils having very slow infiltration rates when thoroughly wetted. These consist chiefly of very poorly drained mineral soils or organic soils. These soils have a very slow rate of water transmission.

Soils in dual hydrologic groups have infiltration rates dependent on depth to water table or bedrock. Onsite analysis is desirable in order to develop a representative composite RCN.

Abenaki	B	Buxton	C
Acton	B	Canaan*	C
Acton Variant	C	Canton	B
Adams	A	Cardigan	B
Agawam	B	Ceasar	A
Alluvial land	C	Charles	C
Au Gres	B	Charlton	B
Bangor	B	Chatfield	B
Becket	C	Chocorua	D
Belgrade	B	Cohas	C
Berkshire	B	Colton	A
Bernardston	C	Croghan	B
Bernardston Variant	C	Dartmouth	B
Biddeford	D	Deerfield	B
Binghamville	D	Dixmont	C
Borofibrists	D	Duane	B
Borohemists*	D	Dumps**	A
Boxford	C	Dutchess	B
Brockport	D	Eldridge	C

Elmridge	C	Moosilauke	C
Elmwood	C	Muck & Peat	A/D
Fresh Water Marsh	D	Mundall	C
Fryeburg	B	Naumburg	C
Glebe	C	Newfields	B
Gloucester*	A	Nicholville	C
Glover	C/D	Nicholville Variant	C
Grange	C	Ninigret	B
Greenwood	A/D	Occum	B
Groveton	B	Ondawa	B
Hadley	B	Ondawa Variant	B
Hartland	B	Ossipee	D
Haven	B	Pawcatuck	D
Hermon	A	Paxton	C
Hinckley	A	Peachum	D
Hitchcock	B	Pemi	C
Hollis	C/D	Pennichuck	B
Hoosic	A	Peru	C
Houghtonville	C	Pillsbury	C
Howland	C	Pipestone	B
Ipswich	D	Pits, gravel	A
Kearsarge	B	Pittstown	C
Kinsman	C	Pittstown Variant	C
Leicester	C	Plaisted	C
Leicester Variant	C	Plaisted Variant	B
Lim	C	Podunk	B
Limerick	C	Podunk Variant	B
Limerick Variant	C	Poocham	B
Lombard	C/D	Pootatuck	B
Londonderry	C/D	Quarries**	D
Lovewell	B	Quonset	A
Lyman	C/D	Raynham	C
Lyme	C	Raynham Variant	C
Machias	B	Redstone	A
Madawaska	B	Ricker	A
Made land**	A	Ridgebury	C
Marlow	C	Rippowam	C
Marsh	D	Riverwash**	C
Masardis	A	Rock Outcrop*	D
Matunuck	D	Roundabout	C
Maybid	D	Rubble land	A
Medomak	D	Rumney	C
Medomak Variant	D	Saco	D
Melrose	C	Saco Variant	D
Merrimac	A	Saddleback	C/D
Metallak	B	Salmon	B
Millis	C	Salmon Variant	B
Mixed Alluvial land	C	Saugatuck	C
Monadnock	B	Scantic*	D
Monarda	D	Scarboro	D
Montauk	C	Scio	B

Scitico	C
Scituate	C
Searsport	D
Sebago	D
Shapleigh	C/D
Sheepscot	B
Sisk	C
Skerry	C
Squamscot	C
Stetson	B
Stissing	C
Stratton	C
Success	A
Sudbury	B
Suffield	C
Sunapee	B
Suncook	A
Sunday	A
Surplus	C
Sutton	B
Swanton*	C/D
Thorndike	C/D
Tidal Marsh	D
Tunbridge	C
Udipsamments	A
Udorthents**	A
Unadilla	B
Unadilla Variant	B
Urban land	D
Vassalboro	D
Walpole	C
Wareham	C
Warwick	A
Waumbek	B
Westbrook	D
Whitman	D
Wilmington	D
Windsor	A
Windsor Variant	C
Winnecook	C
Winooski	B
Woodbridge	C

* See local SCS office for alternative interpretation as some areas have been updated or re-correlated.

** Evaluation of each site is required to determine hydrologic group.

RUNOFF CURVE NUMBER AND RUNOFF

WORKSHEET 1

Project________________________ By________ Date____________

Location________________________ Checked________ Date____________

Circle one: Present Developed ________________________________

1. Runoff curve number (RCN)

Soil name and hydrologic group	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	RCN			Area ☐ acres ☐ mi^2 ☐ %	Product of RCN x area
		Table B	Fig. C	Fig. D		
	Totals =					

RCN (weighted) = $\frac{\text{total product}}{\text{total area}}$ = ________ = ________; Use RCN = ☐

2. Runoff

	Storm #1	Storm #2	Storm #3
Frequency----------yr.			
Rainfall, P (24-hour)----------in.			
Runoff, Q----------in. (Use P and RCN with Table E)			

Adapted from USDA Soil Conservation Service

TIME OF CONCENTRATION (T_C) OR TRAVEL TIME (T_T) WORKSHEET 2

Project ______________________ By __________ Date __________

Location ______________________ Checked __________ Date __________

Circle one: Present Developed ______________________

Circle one: T_C T_t through subarea ______________________

NOTES: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

Sheet Flow (Applicable to T_C only)

	Segment ID	Units		
1. Surface description (Table F)				
2. Manning's roughness coeff., n (Table F)				
3. Flow length, L (total L ≤ 300 ft.)		ft		
4. Two-yr. 24-hour rainfall, P_2		in		
5. Land slope, s		ft/ft		
6. $T_t = \frac{0.007(nL)^{0.8}}{P_2^{0.5} s^{0.4}}$	Compute T_t	hr	+	=

Shallow concentrated flow

	Segment ID	Units		
7. Surface description (paved or unpaved)				
8. Flow length, L		ft		
9. Watercourse slope, s		ft/ft		
10. Average velocity, V (Figure G)		ft/s		
11. $T_t = L/(3600V)$	Compute T_t	hr	+	=

Channel flow

	Segment ID	Units		
12. Cross sectional flow area, a		ft^2		
13. Wetted perimeter, P_W		ft		
14. Hydraulic radius, $r = a/P_w$	Compute r	ft		
15. Channel slope, s		ft/ft		
16. Manning's roughness coeff., n				
17. $V = \frac{1.49 r^{2/3} s^{1/2}}{n}$	Compute V	ft/s		
18. Flow length, L		ft		
19. $T_t = L/(3600V)$	Compute T_t	hr	+	=
20. Watershed or subarea T_C or T_t (add T_t in steps 6, 11, and 19)		hr		

GRAPHICAL PEAK DISCHARGE METHOD — WORKSHEET 3

Project______________________________ By__________ Date__________

Location______________________________ Checked__________ Date__________

Circle one: Present Developed ______________________________

1. Data:

Drainage area	A_m =	______________	mi^2 (acres/640)
Runoff curve number	RCN =	______________	(From Worksheet 1)
Time of concentration	T_c =	______________	hr (From Worksheet 2)
Rainfall distribution type	=	______________	(II, III)
Pond and swamp areas spread throughout watershed	=	______________	percent of A_m (____ acres of mi^2 covered)

		Storm #1	Storm 2	Storm #3
2. Frequency	yr			
3. Rainfall, P (24-hour)	in			
4. Initial abstraction, I (Use RCN with Table H)	in			
5. Compute I_a/P				
6. Unit peak discharge, q_u (Use T_c and I_a/P with Figure J-1 or J-2)	csm/in			
7. Runoff, Q (From Worksheet 1)	in			
8. Pond and swamp adjustment factor, F_p (Use percent pond and swamp area with Table I. Factor is 1.0 for zero percent pond and swamp area.)				
9. Peak discharge, q_p (Where $q_p = q_u A_m Q F_p$)	cfs			

B. THE RATIONAL METHOD

INTRODUCTION

Rainfall intensity is converted into rate of storm runoff by the rational formula:

$Q = C\,i\,A/360$	(Metric)
$Q = C\,i\,A$	(English)

Where:

Q = peak rate of runoff, in cubic meters per second (cubic feet/second), of selected return period.

C = weighted runoff coefficient (average of the coefficients assigned to the different types of contributing areas).

i = average rainfall intensity, in millimeters per hour (inches/hr.) for the selected frequency and for duration equal to the time of concentration.

A = contributing drainage area, in hectares (acres).

NOTE: 1 hectare (ha) = 10 000 m^2

This formula is not dimensionally correct, however a 1 inch depth of rainfall applied at a uniform rate for 1 hour on an area of 1 acre will produce 1.0 cubic foot per second of runoff barring any losses.

The Rational Method is based on the thesis that if a uniform rainfall of intensity (i) were falling on an impervious area (A), the maximum rate of runoff at the outlet to the drainage area would be reached when all portions of the drainage area were contributing; the runoff rate would then become constant. The time required for runoff from the most remote point (point from which the time of flow is greatest) of the drainage area to arrive at the outlet is called the time of concentration (T_c).

Actual runoff is far more complicated than the Rational Method indicates. Rainfall intensity is seldom the same over an area of appreciable size or for any substantial length of time during the same storm. If a uniform intensity of rainfall of duration equal to the time of concentration were to occur on all parts of the drainage area, the rate of runoff would vary in different parts of the area because of differences in the characteristics of the land surface and the nonuniformity of antecedent conditions. Under some conditions maximum rate of runoff occurs before all of the drainage area is contributing.

The temporary storage of storm water enroute toward defined channels and within the channels themselves accounts for a considerable reduction in the peak rate of flow except on very small areas. The error in the runoff estimate increases as the size of the drainage area increases. For these reasons, the Rational Method should not be used to determine the rate of runoff from large drainage areas. For the design of highway drainage structures, the use of the Rational Method should be restricted to drainage areas less than 80 hectares (200 acres).

RUNOFF COEFFICIENT

The runoff coefficient C in the Rational formula is the ratio of the rate of runoff to the rate of rainfall at an average intensity i when all the drainage area is contributing. The coefficient C varies widely from storm to storm, but studies have shown that when rainfall intensity and runoff (based on a 20-year record) were considered separately it was found that the ratio,

$$C = \frac{\text{Peak runoff rate of a given frequency}}{\text{Average rainfall intensity of the same frequency}}$$

remained reasonably constant for the various frequencies.

The range in values of C permits some allowance for land slope, soil type, vegetal cover, surface storage and urban development.

TIME OF CONCENTRATION

The time of concentration (T_c) varies with the size and shape of the drainage area, the land slope, the type of surface, the intensity of rainfall, whether flow is overland or channelized and many other factors. Extreme precision is not warranted in determining time of concentration for the design of drainage channels on rural highways. Time of concentration can be obtained using the Upland Method (Fig. G-2), the Kinematic Wave Equation (See FHWA Hydraulic Design Series No. 4), or if site information is limited Fig. 2-2 may be used. This figure is based on a study of six watersheds which varied in size from 0.5 to 45.3 hectares (1.2 to 112 acres). The watersheds were all located on a single farm in Tennessee. Research is badly needed on the time of concentration of other types of watersheds. The values of Tc from Figure 2-2 are based on meager data, and should only be used when better information is not available. A minimum time of concentration of 10 minutes (rural areas) and 5-minutes (urban areas) is recommended for finding the intensity used for estimating the design discharge.

Use of Fig. 2-2 requires the length (L) of the drainage area measured along the principal drainage line to the most remote (longest T_c) point and the height of this point above the outlet at which the flow is to be estimated.

Fig. 2-2 is based on the following formula:

$$T = \left(\frac{11.9L^3}{H} \right)^{0.385}$$ (English values. Divide T by 2.77 for metric use)

Where:

T = time of concentration, in hours.
L = length of drainage along the stream, in kilometers (miles).
H = height of most remote point in basin above outlet, in meters (feet).

RUNOFF COEFFICIENTS

CHARACTER OF SURFACE	RUNOFF COEFFICIENTS
PAVEMENT	
Asphaltic and Concrete	0.95
Brick	0.85
ROOFS	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
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
URBAN AREAS	
Business	
Downtown areas	0.70 to 0.95
Neighborhood areas	0.50 to 0.70
Residential	
Single-family areas	0.30 to 0.50
Multi units, detached	0.40 to 0.60
Multi units, attached	0.60 to 0.75
Suburban	0.25 to 0.40
Apartment dwelling areas	0.50 to 0.70
Industrial	
Light areas	0.50 to 0.80
Heavy areas	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard areas	0.20 to 0.40

FIGURE 2-1

Unimproved areas	0.10 to 0.30

AGRICULTURAL AREAS:

Topography and Vegetation	Runoff Coefficient C Soil Texture Open Sandy Loam	Clay and Silt Loam	Tight Clay
Woodland			
Flat 0-5% Slope	0.10	0.30	0.40
Rolling 5-10% Slope	0.25	0.35	0.50
Hilly 10-30% Slope	0.30	0.50	0.60
Pasture			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
Cultivated			
Flat	0.30	0.50	0.60
Rolling	0.40	0.60	0.72
Hilly	0.52	0.72	0.82

When a drainage area consists of multiple surface cover types, a weighted composite C value must be used as follows:

$$C = (C_1A_2 + C_2A_2 + ...+ C_nA_n)/(\Sigma A)$$

RAINFALL INTENSITY

Rainfall intensity-frequency data are taken from Weather Bureau Technical Paper 40- Rainfall Frequency Atlas of the United States for Durations from 30 minutes to 24 hours and Return Periods from 1 to 100 years. Charts have been constructed for various sections of the State with different frequency storms (Figures 2-3, 2-4, 2-5). These charts should be used in determining runoff by the Rational Method.

DRAINAGE AREA

The drainage area, in hectares, contributing to the point for which channel capacity is to be determined, can be measured on a topographic map or determined in the field by estimation, pacing, or a survey comparable in accuracy to the stadia-compass traverse. The data required to determine time of concentration and the runoff coefficient should be noted at the time of the preliminary field survey.

For the points along the channel, the design discharge is computed using the longest time of travel to the point for which the discharge is to be determined. Generally, less work is required to compute the runoff from separate areas rather than obtain a weighted C for the whole area.

For some drainage areas, the maximum rate of runoff will occur when only a portion of the area is contributing. When this occurs, use the higher Q for downstream design.

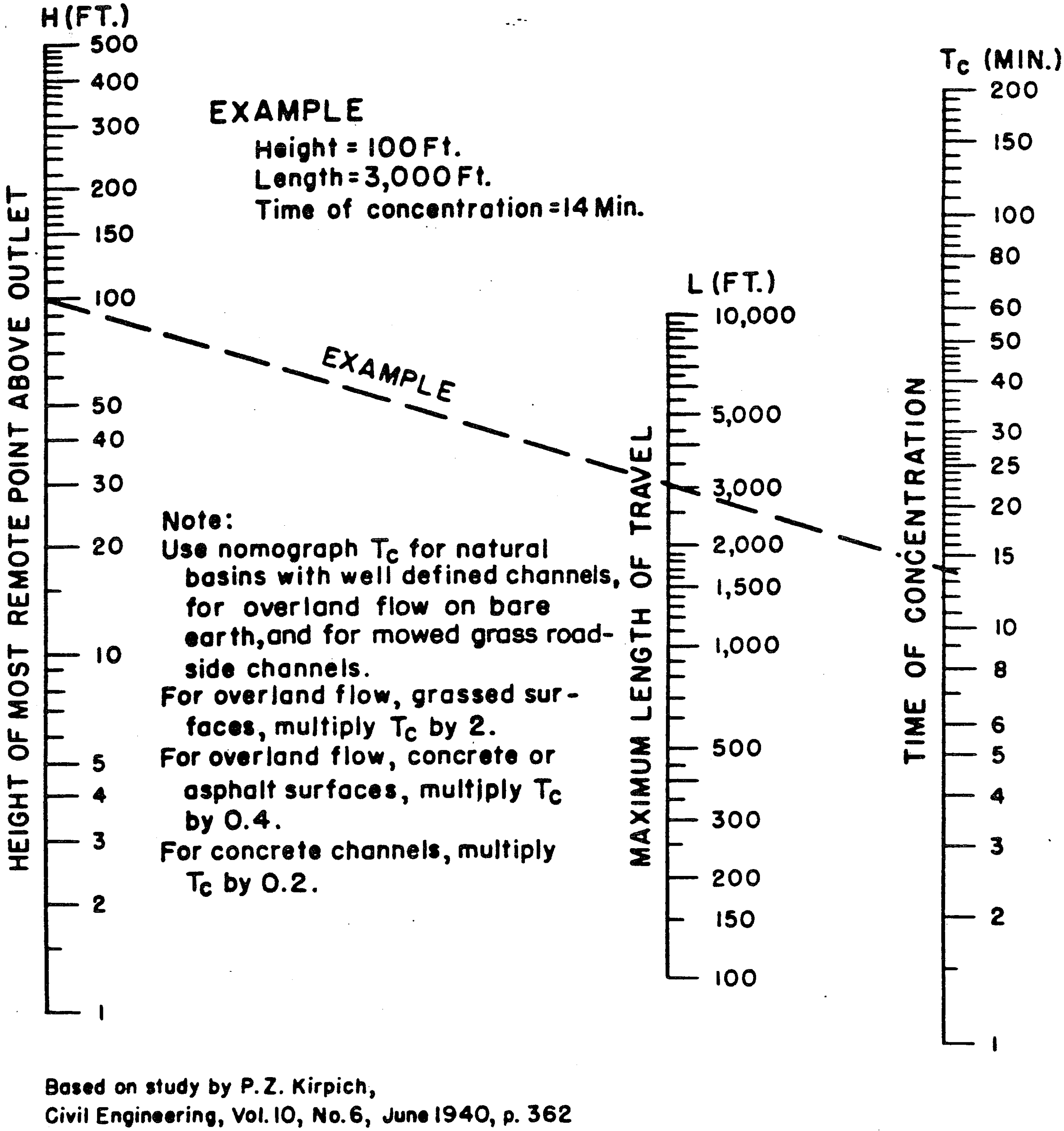
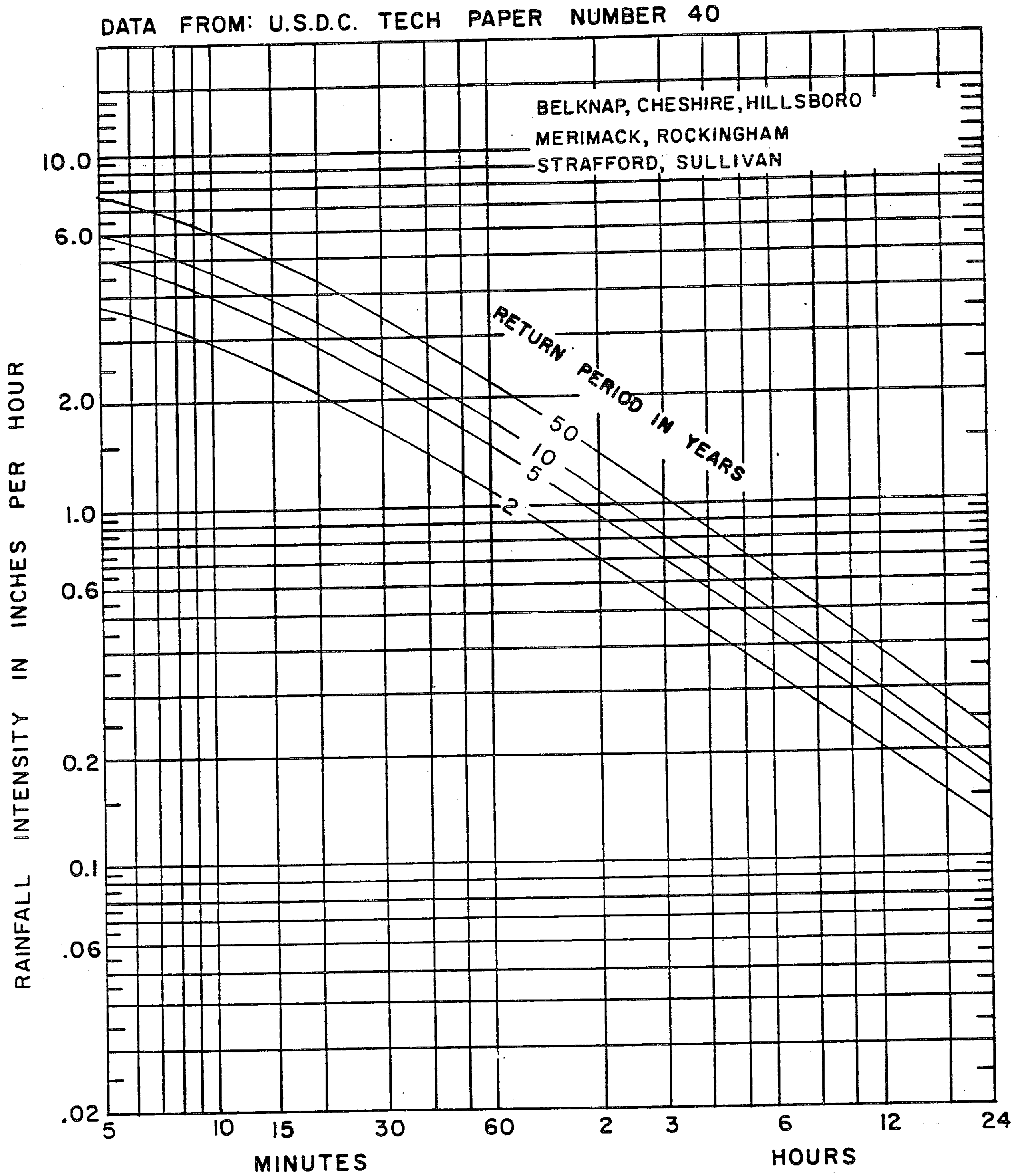


FIGURE 2-2
TIME OF CONCENTRATION OF SMALL DRAINAGE BASINS

NOTE: METRIC CONVERSION

1 inch = 25.4 mm
1 mm = .0394 in.

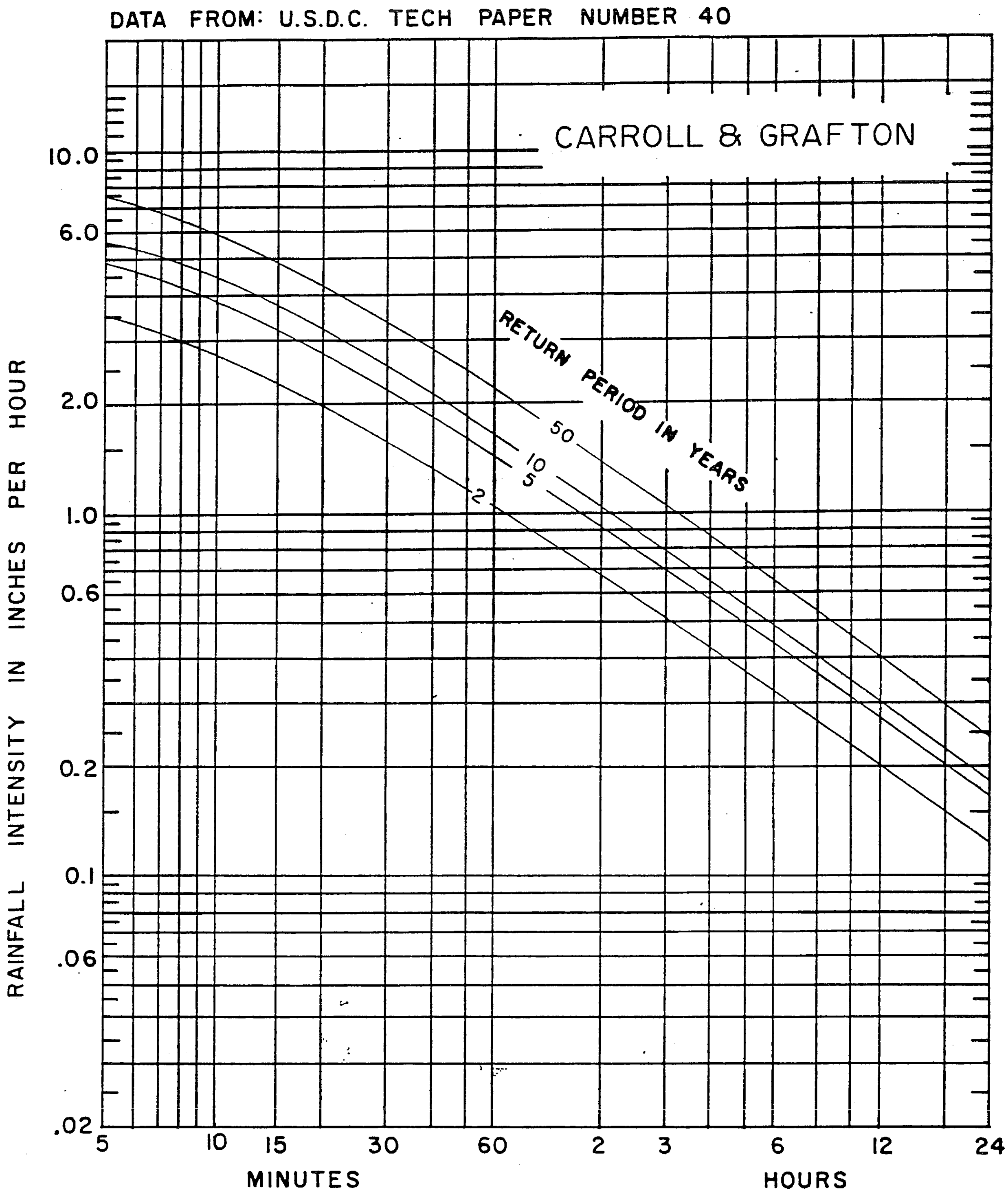
FIGURE 2-3



TIME OF CONCENTRATION

NOTE: METRIC CONVERSION 1 inch = 25.4 mm
1 mm = .0394 in.

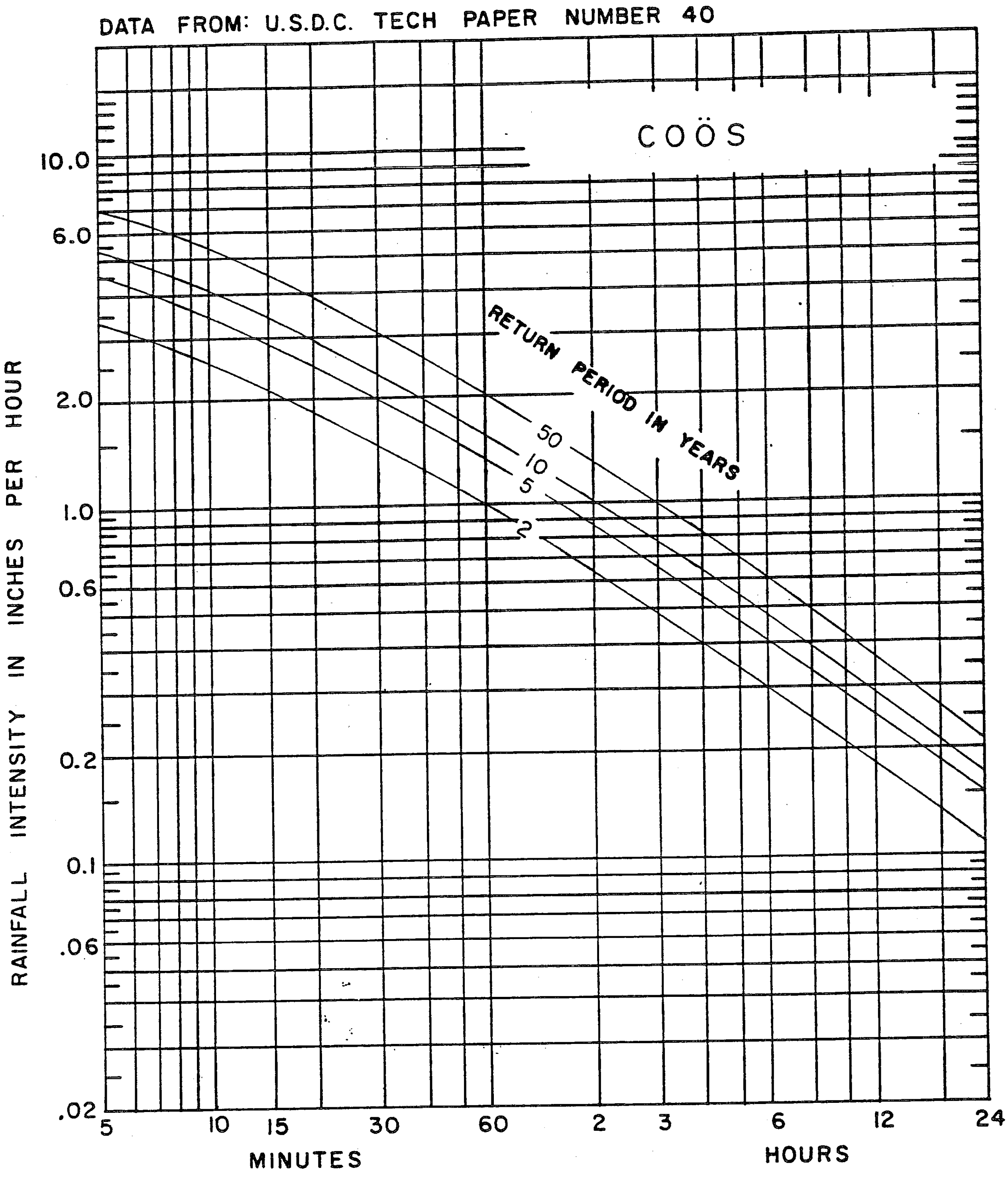
FIGURE 2-4



TIME OF CONCENTRATION

NOTE: METRIC CONVERSION 1 INCH = 25.4 mm
1 mm = .0394 IN.

FIGURE 2-5



TIME OF CONCENTRATION

NOTE: METRIC CONVERSION 1 INCH = 25.4 mm
1 mm = .0394 IN.

C. N.E.H.L. - A.W.M. METHOD

INTRODUCTION

The attached charts giving estimated peak flow runoff are an extension of the several studies made by Public Roads, to wit: "Peak Flows - New England Hill and Lowland, Area B-2", "Peak Flows from Adirondack - White Mountain - Maine Woods, Area B-3" and "Glaciated Sandstone and Shale Area." The two former studies, first distributed in 1957, were found to have greater limitations in their use than the basic data justified because of the mechanical arrangement of the charts. This limitation has been removed. The curves have been extended downward by the use of a limited amount of data for areas less than 390 hectares (1.5 sq. miles).

The area designations in the original studies B2 and B3 are taken from Soil Conservation Problem Maps which are based upon studies of relative erosion, which generally reflects the underlying lithology. These two areas are separated by some relatively narrow corridors. Studies of the runoff in these corridors indicate that runoff there is usually somewhat different than the runoff in the B2 and B3 areas. They have been assigned values in Fig. 2-6 in accordance with their nearest applicable characteristics. This adjustment permits the entire Public Roads' region to be covered by some criteria for estimating runoff, with the exceptions of southern New Jersey and Long Island.

EFFECT OF STORAGE DISTRIBUTION ON RUNOFF

Natural storage on many streams in this area is a major factor in runoff. Ponds, lakes and marshes are indicative of the geologically youthful topography which is extensive in this Region. If the storage is uniformly distributed along a stream the attached curves will apply. If the storage is largely or entirely located just above the point of interest on a stream, the storage will reduce the peak discharge more than either Fig. 2-7 or Fig. 2-8 indicate. The opposite is also true. If the storage is largely located at the headwaters it will not reduce the peak flow to the extent indicated by the use of Fig. 2-7 or Fig. 2-8. The charts are developed from actual runoff measurements at existing stations with storage averaged, and obviously cannot make allowance therein for variations in relationship of storage to other point of interest. The curves themselves fit actual measurements for their derivation only when adjusted for storage location as noted below.

It is not uncommon for storage in the region to be compounded, i.e., one marsh or pond having one or more other marshes or ponds tributary to it. Actual runoff from the lowest marsh or pond in such a series may be only a few second meters per square kilometer. The charts are obviously not valid for such conditions. If such storage is disregarded, unwarranted dollar cost for small drainages may be very large. Allowance for the above conditions can be made as follows:

1. Storage uniformly distributed in watershed.

The charts as they are will give conservative answers to this condition. Passage of the stream through a large marsh or pond at any point, however, may result in storage effects which should be separately considered.

2. Storage in headwaters.

For our use where design is generally only for peak rates of flow, storage which is at the source of a stream may be ignored as contributing drainage area, provided that the remaining area above the point of interest is approximately equal to or greater than the area eliminated. The reason for this is that peak flow from the area below the storage will pass through before appreciable flow from the storage area arrives.

3. Storage immediately above point of interest, including compounded drainage.

This condition has the maximum effect on peak flow. Runoff curves do not apply to outflow from such storage. Outflow must be computed as from a reservoir.

INSTRUCTIONS FOR USE OF CHARTS

The charts needed have been reduced to four. Fig. 2-6 shows the dividing lines between the several areas and the distribution of the precipitation - index P. Fig 2-7 is used for the determination of peak flow, Q. for the Adirondack - White Mountain - Main Woods area (AWM) where storage affects runoff with a storage factor, K, as small as 1%. Fig. 2-8 and Fig. 2-9 cover the New England Hill and Lowland Area (NEHL). Fig. 2-8 takes into account storage where K exceeds 4.5% and Fig. 2-9 is for areas where storage is less than or equal to 4.5%. The measure of storage is specifically stated below.

To determine peak runoff proceed as follows:

1. Determine the watershed area, taking into account the effect of storage as noted above. Although any accurate drainage map may be used, the U.S. Geological Survey topographic maps are recommended as they are best adapted to the determination of the storage index K. Locate the centroid of the watershed area by eye.

2. Using this centroid point on Chart 1, determine the area in which the drainage area is located and the precipitation index P, for that watershed. If the watershed is located on the boundary between areas the guide for selection of the curve to use is the topographic characteristic of the area. Prevalence of storage along the valley indicates the choice of the AWM curve, since storage is a factor in practically all streams in that area. Essentially narrow valleys indicate the use of the NEHL curve.

3. Determining the storage index K. This is the percentage of the total area covered by swamps, lakes, reservoirs, or valleys. Large storage exists in all streams. This consists of the volume which fills the channels and flood plains below flood crest.

   In valleys not wider than 0.4 kilometer (0.25 mi.), this storage is included in the best curves. Where the average width of the valley bottom exceeds 0.4 kilometers (0.25 mi.), a determination of the area that may be assumed to be covered by an overbank flow should be made from the topographic map. This flooded area should be taken at the total area of the valley floor where its average width exceeds 0.4 kilometer (0.25 mi.). Where railroads or major highways have been constructed on the valley floor, it may be assumed that their embankments limit the extent of lateral flooding and they may be taken as one boundary of the flooded area. The sum of the areas of all swamps, lakes, reservoirs and above flood plains divided by the area of watershed, expressed in the same unit, multiplied by 100, is the storage index K. The further effect of storage distribution has been discussed above. (In measuring the areas, a considerable saving of time results from tracing the boundaries of these areas on tracing paper into one composite area, which can then be planimetered in one operation.)

4. To find the peak runoff where affected by storage, enter the appropriate area chart (Fig. 2-7 or Fig. 2-8) at the bottom with the actual or adjusted drainage area and proceed as outlined thereon through the appropriate precipitation index and storage curves to the Q_{50} curve and left to the runoff for a 50-year frequency, which is the design criteria. For the few cases where a 10-year frequency may be considered, peak flow for this frequency may be found by proceeding vertically from the turning point on the Q_{50} curve. In the New England Hill and Lowland area natural storage below 4.5% appears to have no discernible affect on runoff. In such cases Fig. 2-9 should be used, in which drainage area, precipitation index P and runoff Q are the only elements involved. Use of this chart is similar to the use of Fig. 2-8, and is shown for a given watershed area.

   The method of estimating runoff in the Glaciated Sandstone and Shale Area cannot be readily translated into the method used in the AWM and NEHL Areas. It should, therefore, be used as originally published in Public Roads, Vol. 28, April 1954.

   The complex computations involved in storage for power and water supply are generally unnecessary for our purposes and will seldom arise in drainage areas of 65 kilometers (25 sq. mi.) or less. On areas where the point of interest is below such a condition, remaining flow characteristics affected by such construction can generally be obtained from the agency responsible for those structures.

U.S. Bureau of Public Roads
Region One
October 1960

NOTE: METRIC CONVERSION 1 in = 25.4 mm
1 ft = 0.3048 m

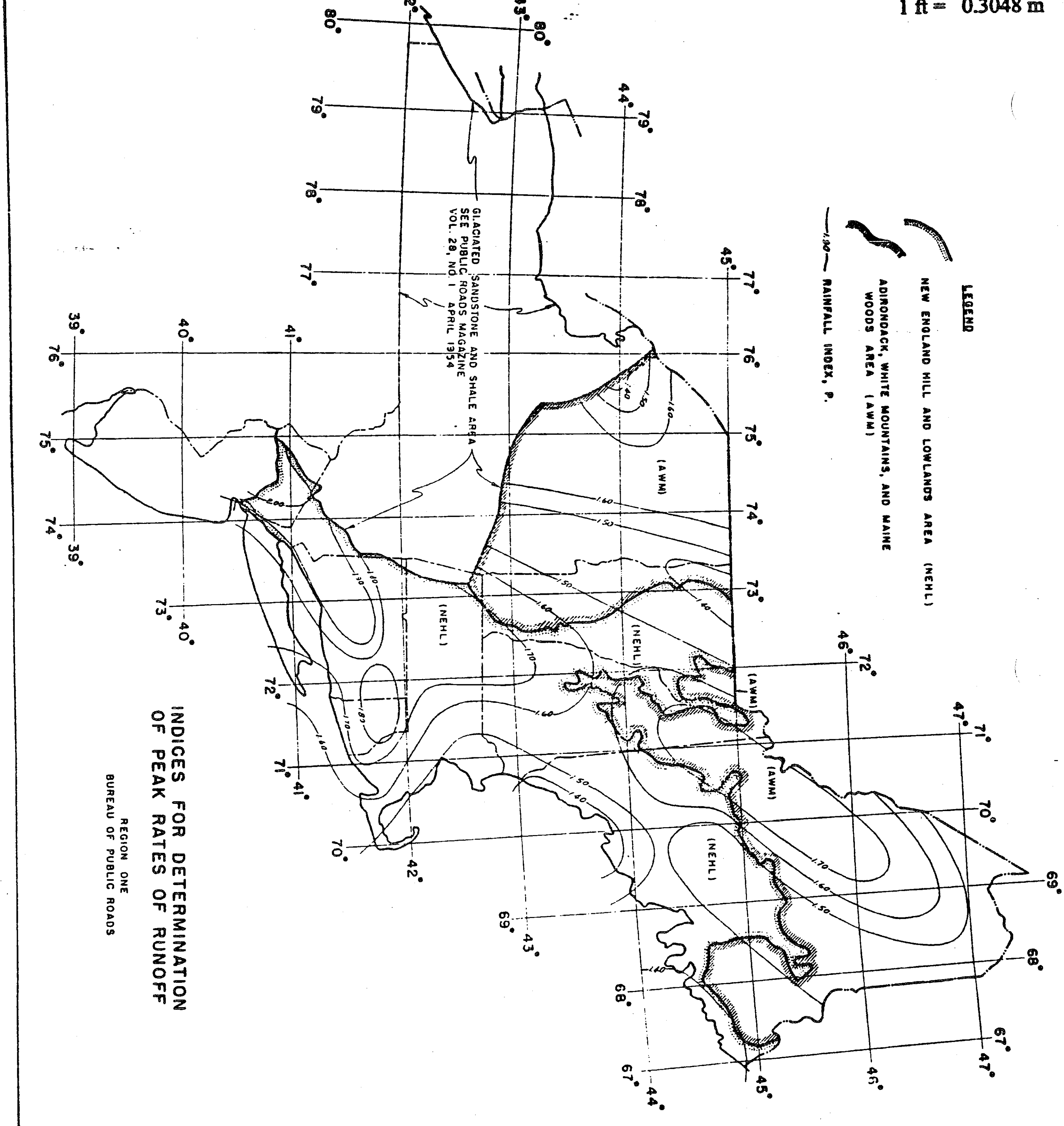
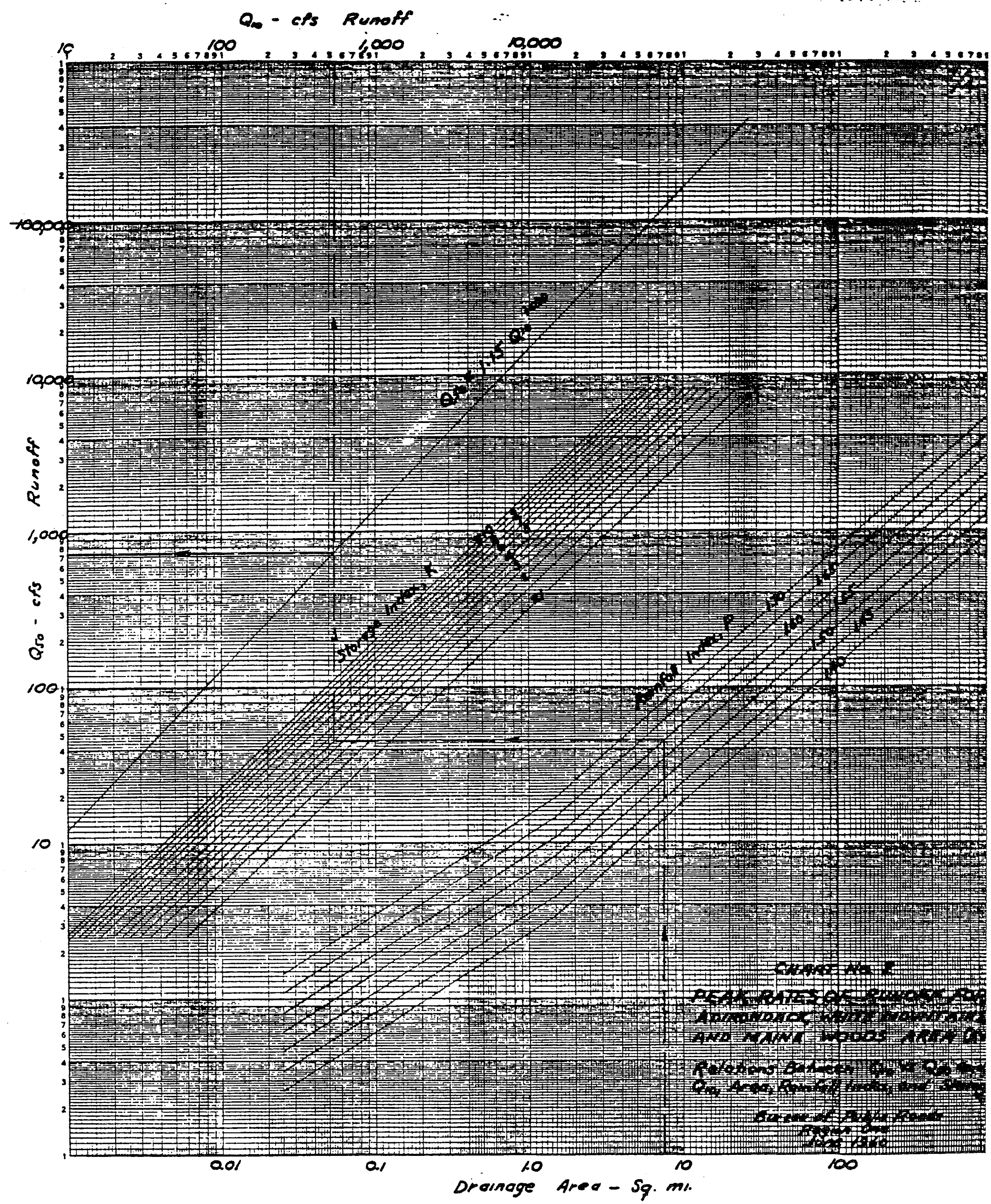
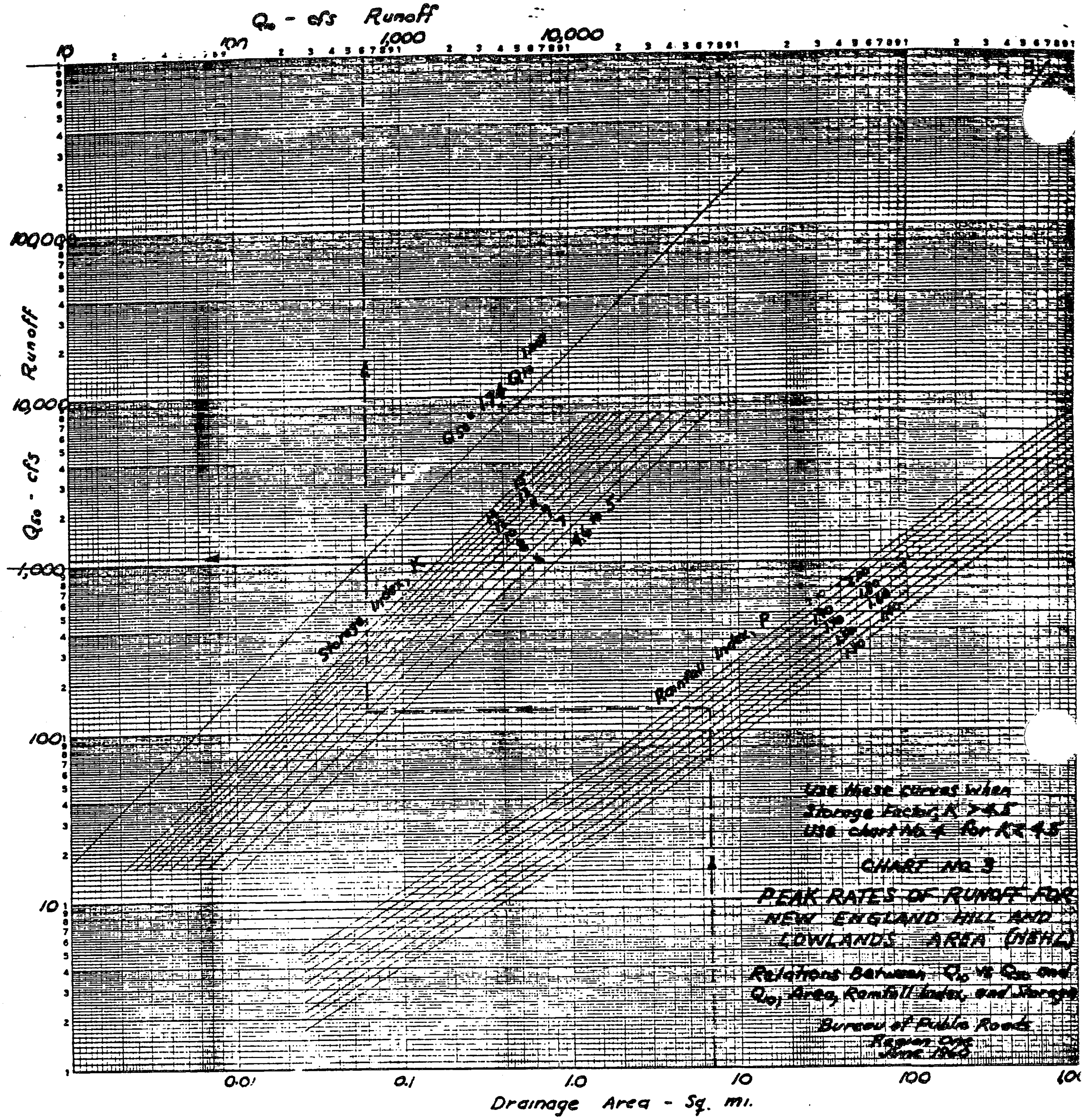


FIGURE 2-6



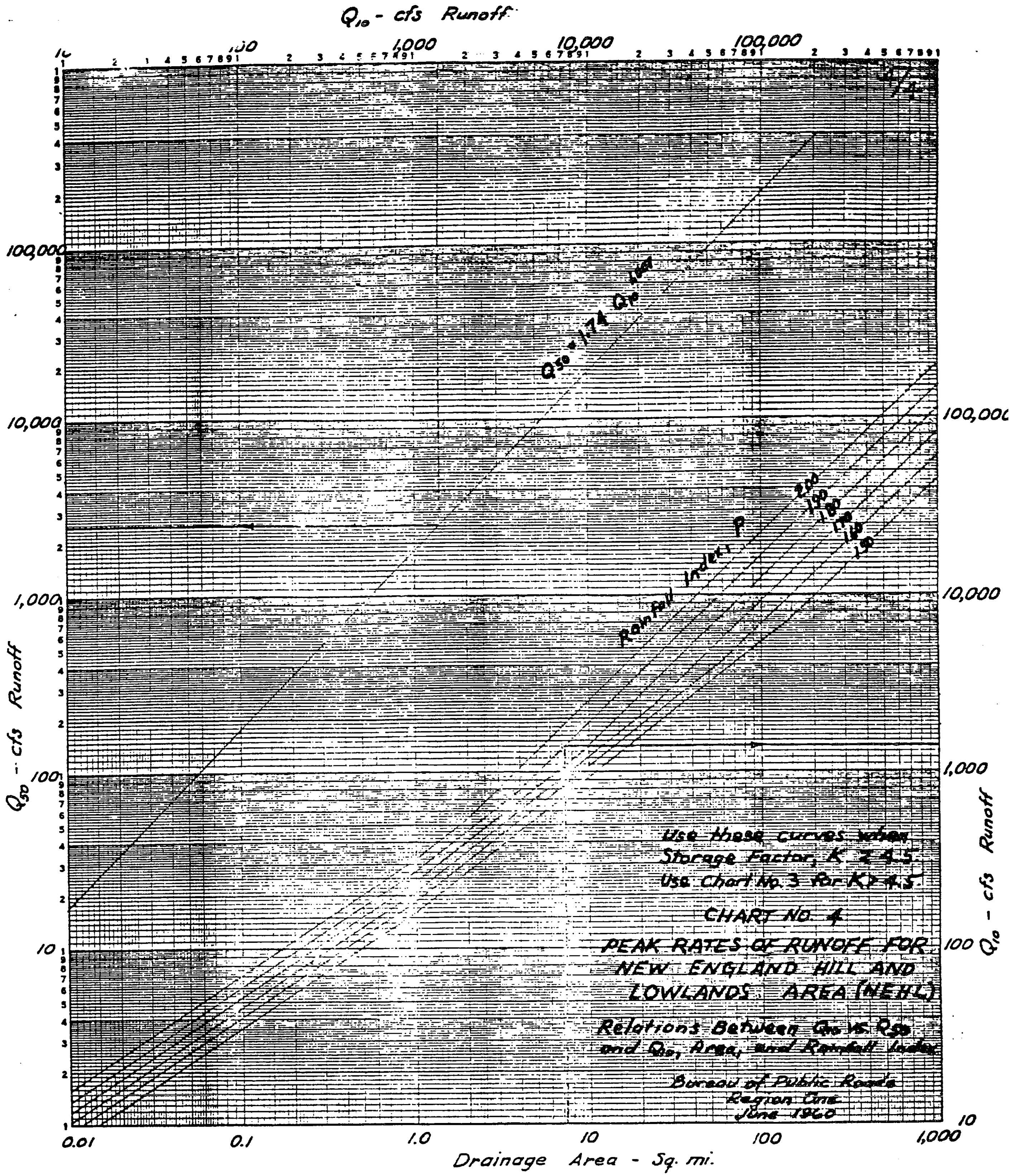
NOTE: METRIC CONVERSION 1 inch = 25.4 mm
1 ft = 0.3048 m

FIGURE 2-7



NOTE: METRIC CONVERSION 1 in = 25.4 mm
1 ft = 0.3048 m

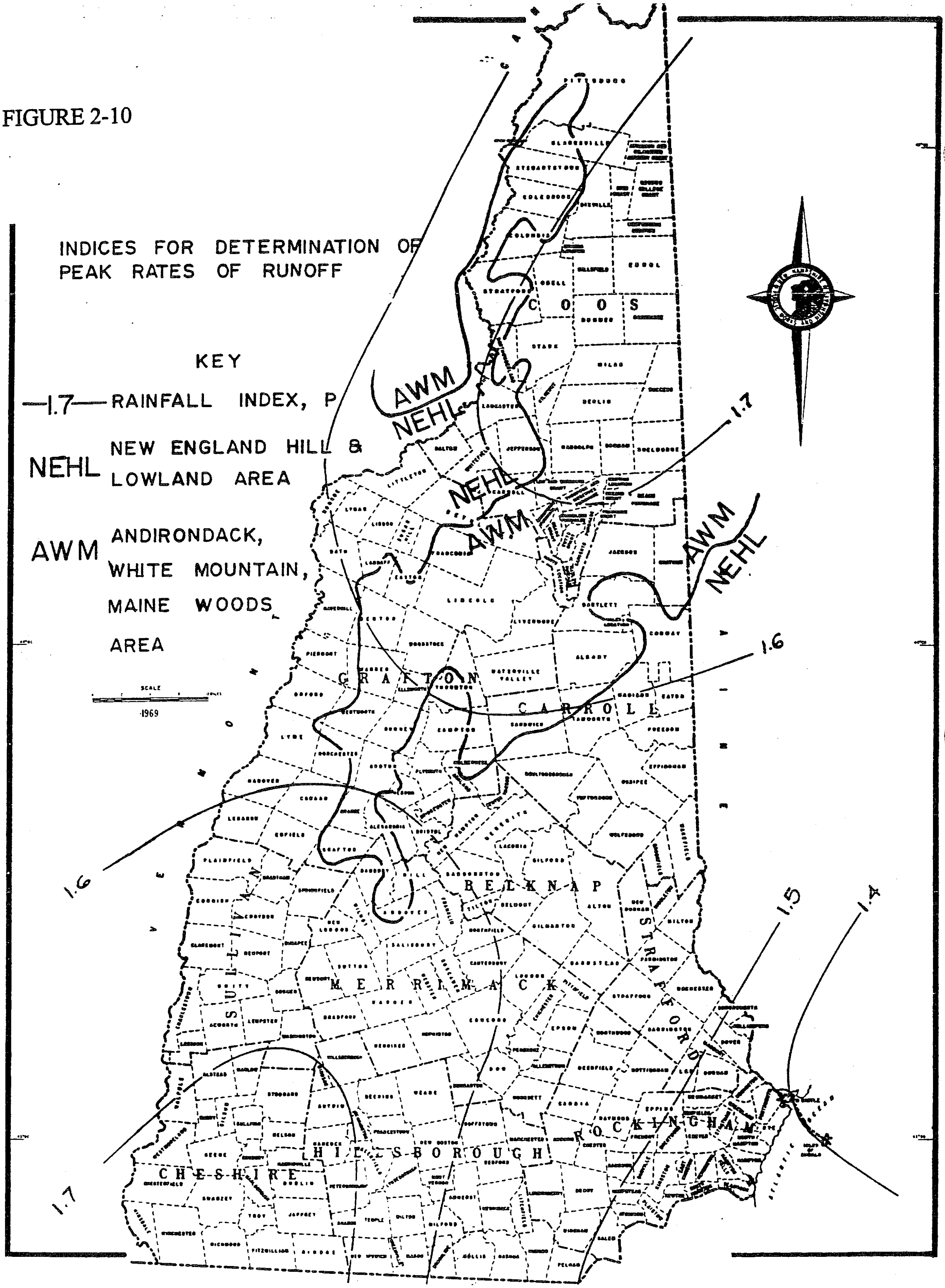
FIGURE 2-8



NOTE: METRIC CONVERSION 1 in. = 25.4 mm
1 ft. = 0.3048 m

FIGURE 2-9

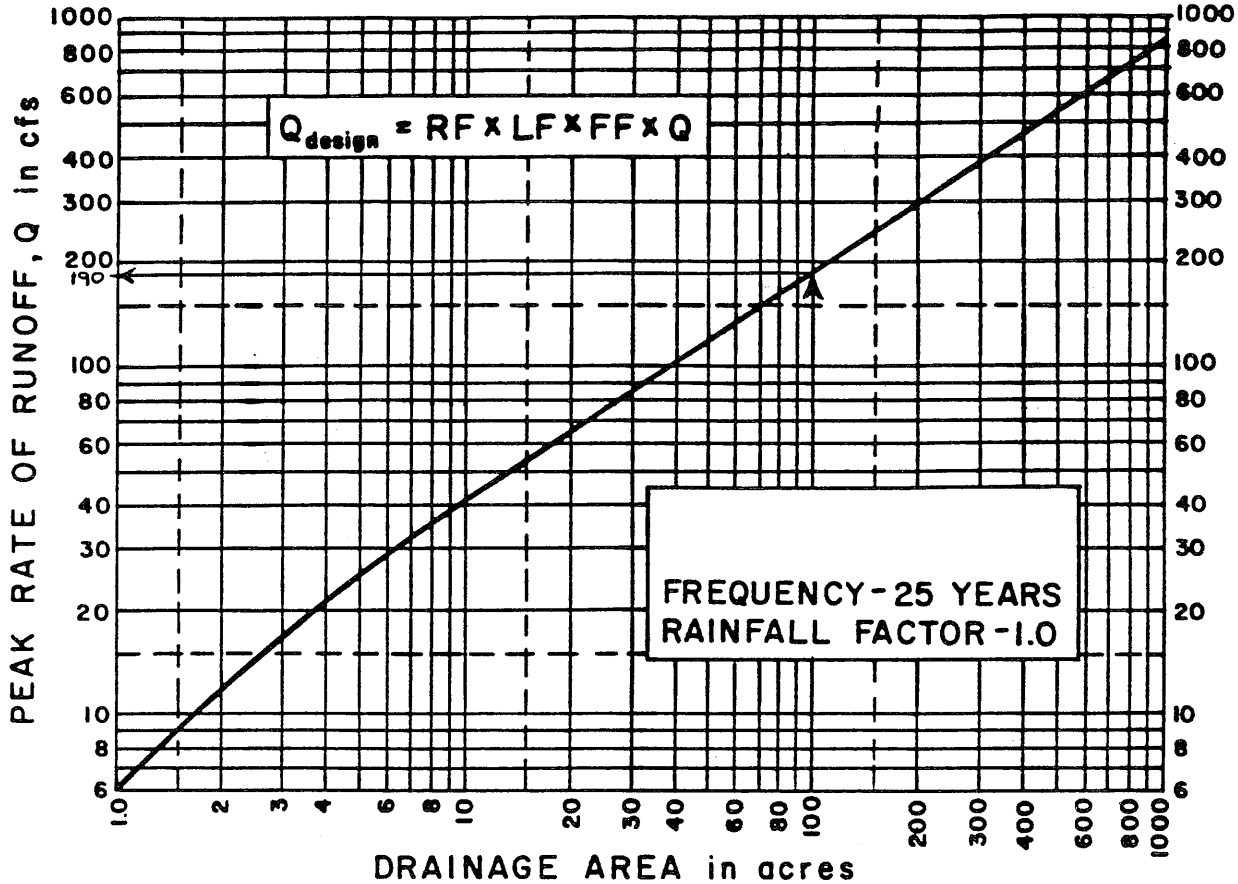
FIGURE 2-10



NOTE: METRIC CONVERSION: 1 INCH = 25.4 mm
1mm = 0.0394 IN.

FIGURE 2-11

POTTER'S METHOD



RAINFALL FACTOR (RF): SEE FIG. 2-12

LAND USE AND SLOPE FACTORS (LF)

Land Slope	Steep over 2%	Flat 0.2%	Very Flat, no ponds
100% Cultivated (row crops)	1.2	0.8	0.25
Mixed cover	1.0	0.6	0.2
Pasture	0.6	0.4	0.1
Woods, deep forest litter	0.3	0.2	0.05

FREQUENCY FACTORS (FF)

Frequency, years	5	10	25	50
Factor	0.6	0.8	1.0	1.2

EXAMPLE

100 acres near Nashville, Tenn., cultivated land sloping about 0.5%, design frequency 10 years

Solution: (see equation on graph)

Q_{10} = 1.2 x 0.9 x 0.8 x 190
= 170 cfs

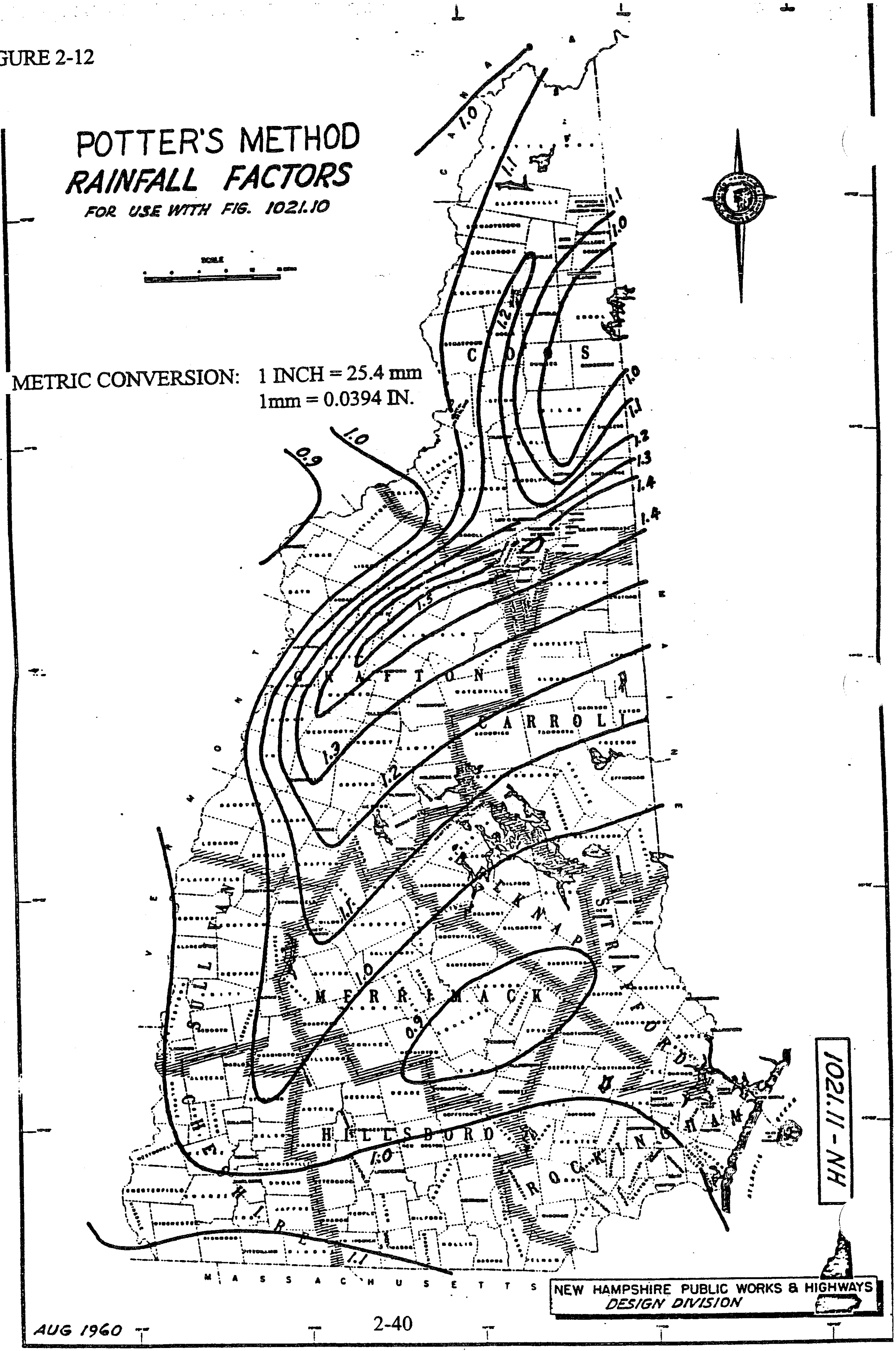
ACCURACY OF BASIC DATA DO NOT JUSTIFY CARRYING MORE THAN TWO SIGNIFICANT FIGURES

SOURCE: ***Derived in part from Potter "Surface Runoff from Small Agricultural Watersheds", Research Report No.II-B, Highway Research Board 1950; Land use and slope factors for flat and very flat land slopes are estimated and subject to revision when observed data become available***

PEAK RATES OF RUNOFF WATERSHEDS UNDER 405 HECTARES (1000 ACRES)

 NOTE: METRIC CONVERSION: 1 INCH = 25.4 mm
1mm = 0.0394 IN

FIGURE 2-12



NOTE: METRIC CONVERSION: 1 INCH = 25.4 mm
1mm = 0.0394 IN.

SECTION 3 - HYDRAULICS OF STORM SEWERS

A. GENERAL

The design of a storm sewer is often of considerable complexity and merits extensive study and sound engineering judgment. Even though no specific set of rules can be laid down as a recipe, a generalized procedure is helpful if the designer is conscious of the various problems that can occur in the system design. The three broad steps that characterize the design method are:

1. Estimating adequate pipe sizes and design discharges at points in the system, from upstream to downstream (Generally computed using the Rational Method);

2. Computing the water surface profile or hydraulic gradeline for the system;

3. Comparing the total time of flow in step 1 with the total time of flow from step 2 to determine if the preceding calculations are accurate enough.

The actual design of the storm drain system is considerably facilitated by the use of two forms for tabulated and calculated data. These forms are shown as fold-out sheets, and sample computations have been worked out on them. The sheets correspond to the plan and profile of Figures 3-1 and 3-2 for a fictitious system. By referring to these sheets, the reader may more easily follow the design procedure given below.

B. EXAMPLE PROBLEM

In this particular system, it is assumed that the field data indicates no special problems, and that all inlets are adequate for the design runoff (in real situations, adequate grate capacity must be verified at each collection point). The minimum cover above pipes has been chosen to be about 1.5 meters. Catch basins have been provided at the locations indicated and care has been taken to not exceed a pipe length of about 90 meters (this is a reasonable maximum length for maintenance purposes). Drainage areas have been estimated, along with their corresponding runoff coefficients, and the pipe type has been designated, along with its corresponding roughness coefficient. It is assumed that design can now be accomplished by simply choosing adequate pipes and checking the water surface profile. This procedure is carried out below.

The first entries on form SS-1 (Figure 3-3) locate the section of pipe under consideration The plan and profile for the area can be used to determine the length (L), the flowline elevations of the pipes, and the pipe slopes.

Next, the drainage area (A_d) is entered along with its corresponding runoff coefficient (C), the product CA_d is computed, and the drainage areas are depicted on a plan. If an area has a composite C-Value, either a weighted C is used or else the product CA is calculated for each sub-area, Then all of the CA_ds draining to the location under study are summed up and entered as ΣCA_d.

The time of concentration (the time required for a particle of the runoff to reach the location from the farthest point of the drainage area) is now determined. For subsequent lengths of pipe along the same line, the time of concentration (t_c) accumulates. Where two or more lines meet, such as at the junction of CB 3, the longest t_c is used for succeeding calculations. For the first drainage area, a minimum t_c of ten minutes should be used. Other than this minimum value, the determination of a reasonable value is a guess based on experience. (REF 6 contains a nomograph for estimating t_c . This nomograph is based on meager experimental data, so it should only be used when better information is not available.)

The rainfall intensity is now determined from the appropriate rainfall intensity-duration frequency curve. This system is designed for a ten-year return period. The discharge is then the product,

$$Q = i \times CA_d.$$

Next, a pipe is chosen such that the normal depth of flow does not exceed the pipe height. This choice is based on an approximation that the flowline slope is almost equal to the slope of the hydraulic gradeline. The normal velocity can be easily found by using the pipe flow charts. By dividing the length of pipe (L) by the normal velocity (V_n), an approximation to the time of flow through the reach of pipe is made, and this time is added to the previous t_c. Computations are then made for succeeding lengths of pipe by methods similar to those explained above.

Example: Pipe sizing

Given: $Q = 25$ cfs
$s = .015$
$n = .012$

Find: V_n, D of pipe

Using Table 3-A below,

$V_n = 10.6$ fps for 24" D pipe
Normal depth of flow = 1.4 ft

PIPE FLOW CHART 24-INCH DIAMETER

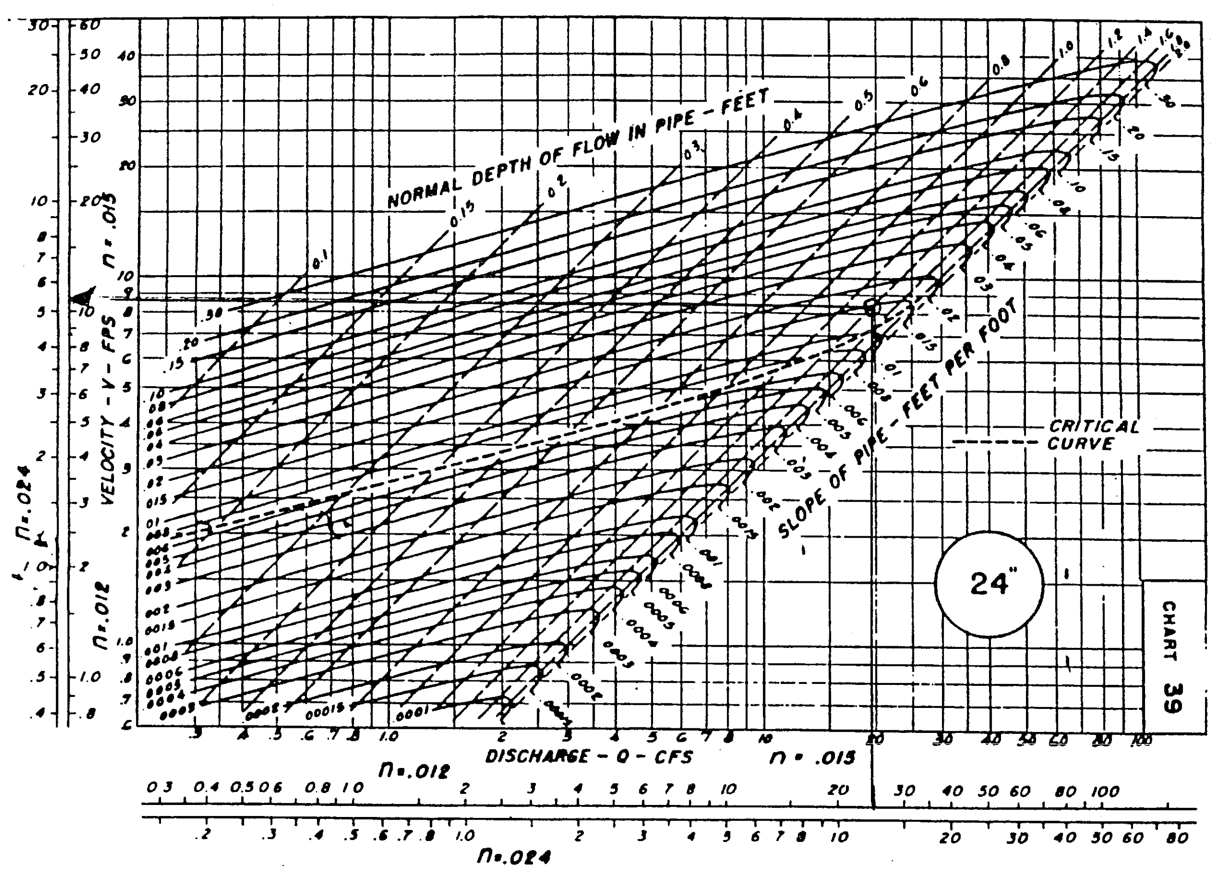


Figure 3-1
PLAN OF GRUNCHVILLE
STORM DRAIN SYSTEM

SCALE 1:1250

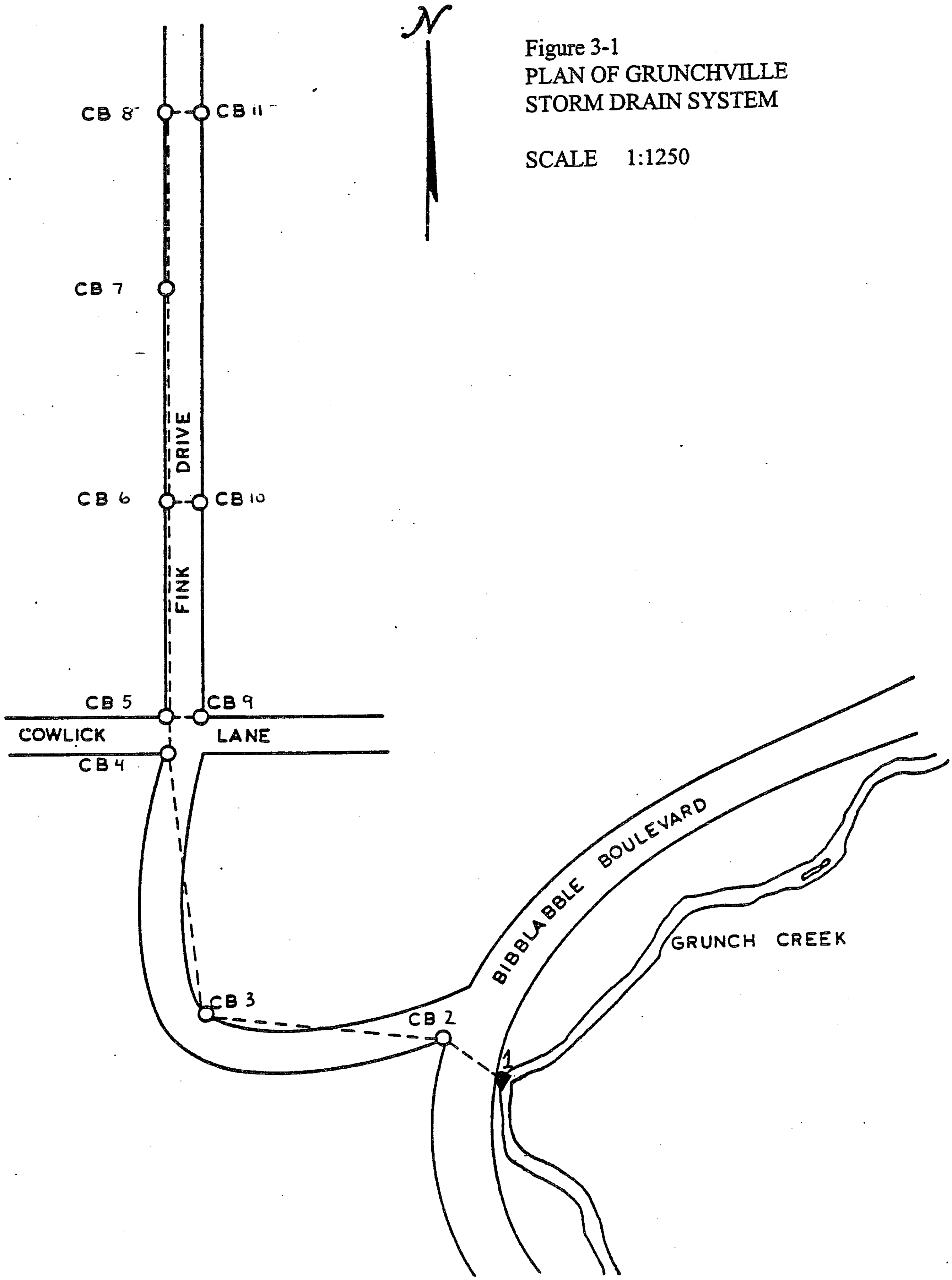
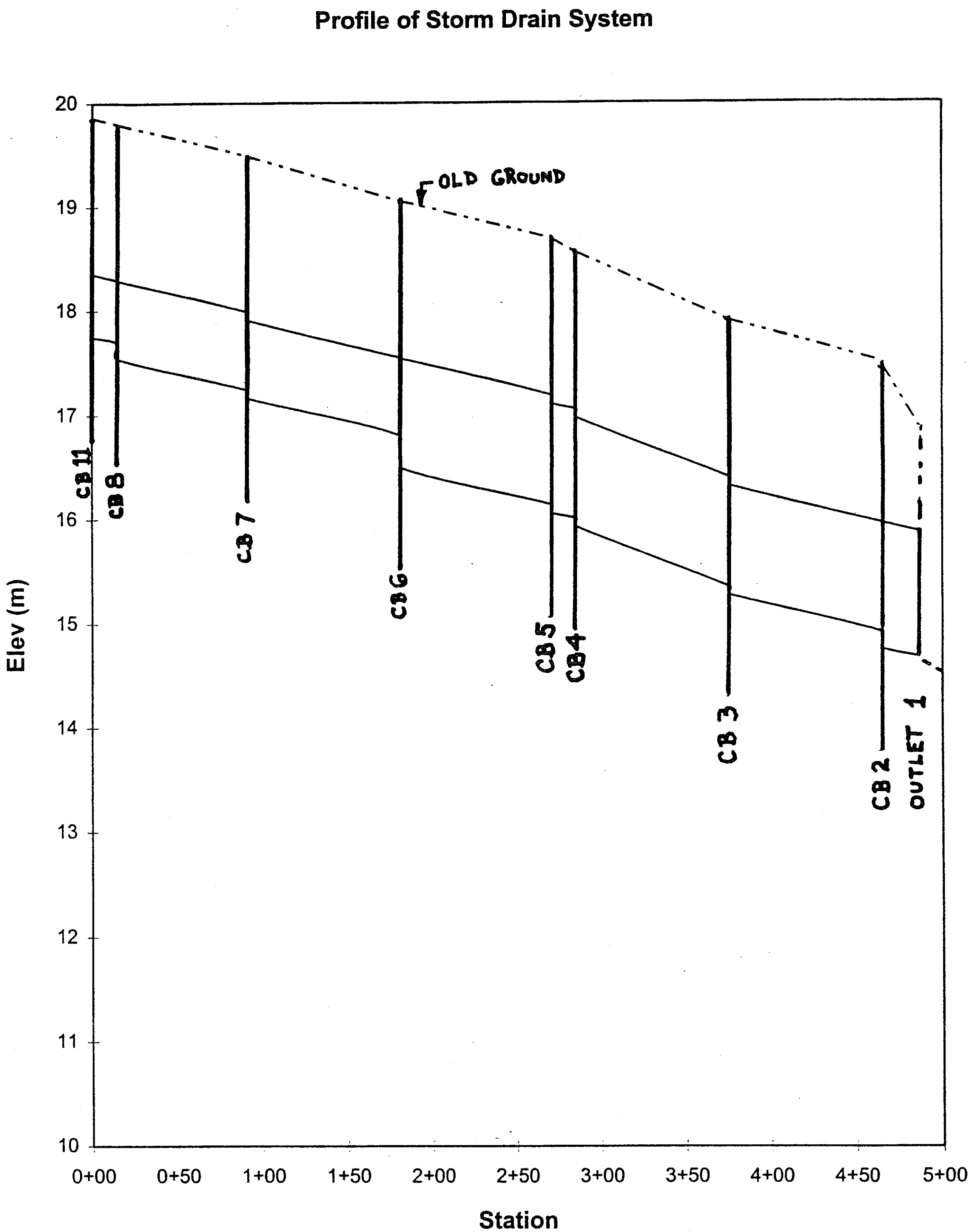


Figure 3-2



Profile of Storm Drain System
20
19
18
17
16
15
14
13
12
11
10
Elev (m)
OLD GROUND
CB 11
CB 8
CB 7
CB G
CB 5
CB 4
CB 3
CB 2
OUTLET 1
0+00
0+50
1+00
1+50
2+00
2+50
3+00
3+50
4+00
4+50
5+00
Station

FORM SS-1 Figure 3-3

Calculations for runoff determination										Comp. by U.R. Alwet		Date 1/29/98				Page 1	
Project No. GS-1000			Location: Fink Drive-Grunchville							Chekd. by I.M. Nott		Date 2/1/98				of 2	
From Pipe Inlet		To Pipe Outlet			Through Reach			(CA_d) (i) / 360 =		Q		Through Reach					
	Flowline		Flowline	A_d	C	CA_d	CA_d	t_c	I	Q	Pipe Size	Q_{full}	S_o	V_n	L	Flow	
Location	Elevation	Location	Elevation	hectares		hectares	hectares	min	mm/hr	cms	n = (.012)	cms	m/m	m/sec	m	min	
CB 11	17.753	CB 8	17.693	3.24	0.3	0.97	0.97	10.00	119.38	0.32	600	0.45	0.0040	1.64	15	0.15	
CB 8	17.541	CB 7	17.237	4.05	0.3	1.22	2.19	10.15	114.30	0.69	750	0.81	0.0040	1.98	76	0.64	
CB 7	17.161	CB 6	16.801	1.21	0.5	0.61	2.79	10.79	111.76	0.87	750	0.81	0.0040	1.98	90	0.76	
CB 10	17.013	CB 6	16.953	3.36	0.5	1.68	4.47			0.00	600		0.0040		15		
CB 6	16.496	CB 5	16.136	1.62	0.5	0.81	5.28	11.55	109.22	1.60	1050	1.90	0.0040	2.44	90	0.61	
CB 9	16.577	CB 5	16.517	2.10	0.5	1.05	6.33			0.00	600		0.0040		15		
CB 5	16.060	CB 4	16.000	1.62	0.5	0.81	7.14	12.16	106.68	2.12	1050	1.90	0.0040	2.44	15	0.10	
CB 4	15.924	CB 3	15.348	0.00	0.5	0.00	7.14	12.27	106.68	2.12	1050	2.52	0.0064	3.29	90	0.46	
CB 3	15.272	CB 2	14.912	1.82	0.5	0.91	8.05	12.72	104.14	2.33	1050	1.90	0.0040	2.44	90	0.61	
CB 2	14.762	1	14.678	2.47	0.5	1.24	9.29	13.34	101.60	2.62	1200	2.80	0.0040	2.74	21	0.13	
OUT. 1	14.678	Outlet to Treatment Area															
Rainfall intensities for Rockingham County																	

C. GRATE CAPACITY

Grate capacity is defined by the amount of water intercepted by the grate under given conditions. Grate inlets will intercept all of the frontal flow (gutter flow passing over the grate) if the grate is sufficiently long and flow velocity is low. As the gutter flow velocity increases, the grate capacity will decrease[1]. Grate capacity design curves have been constructed to assist in approximating grate capacities for different roadway situations. The information in this section was taken from the Hydraulic Engineering Circular No. 12 (HEC 12) and information provided by the Neenah Foundry Co.[2]. For further design assistance and information refer to the HEC 12 manual.

1. Ref., Johnson, Frank L. & Change, Fred F.M., 1984, Hydraulic Engineering Circular No. 12, Federal Highway Administration, McLean, Virginia, 136 pgs.

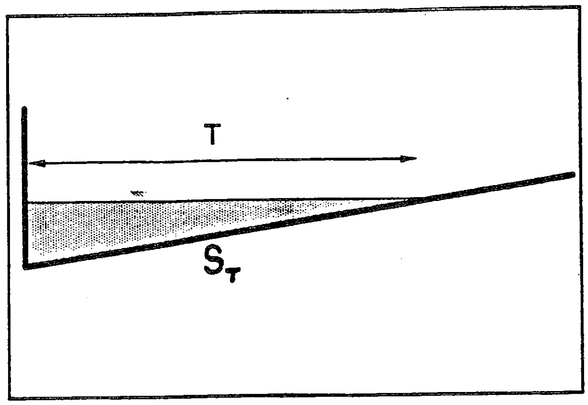
2. Ref., Nennah Foundry Company, 1987, Inlet Grate Capacity for Gutter Flow and Ponded Water, Neenah Foundry Company, 58 pgs.

Determination of Inlet Capacity & Spacing

Method applicable for Type A, B, and F Grates.

Example: Type A Grate & Type B Grate

Given: Contributing area of pavement consists of two 3.6m lanes and two 3.0m shoulders with vertical curb. Location: Farmington, NH (Strafford County)
S_L = Longitudinal Slope = 0.008 m/m
S_T = Transverse Slope = 0.02 m/m
Permissible spread on pavement = 3.0m (water cannot encroach upon traveled way)
Depth at curb, D = 3.0m x 0.02 = 0.06 m



Step 1:

Spread = 3.0m
Depth of water at curb = spread x slope = 3.0 x 0.02 = 0.06m

maximum gutter capacity, $Q = (0.38/n)S_T^{1.67}S_L^{0.5}T^{2.67}$ (Metric)

$Q = (0.56/n)S_T^{1.67}S_L^{0.5}T^{2.67}$ (English)

[Q can be determined graphically using Fig. 3-4]

n = 0.016 (Pavement)
T = Permissible spread in meters

$Q = 0.058\ m^3/s$

Step 2:

Determine capacity and spacing of first inlet on slope.

Pavement area that will produce equivalent discharge from equation:
Q = (CiA)/360

A = (360Q)/(Ci) C = 0.95 (pavement)
i = 150 mm/hr

A = 0.147 ha

Length to first inlet:
L = A/W W = width of roadway = 13.2 m

Length to first inlet on slope, L = (0.147 x 10 000 m^2/ha)/13.2 = **111 m**

Step 3:

Using a Type A grate
K = 18.5 (from Figure 3-4a)

Inlet capacity, $Q = 0.212KD^{1.67}$ (Metric)

D = depth at curb = 0.06 m
$Q = 0.036\ m^3/s$
$Q_{bypass} = 0.058 - 0.036 = \mathbf{0.022\ m^3/s}$

Using a Type B grate
K = 17.8 (from Figure 3-4a)

$Q = 1.05KD^{1.67}$ (English)

D = depth at curb = 0.06 m
$Q = 0.034\ m^3/s$
$Q_{bypass} = 0.058 - 0.034 = \mathbf{0.024\ m^3/s}$

Step 4:

Length to next inlet:

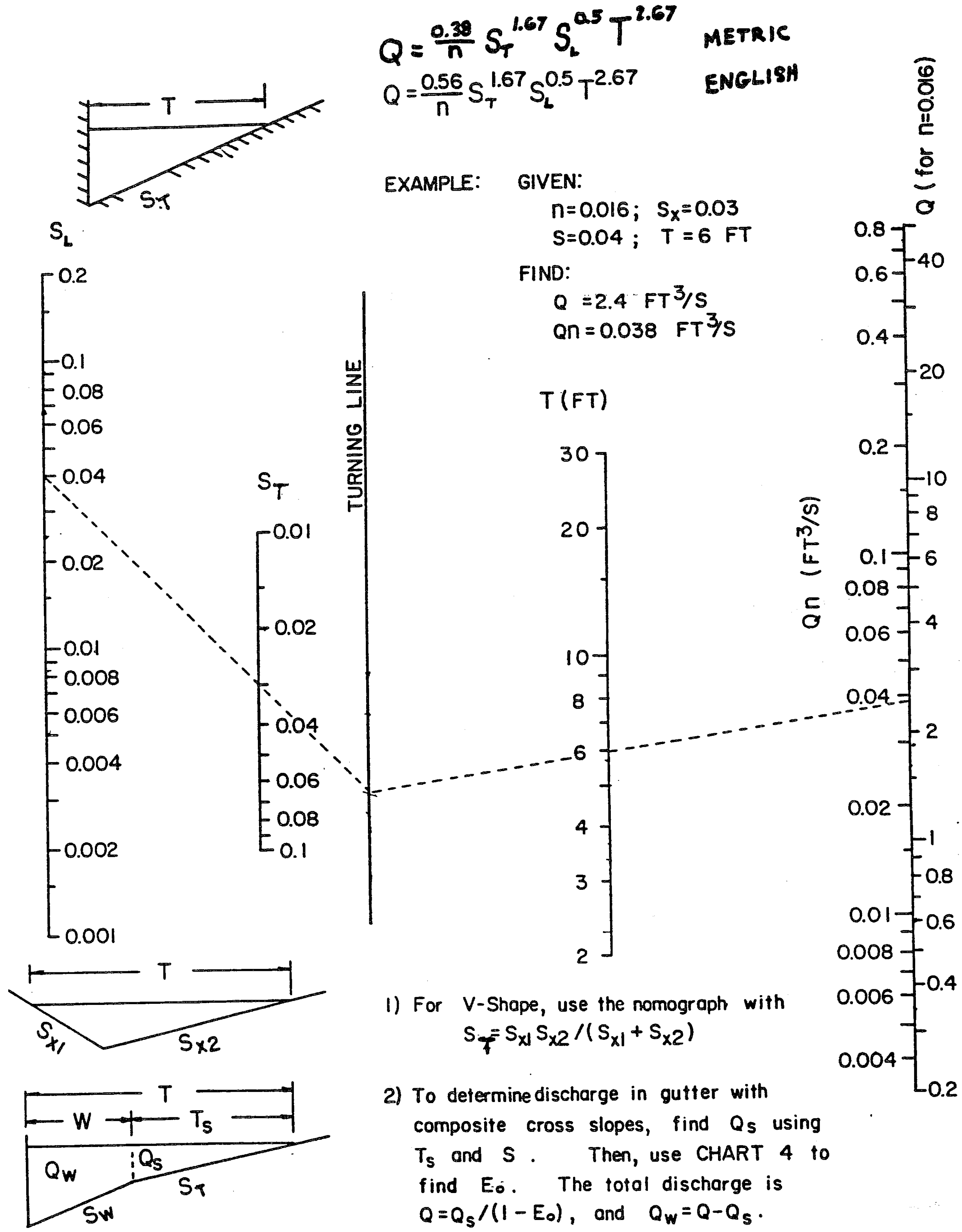
Using: $Q = CiLW/(360 \times 10{,}000\ m^2/hectare)$

$Q = 0.036\ m^3/s$ [Intercepted Flow for given basin] $Q = 0.034\ m^3/s$

$L = 360Q/(iCW)$ $L = 360Q/(iCW)$

$$L = \frac{(0.036)(360)(10\ 000)}{(0.95)(150)(13.2)}$$

L = **69 m** Distance to next inlet L = **65 m**



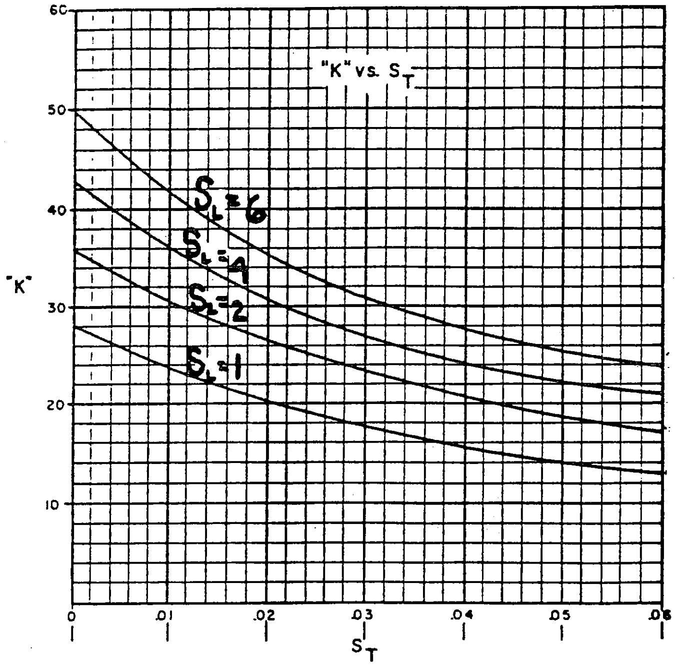
Metric Conversion: 1 ft. = 0.305 m

$1\ ft^3/s = 0.028\ m^3/s$

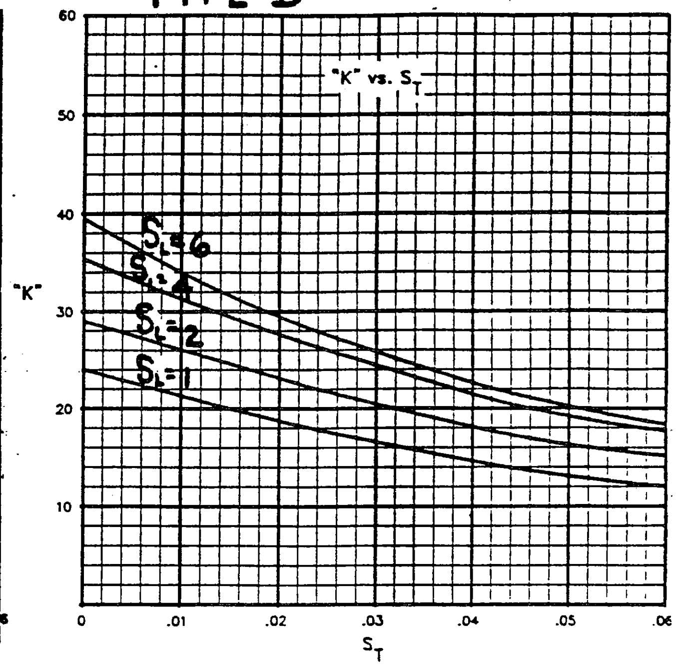
Figure 3-4

FIGURE 3-4a

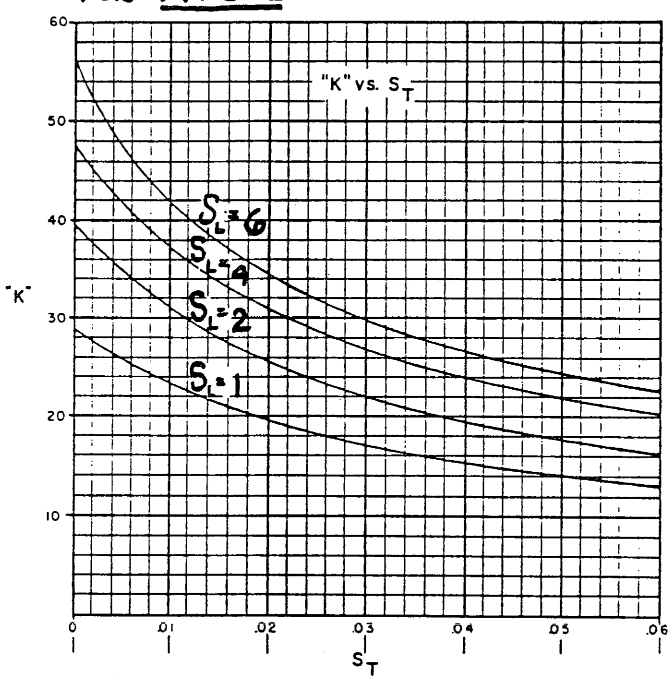
TYPE A



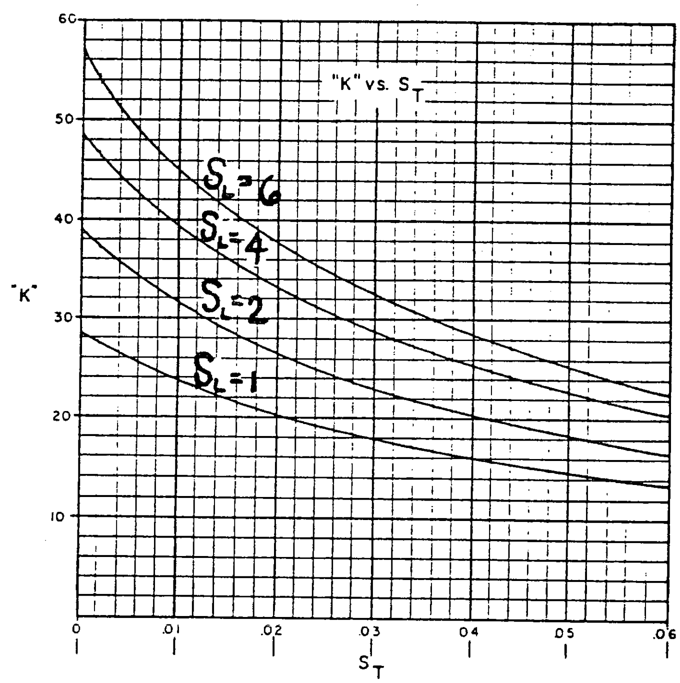
TYPE B



APPROXIMATION FOR TYPE E



TYPE F



S_T = TRANSVERSE GUTTER SLOPE
S_L = LONGITUDINAL GUTTER SLOPE
K = GRATE INLET COEFFICIENT

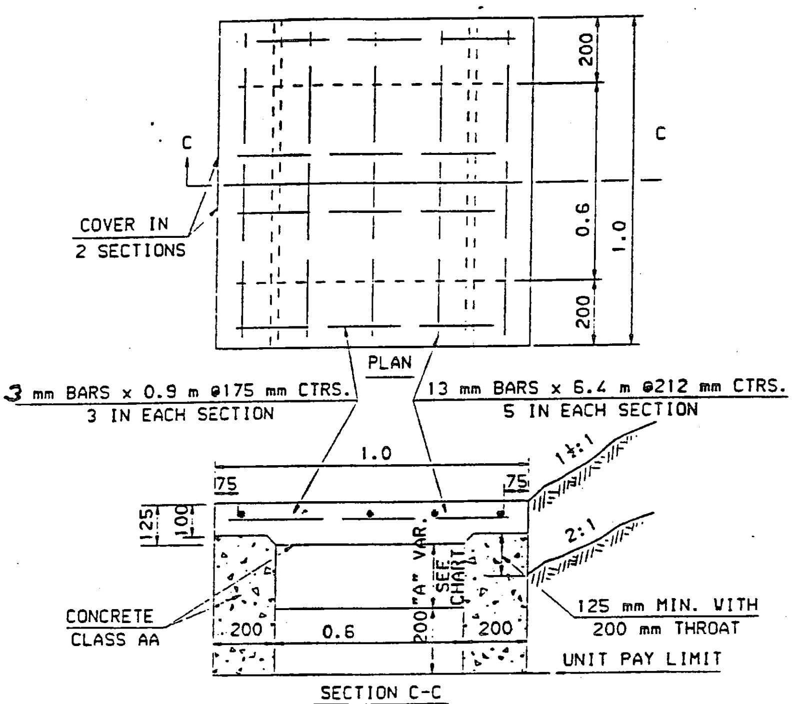
NEENAH FOUNDRY Co. 1987

Grate Capacity

Method used for Type C grate

$Q = CLH^{1.5}$

$C = 1.8$ (3.3 English)
L = Length of opening (usually two sided) (m)
H = water height (head) (m)

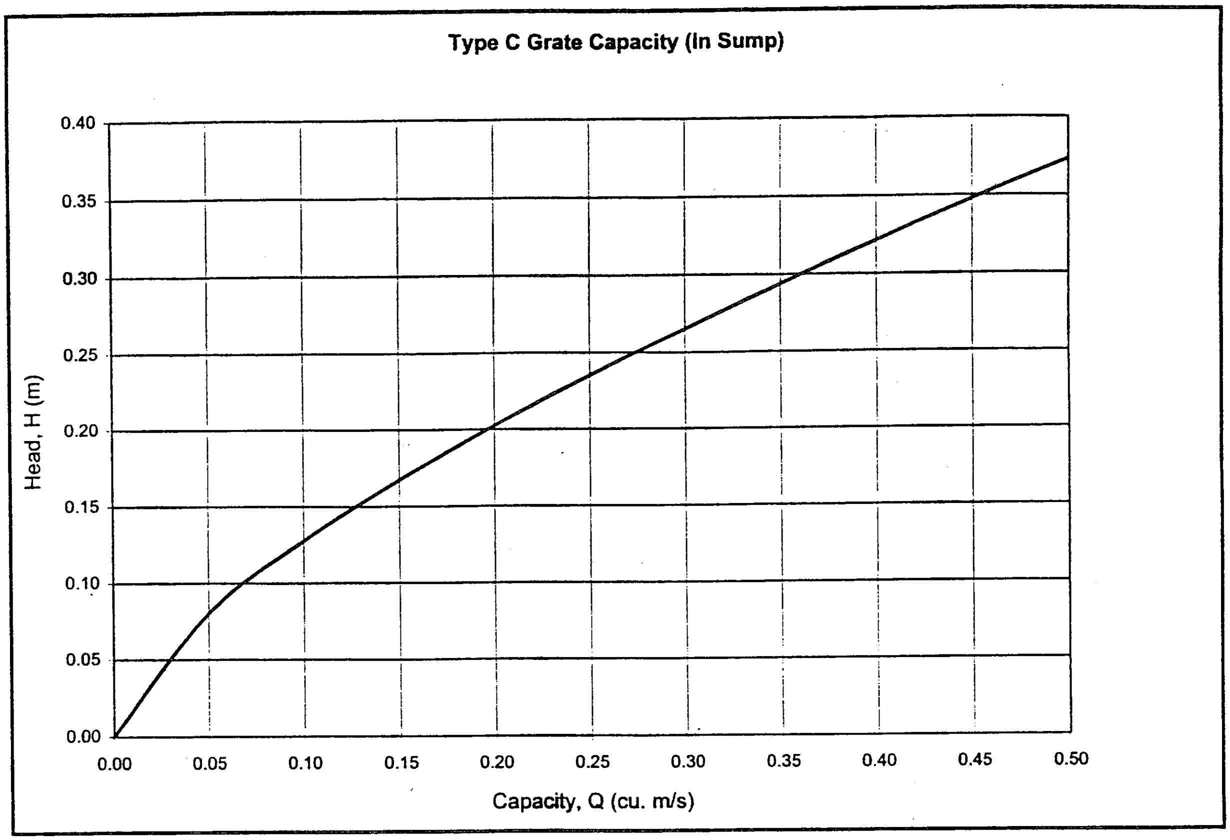


TO BE USED IN BERM DITCHES AND AT LOCATIONS
INACCESSIBLE TO VEHICULAR TRAFFIC

PIPE DIA "D" MILLIMETERS	THROAT DEPTH "A" MILLIMETERS	
	ONE THROAT	TWO THROAT
300	200	200
350	200	200
375	200	200
400	225	200
450	400	200
500	400	225
600	450	400

IN A SERIES OF CONNECTIONS C.B.S OR D.I.S, THE OUTLET PIPES MAY INCREASE IN DIAMETER, BUT THE SURFACE THROAT OPENINGS ARE NOT AFFECTED.

Figure 3-5



<u>**Grate Capacity Example**</u>:

Type C Grate (Total Open Area on Both Sides= 1.2 m)

Depth of water at inlet = 175 mm

Find: Flow entering inlet

From Figure 3-5 $H = 0.175$ m
$Q = 0.16\ m^3/s$

Grate Inlet in Sump Location

Grate inlets act as weirs up to a certain water depth and are dependent upon the open area of the inlet. At greater water depths these drop inlets will act as an orifice.

Capacity in a weir situation:

$$Q = C_W P d^{1.5}$$

P = perimeter of the grate discharging bars and the side against the curb (m)
C_W = 1.8 (3.3 English)
d = depth of water at curb (m)

Capacity in an orifice situation:

$$Q = C_o A (2gd)^{0.5}$$

A = clear open area of grate (m^2)
g = 9.80 m/s^2 (32.2 ft/s^2)
C_o = orifice coefficient = 0.67

Figure 3-6 was created from these equations to simplify the sizing procedure and the following example will show how this chart is utilized.

Example:

Given:

Sag vertical curve with equal flow from both directions. Allow for 25% clogging.
$Q_{by\text{-}pass} = 0.1\ m^3/s$
$Q_{10} = 0.23\ m^3/s$
$Q_{50} = 0.31\ m^3/s$
T = spread = 3.0 m
S_x = cross slope = 0.05
d = depth of water at curb = 0.15 m

Find:

Grate size for Q_{10} and Q_{50}.

Using Q_{10} and d in Figure 3-6 it is found that P = 2.44 m

To account for 25% clogging, the area is reduced by 25% but the perimeter is affected by a lesser amount. For example a Type A grate (606 mm x 606 mm) is 25% clogged. The clogged width is 152 mm which will reduce the area by 25% but will only reduce the perimeter by 17% (454 + 454 + 606).

Installation chosen is two Type A grates.
P = 606 + 606 + 454 + 454 = 2120 mm = 2.12 m
A = 1212 x 454 = 550248 mm = 0.55 m^2

Actual spread for Q_{10}
d = 0.155 m from Figure 3-6 using Q_{10} and A.
T = d/S_x = **3.1 m**

Actual spread for Q_{50}
d = 0.186 m from Figure 3-6 using Q_{50} and A.
T = d/S_x = **3.7 m**

Helpful Hints:

1 m = 3.2808 ft   1 ft = 0.3048 m
1 ft^3/s = .0283 m^3/s

Type A, E & F grate dimensions

2' x 2' = 606 mm x 606 mm
Area = 4 ft^2 = 0.367 m^2
Perimeter = 6 ft = 1.818 m

Type B grate dimensions

1.93' x 1.79' = 589 mm x 533 mm
Area = 3.45 ft^2
Perimeter = 5.51 ft = 1.655 m

Discharge through grates can be determined graphically using Fig. 3-7 (page 3-16).

	Area				Perimeter							
					No Curb				Curb			
	Waterway Opening		Waterway Opening/2		P		P/2		P		P/2	
Grate Type	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m
A	1.73	0.53	0.87	0.27	7.16	2.18	3.58	1.09	5.33	1.62	2.67	0.81
B	2.75	0.84	1.38	0.42	7.45	2.27	3.72	1.13	5.70	1.74	2.85	0.87
C	1.33	0.41	0.67	0.20	See Plate 4 Std. No. 3							
E	1.83	0.56	0.92	0.28	7.35	2.24	3.68	1.12	5.64	1.72	2.82	0.86
F	2.08	0.63	1.04	0.32	7.91	2.41	3.96	1.21	5.94	1.81	2.97	0.91
ALT. B	2.55	0.78	1.28	0.39	7.36	2.24	3.68	1.12	5.61	1.71	2.81	0.86
ALT. E	1.98	0.60	0.99	0.30	7.25	2.21	3.63	1.11	5.56	1.69	2.78	0.85
ALT. F	1.84	0.56	0.92	0.28	7.96	2.43	3.98	1.21	5.97	1.82	2.98	0.91

Area and Perimeter for Grates

Table 3-6-A

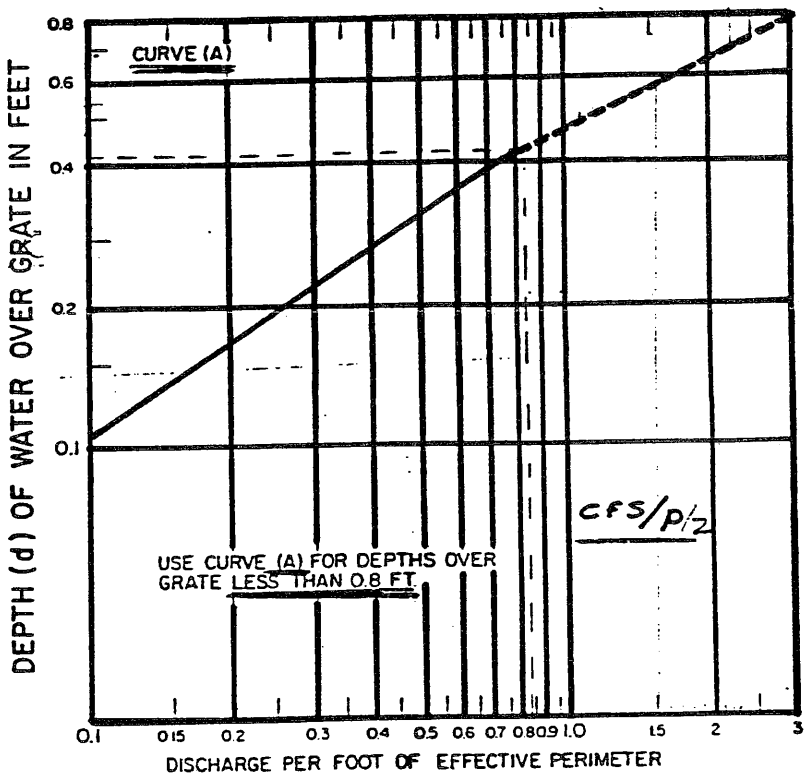


Figure 3-6-B

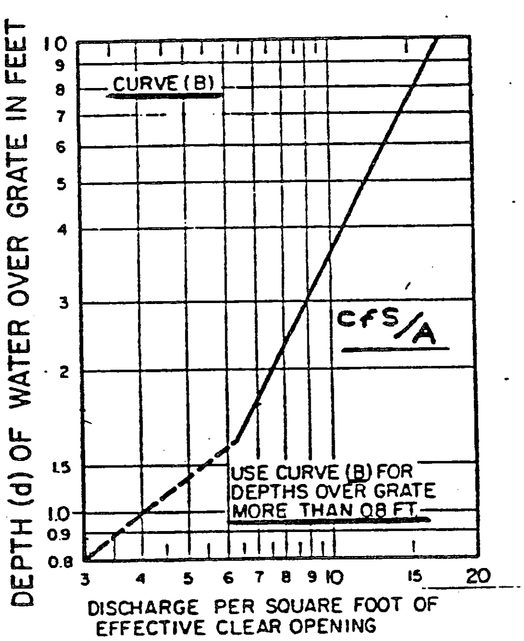
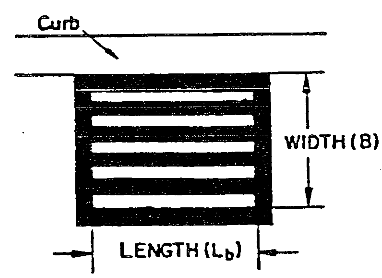


Figure 3-6-C



$P = 2B + L_b$

A = AREA OF CLEAR OPENING IN GRATE

TO ALLOW FOR CLOGGING DIVIDE P OR A BY 2 BEFORE OBTAINING d.

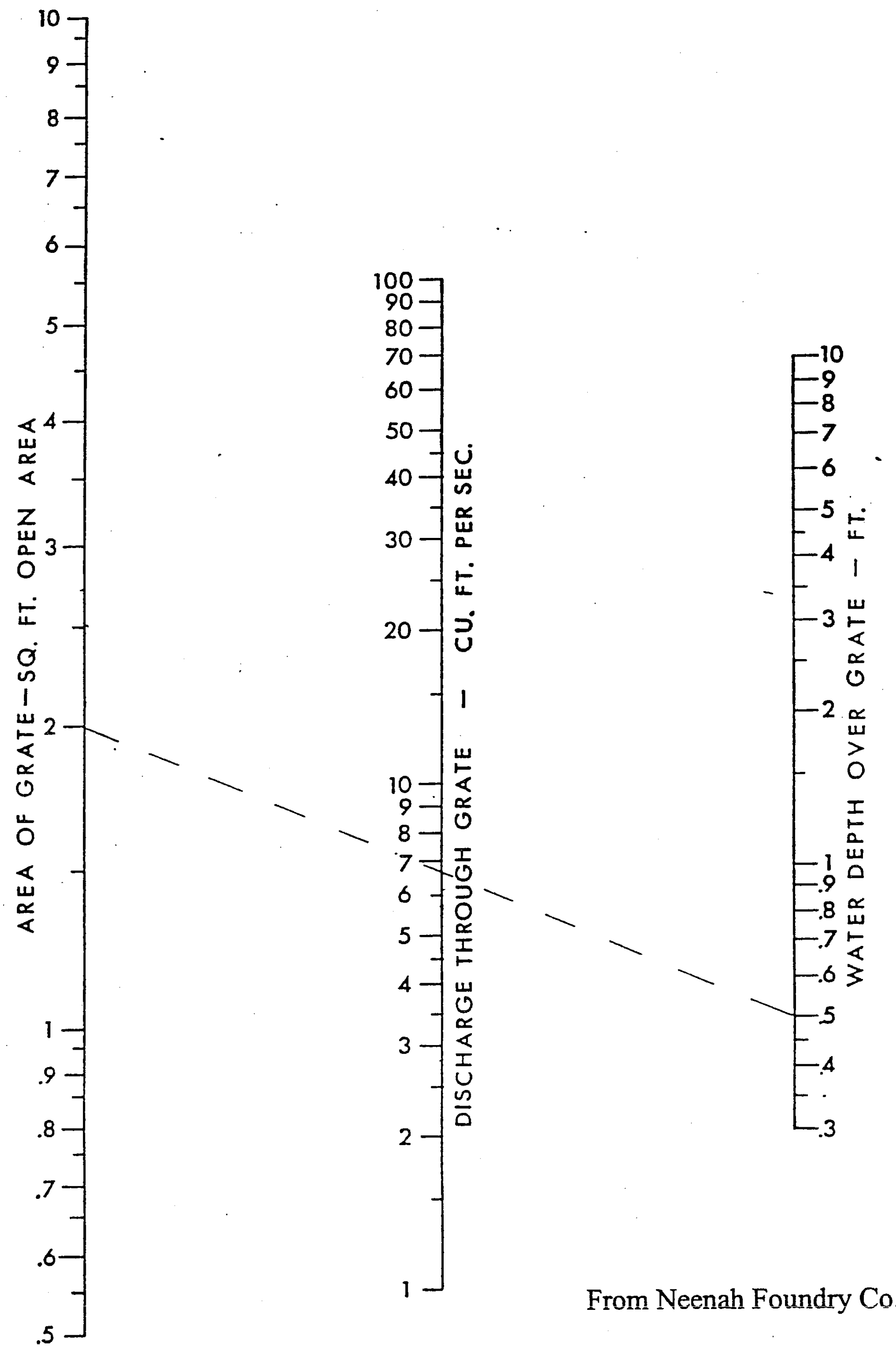
WITHOUT CURB $P = 2(B + L_b)$

METRIC CONVERSION: 1 ft = .3048 m
1 ft^2 = .0929 m^2

Grate inlet capacity in sump conditions

FIGURE 3-6

DISCHARGE vs DEPTH ON GRATE



From Neenah Foundry Co. 1987

Figure 3-7

Metric Conversion: 1 $ft^3/s = 0.028\ m^3/s$
1 $ft^2 = 0.093\ m^2$
1 ft = 0.305 m

SECTION 4 - HYDRAULIC CHARTS FOR THE SELECTION OF HIGHWAY CULVERTS

A. INTRODUCTION

U.S. DEPARTMENT OF COMMERCE

Bureau of Public Roads

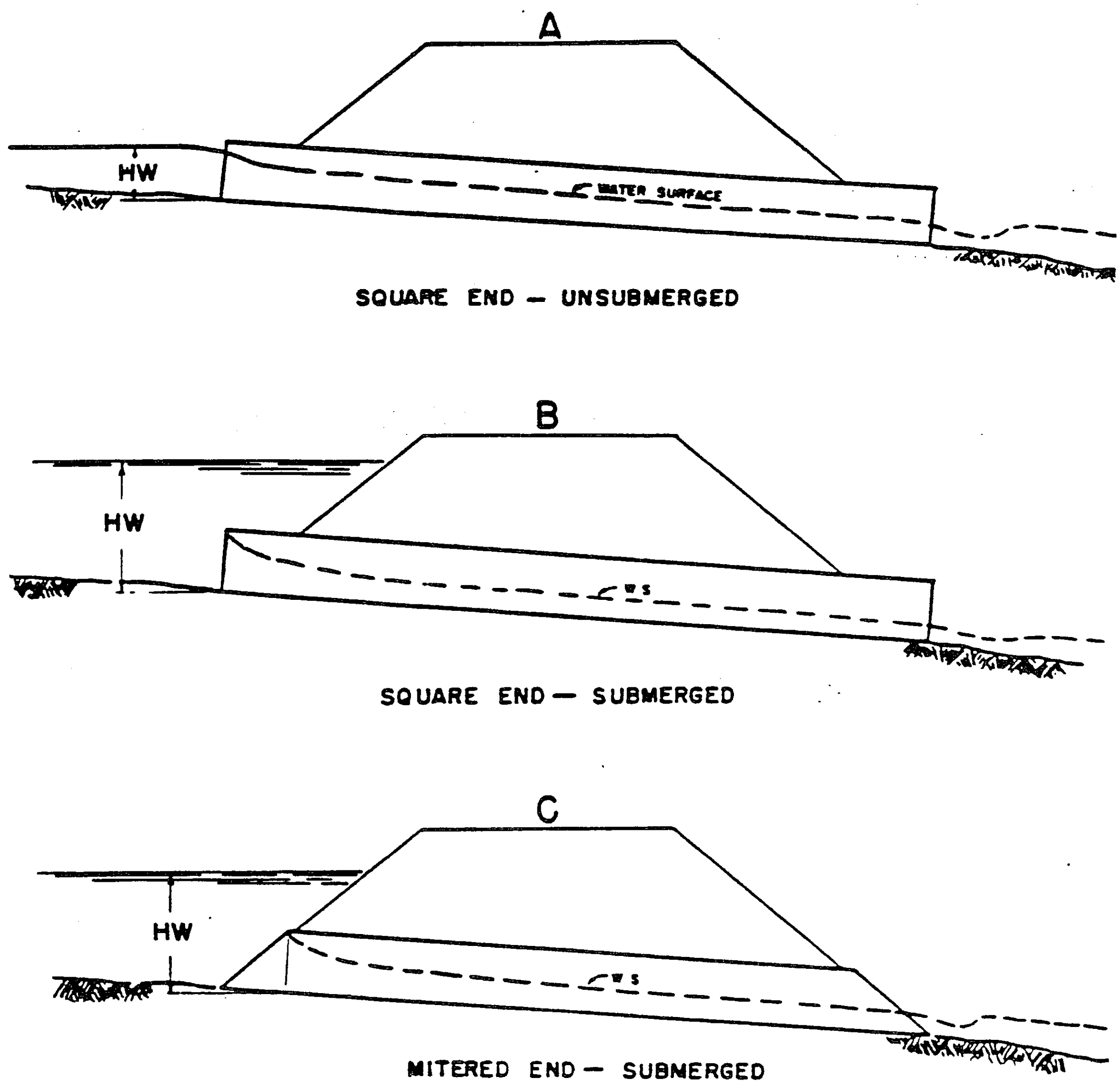
Designing highway culverts involves many factors including estimating flood peaks, hydraulic performance, structural adequacy, and overall construction and maintenance costs. Section 4 contains a brief discussion of the hydraulics of conventional culverts and charts for selecting a culvert size for a given set of conditions. Instructions for using the charts are provided. No attempt is made to cover all phases of culvert design. Other federal manuals will cover culverts with modified inlets and outlets designed to increase performance or to apply to a particular location. Some approximations are made in the hydraulic design procedure for simplicity. These approximations are discussed at appropriate points throughout the section.

For this discussion, conventional culverts include those commonly installed, such as circular, arch and oval pipes, both metal and concrete, and concrete box culverts. All such conventional culverts have a uniform barrel cross section throughout. The culvert inlet may consist of the culvert barrel projected from the roadway fill or mitered to the embankment slope. Sometimes inlets have headwalls, wingwalls and apron slabs, or standard end sections of concrete or metal. The more common types of conventional culverts are considered in this section.

B. CULVERT HYDRAULICS

Laboratory tests and field observations show two major types of culvert flow: (1) flow with inlet control and (2) flow with outlet control. For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross-sectional area of the culvert barrel, the inlet geometry and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the elevation of the tailwater in the outlet channel and the slope, roughness and length of the culvert barrel.

It is difficult in many instances to predict the type of flow likely to occur for any given discharge and culvert installation. The type of flow or the location of the control is dependent on the quantity of flow, roughness of the culvert barrel, type of inlet, flow pattern in the approach channel and other factors. In some instances the flow control varies with change in discharge and occasionally fluctuates from inlet to outlet control and vice versa for a uniform discharge. Thus to design culverts, one should have an understanding of both types of flow so that computations can be made for each type and base the design on the more adverse flow condition. These two types of flow are discussed briefly in subsequent paragraphs.



C. INLET CONTROL

FIGURE 4-1

D. CULVERTS FLOWING WITH INLET CONTROL

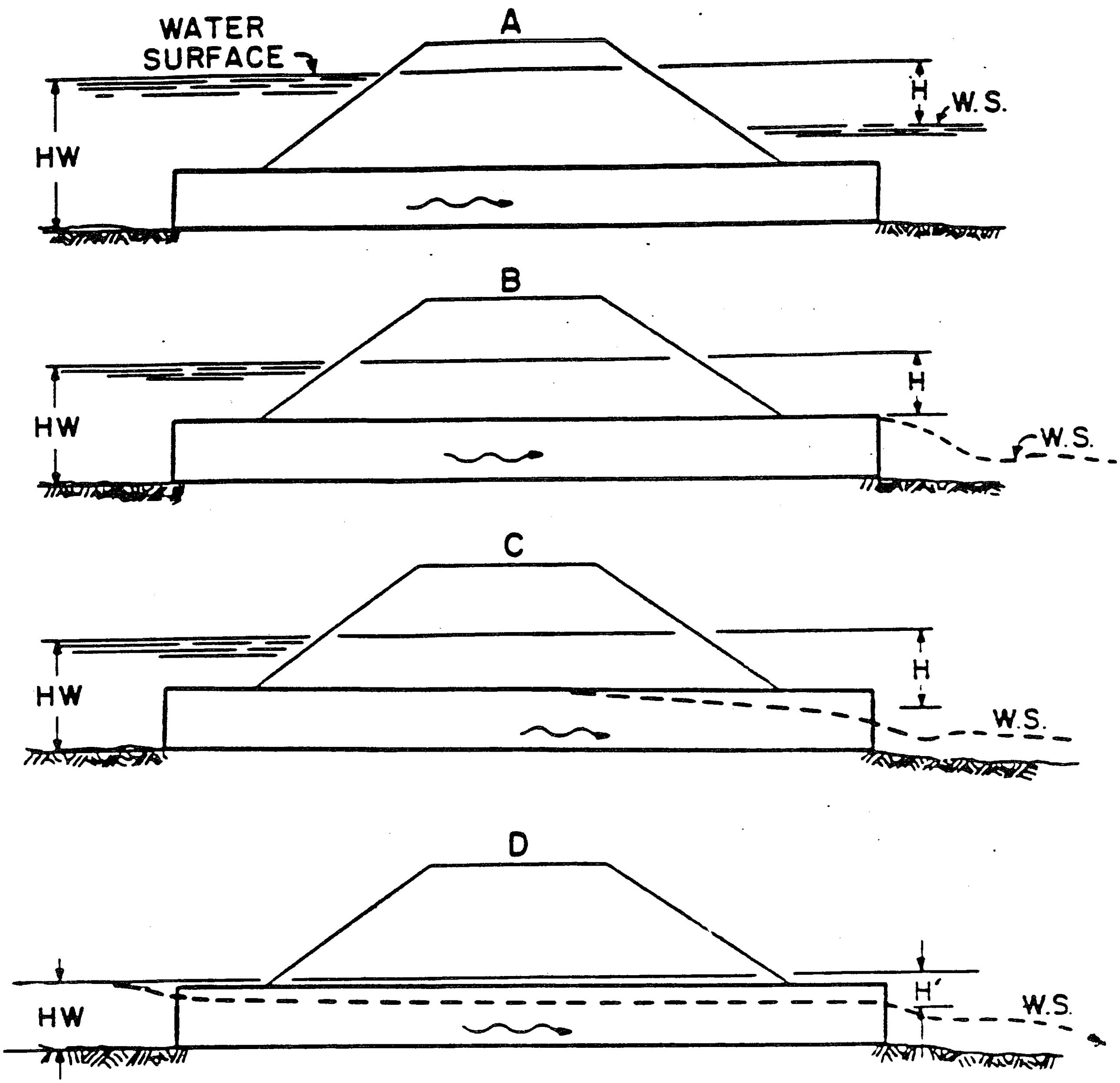
Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater (HW) and the entrance geometry, including the area, shape and type of inlet edge. Types of inlet controlled flow for an unsubmerged and submerged entrance are shown in Fig. 4-1. A mitered (beveled) entrance moves the control downstream to approximately the top of the miter (see Fig. 4-1C).

With inlet control the roughness and length of the culvert barrel and outlet conditions (including depth of tailwater) are not factors in determining culvert capacity. The barrel slope has some effect on discharge but any adjustment for slope is considered minor and can be neglected for conventional culverts flowing with inlet control.

Headwater-discharge relationships for the various types of circular and pipe-arch culverts flowing with inlet control are based on laboratory research of models and verified in some instances by prototype tests. This research is reported in the National Bureau of Standards Report No. 4444 entitled "Hydraulic Characteristics of Commonly Used Pipe Entrance" by John L. French and "Hydraulics of Conventional Highway Culverts" by H. G. Bossy.[1] Experimental data for box culverts with headwalls and wingwalls were obtained from an unpublished report of the United States Geological Survey.

This research data was analyzed, and nomographs for determining culvert capacity for inlet control were developed by the Division of Hydraulic Research, Bureau of Public Roads. These nomographs, Fig. 5-1 or Fig. 5-2, give headwater-discharge relationships for most conventional culverts flowing with inlet control through a range of headwater depths or discharges.

1 Presented at the Tenth National Conference, Hydraulics Division, A.S.C.E. August, 196l. Available on loan from Division of Hydraulic Research, Bureau of Public Roads.



E. OUTLET CONTROL

FIGURE 4-2

F. CULVERTS FLOWING WITH OUTLET CONTROL

Culverts flowing with outlet control can flow with the culvert barrel full or part full for part of the barrel length or for all of it, (see Fig. 4-2). If the entire cross section of the barrel is filled with water for the total length of the barrel the culvert is said to be in full flow or flowing full, Figs. 4-2A and 4-2B. The other two common types of outlet-control flow are shown in Figs. 4-2C and 4-2D. The procedure given in this circular for outlet-control flow does not give an exact solution for a free water surface condition throughout the barrel length shown in Figure 4-2D. However, an approximate solution is given for this case, when the headwater (HW) is .75D and above, where D is the height of the culvert barrel.

The head H required to pass a given quantity of water through a culvert flowing in outlet control with the barrel flowing full throughout its length is made up of three major parts. These three parts are usually expressed in meters (feet) of water and include a velocity head H_v , an entrance loss H_e , and a friction loss H_f . Expressed in equation form

$$H = H_v + H_e + H_f \quad (1)$$

The velocity head H_v equals $V^2 / 2g$, where V is the mean or average velocity in the culvert barrel. The mean velocity is found by dividing the discharge Q by the cross-sectional area A of the flowing water. The velocity head is the kinetic energy of the water in the culvert barrel. This energy is obtained from ponding of water at the entrance (energy from the velocity of flow in the approach channel is neglected in the design procedure for this circular. Also all of the velocity head, H_v is assumed to be lost or, in other words, the exit loss coefficient equals 1.0).

The entrance loss H_e varies with the type or design of the culvert inlet. This loss is expressed as a coefficient k_e times the barrel velocity head or k_e ($V^2/2g$). The coefficients k_e for various types of culvert entrances are given in Table 2, Fig. 4-6 on page 4-13.

The friction head H_f is the energy required to overcome the roughness of the culvert barrel. H_f can be expressed in several ways. Since most highway engineers are familiar with Manning's n, the following expression is used:

$$H_f = [(20n^2L)/R^{1.33}] \; (V^2/2g) \; \{\text{Metric}\} \quad (2)$$

$$H_f = [(29n^2L)/R^{1.33}] \; (V^2/2g) \; \{\text{English}\}$$

where

n = Manning's friction factor (see Table 1, pg. 9-4, for values)
L = length of culvert barrel in meters (feet)
V = mean velocity of flow in culvert barrel in meters/sec. (feet/sec.)
g = acceleration of gravity, 9.80 m/sec.^{2} (32.2 ft/sec^2)
R = hydraulic radius or $\frac{A}{WP}$ in meters (feet)

where

A = area of flow for full cross-section in sq. meters (sq. feet)
WP = wetted perimeter in meters (feet)

Rewriting equation 1 and simplifying, we get for full flow

$$H = [1 + k_e + (20n^2 L)/R^{1.33})] \; V^2/2g \quad \{Metric\} \qquad (3)$$

$$H = [1 + k_e + (29n^2 L)/R^{1.33})] \; V^2/2g \quad \{English\}$$

Equation 3 can be solved readily by the use of the full-flow nomographs, Fig. 6-1 or Fig. 6-2. The equations shown on these nomographs are the same as equation 3 expressed in a different form. Each nomograph is drawn for a single value of n as noted on the respective chart. These nomograph can be used for other values of n by modifying the culvert length as directed in the instructions (pg. 6-1) for the use of the full-flow nomographs.

Finding the value of H from a nomograph is not the complete solution for outlet control type of flow. Headwater must be determined and other factors such as slope of the culvert barrel and outlet conditions enter into this computation.

The value of H in meters must be measured from some "control" elevation at the outlet. This "control" elevation is dependent on the rate of discharge or the elevation of the water surface of the tailwater. For simplicity a value h_o is used as the distance in meters from the culvert invert (flow line) at the outlet to the "control" elevation. The following equation is used to compute headwater (HW):

$$HW = h_o + H - LS_o \qquad (4)$$

where S_o is the slope of the flow line in meters per meter (feet per foot) and all terms are in meters (feet). The determination of h_o is discussed in the following paragraphs for the various flow conditions at the outlet.

If the water surface in the outlet channel (tailwater elevation) is at or above the top of the barrel at the outlet (Fig. 4-2A), the solution for HW is simple. The TW depth is equal to h_o and the relationship of HW to the other terms in equation 4 are illustrated in Fig. 4-A.

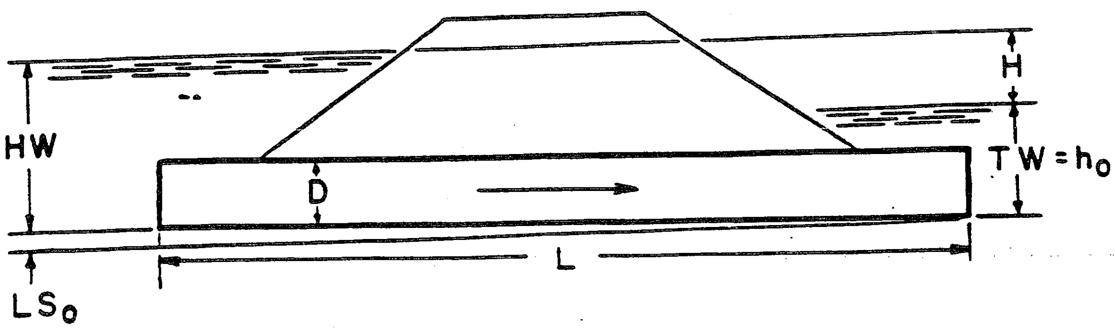


Figure 4-A

If the tailwater elevation is below the top or crown of the culvert at the outlet, the determination of h_o for a given discharge and size of culvert is more difficult. Figs. 4-2B, 4-2C and 4-2D are three common types of flow for outlet control with this low tailwater condition.

In these cases, h_o is found by comparing two values (1) TW depth in the outlet channel and (2) $(d_c + D)/2$ and setting h_o equal to the larger of these values. The fraction $(d_c + D)/2$ is a simplified means of computing h_o, when the tailwater is low and the discharge does not fill the culvert barrel at the outlet. In this fraction d_c is critical depth as determined from Fig. 7-1 or Fig. 7-2 and D is the culvert height. The value of d_c should never exceed D, making the upper limit of this fraction equal to D. The sketch in figure 4-B shows the terms of equation 4 for the cases discussed above.

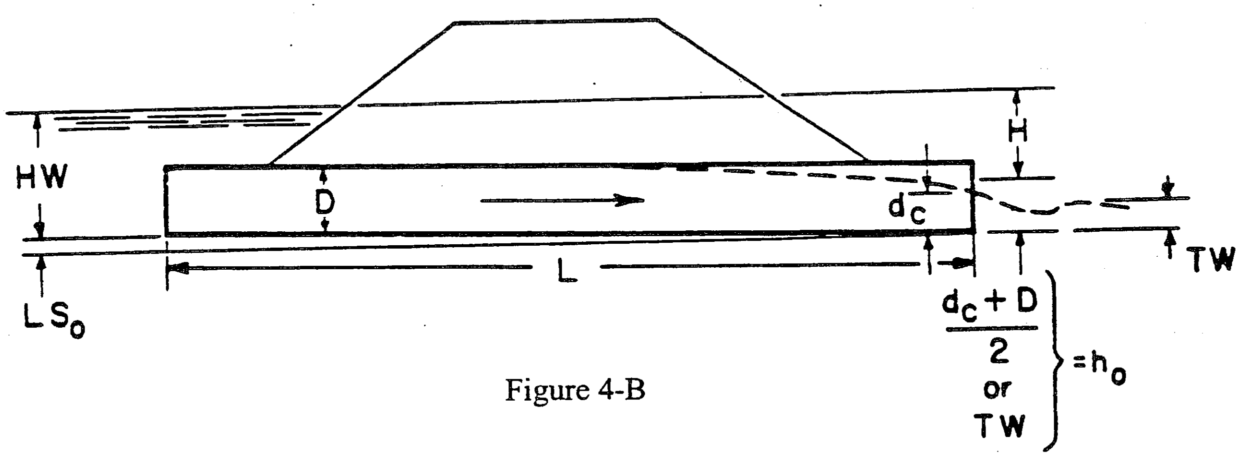


Figure 4-B

For more rigorous solutions it has been found that equation 4 gives accurate answers if the culvert flows full for a part of the barrel length as illustrated by Figure 4-B. This condition of flow will exist if the headwater (as determined by equation 4) is equal to or greater than the quantity.

$$D + (1 + k_e)\frac{V^2}{2g}$$

where V is the mean velocity for the full cross section of the barrel; k_e, the entrance loss coefficient; and D, the culvert height. If the headwater drops below this point the water surface will be free throughout the culvert barrel as in Figure 4-2D and equation 4 gives answers with some error as explained in the next paragraph.

In case 4-2D, equation 4 is used to solve for HW when a free water surface exists through the barrel. Such a computation does not give a true value since the only correct way of finding HW in this case is by a backwater computation starting at the culvert outlet. However, equation 4 will give answers of sufficient accuracy for design purposes if the headwater is limited to values greater than 0.75D. H' is used in Figure 4-2D to show that the head loss here is an approximation of H. No solution is given for HW less than 0.75D.

Although the procedure given in this section is primarily for use in selecting a size of culvert to pass a given discharge at a given headwater, a better understanding of culvert operation is gained by plotting performance curves through some range of discharges and barrel slopes. Such curves can also be used to compare different sizes and types of culverts.

Computing Depth of Tailwater

The depth of tailwater is important in determining the hydraulic capacity of culverts flowing with outlet control. In many cases the downstream channel is of considerable width and the depth of water in the natural channel is less than the height of water in the outlet end of the culvert barrel, making the tailwater ineffective as a control, so that its depth need not be computed to determine culvert discharge capacity or headwater. There are instances, however, where the downstream water-surface elevation is controlled by a downstream obstruction or backwater from another stream. A field inspection of all major culvert locations should be made to evaluate downstream controls and determine water stages.

An approximation of the depth of flow in a natural stream (outlet channel) can be made by using Manning's equation (see pg. 4-9) if the channel is reasonably uniform in cross section, slope and roughness.

Values of n for natural streams in Manning's formula may be found in Table I, Fig. 4-5, pg. 4-12. If the water surface in the outlet channel is established by downstream controls other means must be found to determine the tailwater elevation. Sometimes this necessitates a study of the stage-discharge relationship of another stream into which the stream in question flows or the securing of data on reservoir elevations if a storage dam is involved.

Outlet Velocity and Protection

A culvert, because of its hydraulic characteristics, increases the velocity of flow over that in the natural channel. If the velocity exiting the culvert exceeds the allowable velocity for the soil type (3 m/s or 10 ft/s max.), outlet protection must be provided to prevent erosion or scour. The design of stone outlet aprons is illustrated USDA SCS "Stormwater Management and Erosion and Sediment Control Handbook for Urban and Developing Areas in New Hampshire." For an approximation of stone required, see part I of Section 4, Stone Lining For Ditches.

Energy dissipators for channel flow have been investigated in the laboratory and many have been constructed, especially in irrigation channels. Some of these structures have been modified and at least several hundred have been constructed at the outlets of culverts. All energy dissipators add to the cost of a culvert and engineers should consider using them only when required to prevent a large scour hole or as remedial construction.

The judgment of engineers working in a particular area is required to determine the need for energy dissipators at culvert outlets. As an aid in evaluating this need it is suggested that the outlet velocities be computed. These computed velocities can be compared with outlet velocities of other sizes and types of culverts and with the natural channel velocities. A change in size of culvert does not change outlet velocities appreciably in most cases. For more information refer to HEC-14; "Hydraulic Design of Energy Dissipators for Culverts and Channels."

The outlet velocity can be determined as follows:

1) Calculate full flow discharge by combining Manning's equation and the continuity equation;

$$Q = \frac{0.312}{n} D^{8/3} S^{1/2} \quad \text{(Metric)} \qquad Q = \frac{0.447}{n} D^{8/3} S^{1/2} \quad \text{(English)}$$

2) Calculate full flow velocity, V_f; $V_f = Q_f/A$.

3) Knowing Q/Q_f, use Fig. 4-4A, pg. 4-11-2 to determine V/V_f.
(See culvert design example pg. 4-11-1.)

[2] "Design Charts for Open Channel Flow" by U.S. Bureau of Public Roads, available from Superintendent of Documents, Government Printing Office, Washington, DC.

PROJECT NO.__________ DESIGN FREQUENCY_____yr. DESIGNED BY__________ DATE______

PROJECT NAME__________ CHECKED BY__________ DATE______

	CHECK MAPS USED	NUMBER OF TITLE	SCALE	CONVERSION FACTORS
___	USGS Topog Quad	___	1:24000	1 cm^2 = 0.0576 km^2 = 5.76 ha
___	USGS Topog Quad	___	1:25000	1 cm^2 = 0.0625 km^2 = 6.25 ha
___	USGS Topog Quad	___	1:62500	1 cm^2 = 0.3906 km^2 = 39.06 ha
___	Forest Service Quad	___	1:31680	1 cm^2 = 0.0999 km^2 = 9.99 ha
___	National Forest	___	1:126720	1 cm^2 = 1.6052 km^2 = 160.53 ha
___	BLM or Soil Conservation Quad	___	1:31680	1 cm^2 = 0.0999 km^2 = 9.99 ha
___	BLM Grazing	___	1:63360	1 cm^2 = 0.4007 km^2 = 40.07 ha
___	USGS Army Topog	___	1:250000	1 cm^2 = 6.25 km^2 = 625.00 ha
___	County	___	1:126720	1 cm^2 = 1.606 km^2 = 160.58 ha
___	Other	___	___	1 cm^2 = ___ km^2 = ___ ha

DESCRIPTION OF TERRAIN AND LAND FACTOR(S) USED: __________

DESCRIPTION OF TOPOGRAPHY AND K-FACTOR(S) USED: __________

RAINFALL INTENSITIES

I_2 = ______ in/hr

I_{100}/I_2 = ______

I_{100} = ______ in/hr

I_f = ______ in/hr

I_{25} = ______ in/hr

F.F. = ______

1 in = 2.54 cm

COMMENTS AND EXPLANATIONS: __________

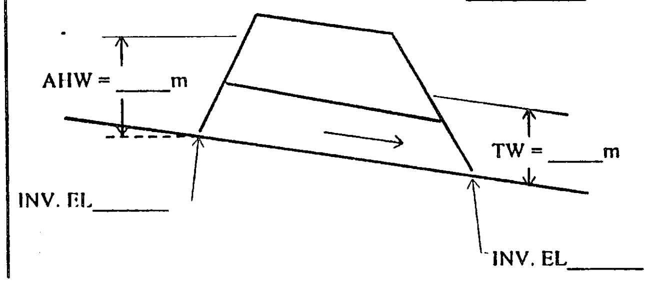
FIGURE 4-3

PROJECT NO.______________ STATION__________ CA. or CLASS____________ DESIGNED BY__________________ DATE_______
PROJECT NAME______________________________ CROSSING ANGLE_________ CHECKED BY__________________ DATE_______

HYDROLOGIC & CHANNEL INFORMATION

Q_{10}= See pg. 1-1 for appropriate
Q_{25}= design frequency.
Q_{50}=

CROSS SECTIONAL DIAGRAM



Allowable outlet velocity = ____________m/s
Length of pipe = __________m
Slope of pipe = __________ m/m

Allowable outlet velocity based on soil type. Check USDA SCS for allowable velocities.
$3\frac{m}{s}$ ($10\frac{ft}{s}$) MAX.

Culvert Type	Size	Q_d	Inlet Control HW/D	HW	Outlet Control (For h_o use>value of (1) or (2)) K_e	d_c	d_c+D/2	h_o(T.W.)	HW= $H+h_o-LS_o$ H	LS_o	HW	Controlling HW	Outlet V_o	Cost. Comments
RCP	TRIAL SIZE	pg. 2-2	pg. 5-2	Headwater for inlet control	pg. 4-13	pg. 7-1		Make reasonable guess or use flow depth in channel.	pg. 6-3	Pipe length x slope ($\frac{m}{m}$)	$H+h_o-LS_o$ = Headwater for Outlet Control	Compare HW for inlet and outlet control - Use larger value.	pg. 4-9 $Q=\frac{0.312}{n}D^{8/3}S^{1/2}$ Fig. 4-4A See example, next page.	
					For outlet control, use greater value of d_c+D/2 and h_o for h_o value in HW equation.									

FIGURE 4-4

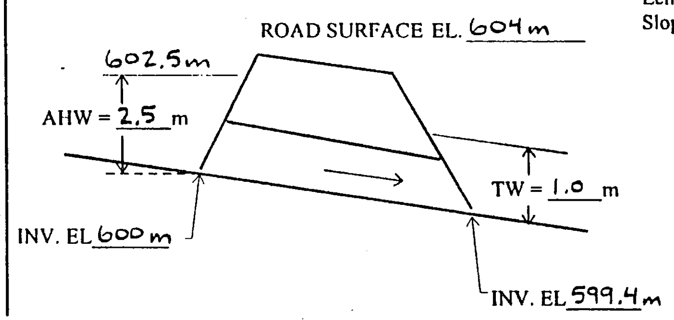
PROJECT NO.______ STATION______ CA. or CLASS______ DESIGNED BY______ DATE______
PROJECT NAME______ CROSSING ANGLE______ CHECKED BY______ DATE______

HYDROLOGIC & CHANNEL INFORMATION

Q_{10}=
Q_{25}= 6.0 m³/sec
Q_{50}=

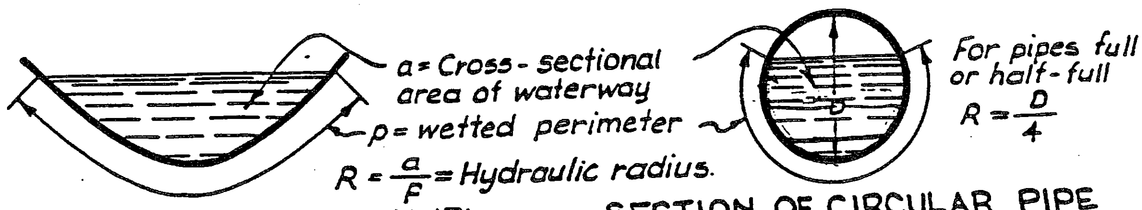
CROSS SECTIONAL DIAGRAM

Allowable outlet velocity = 3.0 m/s
Length of pipe = 60 m
Slope of pipe = .01 m/m



Culvert Type	Size	Q_d	Inlet Control		Outlet Control							Controlling HW	Outlet V_o	Cost. Comments
					For h_o use>value of (1) or (2)				HW= $H+h_o-LS_o$					
			HW/D	HW	K_e	d_c	d_c+D/2	h_o(T.W.)	H	LS_o	HW			
RCP	1500 mm	6.0 m³/s	1.42	2.13	.2	1.22	1.36	1.0	.95	.6	1.71	2.13	4.77	Consider energy dissipation or use larger culvert size.

EXAMPLE PROBLEM
4-11-1

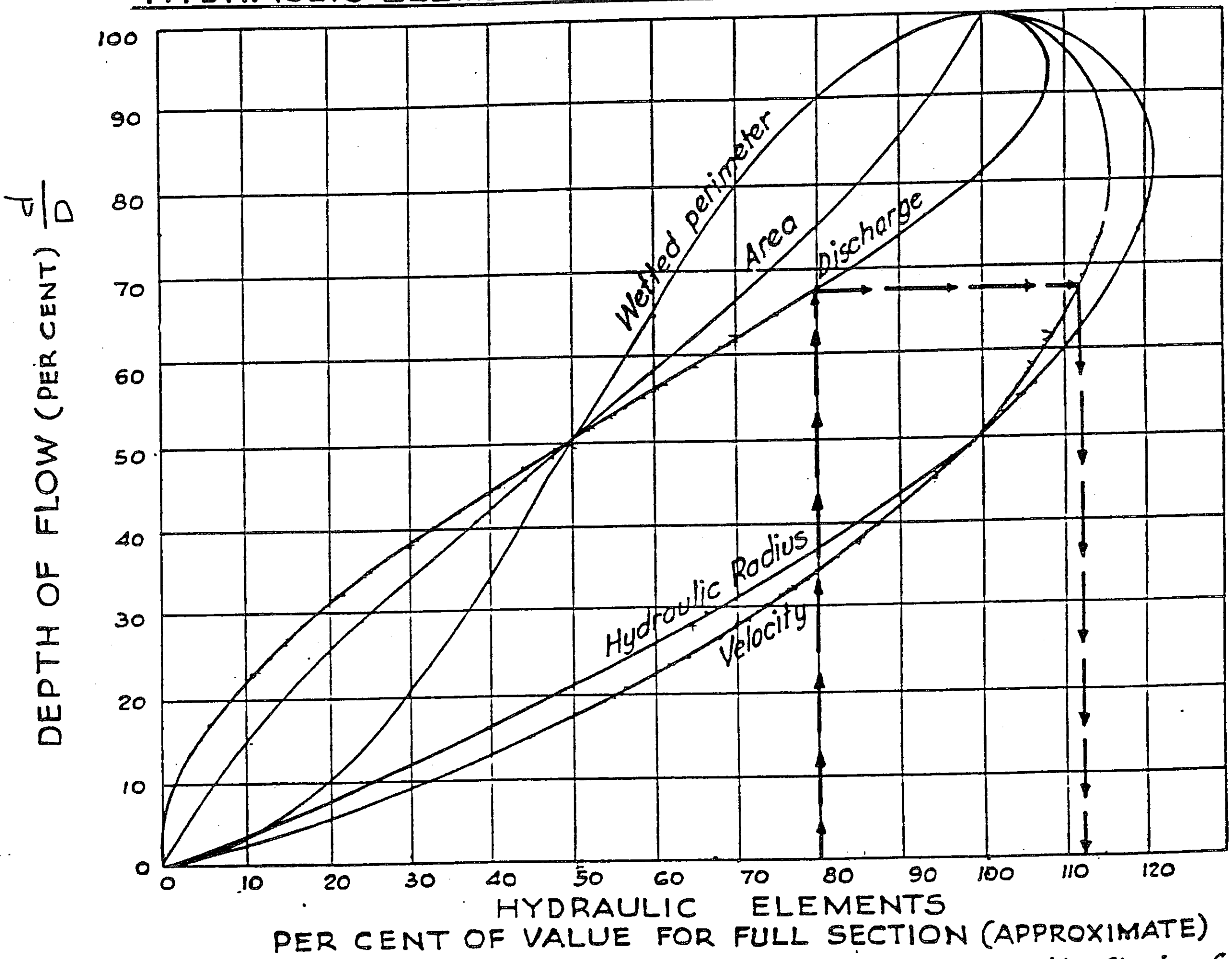


SECTION OF ANY OPEN CHANNEL

SECTION OF CIRCULAR PIPE

V = Average or mean velocity in feet per second.
Q = a V = Discharge of pipe or channel in cubic feet per second (c.f.s.).
n = Coefficient of roughness of pipe or channel surface, see Table A-Pg. 18-68.
S = Slope of Hydraulic Gradient (water surface in open channels or pipes not under pressure, same as slope of channel or pipe invert only when flow is uniform in constant section.

HYDRAULIC ELEMENTS OF CHANNEL SECTIONS.



EXAMPLE: Given: Discharge = 12 c.f.s. through a pipe which has capacity flowing full of 15 c.f.s. at a velocity of 7.0 ft. per sec. Required to find V for Q = 12 c.f.s.
∴ Percentage of full discharge = $\frac{12}{15}$ = 80%. Enter chart at 80% of value for full section of Hydraulic Elements, find V = 112.5% × 7 = 7.9 ft. per sec.

VALUES OF HYDRAULIC ELEMENTS OF CIRCULAR SECTION FOR VARIOUS DEPTHS OF FLOW.

Figure 4-4A

TABLE 1. - MANNING'S N FOR NATURAL STREAM CHANNELS (SURFACE WIDTH AT FLOOD STAGE LESS THAN 30 M.)

1. Fairly regular section:
   a. Some grass and weeds, little or no brush---------------------------------- 0.03-0.035

   b. Dense growth of weeds, depth of flow materially greater than weed height--0.035-0.05

   c. Some weeds, light brush on banks ---0.035-0.05

   d. Some weeds, heavy brush on banks---0.05 -0.07

   e. Some weeds, dense willows on banks---0.06 -0.08

   f. For trees within channel, with branches submerged at high stage, increase all above values by--0.01 -0.02

2. Irregular sections, with pools, slight channel meander; increases values given above by---0.01 -0.02

3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:

   a. Bottom of gravel, cobbles, and few boulders---------------------------------0.04-0.05

   b. Bottom of cobbles, with large boulders---------------------------------------0.05-0.07

FIGURE 4-5

TABLE 2. - ENTRANCE LOSS COEFFICIENTS

Coeffecient k_e to apply to velocity head $\frac{V^2}{2g}$ to determine loss of head at entrance of a structure, such as a culvert or conduit, operating full or partly full with <u>control at the outlet.</u>

$$\text{Entrance head loss } H_e = ke \frac{V^2}{2g}$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end	0.2
Projecting from fill, sq. cut end	0.5
Headway or headway and wingwalls	
Socket end of pipe	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.1
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headway)	0.9
Headway or headway and wingwalls	
Square-edge	0.5
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Box, Reinforced Concrete	
Headway parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of-1/12 barrel dimension	0.2
Wingwalls at 10° to 30° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7

*Note: "End Section conforming to fill slope", made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headway in both <u>inlet</u> and <u>outlet</u> control. Some end sections, incorporating a <u>closed</u> taper in their design have a superior hydraulic performance. These latter sections will be discussed in a subsequent section.

Figure 4-6

H. PROCEDURE FOR SELECTION OF CULVERT SIZE

Step l: List given data

a. Design discharge Q, in cms.

b. Approximate length of culvert, in meters.

c. Allowable headwater depth, in meters, which is the vertical distance from the culvert invert (flow line) at the entrance to the water surface elevation permissible in the approach channel upstream from the culvert.

d. Type of culvert, including barrel material, barrel cross sectional shape and entrance type.

e. Slope of culvert. (If grade is given in percent, convert to slope in m/m)

f. Allowable outlet velocity (if scour is a problem).

Note: It is suggested that culvert design sheets, similar to Fig. 4-4 be used to record design data.

Step 2: Determine a trial size culvert.

a. Refer to the inlet control nomograph (Fig. 5-l or Fig. 5-2) for the culvert type selected.

b. Using an $\frac{HW}{D}$ of approximately 1.5 and the scale for the entrance type to be used, find a trial size culvert by following the instructions for use of these nomographs. If reason for lesser or greater relative depth of headwater in a particular case should exist, another value of $\frac{HW}{D}$ may be used for this trial selection.

c. If the trial size for the culverts is obviously too large in dimension because of limited height of embankment or availability of size, try a different $\frac{HW}{D}$ value or multiple culverts by dividing the discharge equally for the number of culverts used. Raising the embankment height or the use of pipe arch and box culverts with width greater than height should be considered. Selection should be based on an economic analysis.

Step 3. Find headwater HW depth for the trial size culvert.

a. Determine and record headwater HW depth by use of the appropriate inlet control nomograph (Fig. 5-1 or Fig. 5-2). Tailwater TW conditions are to be neglected in this determination. HW in this case is found by simply multiplying $\frac{HW}{D}$ obtained from the nomograph by D.

b. Compute and record HW for outlet control as instructed below:

(1) Approximate the depth of tailwater TW for the design flood condition in the outlet channel. The TW depth may also be due to backwater caused by another stream or some control downstream. An estimate of TW depth can be made by use of channel flow formulas or charts (see general discussion on tailwater pg. 4-8).

(2) For tailwater TW depths equal to or above the depth of the culvert at the outlet set TW equal to h_o and find HW by the following equation.

$$HW = h_o + H - S_oL$$

where:

HW = vertical distance in meters (feet) from culvert invert (flow line) at entrance to pool surface upstream.

H = head loss in meters (feet) as determined from the appropriate nomograph (Fig. 6-1 or Fig. 6-2).

h_o = vertical distance in feet from culvert flow line at outlet to "control" point. (In this case h_o equals TW.)

S_o = slope of barrel in meter/meter (feet/foot)

L = culvert length in meters

(3) For tailwater TW elevations below the crown of the culvert at the outlet, use the following equation to find headwater HW (it should be noted that this computa tion may contain approximations which are discussed under the head "Culverts Flowing with Outlet Control" pg. 4-5).

$$HW = h_o + H - S_oL$$

where:

$h_o = \frac{d_c + D}{2}$ or TW, whichever is the greater*

d_c = critical depth (mm) (Fig. 7-1 or Fig. 7-2)
D = culvert height (mm)

Other items are as defined in (1) above.

* Note: When d_c exceeds D in a rectangular section h_o should be set equal to D.

c. Compare the headwaters found in Step 3a and Step 3b (Inlet Control or Outlet Control). The higher headwater governs and indicates the flow control existing under the given conditions.

d. Compare the higher HW above with that allowable at the site. If HW is greater than the allowable, repeat the procedure using a larger culvert. If the HW is less than the allowable, repeat the procedure to investigate the possibility of using a smaller size.

Step 4. Check outlet velocities for size selected.

a. If outlet control governs in c above, outlet velocity equals $\frac{Q}{A}$, where A is the cross-sectional area of flow at the outlet. If d_c or TW is less than the height of the culvert barrel use A corresponding to d_c or TW depth, whichever gives the greater area of flow.

b. If inlet control governs in c above, outlet velocity can be assumed to equal normal velocity in open-channel flow as computed by Manning's equation for the barrel size, roughness and slope of culvert selected.

Note: In computing outlet velocities charts and tables[3] are helpful.

Step 5: Try a culvert of another type or shape and determine size and HW by the above procedure.

Step 6: Record final selection of culvert with size, type, outlet, velocity, required HW and economic justification.

3 "Hydraulic Tables", Corps of Engineers, U. S. Army, Superintendent of Documents, Government Printing Office, Washington 25, D.C.

"Hydraulic and Excavation Tables", U. S. Bureau of Reclamation, Superintendent of Documents, Government Printing Office, Washington 25, D. C., $1.50

"Handbook of Hydraulics", by H. W. King, McGraw-Hill Book Company, New York City.

"Design Charts for Open-Channel Flow", U. S. Department of Commerce, Bureau of Public Roads, Superintendent of Documents, Government Printing Office, Washington 25, D. C.

I. STONE LINING FOR DITCHES

If the velocity flowing through a ditch or exiting a culvert exceeds stone may be required in drainage ditches if the velocity is high enough to cause erosion to the soil. For a complete ditch design, refer to HEC-15; "Design of Roadside Channels with Flexible Linings". The following process provides an approximation for the lining required:

1. Determine the depth of flow in the channel using the following equations:

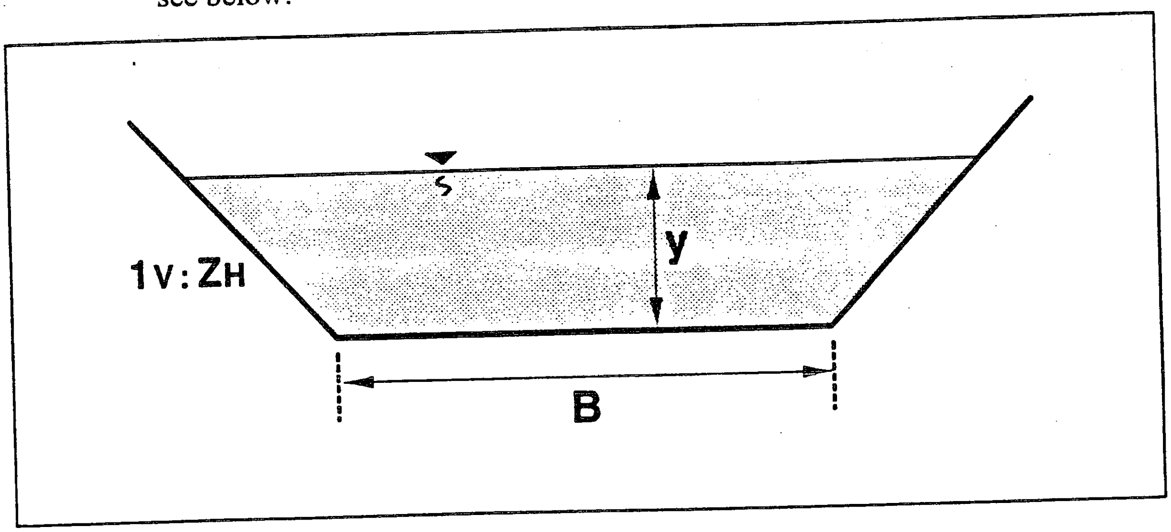
Manning's Equation: $V=(1/n)R^{2/3}S^{1/2}$

Continuity Equation: $Q=VA$

Combine equations: $Q=(1/n)AR^{2/3}S^{1/2}$ where, (1)

$$\text{Hydraulic radius, } R = \frac{\text{Area}}{\text{Wetted Perimeter}} = \frac{By+Zy^2}{B+2y(1+z^2)^2}$$

see below:

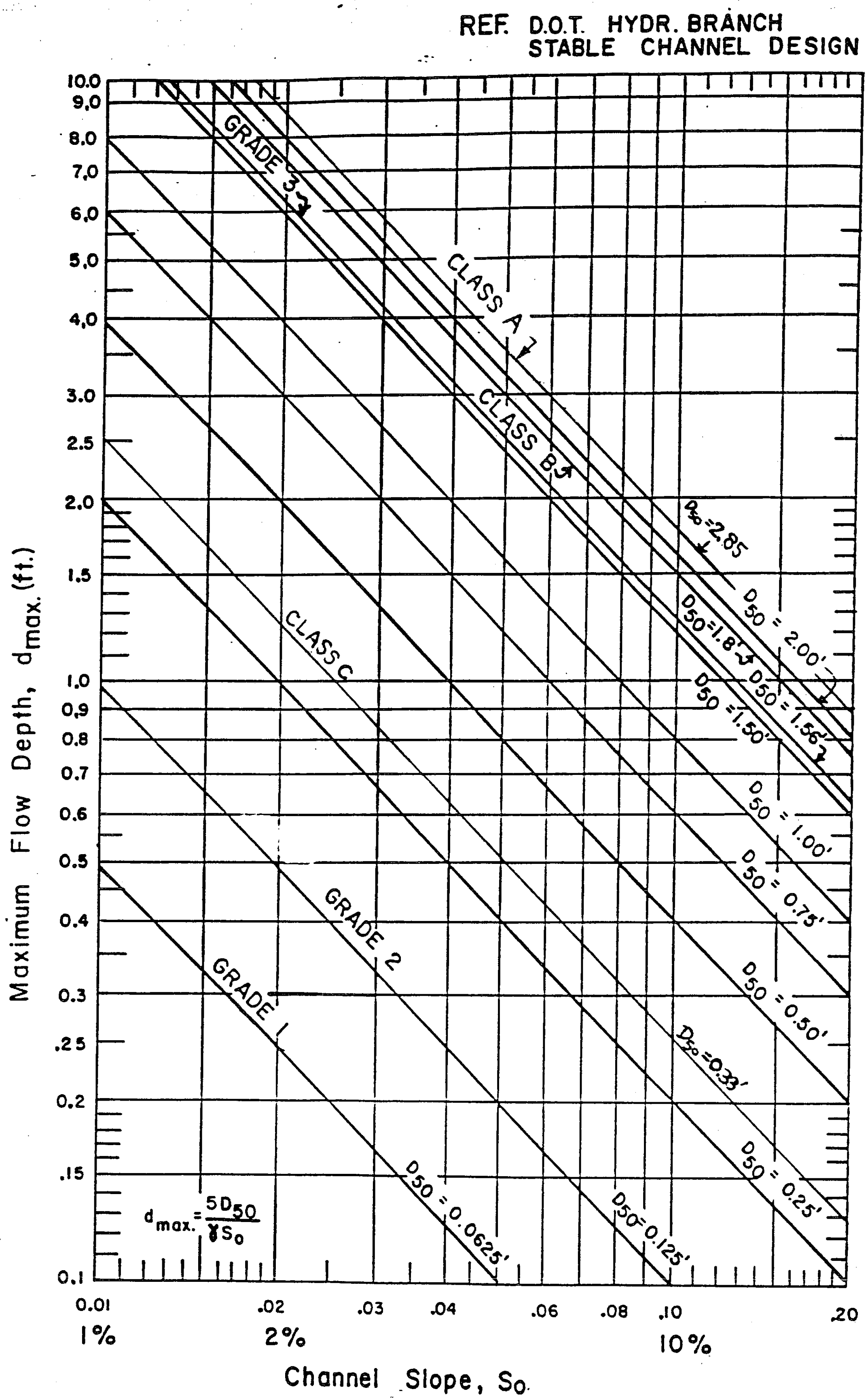


Solve equation (1) using trial and error by assuming values of y.

2. For depth of flow calculated use Fig. 4-7 to determine the required stone size for the slope (S_o) of outlet or channel.

Source: Hydraulic Design Series No. 4

Figure 4-7



Metric Conversion: 1 in = 25.4 mm
1 ft = 0.3048 m

Maximum Permissible Depth of Flow for Stone Lined Channels

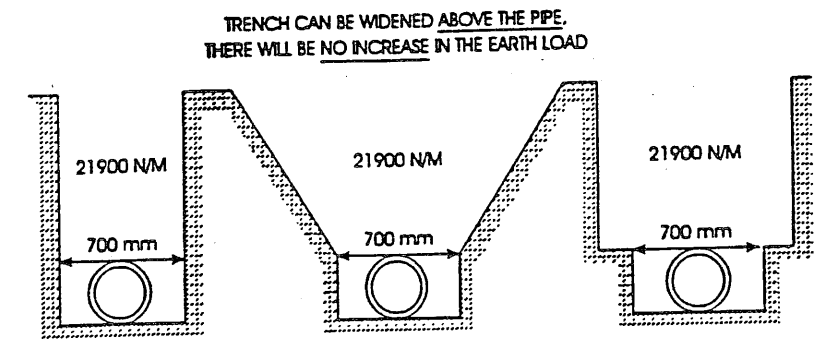
I. STRENGTH REQUIREMENTS

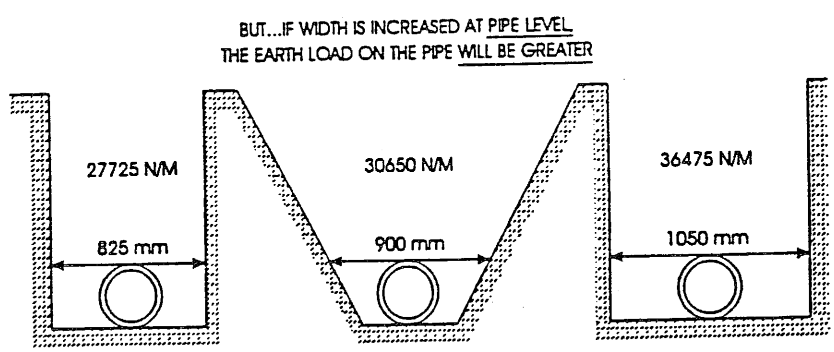
When the height of fill over a pipe is 6 m (20 ft.) or more, a special method of installation known as "Imperfect Trench" must be used to protect the pipe from the weight of the fill. It includes excavating a trench over the top of the installed pipe and backfilling with loose excavated material to form a cushion. The remaining compacted embankment proceeds vertically as normal. The compacted material over the cushion transfers the load to the sides of the trench by arch action in effect, reducing the load on the pipe and preventing it from being crushed.

Use the following chart as a guide:

Max. Height of Fill Over RCP Pipe	Normal Construction	Imperfect Trench
3.9 m (13 ft.)	100 kPa (2000D)	-
6.0 m (20 ft.)	150 kPa (3000D)	-
10.5 m (35 ft.)	-	150 kPa (3000D)
15.0 m (50 ft.)	-	200 kPa (4000D)
22.5 m (75 ft.)	-	250 kPa (5000D)

The width of the trench must never exceed that specified in the Standard Specifications since pipe load increases with wider trenches.





Metric Conversion:

1 meter (m) = 3.28 feet (ft)
1 Newton (N) = 0.225 pounds of force (lbf)
1 kilopascal (kPa) = 0.145 pounds per sq. inch (lbf/in^2)

NOTE: For more information on determining strength of pipe (excessive and insufficient ground cover) refer to the American Concrete Pipe Association; "Concrete Pipe Design Manual"; Chapter 4.

If corrugated steel pipe is to be used, specify according to the following chart:

Fill Limits for Corrugated Steel Pipe

Minimum Cover over pipe is 300 mm (12in.)

		Max. Cover	
		Thickness	
Diameter	1.63 mm (0.064 in)	2.01 mm (0.079 in)	4.27 mm (0.168)
150 mm (6 in)	75 m (248 ft)	-	-
300 mm (12 in)	75 m (248 ft)	-	-
375 mm (15 in)	60 m (199 ft)	-	-
450 mm (18 in)	50 m* (166 ft)	-	-
600 mm (24 in)	35 m (124 ft)	45 m* (155 ft)	-
1050 mm (42 in)	-	-	60 m (195 ft)

* Corrugated Aluminized Steel Pipe (Type II)

NOTE: For larger pipes and arch culverts refer to American Iron and Steel Institute; "Handbook of Steel Drainage & Highway Construction Products," 1994.

SECTION 5 - INLET-CONTROL NOMOGRAPHS

Instructions for Use

Note: For oval concrete culverts, corrugated metal pipe, and arch pipe culverts refer to the previous version of the "Manual on Drainage Design for Highways."

1. To determine headwater (HW)

   a. Connect with a straightedge the given culvert diameter or height (D) and the discharge Q, or $\frac{Q}{B}$ for box culverts; mark intersection of straightedge on $\frac{HW}{D}$ scale marked (1).

   b. If $\frac{HW}{D}$ scale marked (1) represents entrance type used, read $\frac{HW}{D}$ on scale (1).

      If some other entrance type is used extend the point of intersection in (a) horizontally to scale (2) or (3) and read $\frac{HW}{D}$.

   c. Compute HW by multiplying $\frac{HW}{D}$ by D.

2. To determine culvert size

   a. Given an $\frac{HW}{D}$ value, locate $\frac{HW}{D}$ on scale for appropriate entrance type. If scale (2) or (3) is used extend $\frac{HW}{D}$ point horizontally to scale (1).

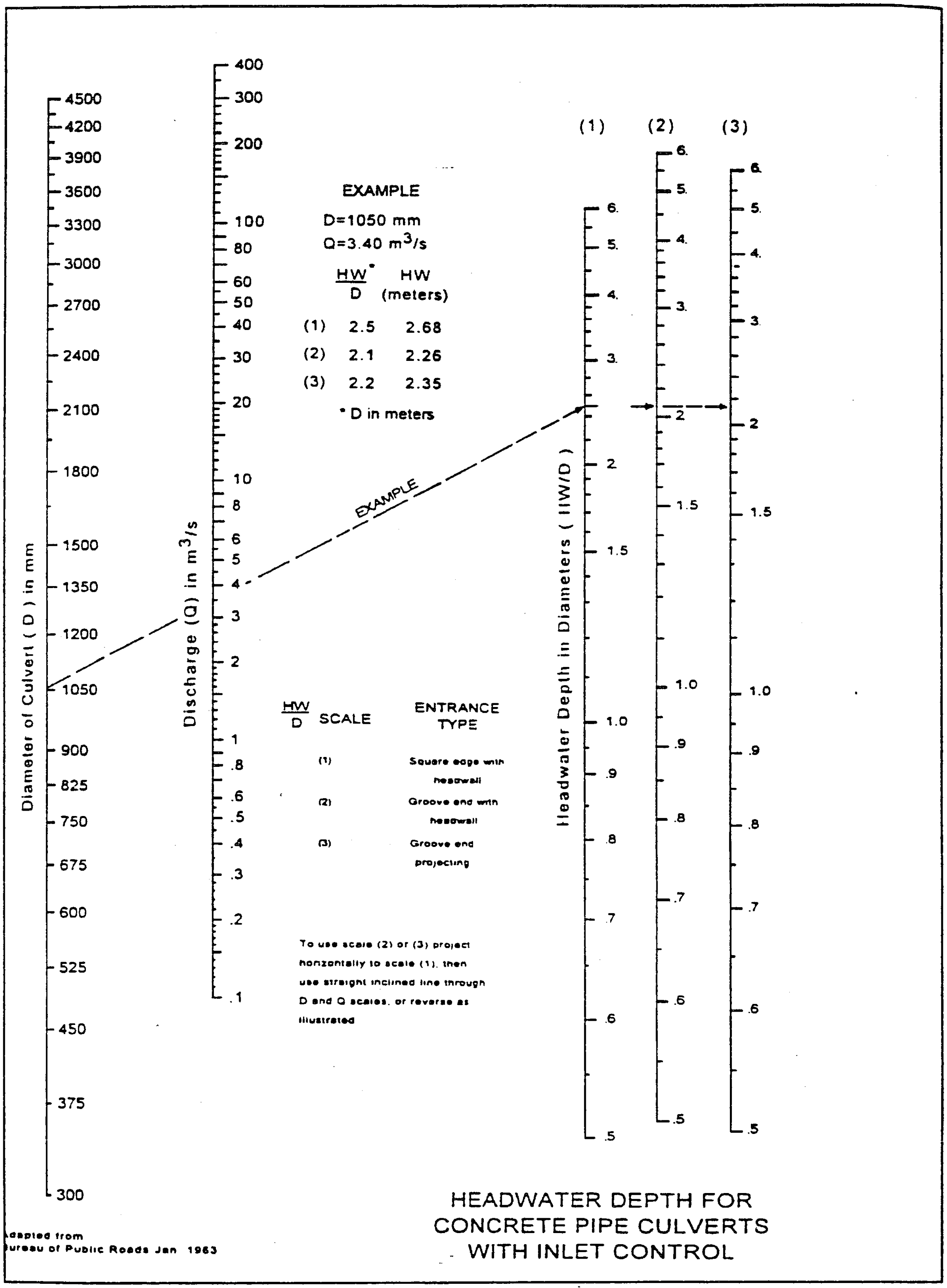
   b. Connect point on $\frac{HW}{D}$ scale (1) as found in (a) above to given discharge and read diameter, height or size of culvert required.

3. To determine discharge (Q)

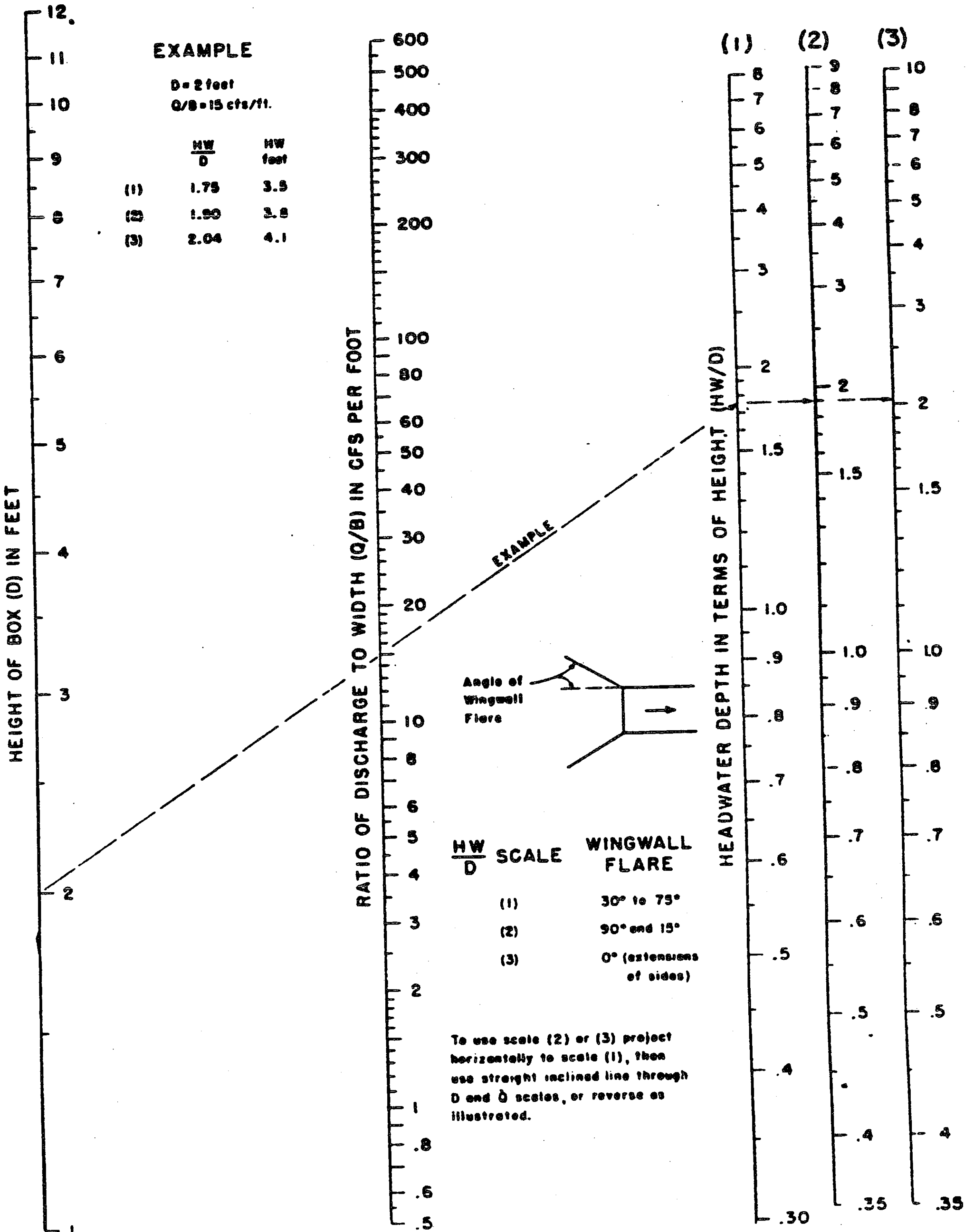
   a. Given HW and D, locate $\frac{HW}{D}$ scale for appropriate entrance type. Continue as in 2a.

   b. Connect point on $\frac{HW}{D}$ scale (1) as found in (a) above and the size of culvert on on the left scale and read Q or $\frac{Q}{B}$ on the discharge scale.

   c. If $\frac{Q}{B}$ is read in (b) multiply by B to find Q.



NOTE: METRIC CONVERSION 1 in = 25.4 mm
1 ft = 0.3048 m



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS
OCTOBER 1960

NOTE: METRIC CONVERSION 1 inch = 25.4 mm
1 ft = 0.3048 m

Instructions for Use

Note: For oval concrete culverts, corrugated metal pipe, and arch pipe culverts refer to the previous version of the "Manual on Drainage Design for Highways."

These nomographs solve equation 3, pg. 4-6, for head H when culverts flow full with outlet control. They are also used in approximating the head for some part-full flow conditions with outlet control. These nomographs do not give a complete solution for finding headwater HW (see "Procedure for Selection of Culvert Size", pg. 4-14, and "Culvert Hydraulics", pg. 4-1).

1. To determine head H for a given culvert and discharge Q.

   a. Locate appropriate nomograph for type of culvert selected.

   b. Begin nomograph solution by locating starting point on length scale.

      (1) To locate the proper starting point on the length scales follow instructions below:

         (a) If the n value of the nomograph corresponds to that of the culvert being used, find the proper k_e from Table 2, Fig. 4-6, pg. 4-13 and on the appropriate nomograph locate starting point on length curve for that k_e. If a k_e curve is not shown for the selected k_e, see b below. If the n value for the culvert selected differs from that of the nomograph, see c below.

         (b) For the n of the nomograph and a k_e intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two chart scales in proportion to the k_e values.

         (c) For a different value of roughness coefficient n_1 than that of the chart n, use the length scales shown with an adjusted length L_1, calculated by the formula.

$$L_1 = L\,(n_1/n)^2$$

See instruction 2 for n values.

c. Using a straightedge, connect point on length scale to size of culvert barrel and mark the point of crossing on the "turning line." See instruction 3 below for size considerations for rectangular box culvert.

d. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in meters on the head (H) scale. For values beyond the limit of the chart scales, find H by solving equation given on nomograph or by $H = KQ^2$ where K is found by substituting values of H and Q from chart.

2. To find the n value for the culvert selected refer to table below:

Concrete		Corrugated Metal	
Pipe	Boxes	Small Corrugations	Large Corrugations
.012	.012	Unpaved ---- .024 25% paved -- .021	Unpaved ---- .030 25% paved -- .025

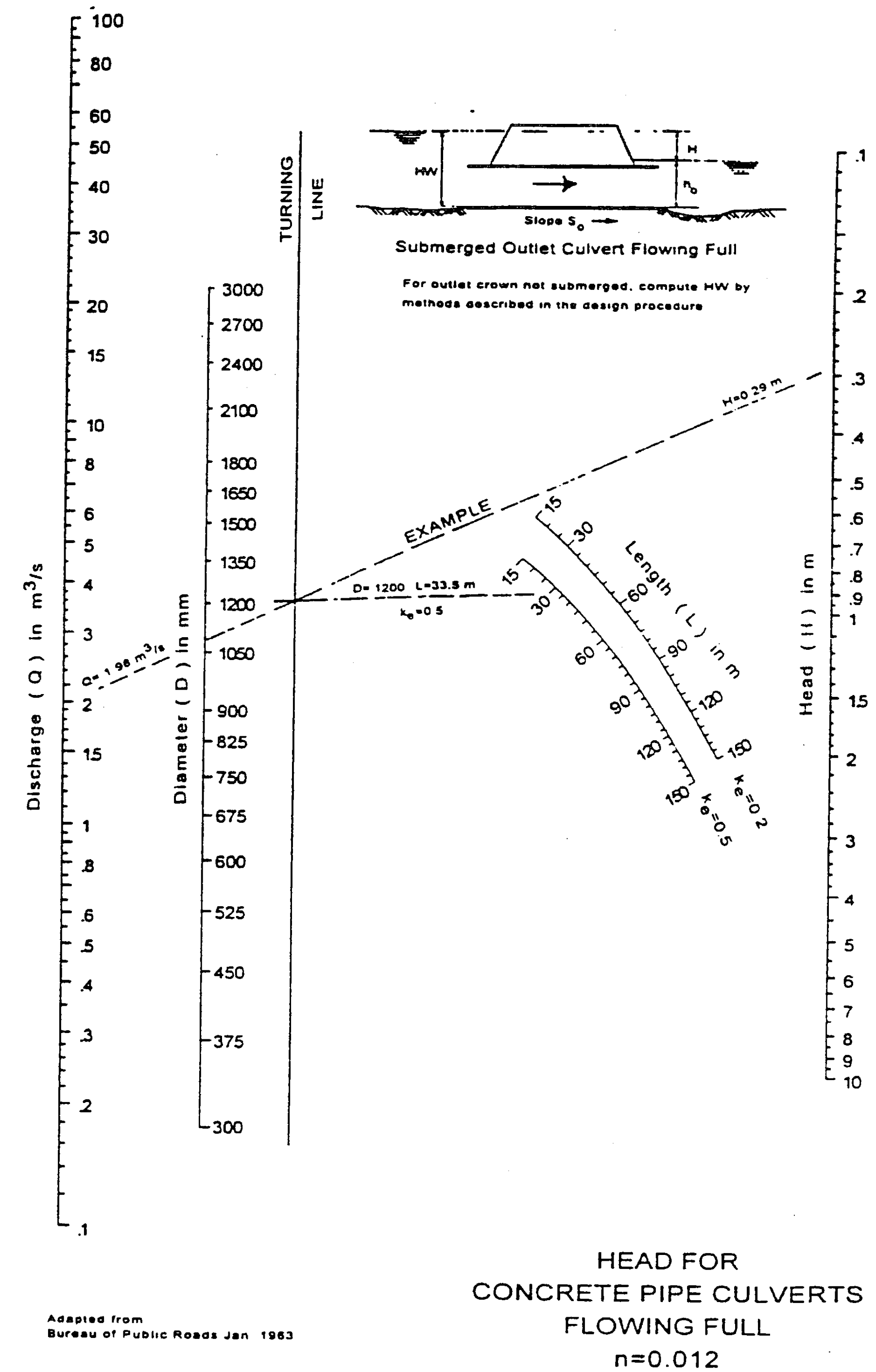
3. To use the box culvert nomograph, Fig. 6-2, for full-flow for other than square boxes.

   a. Compute cross-sectional area of the rectangular box[1].

   b. Connect proper point (see instruction 1) on length scale to barrel area and mark point on turning line.

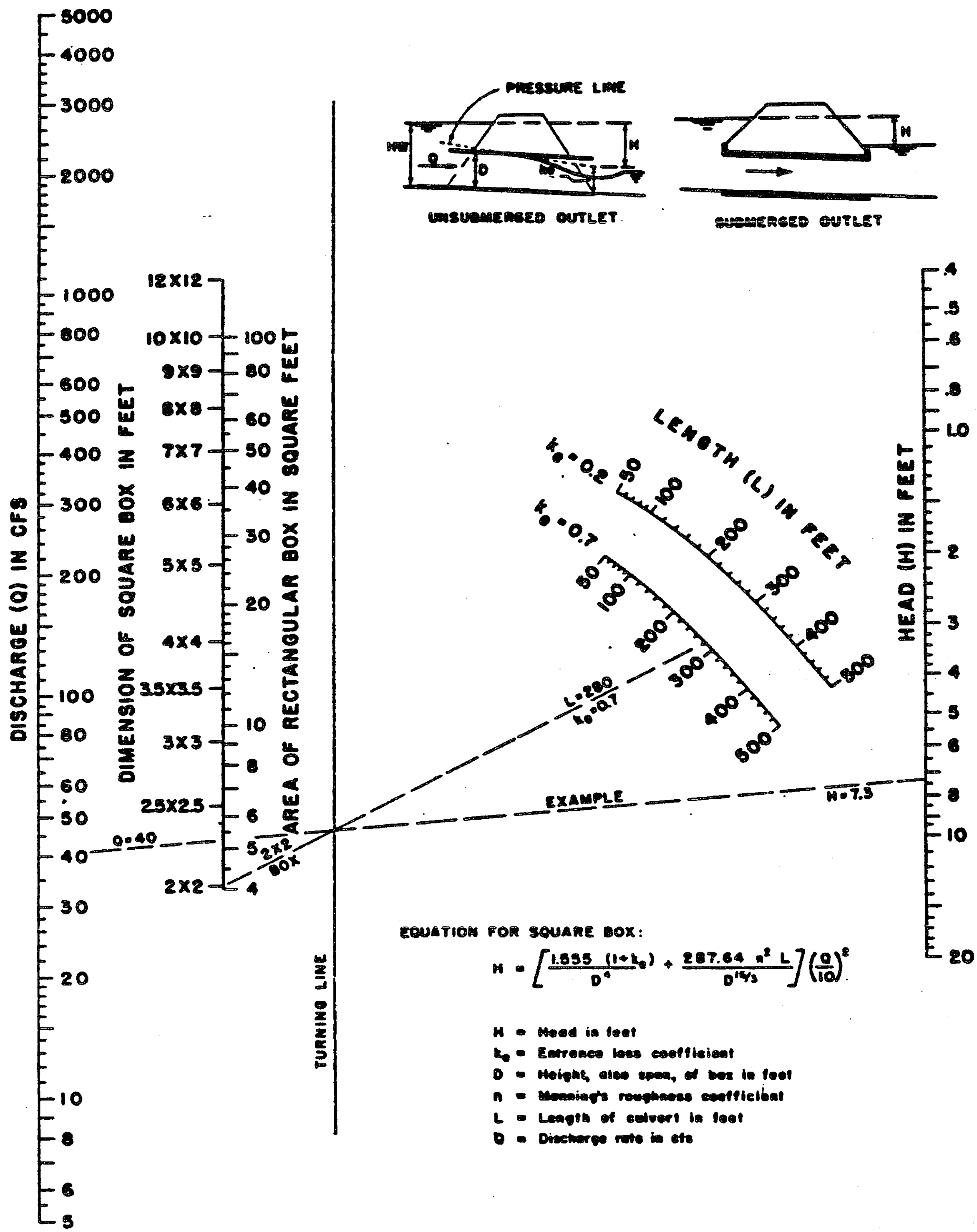
   c. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in meters on the head (H) scale.

1 The area scale on the nomograph is calculated for barrel cross-sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 2/3D. For other box proportions use equation shown on nomograph for more accurate results.



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n=0.012

NOTE: METRIC CONVERSION 1 in. = 25.4 mm
1 ft. = 0.3048 m



BUREAU OF PUBLIC ROADS
OCTOBER 1960

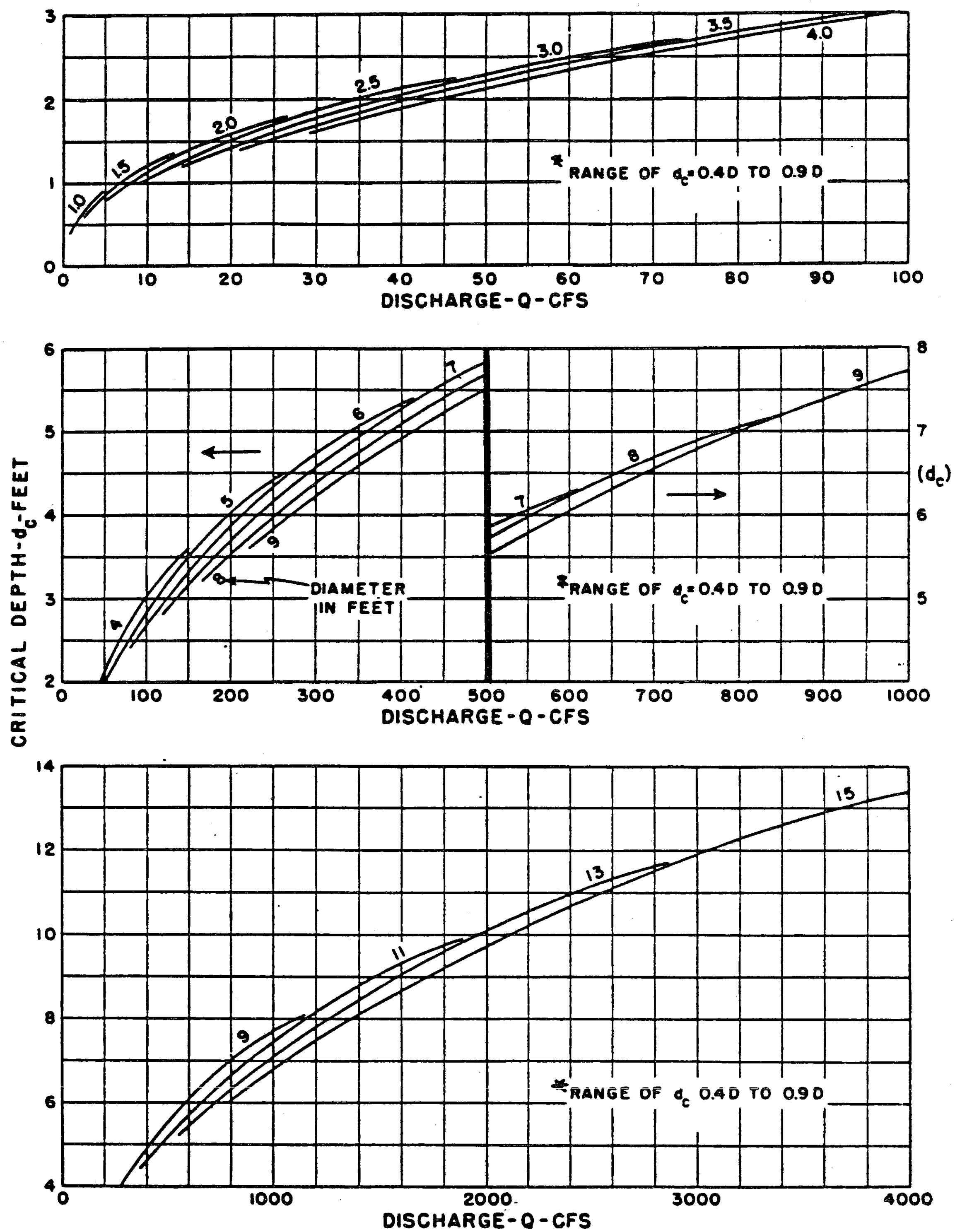
HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
$n = 0.012$

NOTE: METRIC CONVERSION 1 in. = 25.4 mm
1 ft. = 0.3048 m

SECTION 7 - CRITICAL DEPTH CHARTS

Note: For oval concrete pipe, riveted C.M. pipe-arch, and field bolted C.M. pipe-arch critical depths refer to the previous version of the "Manual on Drainage Design for Highways."

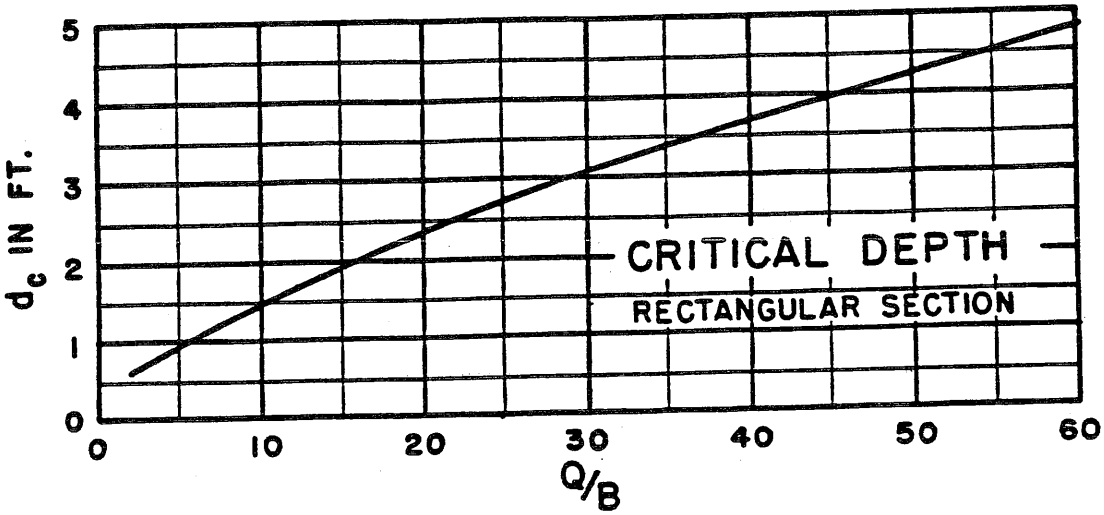
Figure 7-1

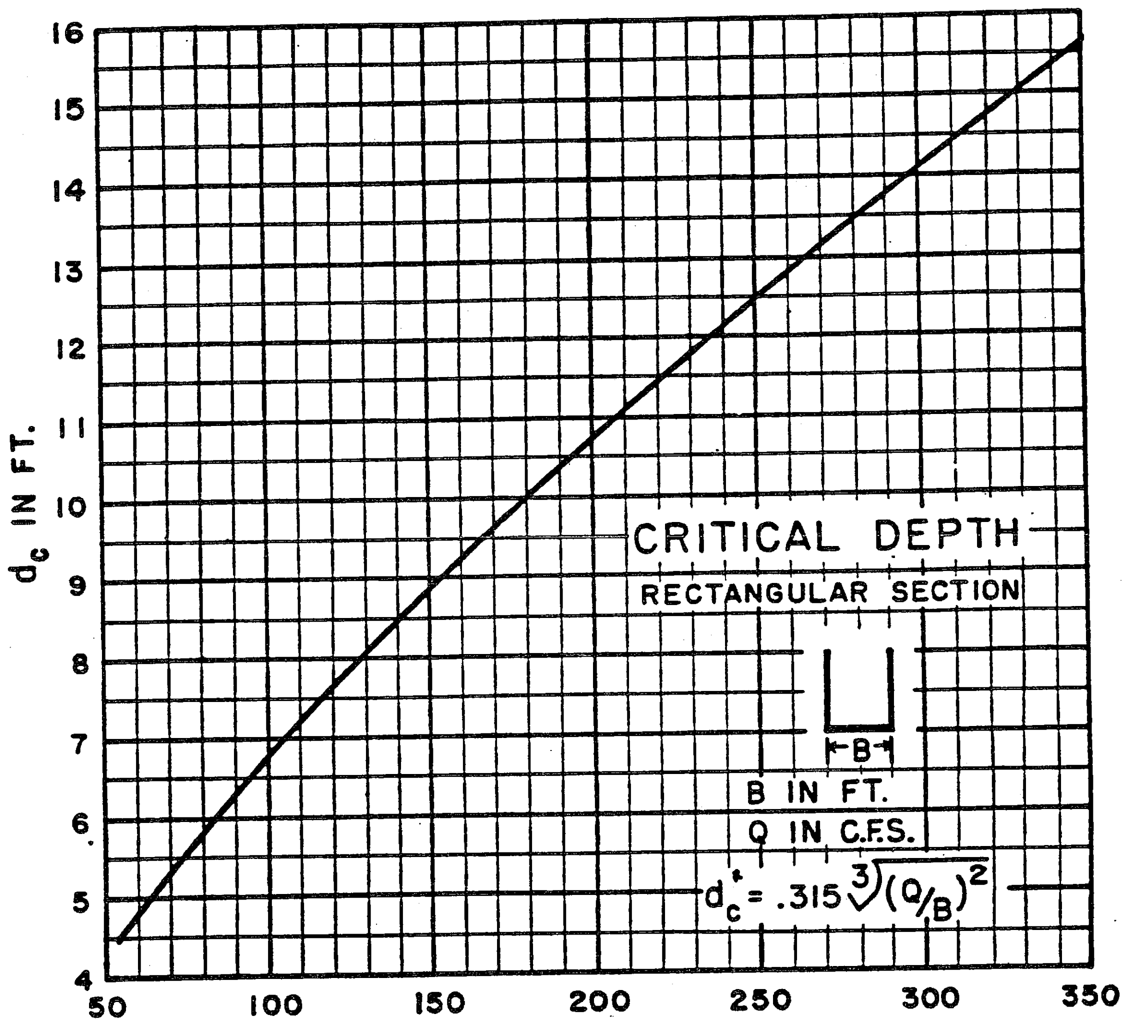


*NOTE: FOR VALUES OF d_c ABOVE CURVE, USE d_c = D

CIRCULAR PIPE CRITICAL DEPTH

NOTE: METRIC CONVERSION 1 in. = 25.4 mm
1 ft. = 0.3048 m





*NOTE: d_c CANNOT EXCEED D

BUREAU OF PUBLIC ROADS
NOVEMBER, 1960

NOTE: METRIC CONVERSION 1 in. = 25.4 mm
1 ft. = 0.3048 m

NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION

EROSION & SEDIMENTATION CONTROL AND STORMWATER MANAGEMENT POLICY

The New Hampshire Department of Transportation (NHDOT) is environmentally conscientious and strongly committed to the prevention of erosion and sedimentation and to the protection of water and other natural resources.

*It is the **policy** of the NHDOT to:*

- Meet or exceed State and Federal environmental laws and regulations
- Incorporate state of the art erosion/sedimentation controls and stormwater management measures into the design of its projects
- Ensure erosion/sedimentation controls and stormwater management measures are properly implemented and maintained during construction
- Properly maintain erosion/sedimentation controls and stormwater management measures after project construction
- Be vigilant in identifying and addressing erosion/sedimentation and stormwater concerns in the State/Federal transportation system
- Support and provide continuing education for all appropriate personnel
- Empower Department Contract Administrators and other supervisory personnel to enforce this policy

Leon S. Kenison, Commissioner

Carol A. Murray, Asst. Commissioner

April 22, 1999

PERMANENT MEASURES FOR EROSION & SEDIMENT CONTROL

PERMANENT MEASURES (1)

FIGURE 8-1

	PLANTING & SEEDING(2)				STRUCTURES																			
	CROWN VETCH	GRASSES (3)	SOD	VEGETATIVE COVER(4)	SECTIONAL DOWN DRAIN	LEVEL SPREADER	TERRACE	INTERSEPTOR DIKE	CHUTES & FLUMES	BERM	ENERGY DISSIPATOR(1)	STONE PROTECTION	STONE DIVERSION DIKE	GRAVEL	CLASS A	CLASS B	RIPRAP	JETTY	GABIONS	RETAINING WALL	STRAIGHT DROP SPILLWAY(5)	BOX INLET DROP SPILLWAY(5)	DROP BOX INLET	OTHER TECHNIQUES(6)
AREAS																								
SLOPES																								
1. CUT OR BACK																								
A - LEDGE																								
B - EARTH	■	■	■	■	■	■	■	■	■															
2. FILL	■	■	■	■	■				■															
3. TOE OF															■	■								
4. SHOULDERS		■	■	■																				
DITCHES																								
1. MEDIAN		■	■	■							■				■									■
2. ROADSIDE		■	■	■	■						■				■									■
3. TERRACE		■	■	■	■	■									■									■
4. BERM		■	■	■	■	■									■									■
5. OUTLET		■	■	■							■	■												■
CULVERTS																								
1. INLETS																					■	■	■	■
2. BARREL											■													
3. OUTLETS											■			■	■	■	■		■	■				■
CHANNEL RELOCATIONS																								
1. STREAMS		■	■	■								■												■
2. RIVERS		■	■	■								■												■
3. SIDE BANKS		■	■	■										■	■	■	■		■	■				■
4. STREAM BED															■	■	■	■	■					■
5. MEANDERS				■											■	■	■		■	■				■
GRAVEL SURFACE RDS.														■	■									■
MISC.																								
1. STORM DRAINS																								
A - INLETS											■													
B - OUTLETS									■		■			■	■	■	■							
2. TEMP. BORROW AREA	■	■	■	■		■	■	■												■				■
3. SPOILS AREA	■	■	■	■				■																■
PROTECTION AGAINST																								
EROSION AT:																								
1. BRIDGE ABUTMENT												■												■
2. PIERS												■												■
3. DIKES		■	■	■								■												■
4. LAKE SHORES		■	■	■								■												■

*( )*SEE NOTES ON PAGE 8-3*

TEMPORARY MEASURES FOR EROSION & SEDIMENT CONTROL

TEMPORARY MEASURES (7) **FIGURE 8-2**

AREAS		MULCHS & MATTINGS				PLANTING & SEEDING			STRUCTURES													
	CHEMICAL STABILIZER(8)	JUTE MATTING	STRAW OR HAY	WOOD CHIPS	OTHER PRODUCTS (9)	CROWN VETCH	ANNUALS	MISC. MEASURES SEE SPEC.	FLEXIBLE DOWN DRAIN	LEVEL SPREADER	CHECK DAMS(10)	STRAW OR HAY BALES	STACKED SAND BAGS	CHUTE, FLUME, 1/2 CIRCLE SLUICE	ROADSIDE BERM	FILTER BERM	FILTER INLET/TRASH RACK	SEDIMENT RETENTION BASIN	DIAPER (FLORIDA)	STONE PROTECTION	TEMPORARY BRIDGE	COFFER DAM
CLEARED & GRUBBED AREAS	(5)		(5)	(5)	(5)		■	■														
ROADWAY SURFACES DURING CONST.														■						■		
SLOPES																						
1. CUT OR BACK																						
A - LEDGE																						
B - EARTH	■		■	■	■	■	■	■	■	■				■						■		
2. FILL	■		■	■	■	■	■	■	■					■						■		
3. TOE OF	■	■	■	■	■	■	■	■				■										
4. SHOULDERS	■		■	■	■		■	■					■		■							
DITCHES																						
1. MEDIAN	(6)	■	■		■		■	■			■									■		
2. ROADSIDE	(6)	■	■		■		■	■		■	■									■		
3. TERRACE	(6)	■			■		■	■	■	■										■		
4. BERM	(6)	■			■		■	■	■	■										■		
5. OUTLETS	(6)	■			■		■	■			■		■							■		
CULVERTS																						
1. INLET DITCHES		■			■						■		■				■	■		■		
2. OUTLET DICHES		■			■								■					■		■		
CHANNEL RELOCATIONS																						
1. STREAMS											■								■	■	■	■
2. RIVERS																			■	■	■	■
3. SIDE BANKS	■		■	■	■	■	■	■												■		
4. STREAM BED											■									■		
5. MEANDERS	■		■	■	■	■	■	■												■		
MISC.																						
1. STORM DRAINS																	■					
A - INLETS																						
B - OUTLETS	■	■			■						■		■	■				■	■	■		
2. TEMP. BORROW AREA	(7)	(7)	(7)	(7)	(7)	(7)	(7)	(7)								■		■				
3. HAUL ROADS															■	■		■				
4. ON SITE STORAGE											■					■		■				
5. SPOILS AREA											■				■	■		■				
6. STREAM CROSSING																				■	■	■
WORK IN																						
1. RIVERS																			■	■		
2. LAKES																			■			

*( ) SEE NOTES ON 8-3, 8-4

EROSION CONTROL MEASURES - NOTES:

PERMANENT

1. Refer to applicable appendix for specifications and use of the permanent measures. These being applied to a given area at the discretion of the Engineer.

2. Refer to Roadside Development Section for details on vegetative ground cover and landscaping techniques.

3. Grasses and Herbaceous Vegetation most often used:

   A - Lawn/slope grasses (Ryegrass, Fescue, Bluegrass, Bermuda Grass)
   B - Legume (Clover, Crownvetch, Trefoil)
   C - Marsh grasses/reeds (Reed Canary Grass)
   D - Other warm season grasses (Switchgrass)
   E - Cereals (Oat, Winter Rye, Buckwheat)
   F - Wildflowers
   G - Forbes

4. Vegetative ground cover:

   A - vines, shrubs & trees
   B - herbaceous plants (annuals/perennials), (bare root, potted, plug forms)
   C - Whips, liners, canes

5. Used as both stream channel degradation and sediment control.

6. A variety of commercial products are available, Bureau of Public Works and Roadside Development Section should be contacted for specifications and use.

TEMPORARY

7. Refer to applicable literature for specifications and use of the temporary measures. These being applied to a given area at the discretion of the Engineer, as the situation dictates.

8. Applied to denuded earth or mulch:

   A - Areospray® 52 Binder
   B - Aquatain
   C - Curasol® AE
   D - Curasol® AH
   E - DCA 70
   F - Liquid Asphalt
   G - Petroset® SB
   H - Terra Tack
   I - Other Commercial Products

9. Available alternatives:

A - Excelsior Blanket
B - Fiberglass Matting
C - Glass Root®
D - Mulch Blanket
E - Plastic Filter Sheet
F - Wood Fiber Mulch

10. Various types of check dams:

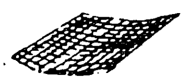
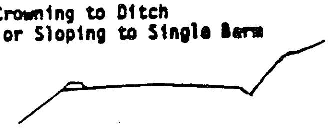
A - Straw or Hat Bale
B - Log & Post
C - Plastic Liner & Fence
D - "Snow Fence"

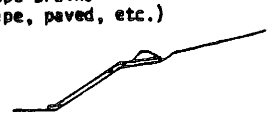
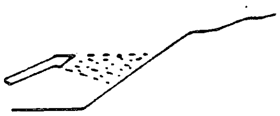
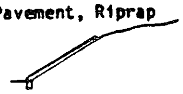
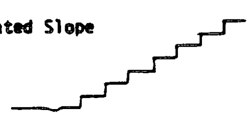
11. Temporary covering may be required if area is to be left denuded for an extended time period.

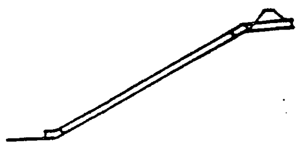
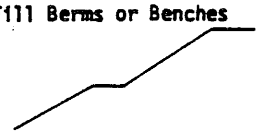
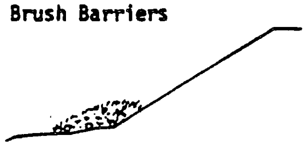
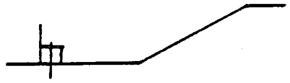
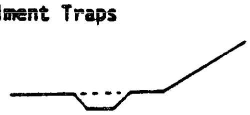
12. It may be necessary to stabilize the soil prior to the establishment of vegetation.

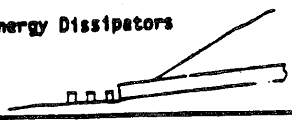
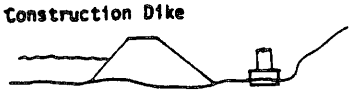
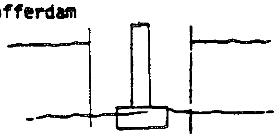
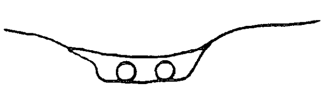
13. It may be necessary to stabilize the borrow area if left unattended during a halt of operations.

ENGINEERING EROSION CONTROL CRITERIA (FIGURE 8-3)

Treatment Practice	Advantages	Problems
ROADWAY DITCHES		
Check Dams 	Maintain low velocities Catch sediment Can be constructed of logs, shot rock, lumber, masonry or concrete	Close spacing on steep grades Require clean-out Unless keyed at sides and bottom, erosion may occur
Sediment Traps/ Straw Bale Filters 	Can be located as necessary to collect sediment during construction Clean-out often can be done with on-the-job equipment Simple to construct	Little direction on spacing and size Sediment disposal may be difficult Specification must include provisions for periodic clean-out May require seeding, sodding or pavement when removed during final cleanup
Sodding 	Easy to place with a minimum of preparation Can be repaired during construction Immediate protection May be used on sides of paved ditches to provide increased capacity	Requires water during first few weeks Sod not always available Will not withstand high velocity or severe abrasion from sediment load
Seeding with Mulch and Matting 	Usually least expensive Effective for ditches with low velocity Easily placed in small quantities with inexperienced personnel	Will not withstand medium to high velocity
Paving, Riprap, Rubble	Effective for high velocities May be part of the permanent erosion control effort*	Cannot always be placed when needed because of construction traffic and final grading and dressing Initial cost is high
ROADWAY SURFACE		
Crowning to Ditch or Sloping to Single Berm 	Directing the surface water to a prepared or protected ditch minimizes erosion	None - should be part of good construction procedures
Compaction	The final lift of each day's work should be well compacted and bladed to drain to ditch or berm section. Loose or uncompacted material is more subject to erosion	None - should be part of good construction procedures
Aggregate Cover 	Minimizes surface erosion Permits construction traffic during adverse weather May be used as part of permanent base construction	Requires reworking and compaction if exposed for long periods of time Loss of surface aggregates can be anticipated
Seed/Mulch	Minimizes surface erosion	Must be removed or is lost when construction of pavement is commenced

Treatment Practice	Advantages	Problems
CUT SLOPES		
Berm @ top of cut 	Diverts water from cut Collects water for slope drains/paved ditches May be constructed before grading is started	Access to top of cut Difficult to build on steep natural slope or rock surface Concentrates water and may require channel protection or energy dissipation devices Can cause water to enter ground, resulting in sloughing of the cut slope
Diversion Dike 	Collects and diverts water at a location selected to reduce erosion potential May be incorporated in the permanent project drainage	Access for construction May be continuing maintenance problem if not paved or protected Disturbed material or berm is easily eroded
Slope Benches 	Slows velocity of surface runoff Collects sediment Provides access to slope for seeding, mulching, and maintenance Collects water for slope drains or may divert water to natural ground	May cause sloughing of slopes if water infiltrates Requires additional ROW Not always possible due to rotten material etc. Requires maintenance to be effective Increases excavation quantities
Slope Drains (pipe, paved, etc.) 	Prevents erosion on the slope Can be temporary or part of permanent construction Can be constructed or extended as grading progresses	Requires supporting effort to collect water Permanent construction is not always compatible with other project work Usually requires some type of energy dissipation
Seeding/Mulching 	The end objective is to have a completely grassed slope. Early placement is a step in this direction. The mulch provides temporary erosion protection until grass is rooted. Temporary or permanent seeding may be used. Mulch should be anchored. Larger slopes can be seeded and mulched with smaller equipment if stage techniques are used.	Difficult to schedule high production units for small increments Time of year may be less desirable May require supplemental water Contractor may perform this operation with untrained or unexperienced personnel and inadequate equipment if stage seeding is required
Sodding 	Provides immediate protection Can be used to protect adjacent property from sediment and turbidity	Difficult to place until cut is complete Sod not always available May be expensive
Slope Pavement, Riprap 	Provides immediate protection for high risk areas and under structures May be cast in place or off site	Expensive Difficult to place on high slopes May be difficult to maintain
Temporary Cover	Plastics are available in wide rolls and large sheets that may be used to provide temporary protection for cut or fill slopes Easy to place and remove Useful to protect high risk areas from temporary erosion	Provides only temporary protection Original surface usually requires additional treatment when plastic is removed Must be anchored to prevent wind damage
Serrated Slope 	Lowers velocity of surface runoff Collects sediment Holds moisture Minimizes amount of sediment reaching roadside ditch	May cause minor sloughing if water infiltrates Construction compliance

Treatment Practice	Advantages	Problems
FILL SLOPES		
Berms at Top of Embankment 	Prevent runoff from embankment surface from flowing over face of fill Collect runoff for slope drains or protected ditch Can be placed as a part of the normal construction operation and incorporated into fill or shoulders	Cooperation of construction operators to place final lifts at edge for shaping into berm Failure to compact outside lift when work is resumed Sediment buildup and berm failure
Slope Drains 	Prevent fill slope erosion caused by embankment surface runoff Can be constructed of full or half section pipe, bituminous, metal, concrete, plastic, or other waterproof material Can be extended as construction progresses May be either temporary or permanent	Permanent construction as needed may not be considered desirable by contractor Removal of temporary drains may disturb growing vegetation Energy dissipation devices are required at the outlets
Fill Berms or Benches 	Slows velocity of slope runoff Collects sediment Provides access for maintenance Collects water for slope drains May utilize waste	Requires additional fill material if waste is not available May cause sloughing Additional ROW may be needed
Seeding/Mulching	Timely application of mulch and seeding decreases the period a slope is subject to severe erosion Mulch that is cut in or otherwise anchored will collect sediment. The furrows made will also hold water and sediment	Seeding season may not be favorable Not 100 percent effective in preventing erision Watering may be necessary Steep slopes or locations with low velocities may require supplemental treatment
PROTECTION OF ADJACENT PROPERTY		
Brush Barriers 	Use slashing and logs from clearing operation Can be covered and seeded rather than removed Eliminates need for burning or disposal off ROW	May be considered unsightly in urban areas
Straw Bale Barriers 	Straw is readily available in many areas When properly installed, they filter sediment and some turbidity from runoff	Require removal Subject to vandal damage Flow is slow through straw requiring considerable area
Sediment Traps 	Collect much of the sediment spill from fill slopes and storm drain ditches Inexpensive Can be cleaned and expanded to meet need	Do not eliminate all sediment and turbidity Space is not always available Must be removed (usually)
Sediment Pools 	Can be designed to handle large volumes of flow Both sediment and turbidity are removed May be incorporated into permanent erosion control plan	Require prior planning, additional ROW and/or flow easement If removal is necessary, can present a major effort during final construction stage Clean-out volumes can be large Access for clean-out not always convenient

Treatment Practice	Advantages	Problems
PROTECTION OF ADJACENT PROPERTY (continued)		
Energy Dissipators 	Slow velocity to permit sediment collection and to minimize channel erosion off project	Collect debris and require cleaning Require special design and construction of large shot rock or other suitable material from project
Level Spreaders 	Convert collected channel or pipe flow back to sheet flow Avoid channel easements and construction off project Simple to construct	Adequate spreader length may not be available Sodding of overflow berm is usually required Must be a part of the permanent erosion control effort Maintenance forces must maintain spreader until no longer required
PROTECTION OF STREAM		
Construction Dike 	Permits work to continue during normal stream stages Controlled flooding can be accomplished during periods of inactivity	Usually requires pumping of work site water into sediment pond Subject to erosion from stream and from direct rainfall on dike
Cofferdam 	Work can be continued during most anticipated stream conditions Clear water can be pumped directly back into stream No material deposited in stream	Expensive
Temporary Stream Channel Change	Prepared channel keeps normal flows away from construction	New channel usually will require protection Stream must be returned to old channel and temporary channel refilled
Riprap	Sacked sand with cement or stone easy to stockpile and place Can be installed in increments as needed	Expensive
Temporary Culverts for Haul Roads 	Eliminate stream turbulence and turbidity Provide unobstructed passage for fish and other water life Capacity for normal flow can be provided with storm water flowing over the roadway	Space not always available without conflicting with permanent structure work May be expensive, especially for larger sizes of pipe Subject to washout
Rock-lined Low-Level Crossing 	Minimizes stream turbidity Inexpensive May also serve as ditch check or sediment trap	May not be fordable during rainstorms During periods of low flow passage of fish may be blocked

Treatment Practice	Advantages	Problems
BORROW AREAS		
Selective Grading and Shaping	Water can be directed to minimize off-site damage Flatter slopes enable mulch to be cut into soil	May not be most economical work method for contractor
Stripping and Replacing of Topsoil	Provides better seed bed Conventional equipment can be used to stockpile and spread topsoil	May restrict volume of material that can be obtained for a site Topsoil stockpiles must be located to minimize sediment damage Cost of rehandling material
Dikes, Berms Diversion Ditches Settling Basins Sediment Traps Seeding & Mulch	See other practices	See other practices

OTHER MANUALS APPROVED BY THE DEPARTMENT FOR EROSION CONTROL/WATER QUALITY - STANDARD PROCEDURES:

A) Volume III, AASHTO Highway Drainage Guidelines, 1992
B) EPA, Storm Water Management for Construction Activities, September 1992
C) New Hampshire Department of Environmental Services, Rockingham County Conservation District, USDA Soil Conservation Service "Stormwater Management and Erosion and Sediment Control Handbook for Urban and Developing Areas in New Hampshire", August 1992
D) U.S. Department of Transportation, Best Management Practices for Erosion and Sediment Control, June 1995

a/ Highway Research Board, "National Cooperative Highway Research Program Synthesis of Highway Practice No. 18", "Erosion Control on Highway Construction," Division of Engineering, National Research Council, National Academy of Sciences - National Academy of Engineering, pp. 42-46 (1973).

SECTION 9 -ROADSIDE DITCH CHANNEL CHARTS

STATE OF NEW HAMPSHIRE
DEPARTMENT OF TRANSPORTATION
Bureau of HighwayDesign

I. CHART NUMBERS Figs. 2 through 7

The above listed charts were prepared by the Ohio Department of Highways. These charts can be used in the design of roadway ditches in the State of New Hampshire, whenever the side slopes of the roadway being designed correspond with the slopes shown on one of the above mentioned charts.

II. EXPLANATION OF CHARTS

The abscissa scale is discharge, Q and the ordinate scale is velocity. Both scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth of flow, and the slightly inclined lines represent channel slope.

Auxiliary abscissa and ordinate scales are provided for use with values of **n** other than those used in preparing the charts.

In using these charts, interpolations may be made, not only on the ordinate and abscissa scales, but between the inclined lines representing depth and slope.

III. INSTRUCTIONS FOR USING CHARTS

These charts provide a solution of the Manning equation for flow in open channels of uniform slope, cross sections, and roughness, provided the flow is not affected by backwater and the channel has a length sufficient to establish uniform flow. These charts will provide sufficient accuracy for the design of highway drainage channels of fairly uniform cross section and slope.

III-A. USE OF CHARTS WITH BASIC CHART-DESIGN VALUE OF n

For a given slope and cross section of ditch when n = 0.03, the depth and velocity of uniform flow may be read directly from the chart for that size ditch. The initial step is to locate the intersection of a vertical line through the discharge (abscissa) and the appropriate slope line. At this intersection, the depth of flow is read from the depth lines; and the mean velocity is read on the ordinate scale opposite the point of intersection (see Example 1).

The procedure is reversed to determine the discharge at a given depth of flow (see Example 2).

Example 1

Given: A roadside ditch with side slopes of 6:1 and 4:1 and a rounded bottom, with n = 0.03, on 4% slope (S = 0.04), discharging 4.25 m^3/s.

Find: Depth, velocity

1. Select the appropriate chart, (Fig. 2)

2. From 4.25 m^3/s on the Q scale move vertically to intersect the slope line S = 0.04, and from the depth lines read d = 0.45 meter.

3. Move horizontally from the same intersection and read the normal velocity, V = 2.8 m/s, on the ordinate scale.

Example 2

Given: A roadside ditch with side slopes of 4:1 and 3:1 and a rounded bottom, with n = 0.03, on a 1% slope S = (0.01), with a depth of flow of 0.23 meter.

Find: Discharge and Velocity

1. Select the appropriate chart, (Fig. 3)

2. Locate the intersection of the 0.23 meter depth line and slope line S = 0.01 and, moving vertically to the abscissa scale, read the corresponding discharge, Q = 0.48 m^3/s.

3. Move horizontally from the intersection and read the normal velocity, V = 0.95 m/s, on the ordinate scale.

III-B. USE OF CHARTS WITH OTHER THAN BASIC CHART-DESIGN VALUE OF n

Auxiliary scales, labeled Qn (abscissa) and Vn (ordinate), are provided on the charts so that they may be used for values of n other than those for which the charts were basically prepared. To use the auxiliary scales, multiply the discharge by the value of n and use the Qn and Vn scales instead of the Q and V scales. To obtain normal velocity V from a value of the Vn scale, divide the value of n (see Example #3).

Example 3

Given: Suppose that you decide that the velocity of flow found in Example 1 was excessive and would cause erosion in the ditch. You now decide to pave the ditch with a concrete, float finished. The new value of n is 0.015, other given data remains the same, that is: side slopes 6:1 and 4:1, rounded bottom of a 4% slope ($S = 0.04$), discharging 4.25 m^3/s.

Find: Depth, Velocity

1. Select the appropriate chart, (Fig. 2)

2. Multiply Q times n: $(Qn) = (4.25)(0.015) = 0.06375$

3. From 2.25 on the Qn scale move vertically to intersect the slope line $S = 0.04$, and from the depth lines read **d** = 0.326 meter.

4. Move horizontally from the same intersection and read the Vn scale, Vn = 0.22

5. Divide Vn by n: $\frac{(Vn)}{n} = \frac{0.067}{.015} = 4.47$ The new velocity is then

   4.48 m/s

Note: The velocity has now increased because the friction factor was decreased as a result of paving. However the channel is now protected against erosion. The ditch should be paved to at least a depth of 0.326 meter for adequate protection.

SUMMARY

Table 1 attached gives Manning roughness coefficients, (n) for various channel linings.

If you do not find attached a chart for the geometrically shaped channel you are designing, you may find it in the Bureau of Public Roads Hydraulic Design Series No. 3 "Design Charts for Open-Channel Flow" which is located in the Hydraulic Library.

Charts for the following shaped channels can be found in Hydraulic Design Series No. 3:

a. Rectangular
b. Trapezoidal
c. Triangular
d. Pipe Arch
e. Oval Concrete
f. Circular Pipe

You will find the Circular Pipe Charts particularly useful in designing storm drainage sewers.

Table 1. -- Manning roughness coefficients, n [1]

	Manning's n range[2]
I. Closed conduits	
A. Concrete Pipe	0.011-0.013
B.Corragated-metal or pipe-arch:	
1. 2 2/3 by ½-inch corrugation (riveted pipe):[3]	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 25 and 50 per-cent of circumference paved):	
(1) Flow full depth	0.021-0.018
(2) Flow 0.8 depth	0.021-0.016
(3) Flow 0.6 depth	0.019-0.013
2. 6 by 2-in. corrugation (field bolted)	0.03
C. Vitrified clay pipe	0.012-0.014
D. Cast-iron pipe, uncoated	0.013
E. Steel Pipe	0.009-0.011
F. Brick	0.014-0.017
G. Monolithic concrete:	
1. Wood forms, rough	0.015-0.017
2. Wood forms, smooth	0.012-0.014
3. Steel forms	0.012-0.013
H. Cemented rubble masonry walls	
1. Concrete floor and top	0.017-0.022
2. Natural floor	0.0190-0.025
I. Laminated treated wood	0.015- 0.017
J. Vitrified clay liner plates	0.015
II. Open channels, lined[4] (straight alinement):[5]	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013-0.017
2. Trowel finish	0.012-0.014
3. Float finish	0.013-0.015
4. Float finish, some gravel on bottom	0.015-0.017
5. Gunite, good section	0.016-0.019
6. Gunite, wavy section	0.018-0.022
B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar	0.015-0.017
2. Random stone in mortar	0.017-0.020
3. Cement rubble masonry	0.020-0.030
4. Cement rubble masonry, plastered	0.016-0.020
5. Dry rubble (riprap)	0.020-0.030
C. Gravel bottom, sides as indicated	
1. Formed concrete	0.017-0.020
2. Random stone in mortar	0.020-0.023
3. Dry rubble (riprap)	0.023-0.033
D. Brick	0.014-0.017
E. Asphalt:	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011-0.013
G. Concrete-lined excavated rock	
1. good section	0.017-0.020
2. Irregular section	0.022-0.027
III. Open channels, excavated[4] (straight alinement[5], natural lining):	

	Manning's n range
A. Earth, uniform section:	
1. Clean, recently completed	0.016-0.018
2. Clean, after weathering	0.018-0.020
3. With short grass, few weeds	0.022-0.027
4. In gravelly soil, uniform section, clean	0.022-0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022-0.025
2. Grass, some weeds	0.025-0.030
3. Dense weeds or aquatic plants in deep channels	0.030-0.035
4. Sides clean, gravel bottom	0.025-0.030
5. Sides clean, cobble bottom	0.030-0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028-0.033
2. Light brush on banks	0.035-0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035-0.040
b. Jagged and irregular	0.040-0.045
E. Channels not maintained, weeds and brush uncut:	
1. Dense weeds, high as flow depth	0.08-0.12
2. Clean bottom, brush on sides	0.05-0.08
3. Clean bottom, brush on sides, highest stage of flow	0.07-0.11
4. Dense brush, high stage	0.10-0.14
IV. Highway Channels and swales with maintained vegetation[5,6] (values shown are for velocities of 2 and 6 ft./s)	
A. Depth of flow up to 0.7 ft.:	
1. Bermuda grass, Kentucky bluegrass, Buffalo grass:	
a. Mowed to 2 inches	0.07-0.045
b. length 4 to 6 inches	0.09-0.05
2. Good stand, any grass:	
a. Length about 12 inches	0.18-0.09
b. Length about 24 inches	0.30-0.15
3. fair stand, any grass:	
a. Length about 12 inches	0.14-0.08
b. Length about 24 inches	0.25-0.13
B. Depth of flow 0.7-1.5 ft.	
1. Bermuda grass, Kentucky bluegrass, Buffalo grass:	
a. Mowed to 2 inches	0.05-0.035
b. length 4 to 6 inches	0.06-0.04
2. Good stand, any grass:	
a. Length about 12 inches	0.12-0.07
b. Length about 24 inches	0.20-0.10
3. fair stand, any grass:	
a. Length about 12 inches	0.10-0.06
b. Length about 24 inches	0.17-0.09
V. 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. For gutters with small slope, where sediment may accumulate, increase all above *n* values by	0.002

VI. Natural stream channels

A. Minor streams (surface width at flood stage less than 100 ft.)	
1. Fairly regular section:	0.030-0.035
a. Some grass and weeds, little or no brush	0.035-0.05
b. Dense growth of weeds, depth of flow materially greater than weed height.	0.035-0.05
c. Some weeds, light brush on banks	0.05-0.07
d. Some weeds, heavy brush on banks	0.06-0.05
e. Some weeds, dense willows on bank	0.01-0.02
f. For trees within channel, with branches submerged at high stage, increase all above values by	0.01-0.02
2. Irregular sections, with pools, slight channel meander; increase values given in 1a-e about	
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	0.04-0.05
a. Bottom of gravel, cobbles, and few boulders	0.04-0.05
b. Bottom of cobbles, with large boulders	0.05-0.07
a. Winter	0.07-0.11
b. Summer	0.10-0.16
6. Dense willows, summer, not bent over by current	0.15-0.20
7. Cleared land with tree stumps, 100-150 per acre:	
a. No sprouts	0.04-0.05
b. With heavy growth of sprouts	0.06-0.08
8. Heavy stand of timber, a few down trees, little undergrowth	
a. Flood depth below branches	0.10-0.12
b. Flood depth reaches branches	0.12-0.16
C. Major streams (surface width at flood stage more than 100ft.): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks r vegetation on banks. Values of n may be somewhat reduced. Follow recommendation in publication cited [8] if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028-0.33

Footnotes to table:

[1] Estimates are by Bureau of Public Roads unless otherwise noted and are for straight alinement. A small increase in *n* may be made for channel alinement other than straight.

[2] Ranges for secs. I through III are for good to fair construction. For poor quality construction, use larger *n* values.

[3] *Friction Losses in Corrugated Metal Pipe*, by M.J. Webster and L.R. Metcalf, Corp. of Engineers, Department of the Army; published in Journal of the Hydraulics Division, Proceedings of ASCE, Vol. 85, No. HY 9, September 1959, Paper No. 2148, pp.35-67.

[4] For important work and where accurate determination of water profiles is necessary, the designer is urged to consult the following references and to select *n* by comparison of the specific conditions with the channels tested:
Flow of Water in Irrigation and Similar Canals, by F.C. Scobey, US Dept. of Agriculture, Technical Bulletin No. 625, February 1939. *Flow of Water in Drainage Channels*, by C.E. Ramser, US Department of Agriculture, Technical Bulletin No. 129, November 1929.

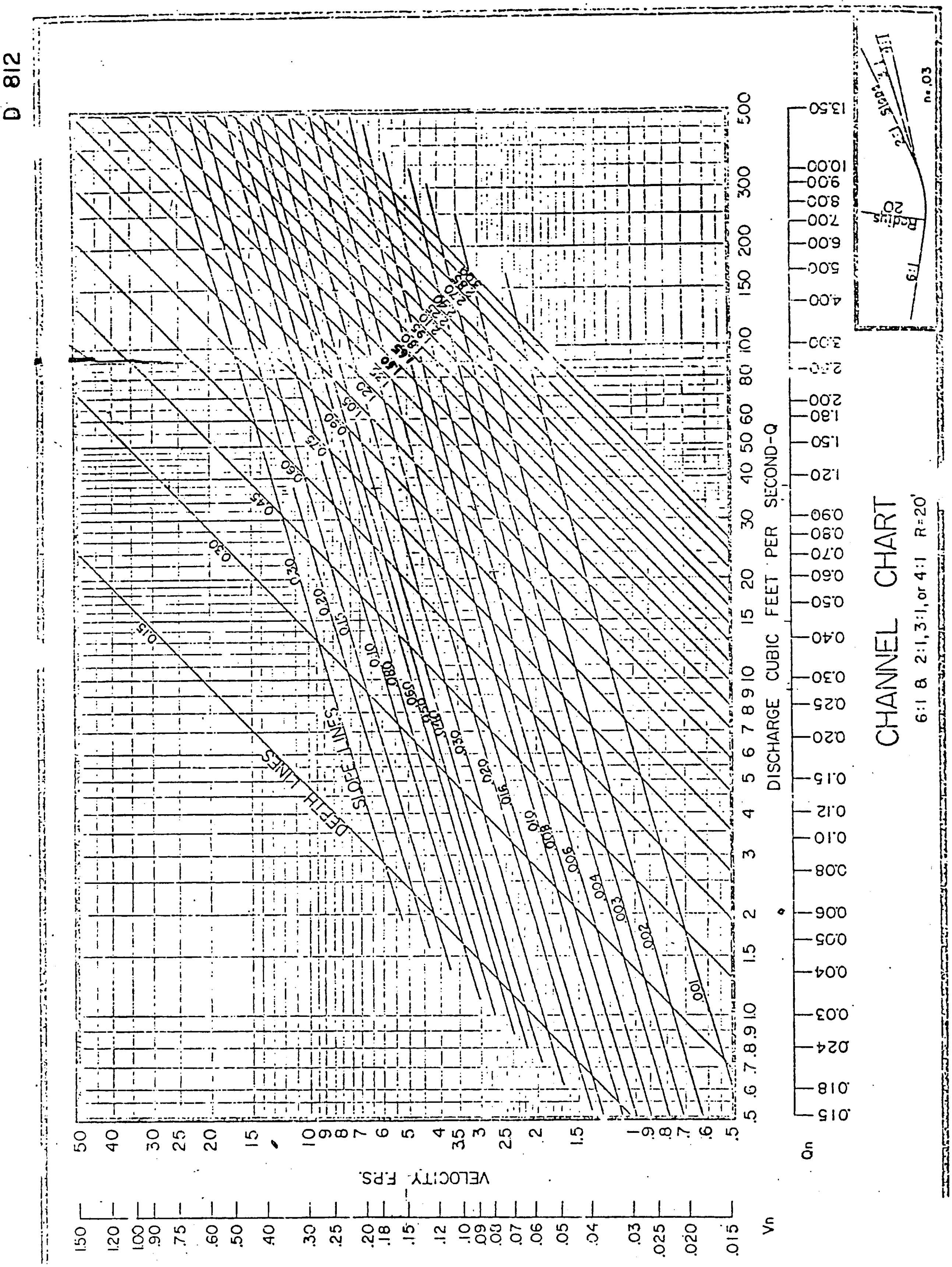
[5] *Handbook of Channel Design for Soil and Water Conservation*, prepared by the Stillwater Outdoor Hydraulic Laboratory in cooperation with the Oklahoma Agricultural Experiment Station. Published by the Soil Conservation Service, US Department of Agriculture, Publ. No. SCS-TP-61, March 1957, rev. June 1954.

[6] *Flow of Water in Channels, Protected by Vegetative Linings*, W.O. Ree and V.J. Palmer, Division of Drainage and Water Control, Research, Soil Conservation Service, US Department of Agriculture, tech. Bull. No. 967, February 1949.

[7] For calculations of stage or discharge in natural stream channels, it is recommended that the designer consult the local District Office of the Surface Water Branch of the US Geological Survey, to obtain data regarding values of *n* applicable to streams or any specific locality. Where this procedure is not followed, the table may be used as a guide. The values of *n* tabulated have been derived from data reported by C.E. Ramser (see footnote 4) and from other incomplete data.

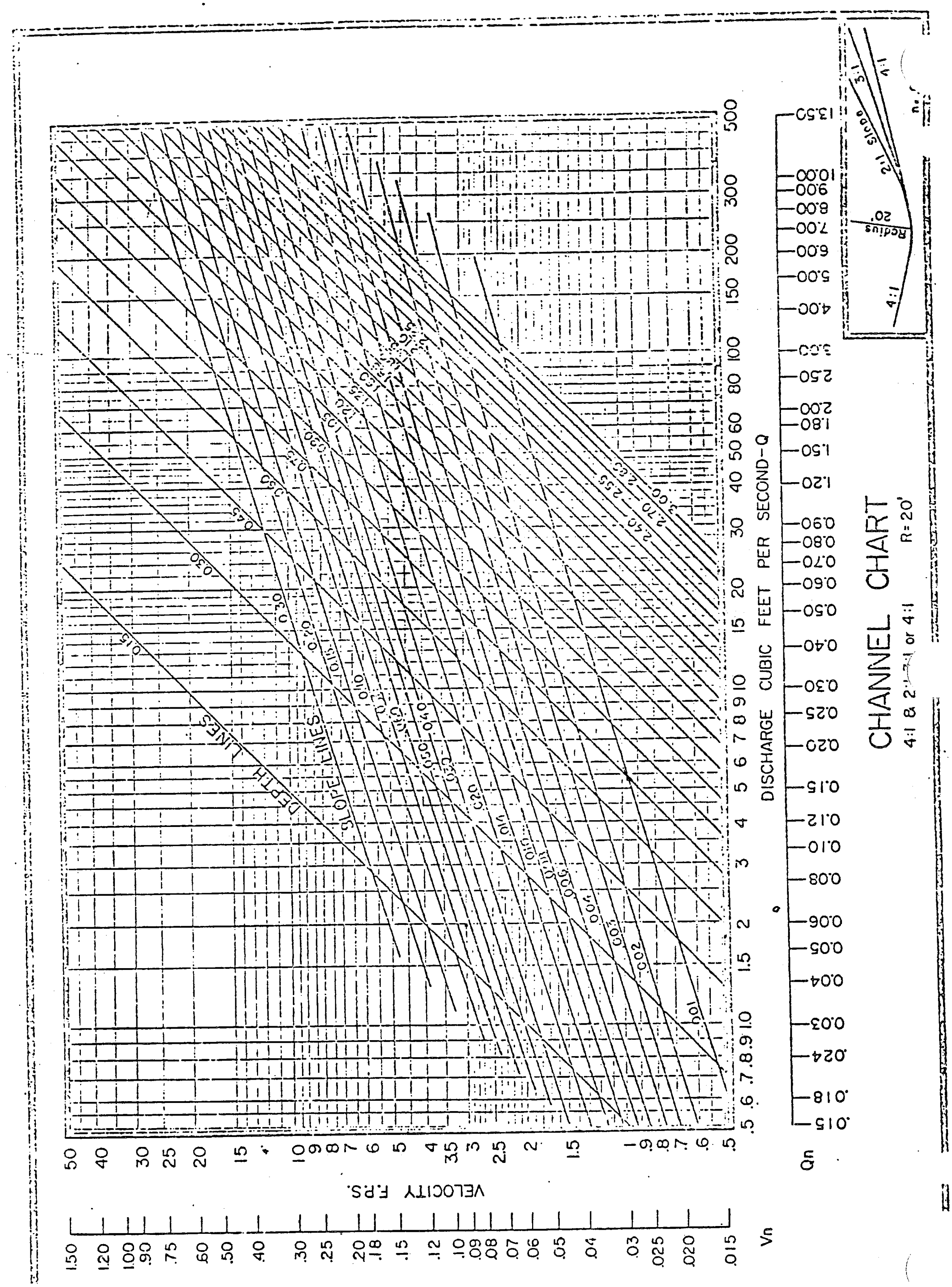
[8] The tentative values of *n* cited are principally derived from measurements made on fairly short but straight reaches of natural streams. Where slopes calculated from flood elevations along a considerable length of channel, involving meanders and bends, are to be used in velocity calculations by the Manning Formula, the value of *n* must be increased to provide for the additional loss of energy caused by the bends. The increase may be in the range of perhaps 3 to 15 percent.

[9] The presence of foliage on trees and brush under flood stage will materially increase the value of *n*. Therefore roughness coefficients for vegetation in leaf will be larger than for bare branches. For trees in channels or on banks, and for brush on banks where submergence of branches increases with depth of flow, *n* will increase with rising stage.



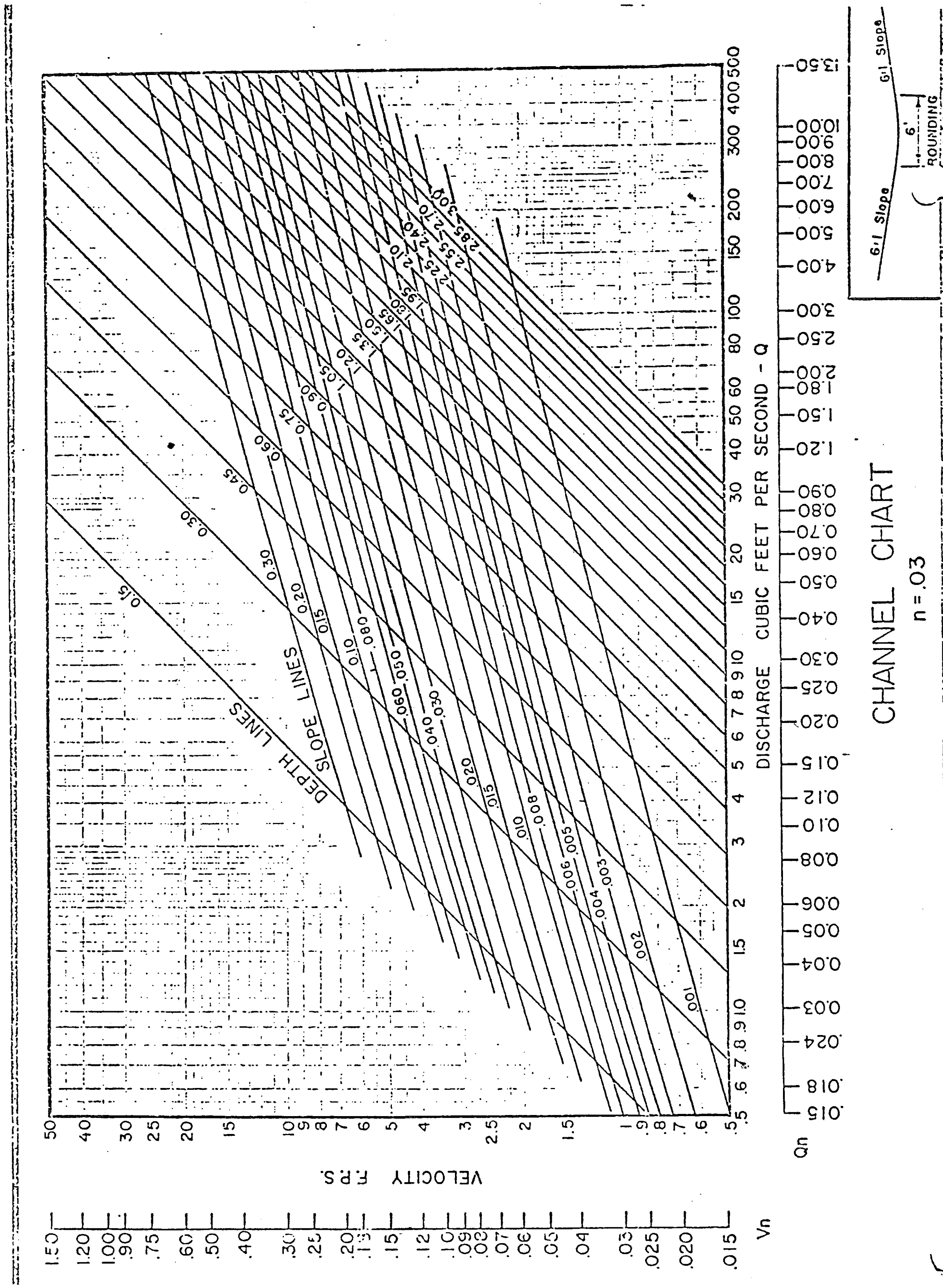
METRIC CONVERSION: 1 in =25.4 mm
1 ft = 0.3048 m

FIGURE 2



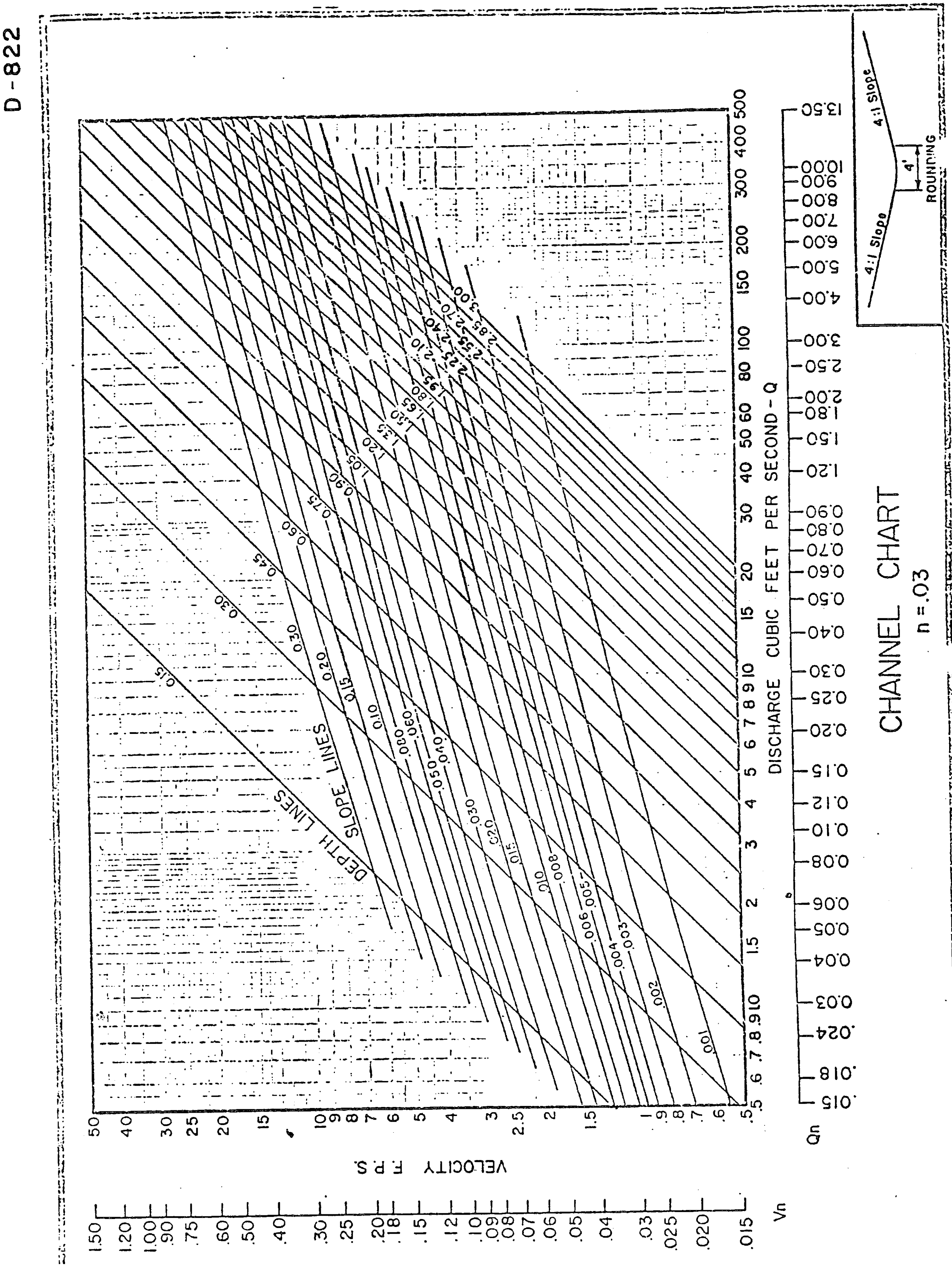
METRIC CONVERSION: 1 in =25.4 mm
1 ft = 0.3048 m

FIGURE 3



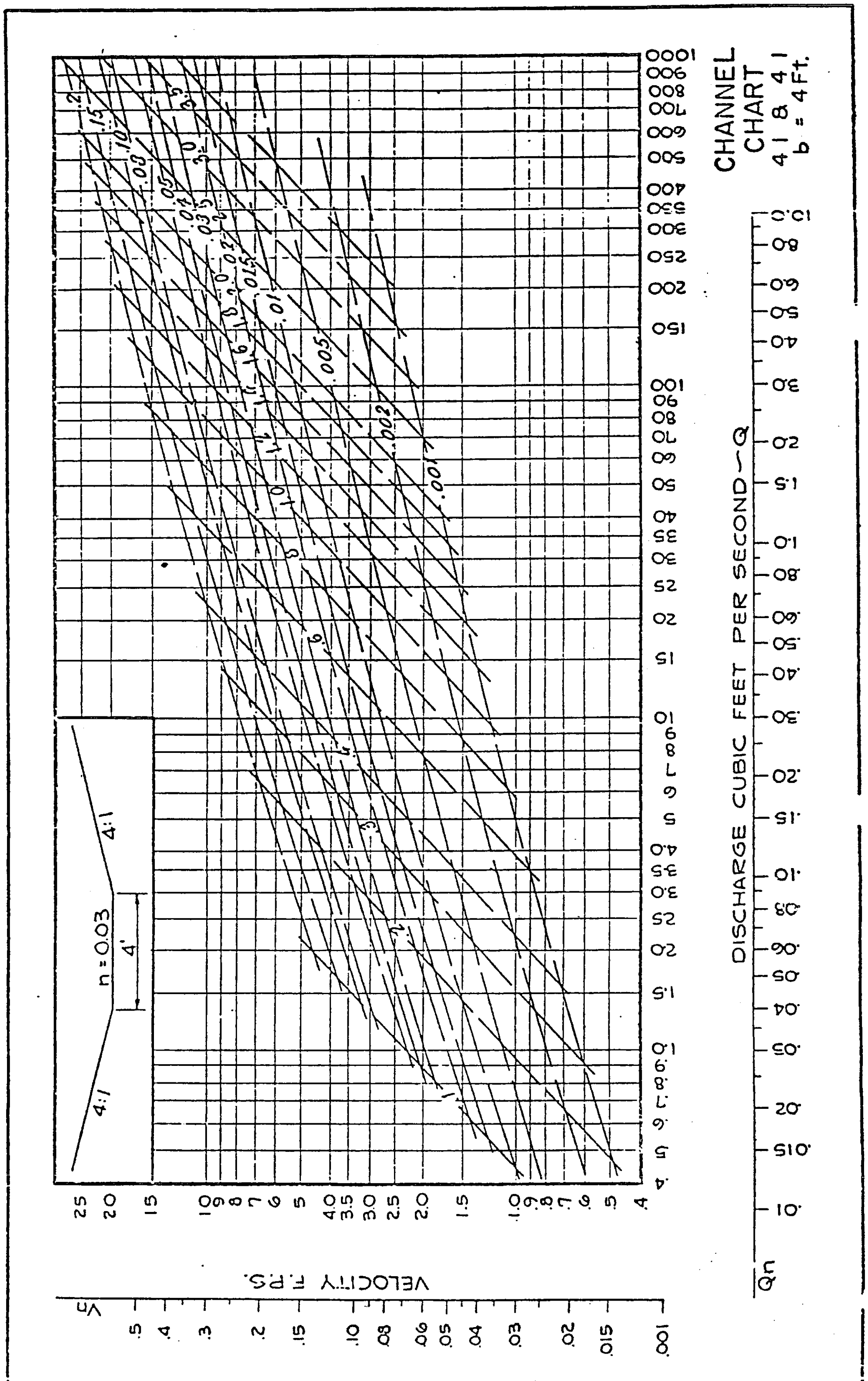
METRIC CONVERSION: 1 in =25.4 mm
1 ft = 0.3048 m

FIGURE 4



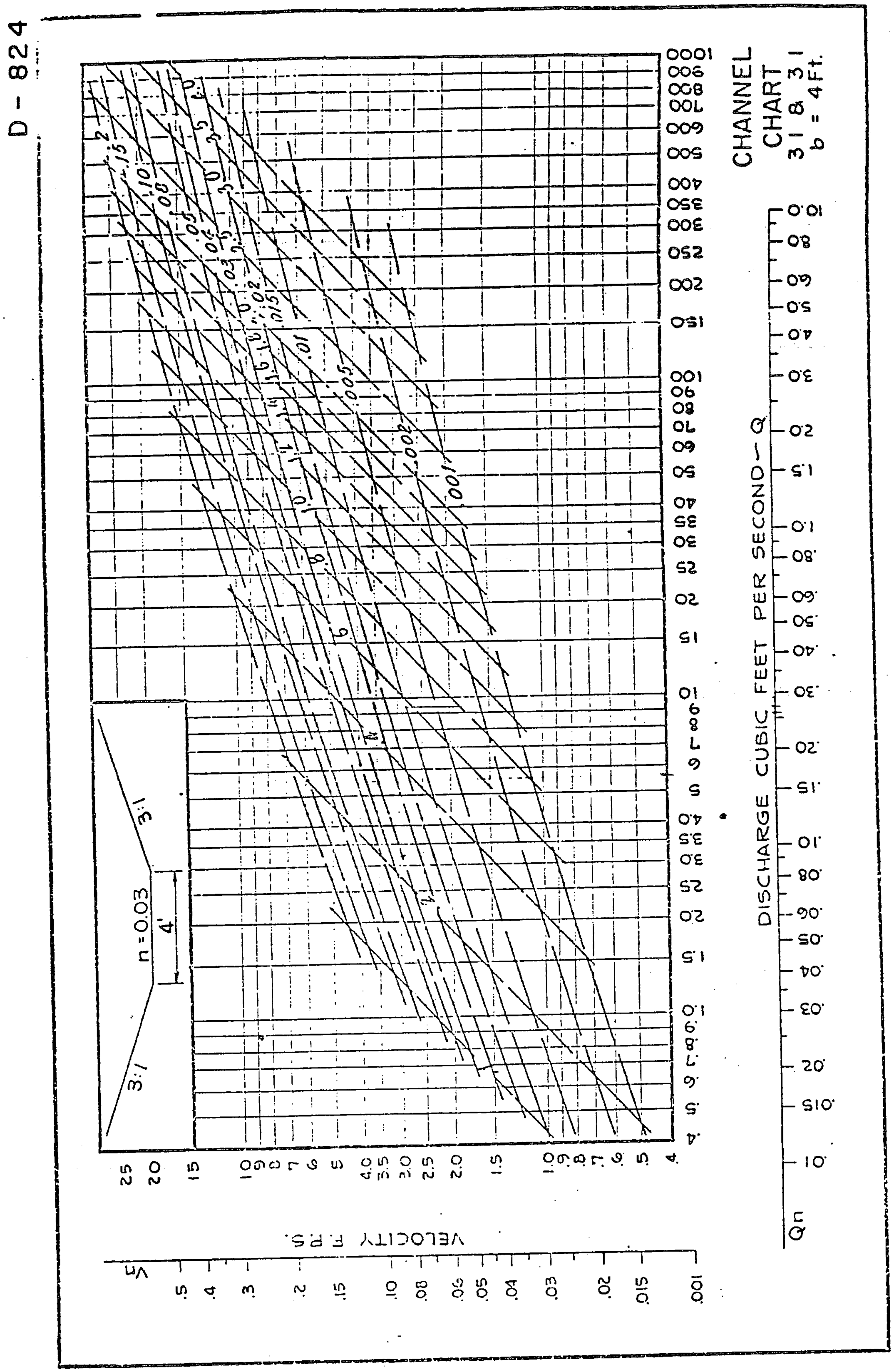
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 5



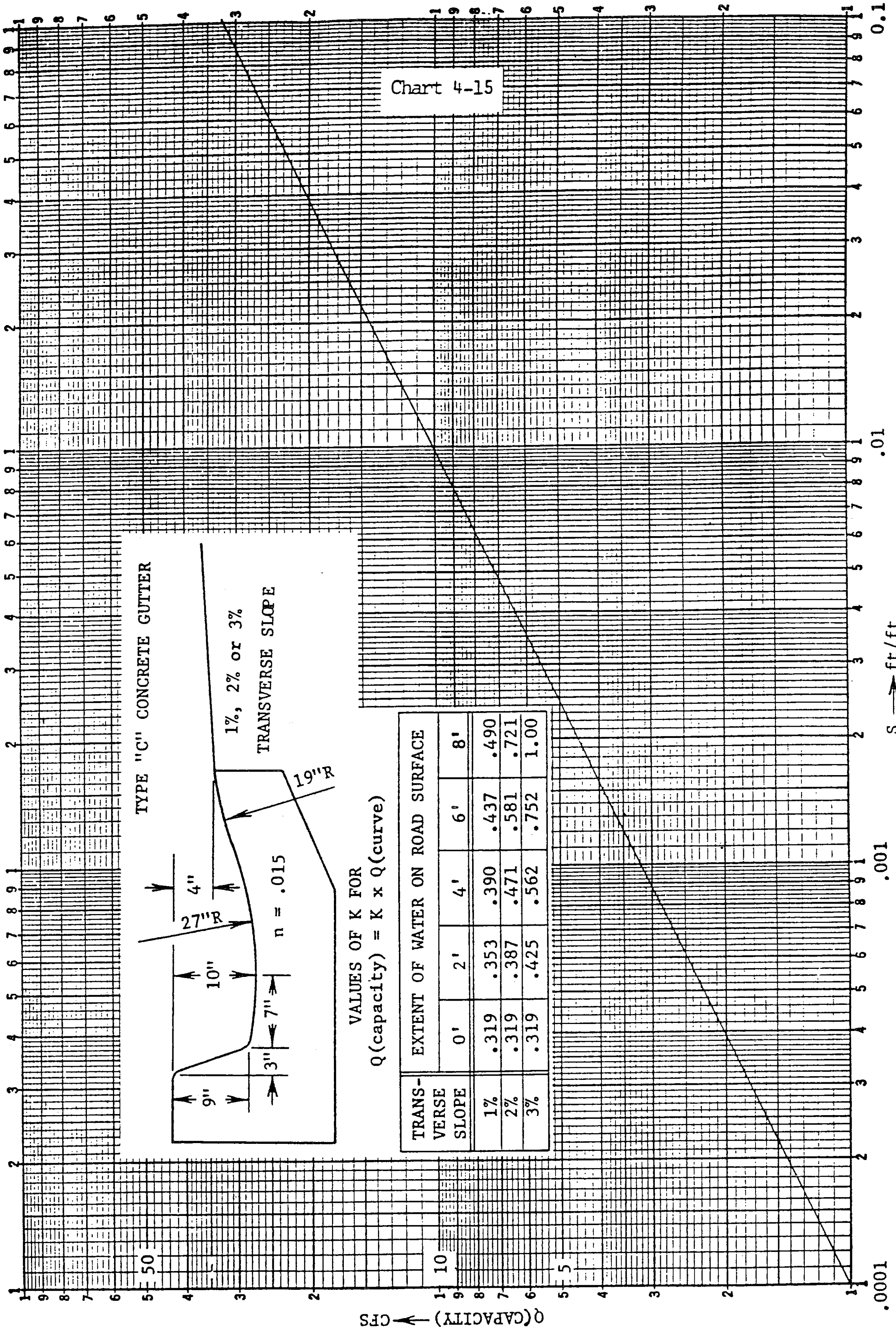
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 6



METRIC CONVERSION: 1 in =25.4 mm
1 ft = 0.3048 m

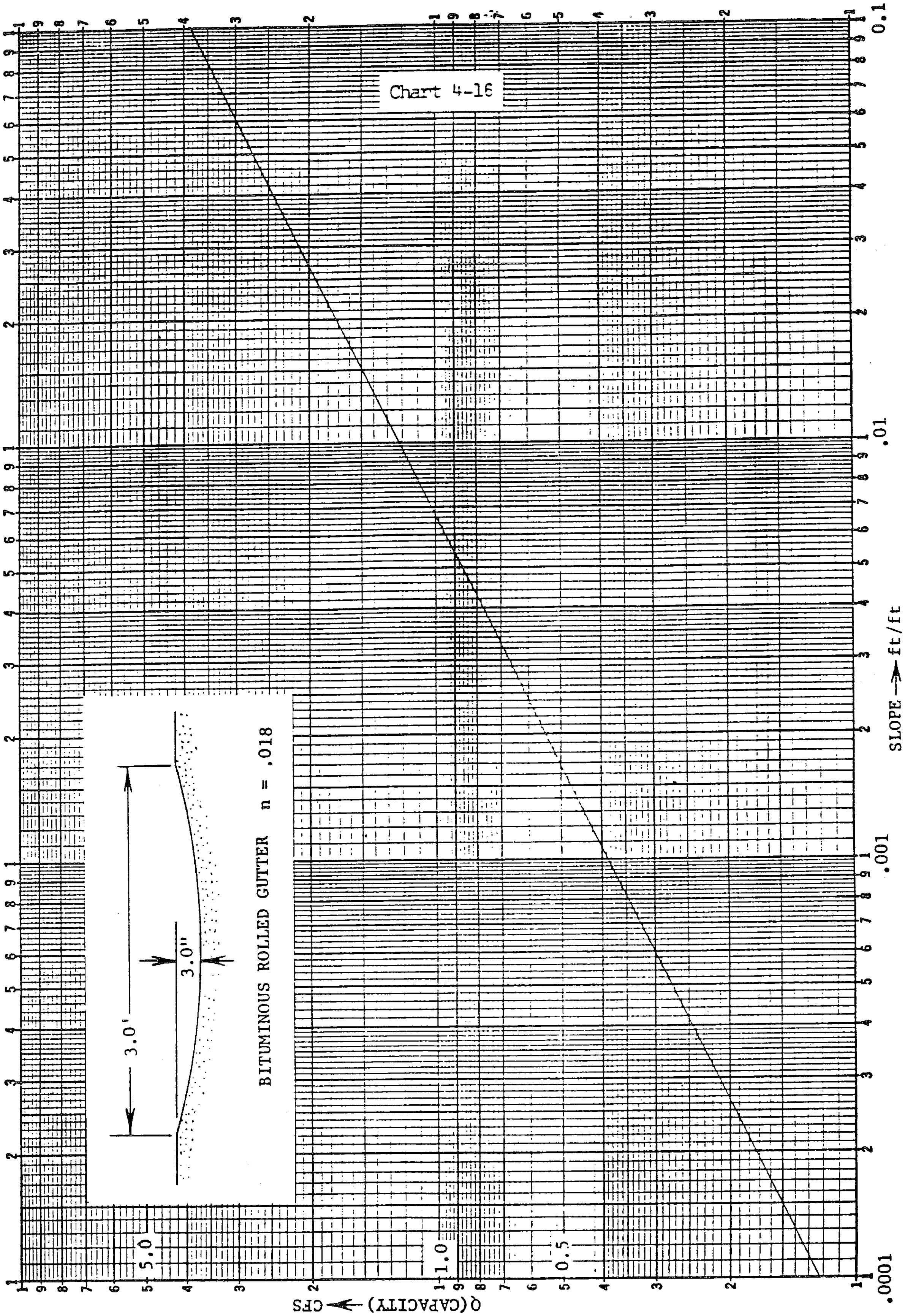
FIGURE 7



TRANSVERSE SLOPE	EXTENT OF WATER ON ROAD SURFACE				
	0'	2'	4'	6'	8'
1%	.319	.353	.390	.437	.490
2%	.319	.387	.471	.581	.721
3%	.319	.425	.562	.752	1.00

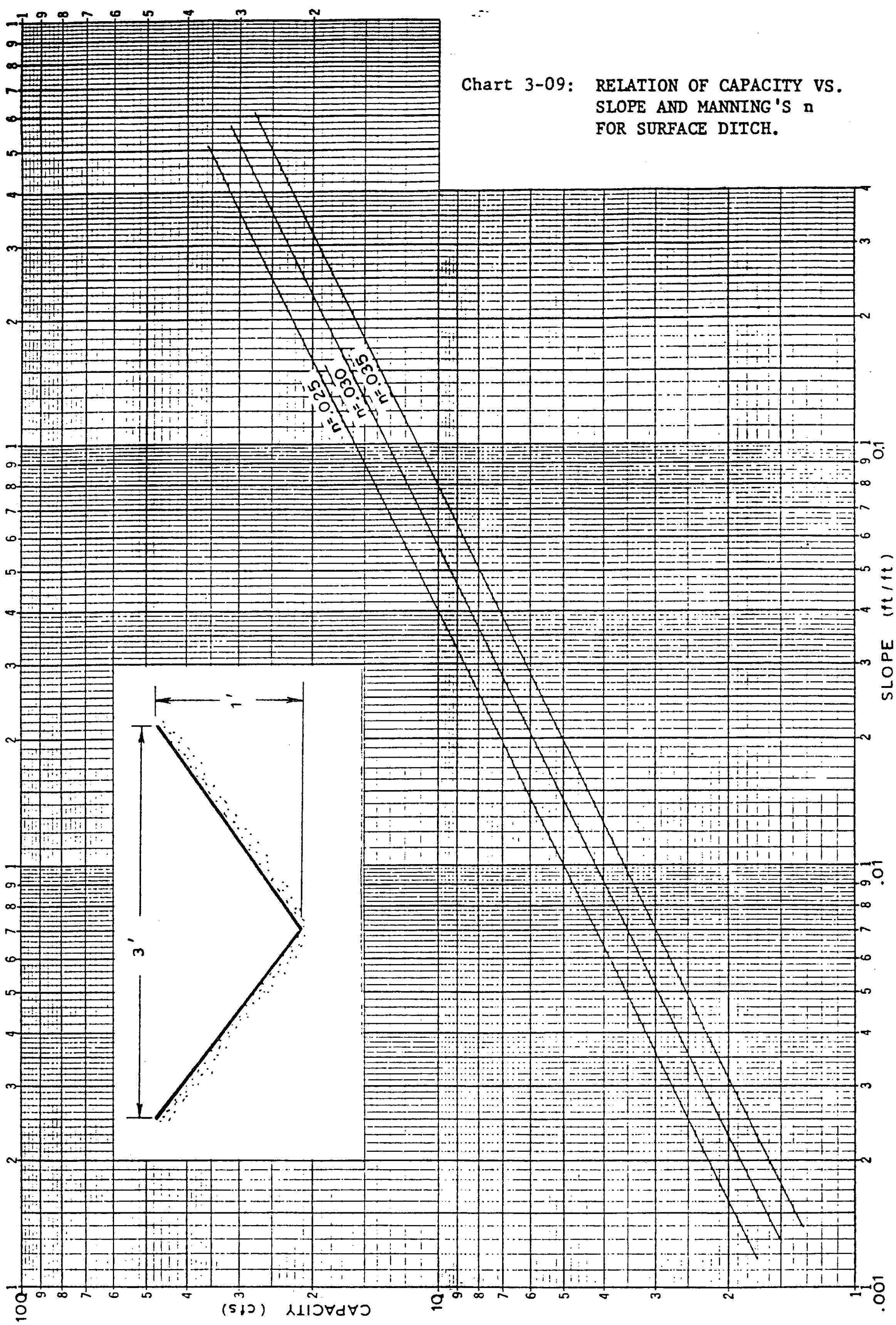
METRIC CONVERSION: 1 in =25.4 mm
1 ft = 0.3048 m

FIGURE 8



METRIC CONVERSION: 1 in =25.4 mm
1 ft = 0.3048 m

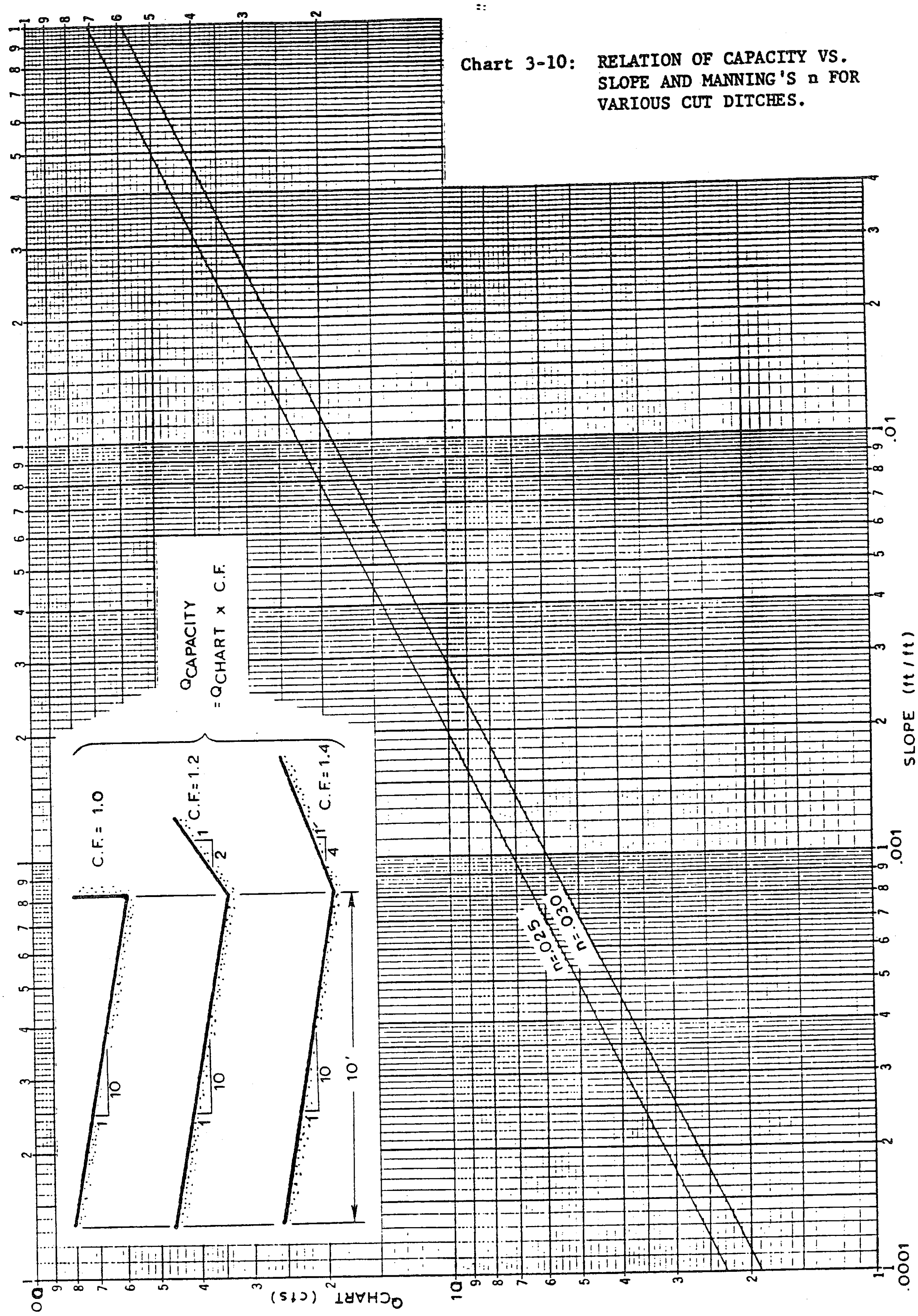
FIGURE 9



METRIC CONVERSION: 1 in =25.4 mm
1 ft = 0.3048 m

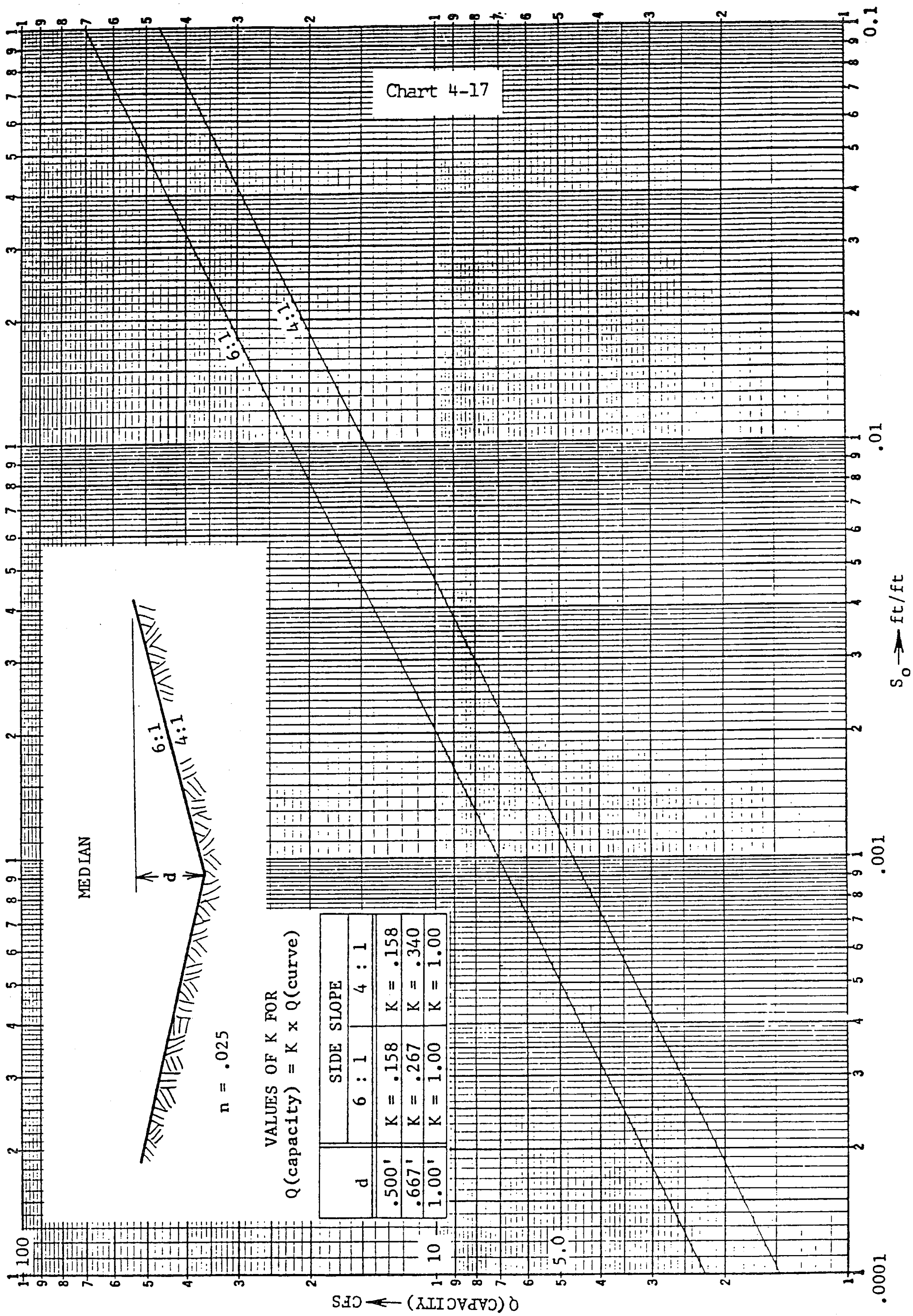
FIGURE 10

Chart 3-10: RELATION OF CAPACITY VS. SLOPE AND MANNING'S n FOR VARIOUS CUT DITCHES.



METRIC CONVERSION: 1 in =25.4 mm
1 ft = 0.3048 m

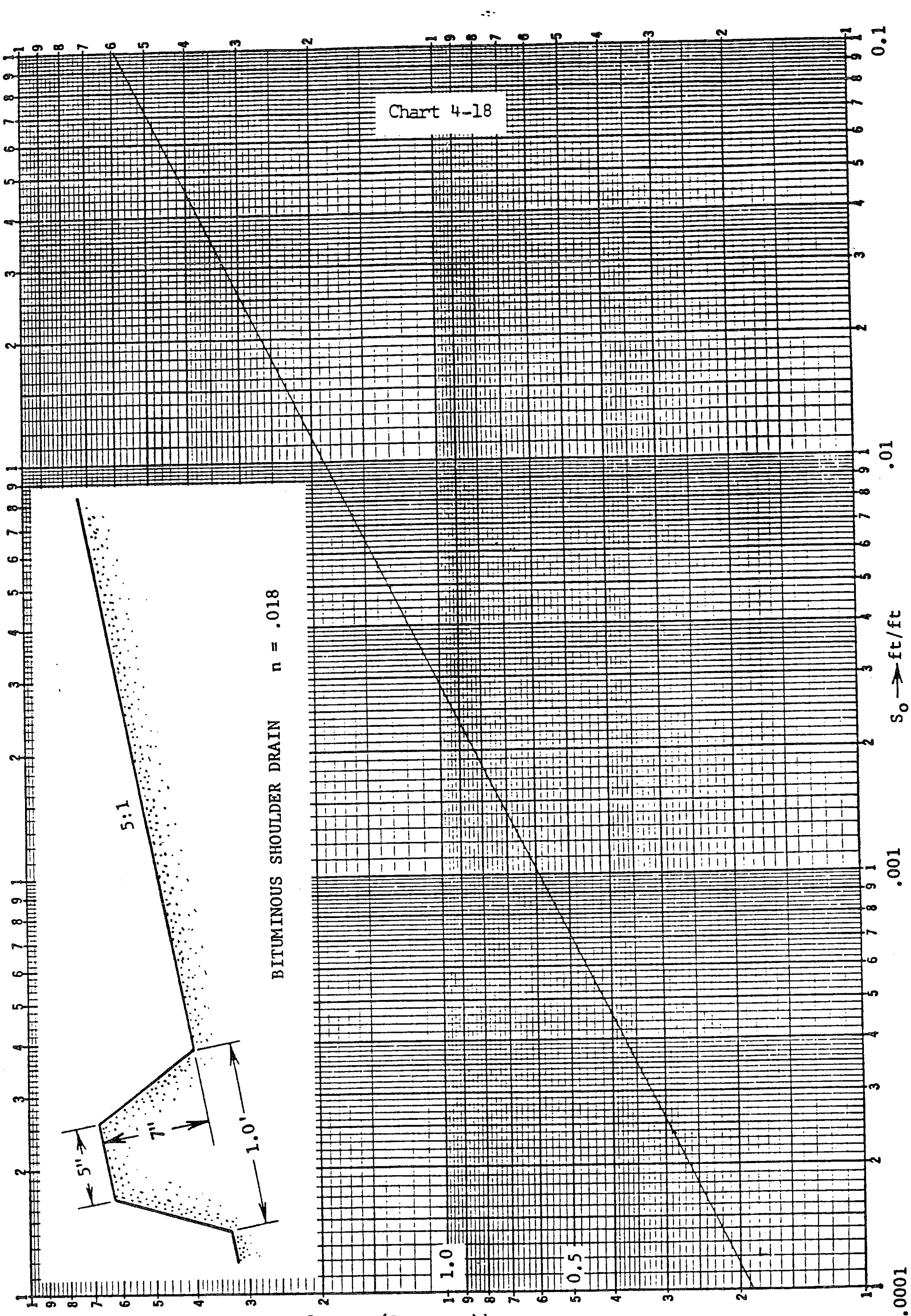
FIGURE 11



d	SIDE SLOPE 6 : 1	SIDE SLOPE 4 : 1
.500'	K = .158	K = .158
.667'	K = .267	K = .340
1.00'	K = 1.00	K = 1.00

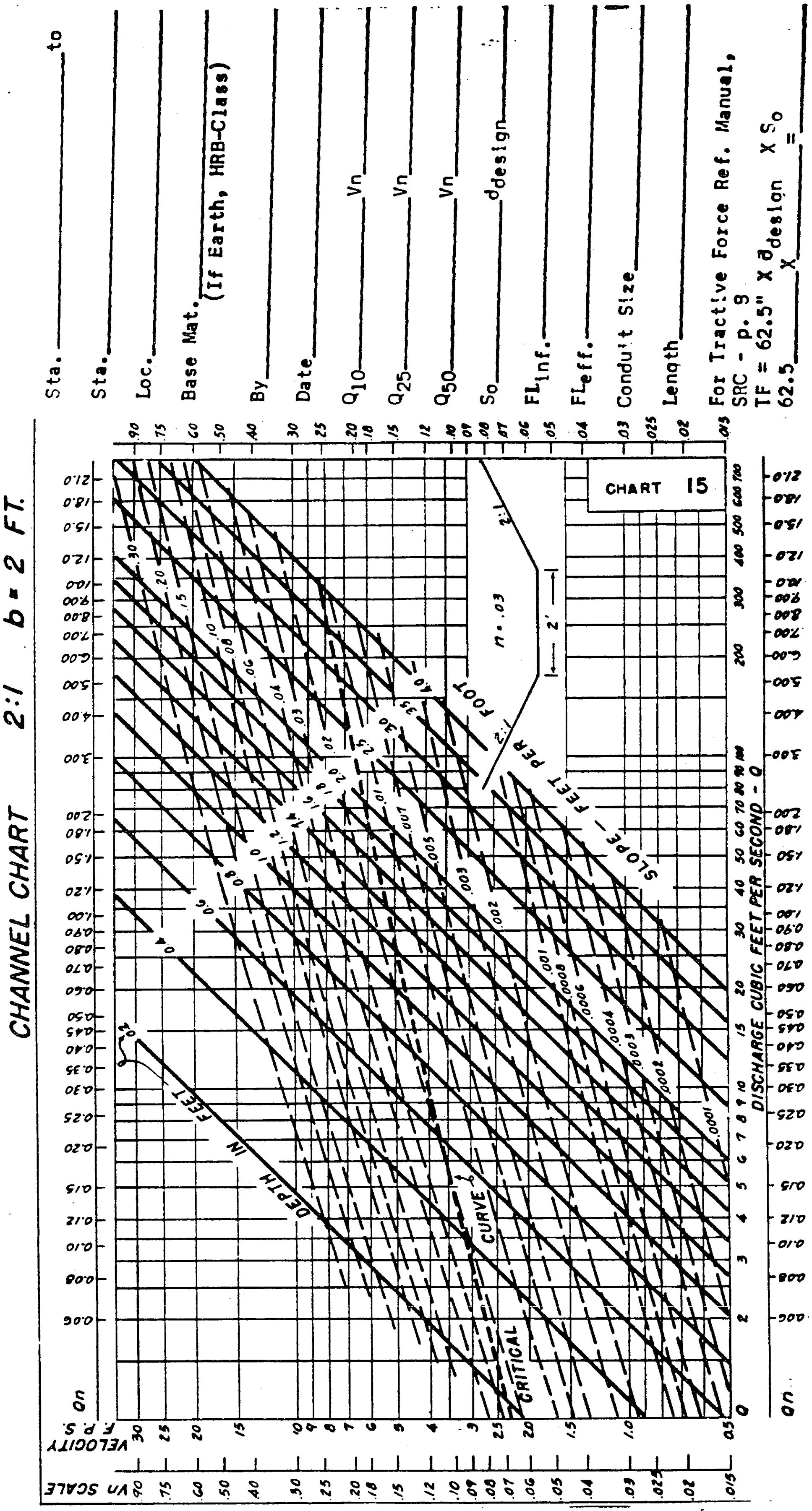
METRIC CONVERSION: 1 in =25.4 mm
1 ft = 0.3048 m

FIGURE 12



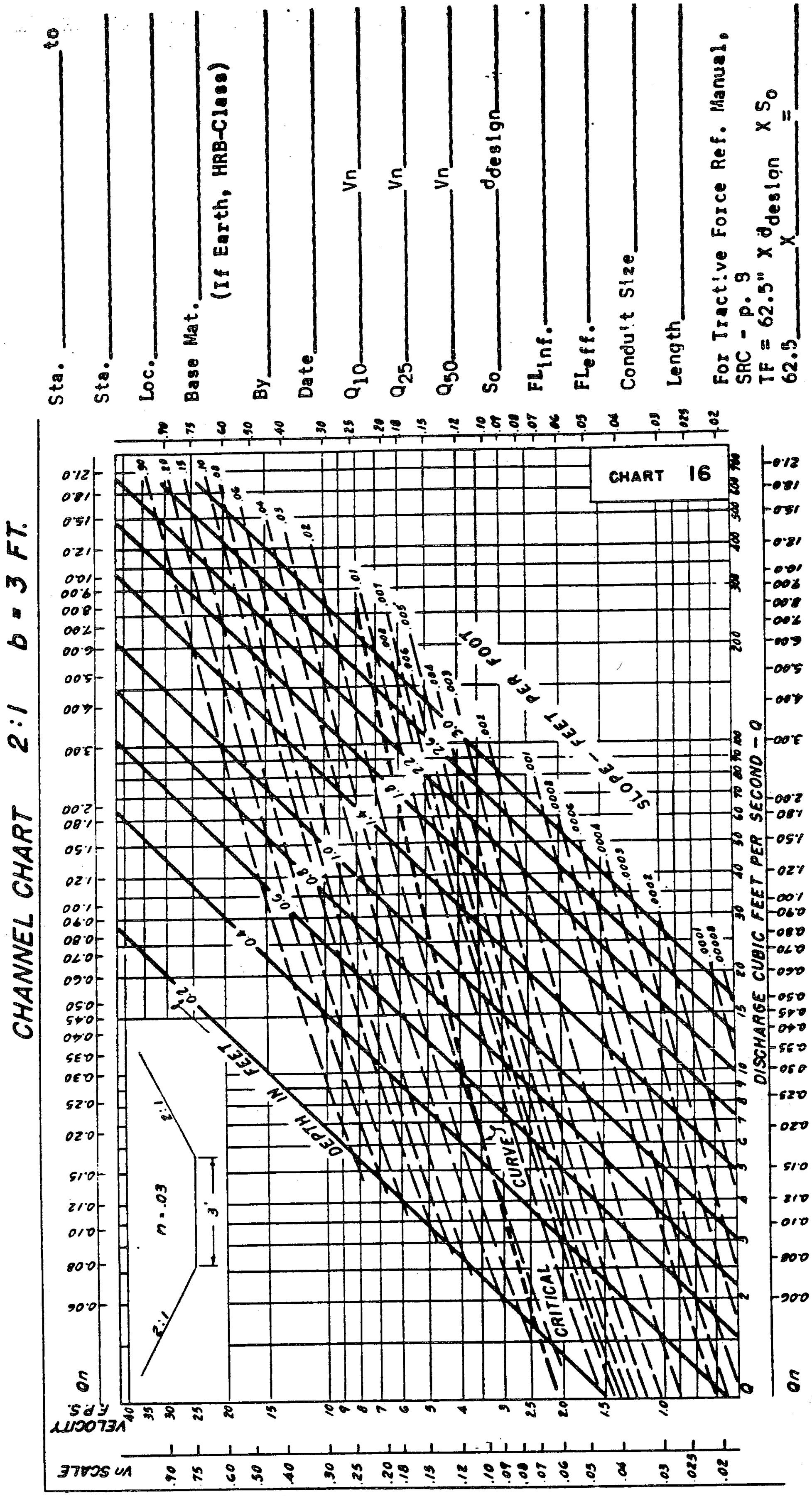
METRIC CONVERSION: 1 in =25.4 mm
1 ft = 0.3048 m

FIGURE 13



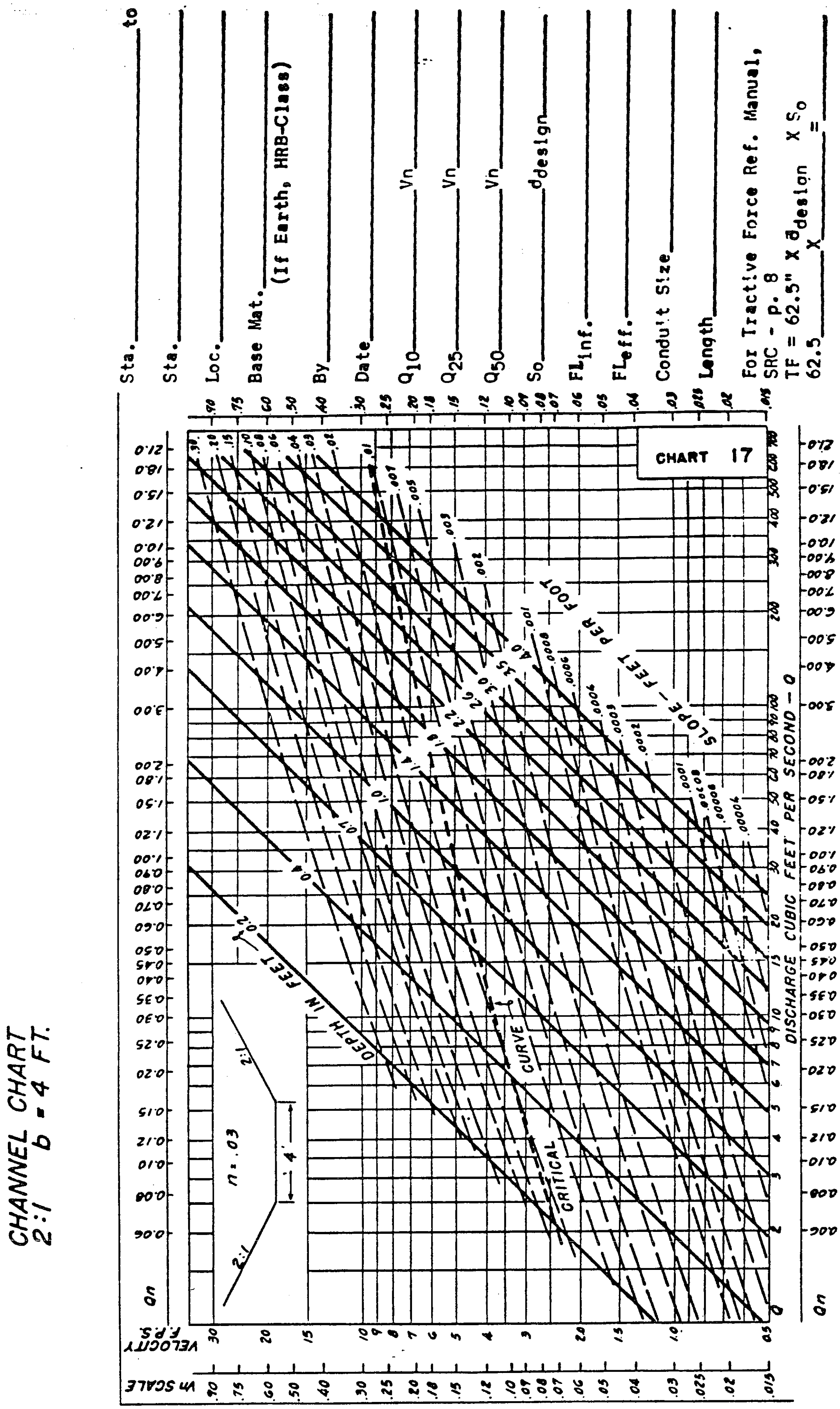
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 14



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

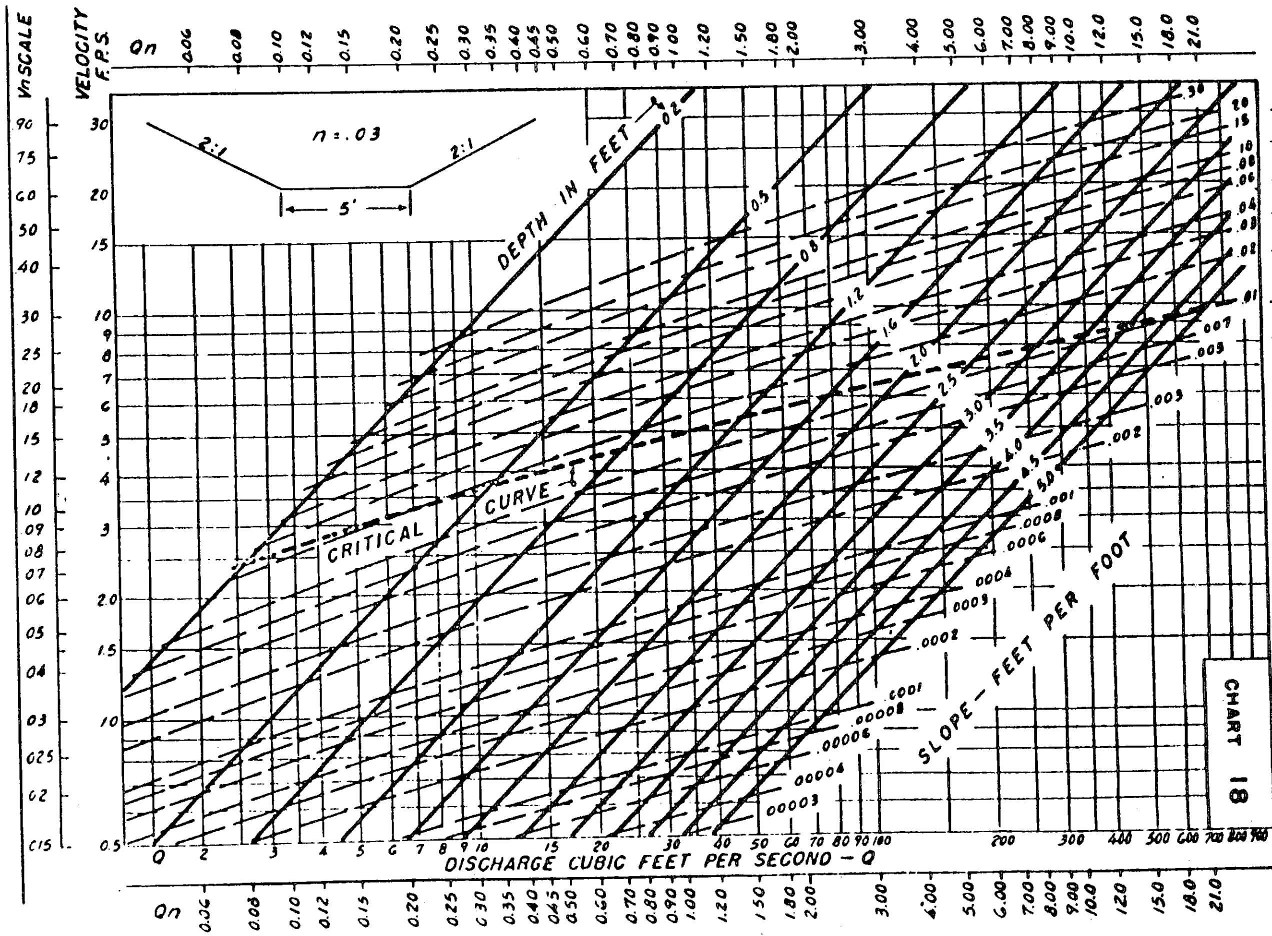
FIGURE 15



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 16

CHANNEL CHART
2:1 b = 5 FT.



Sta. ______________ to

Sta. ______________

Loc. ______________

Base Mat. ______________
(If Earth, HRB-Class)

By ______________

Date ______________

Q_{10} ________ Vn ________

Q_{25} ________ Vn ________

Q_{50} ________ Vn ________

S_o ________ d_{design} ________

$FL_{inf.}$ ______________

$FL_{eff.}$ ______________

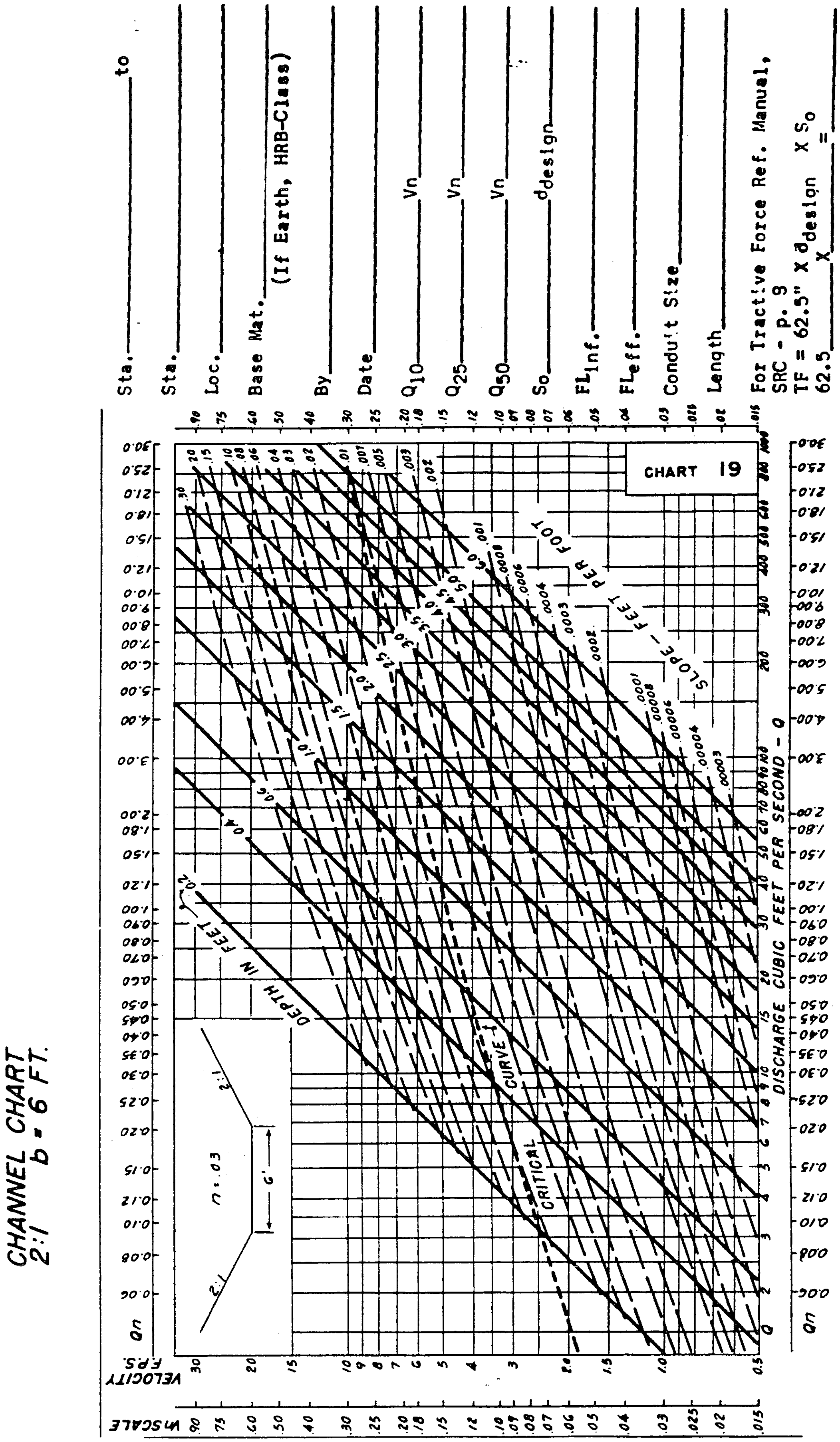
Conduit Size ______________

Length ______________

For Tractive Force Ref. Manual,
SRC - p. 9
TF = 62.5" X d_{design} X S_o
62.5 ______ X ______ = ______

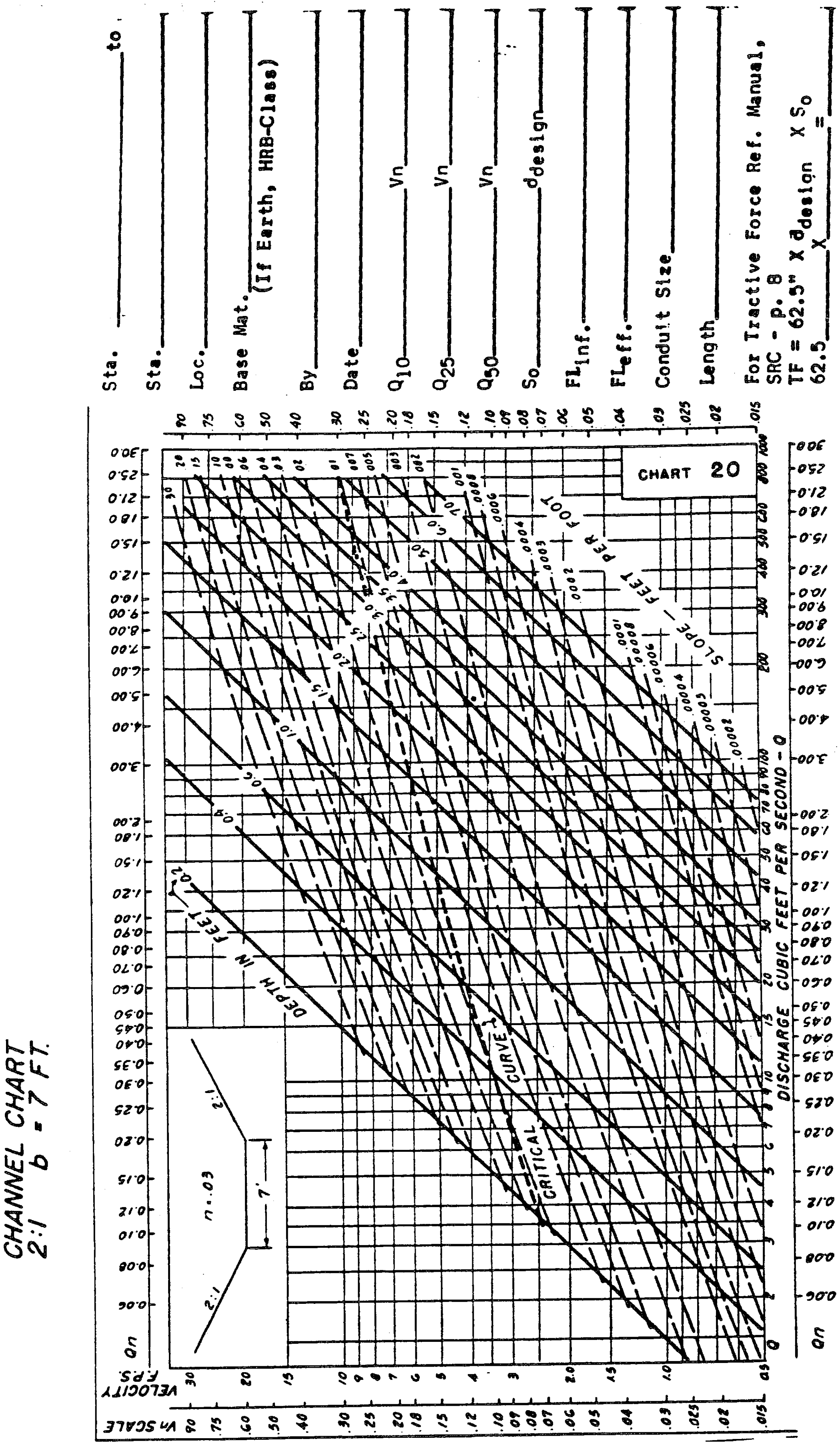
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 17



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

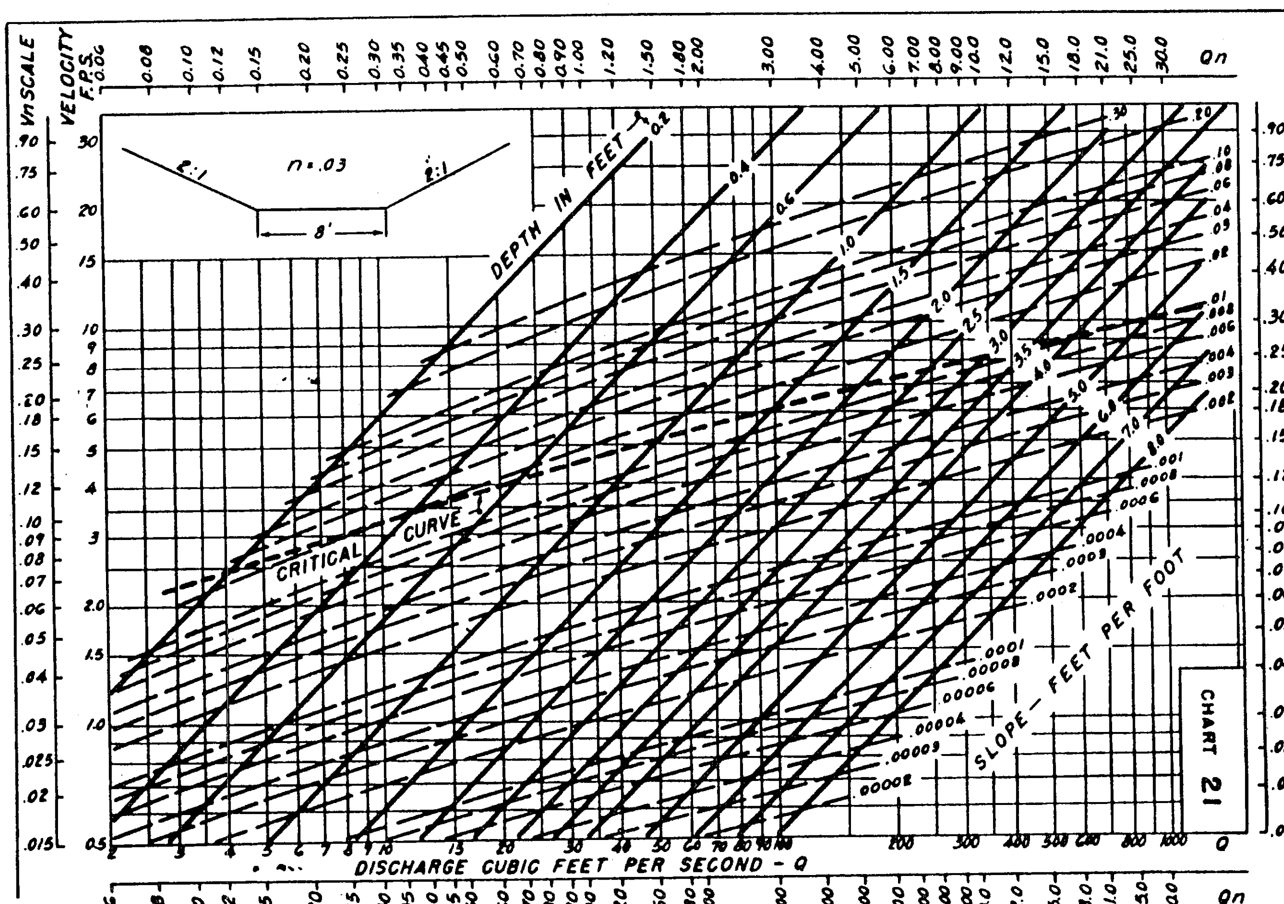
FIGURE 18



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 19

CHANNEL CHART
2:1 b = 8 FT.



Sta.______________________to

Sta.______________________

Loc.______________________

Base Mat.______________________
(If Earth, HRB-Class)

By______________________

Date______________________

Q_{10}__________Vn__________

Q_{25}__________Vn__________

Q_{50}__________Vn__________

So__________d_{design}__________

$FL_{inf.}$______________________

$FL_{eff.}$______________________

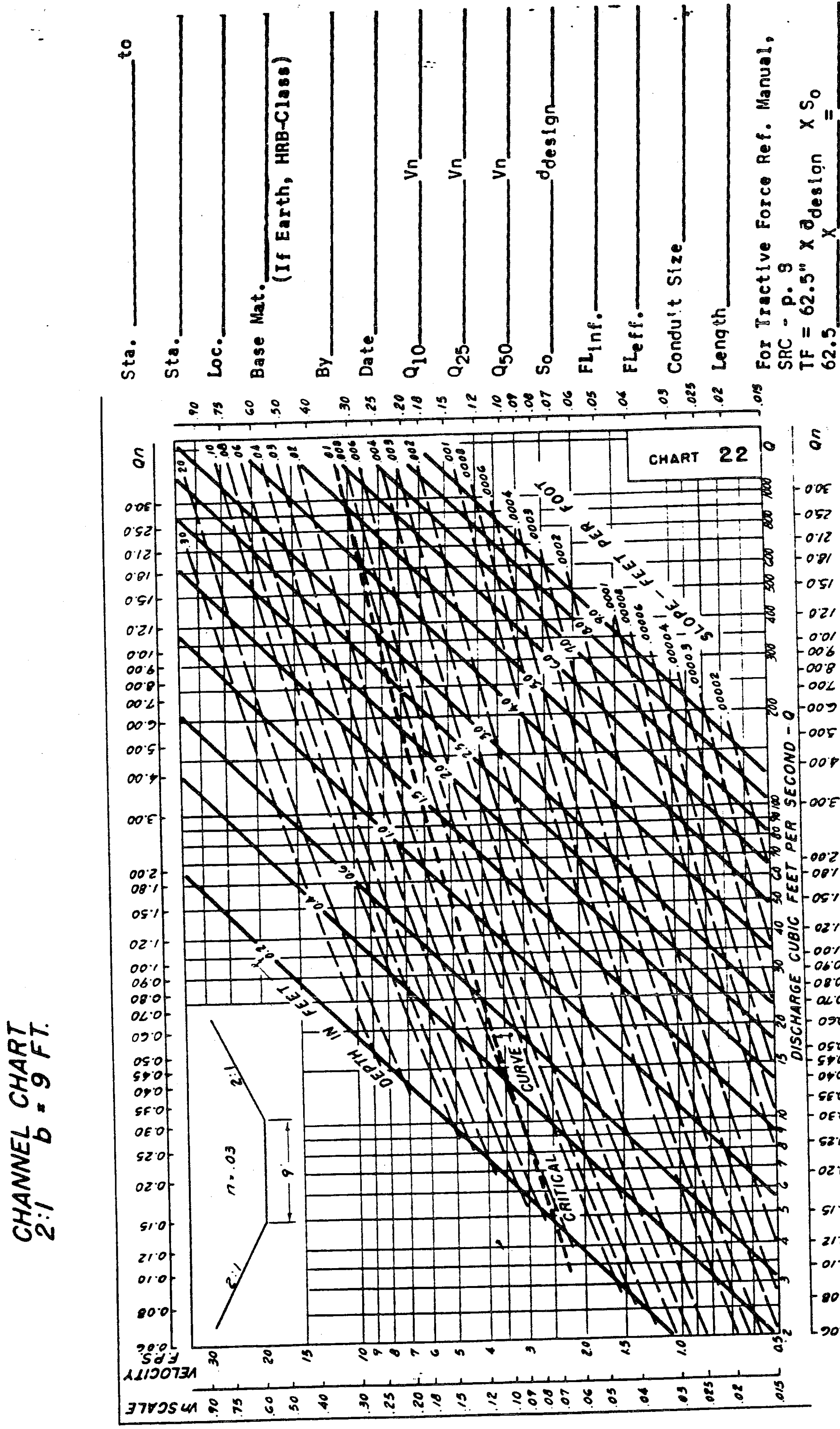
Conduit Size______________________

Length______________________

For Tractive Force Ref. Manual,
SRC - p. 8
TF = 62.5" X d_{design} X S_o
62.5______X______=______

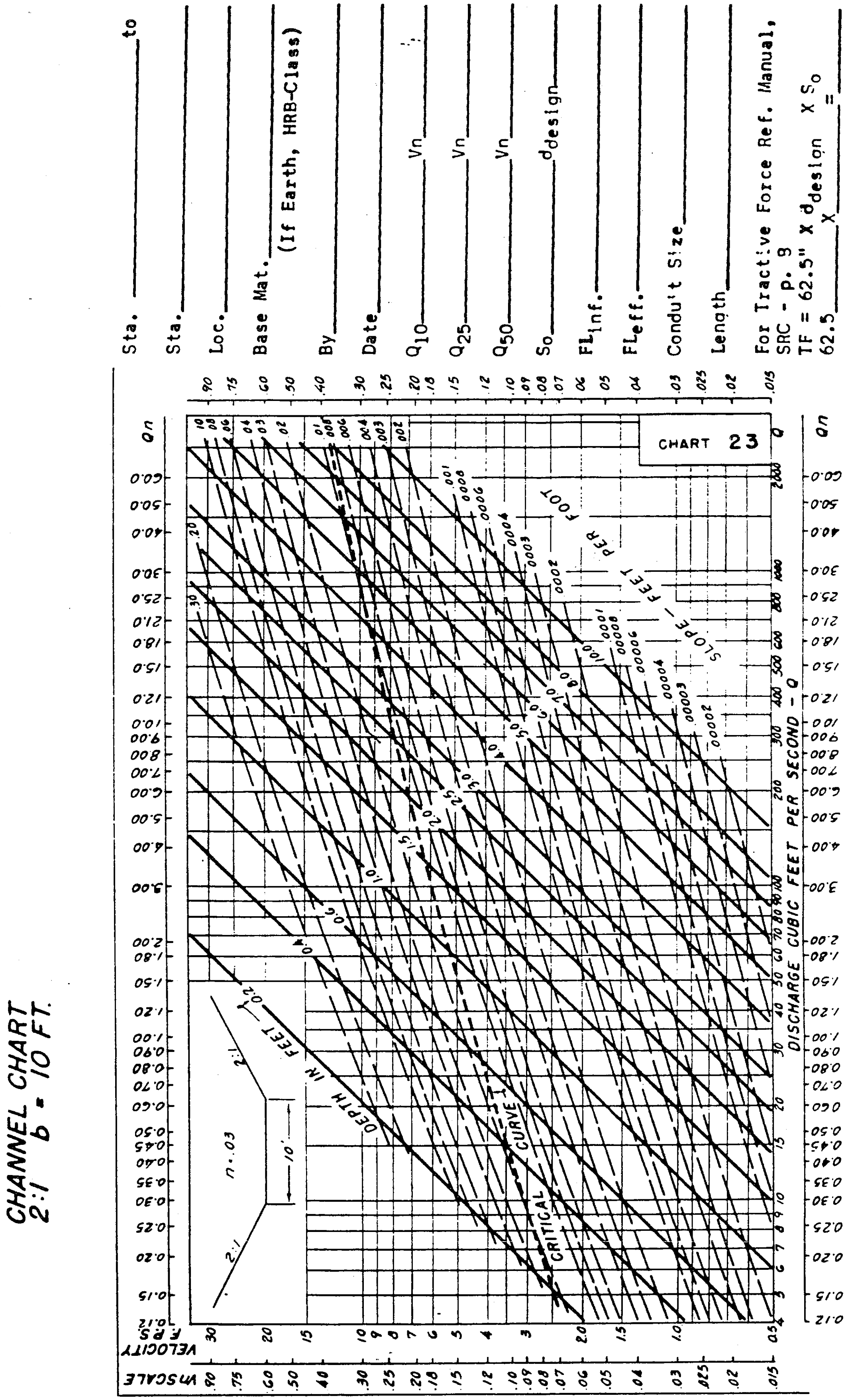
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 20



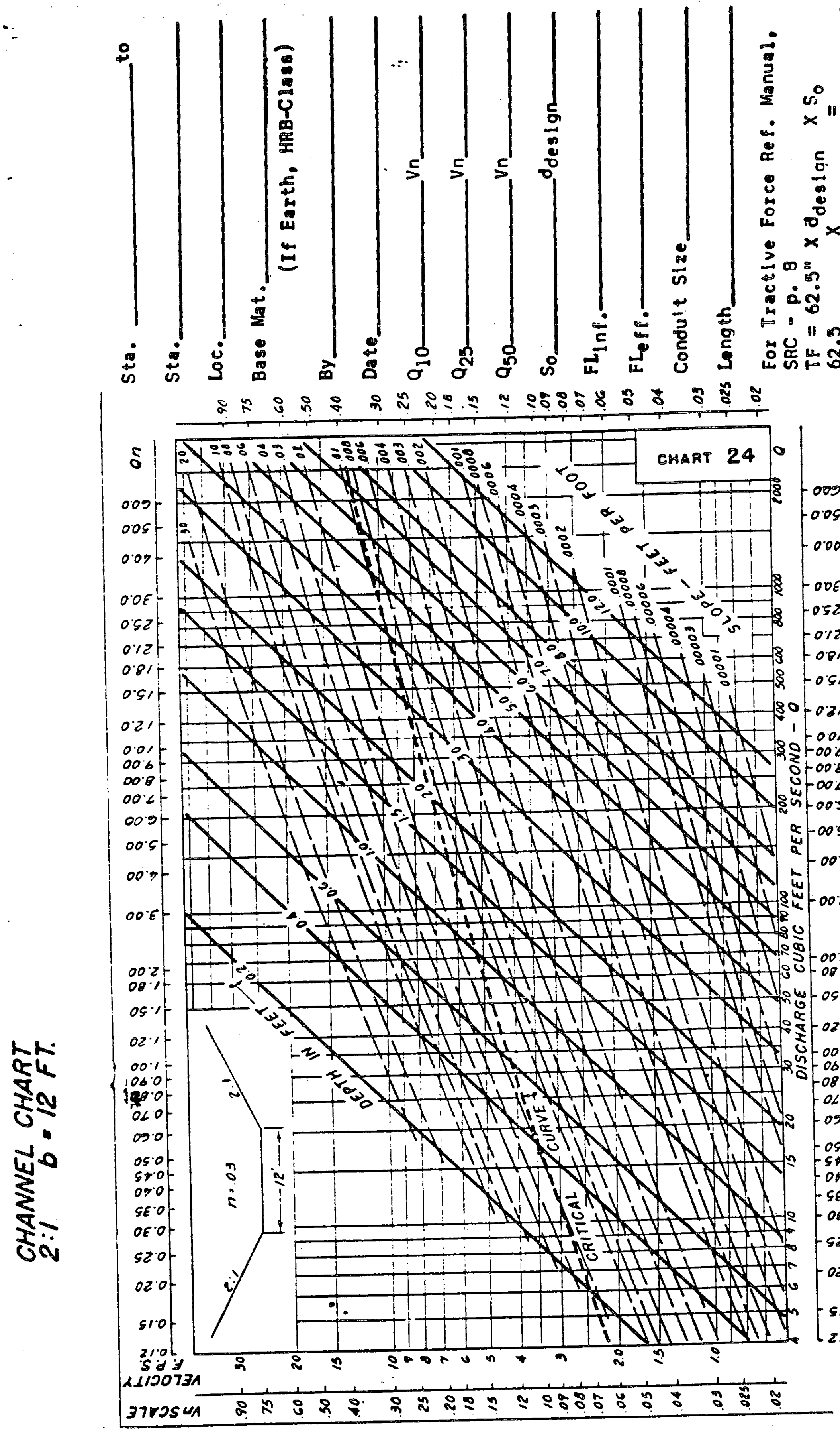
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 21



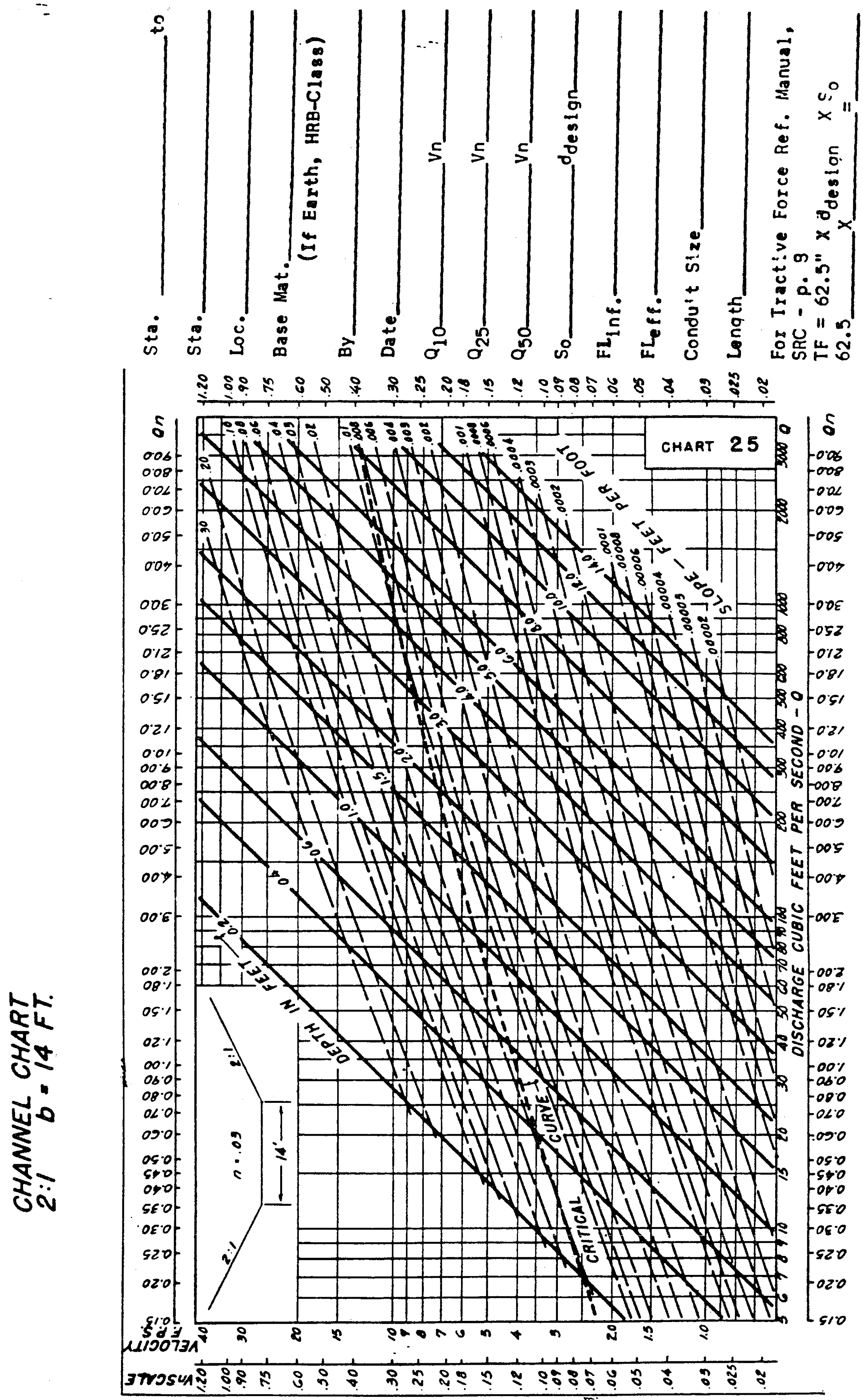
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 22



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

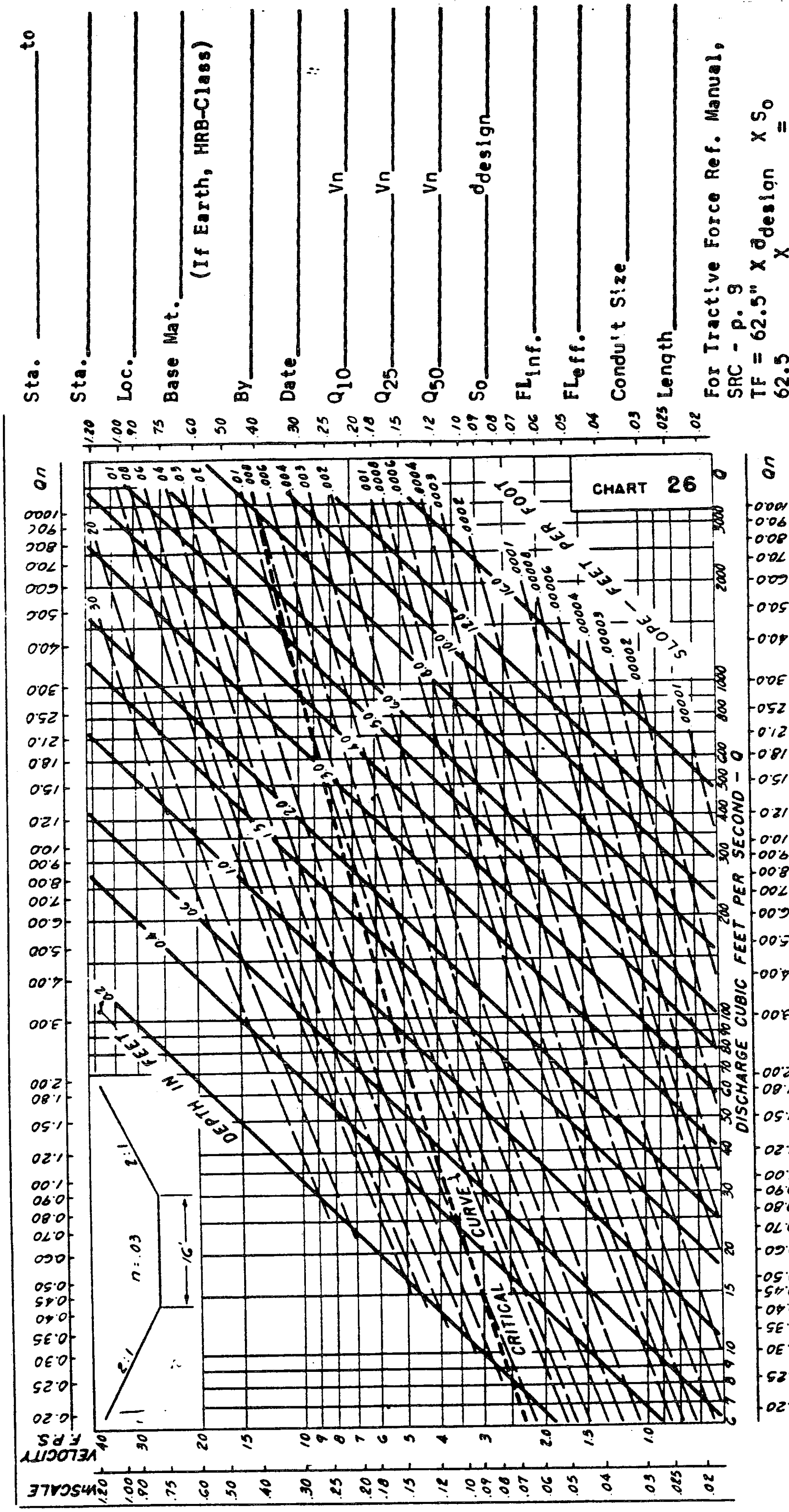
FIGURE 23



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

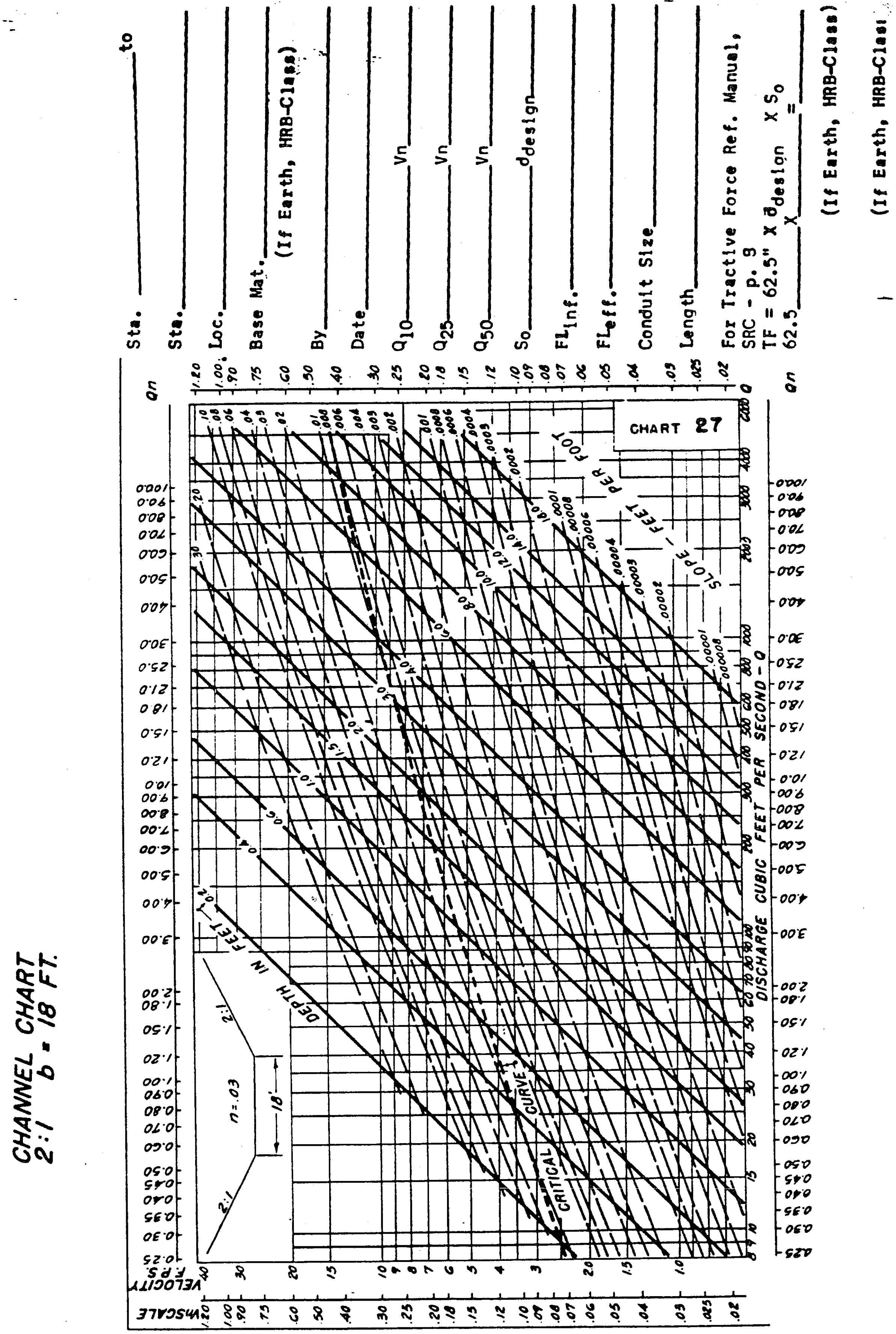
FIGURE 24

CHANNEL CHART
2:1 b = 16 FT.



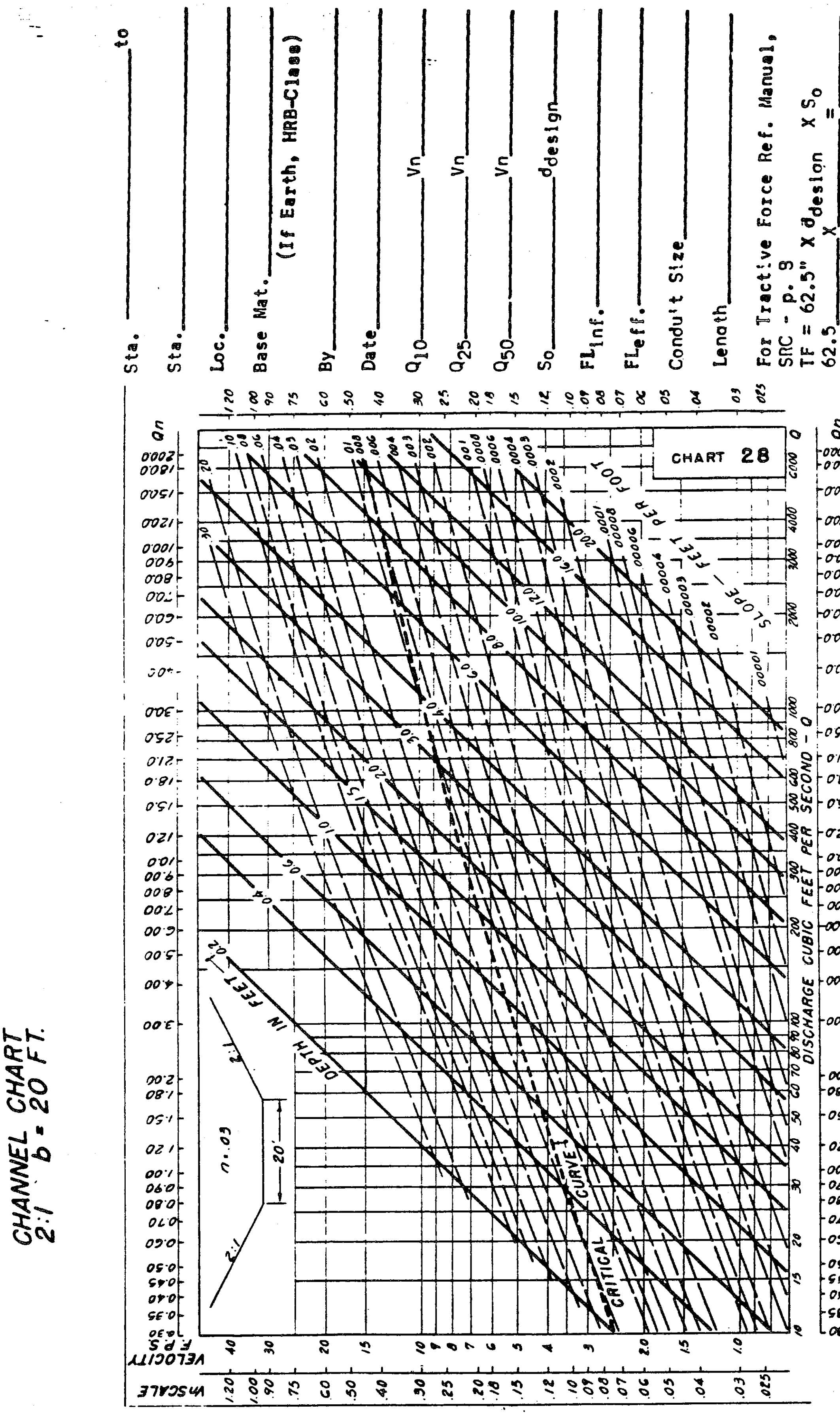
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 25



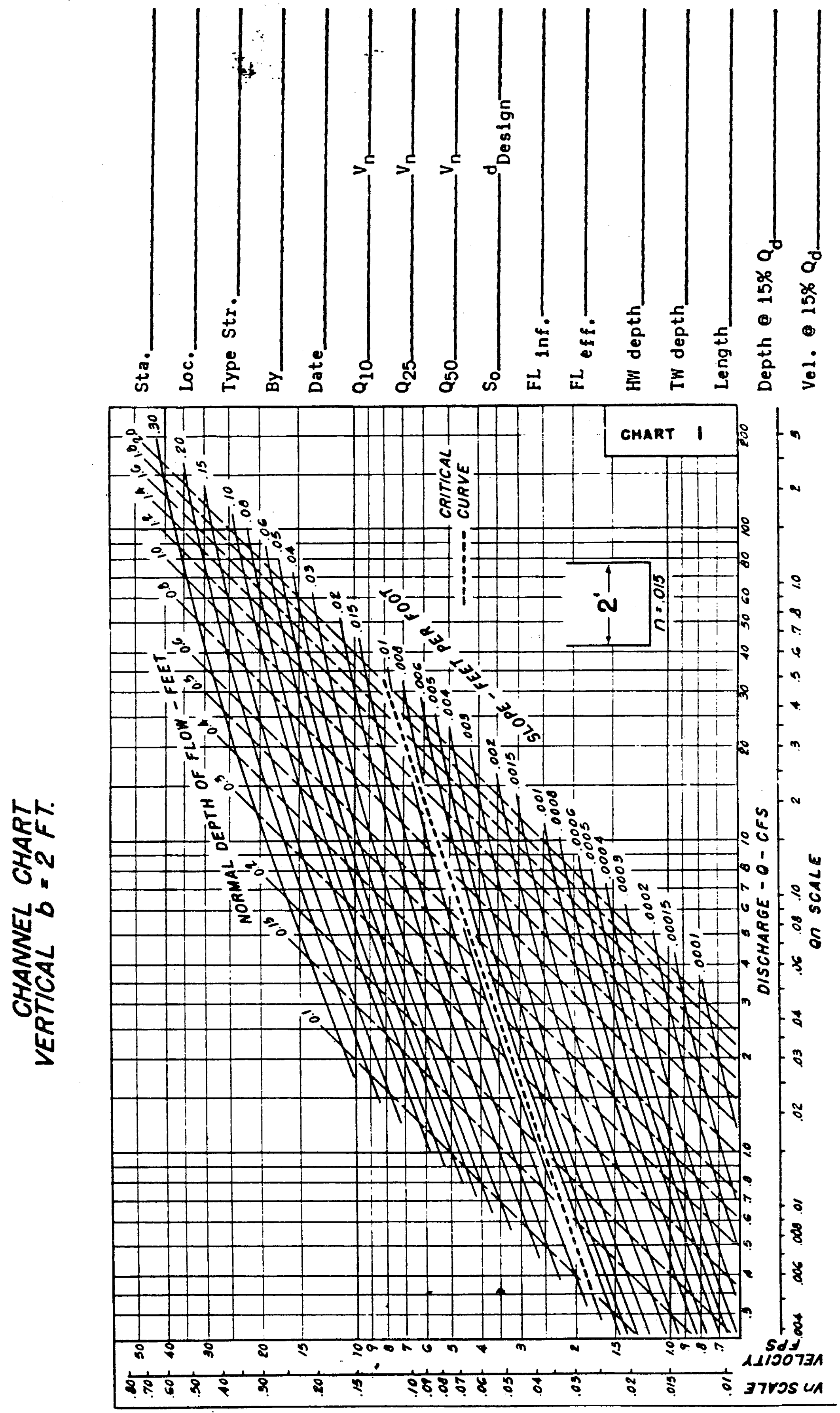
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 26



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

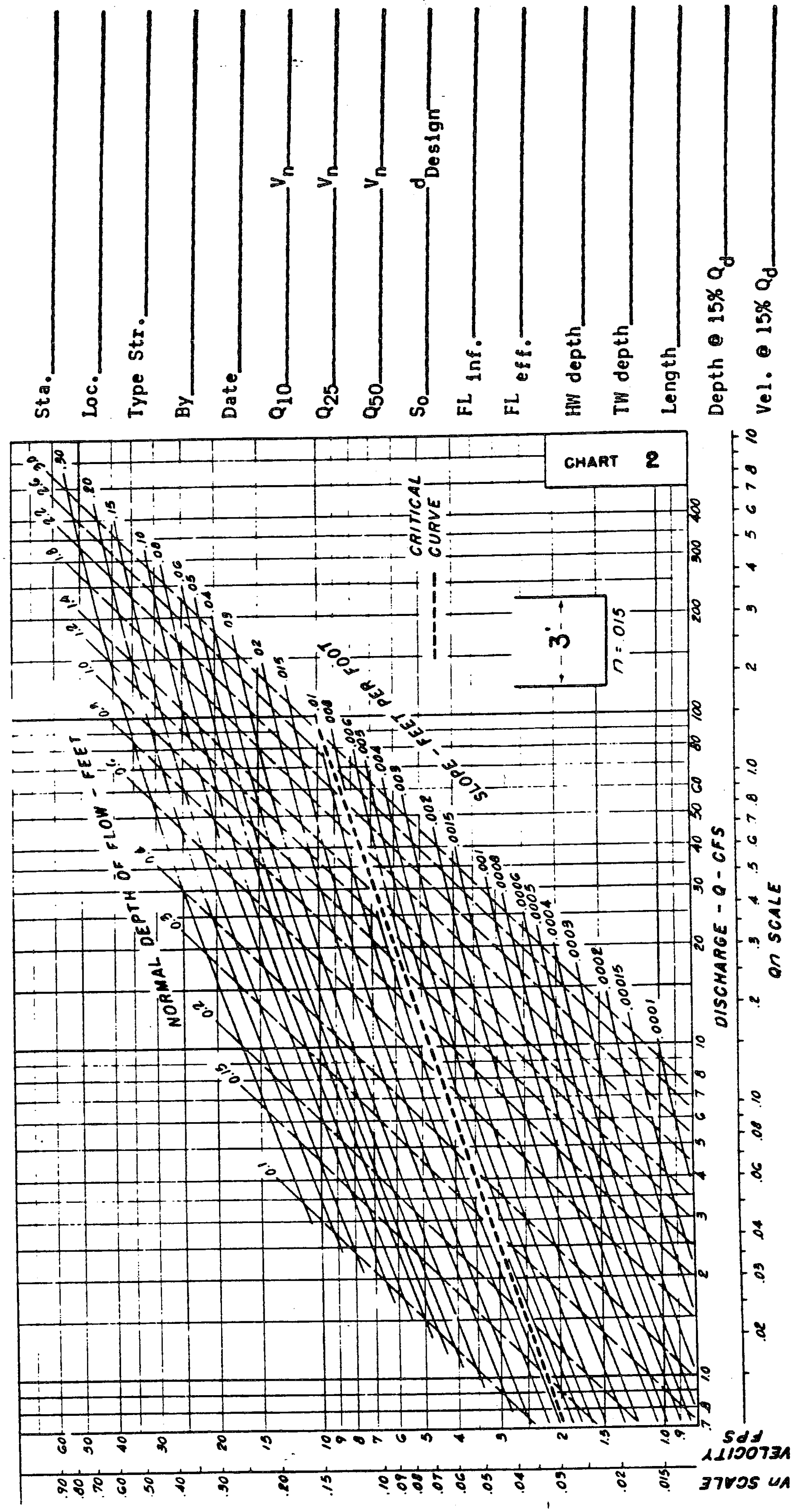
FIGURE 27



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

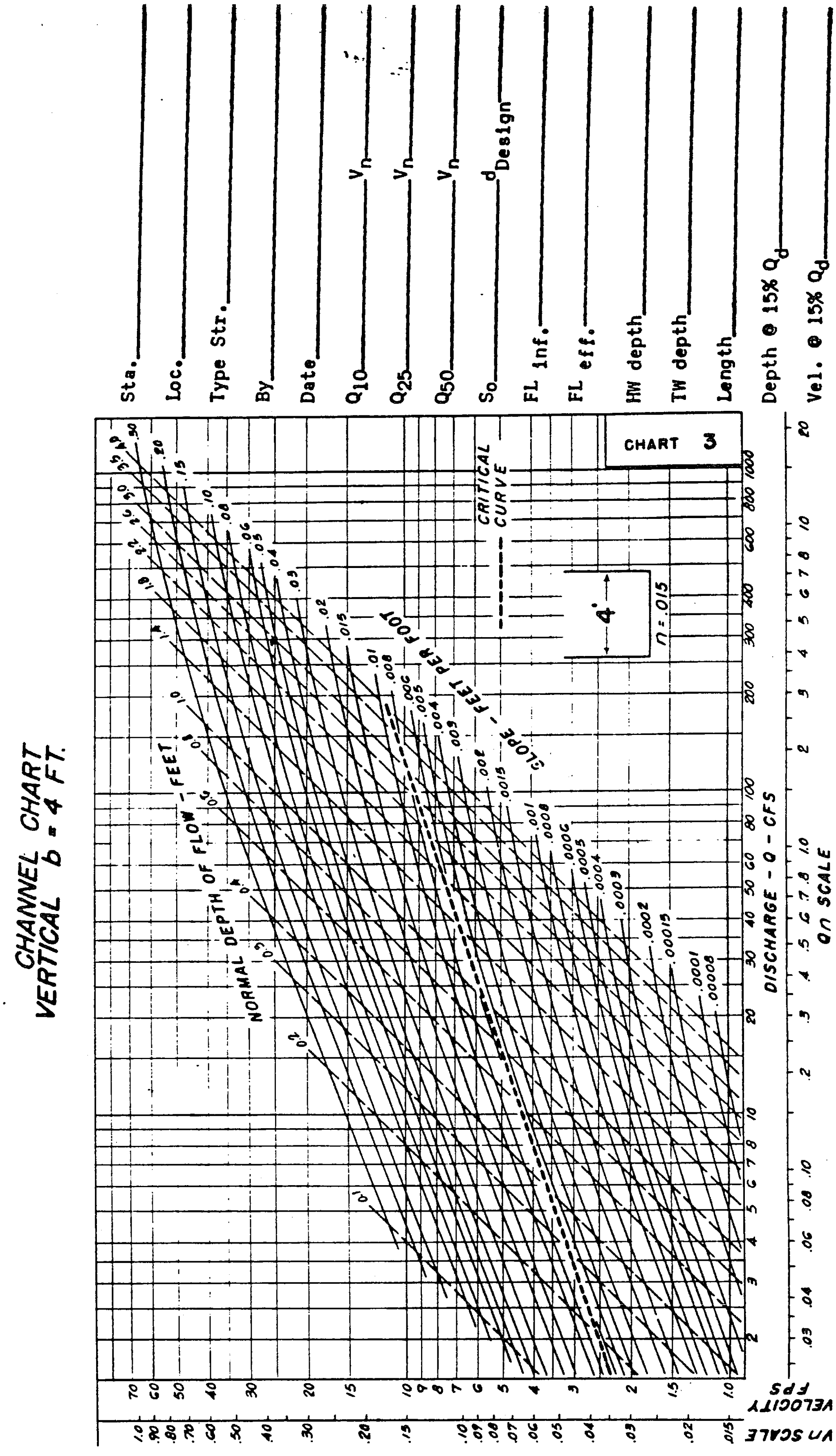
FIGURE 28

CHANNEL CHART
VERTICAL b = 3 FT.



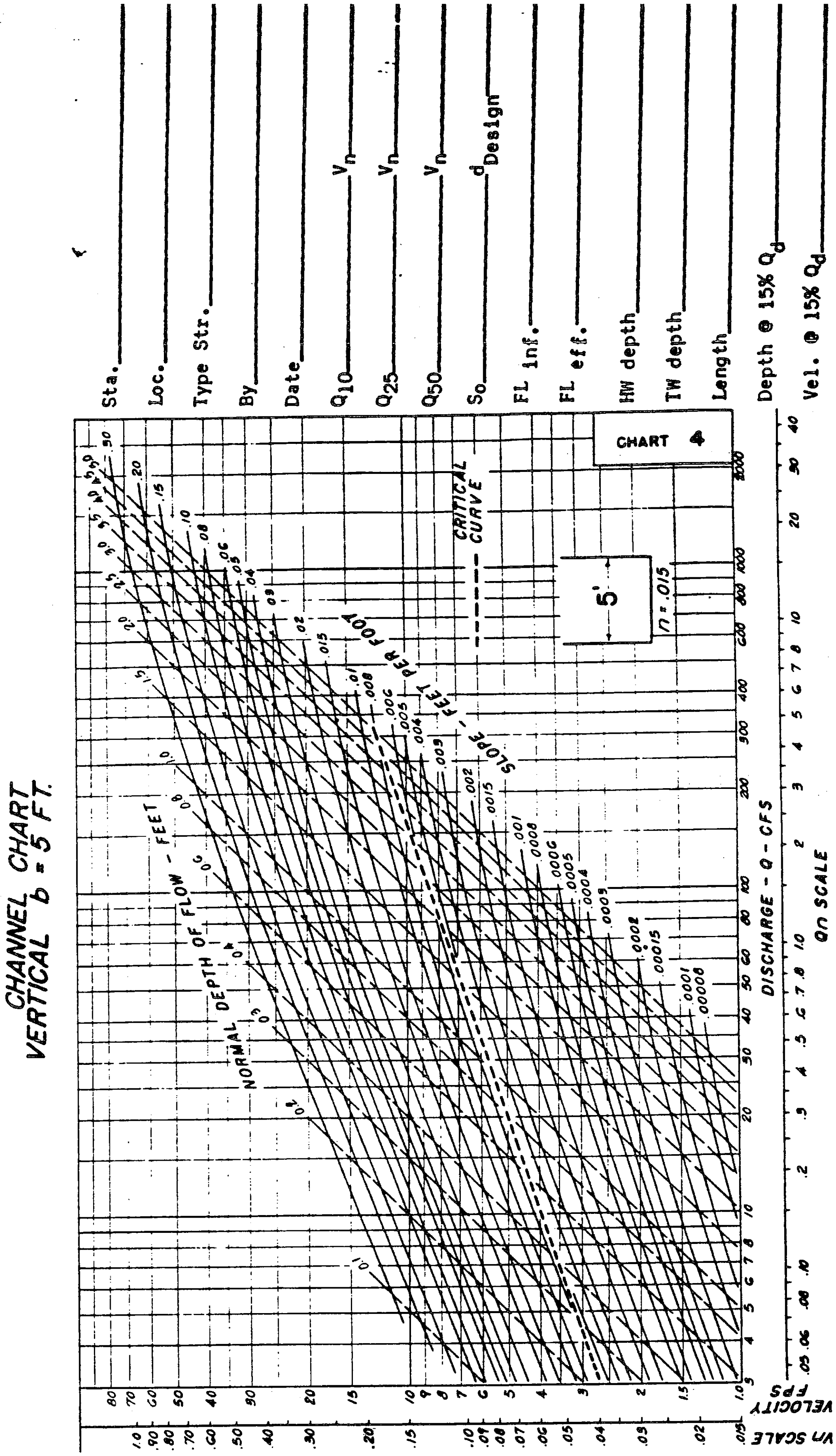
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 29



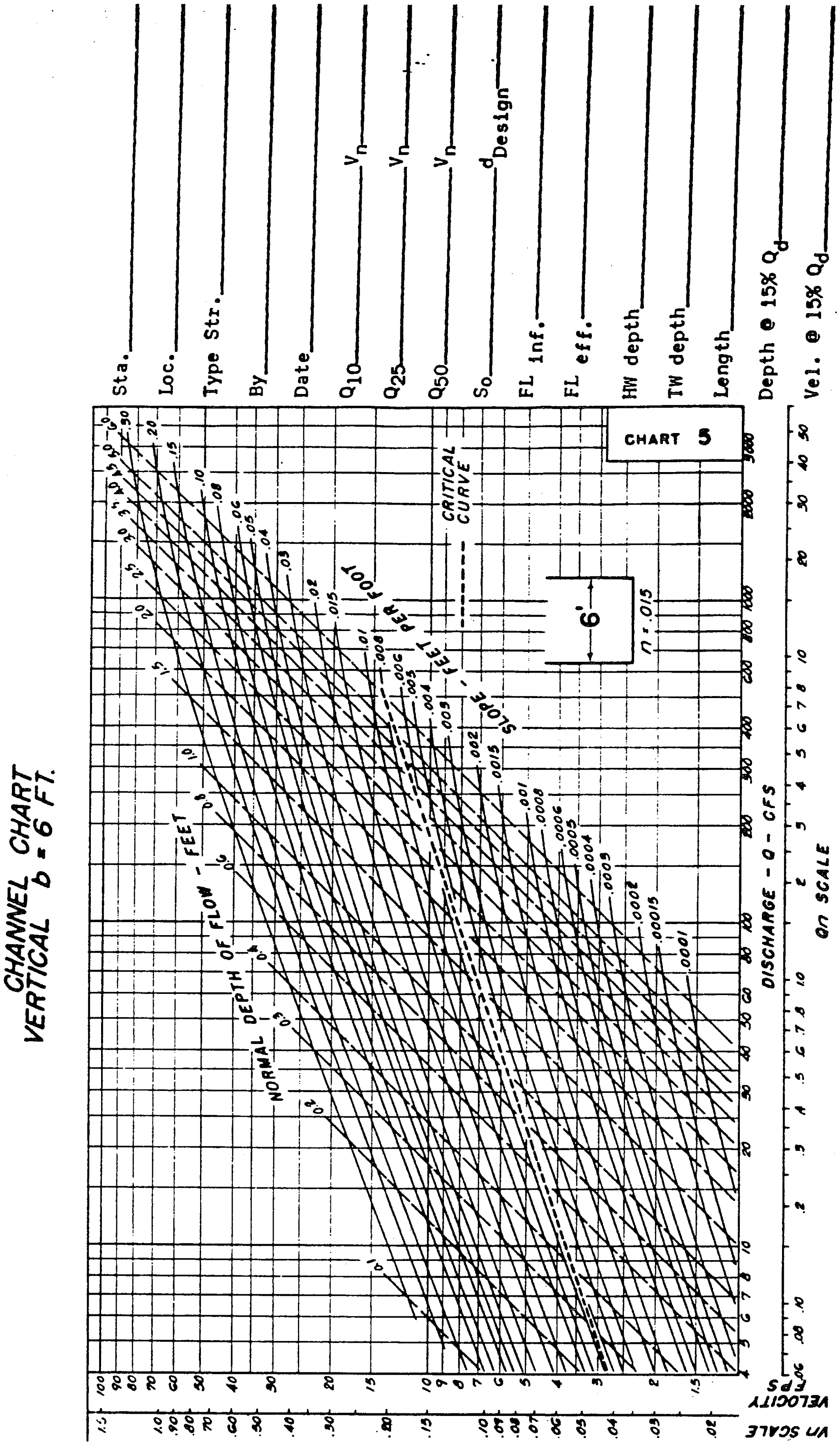
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 30



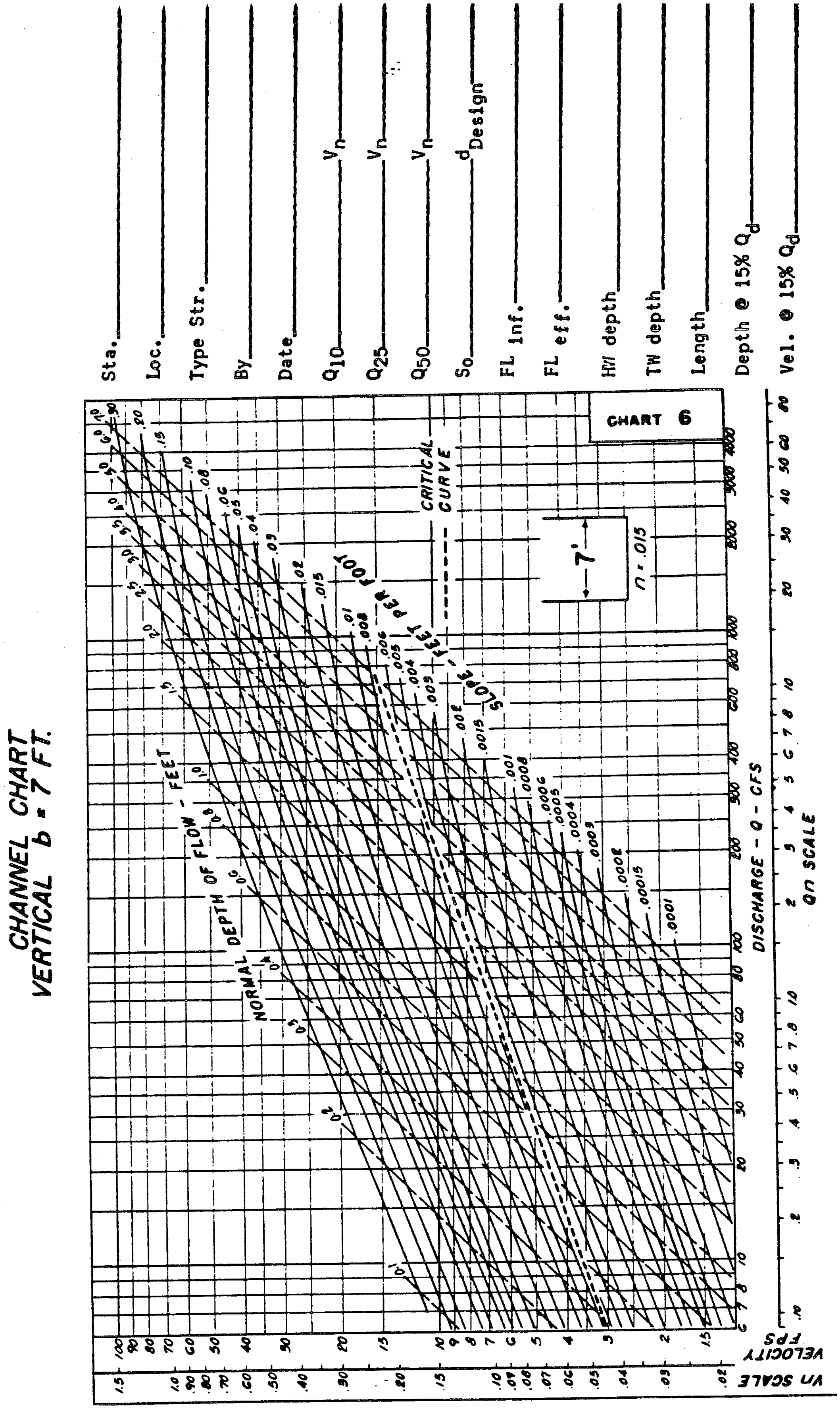
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 31



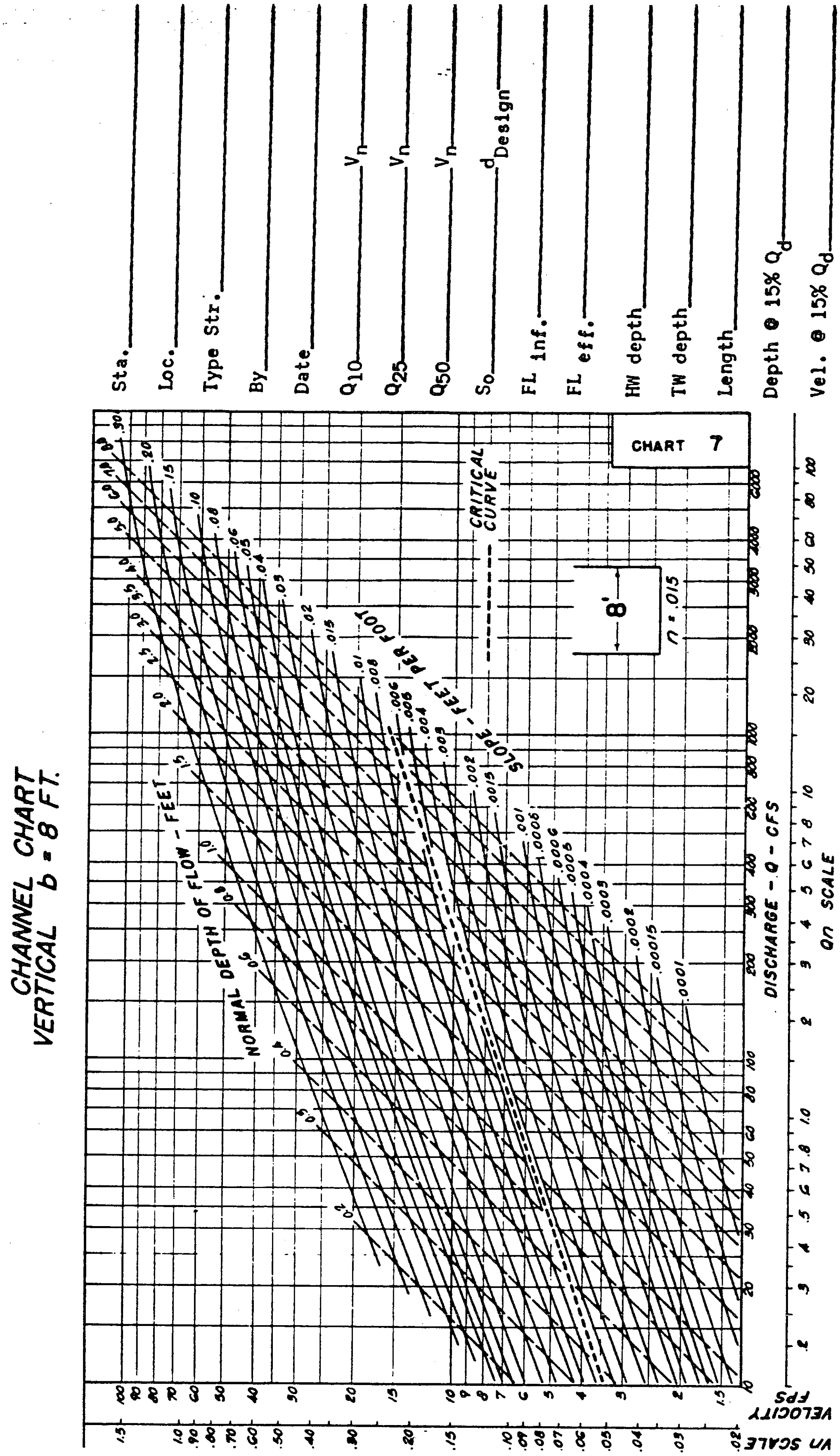
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 32



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

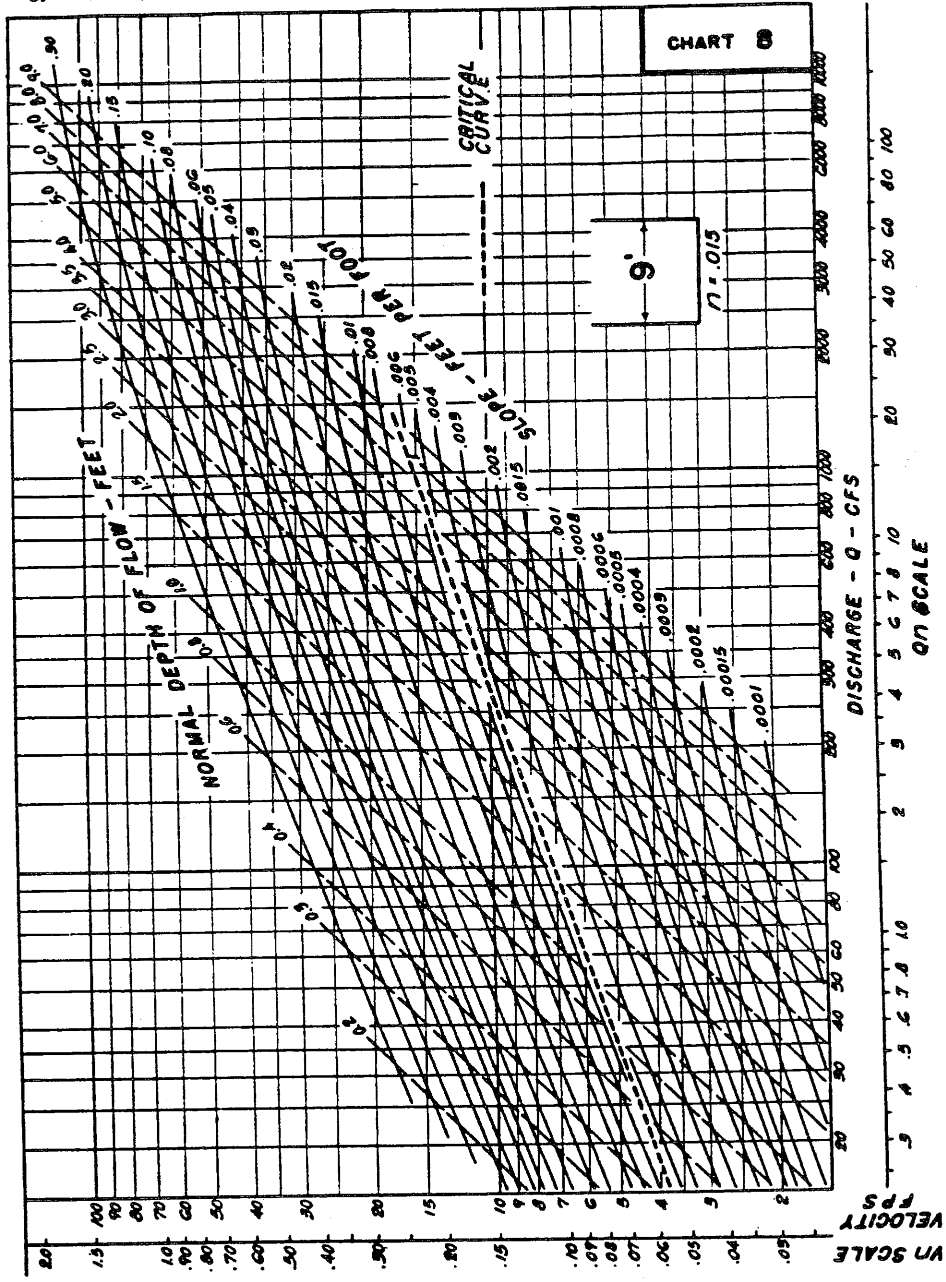
FIGURE 33



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 34

CHANNEL CHART
VERTICAL b = 9 FT.



Sta.________

Loc.________

Type Str.________

By________

Date________

Q_{10}________ V_n________

Q_{25}________ V_n________

Q_{50}________ V_n________

S_o________ d_{Design}________

FL inf.________

FL eff.________

HW depth________

TW depth________

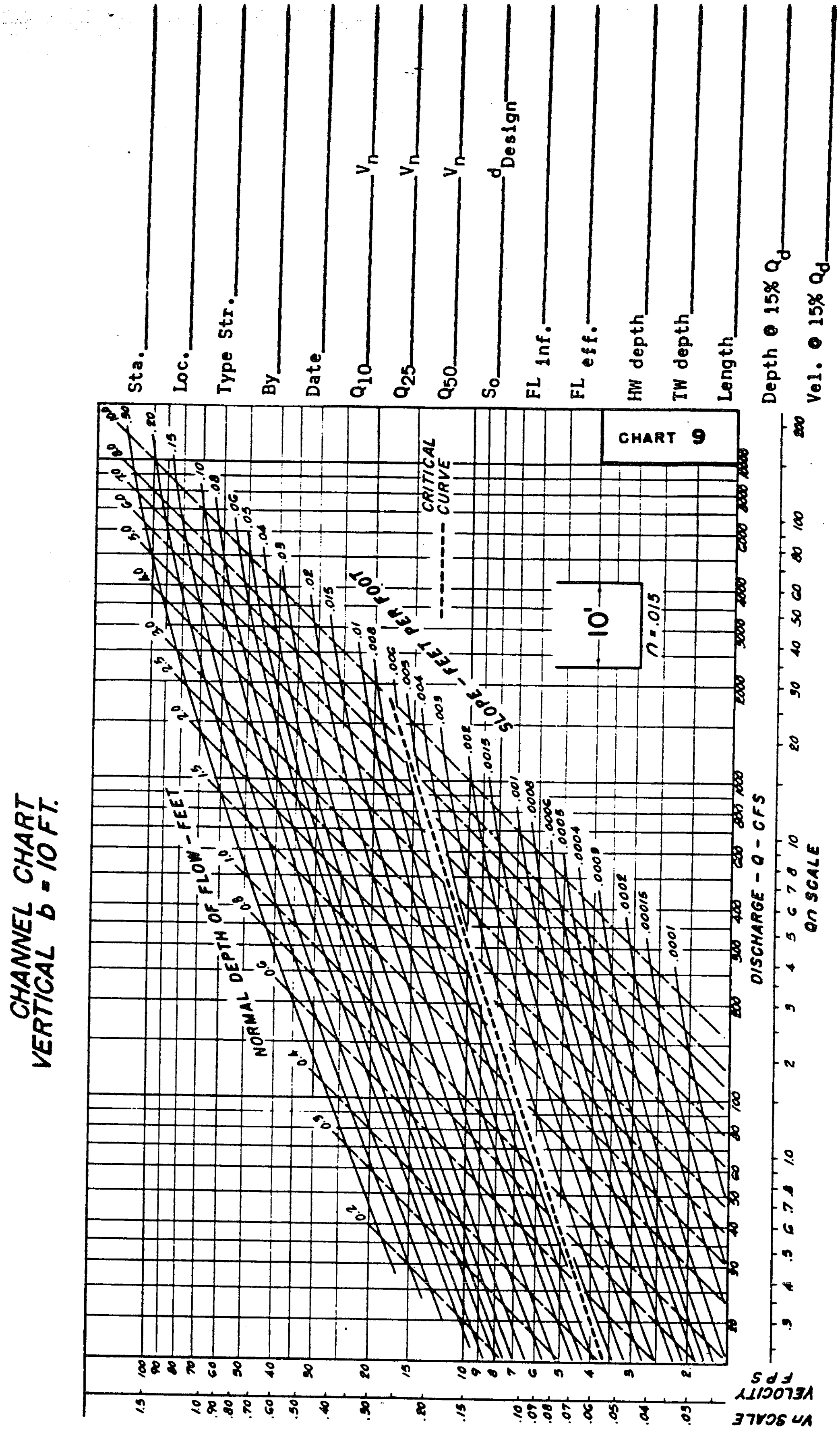
Length________

Depth @ 15% Q_d________

Vel. @ 15% Q_d________

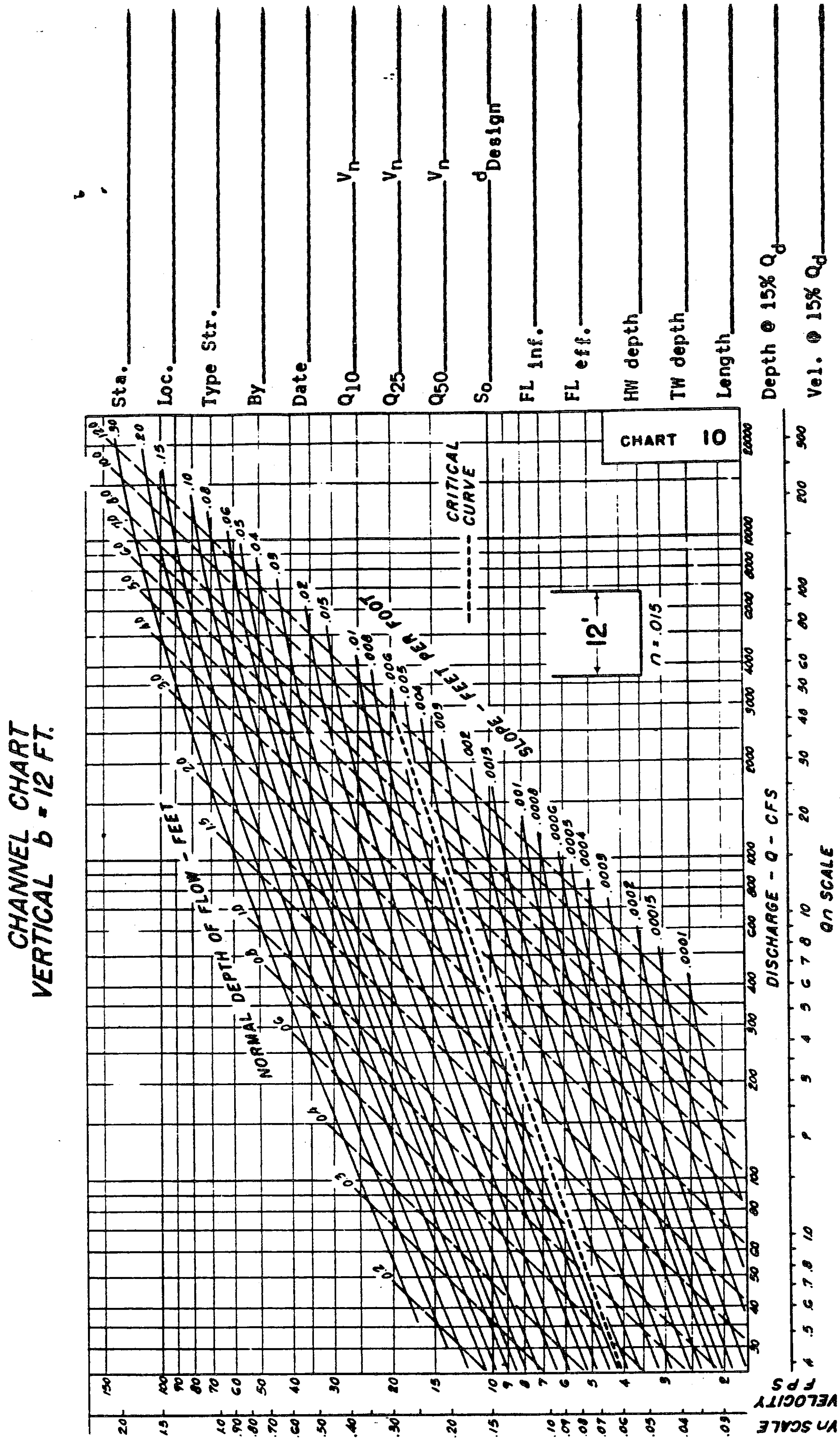
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 35



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

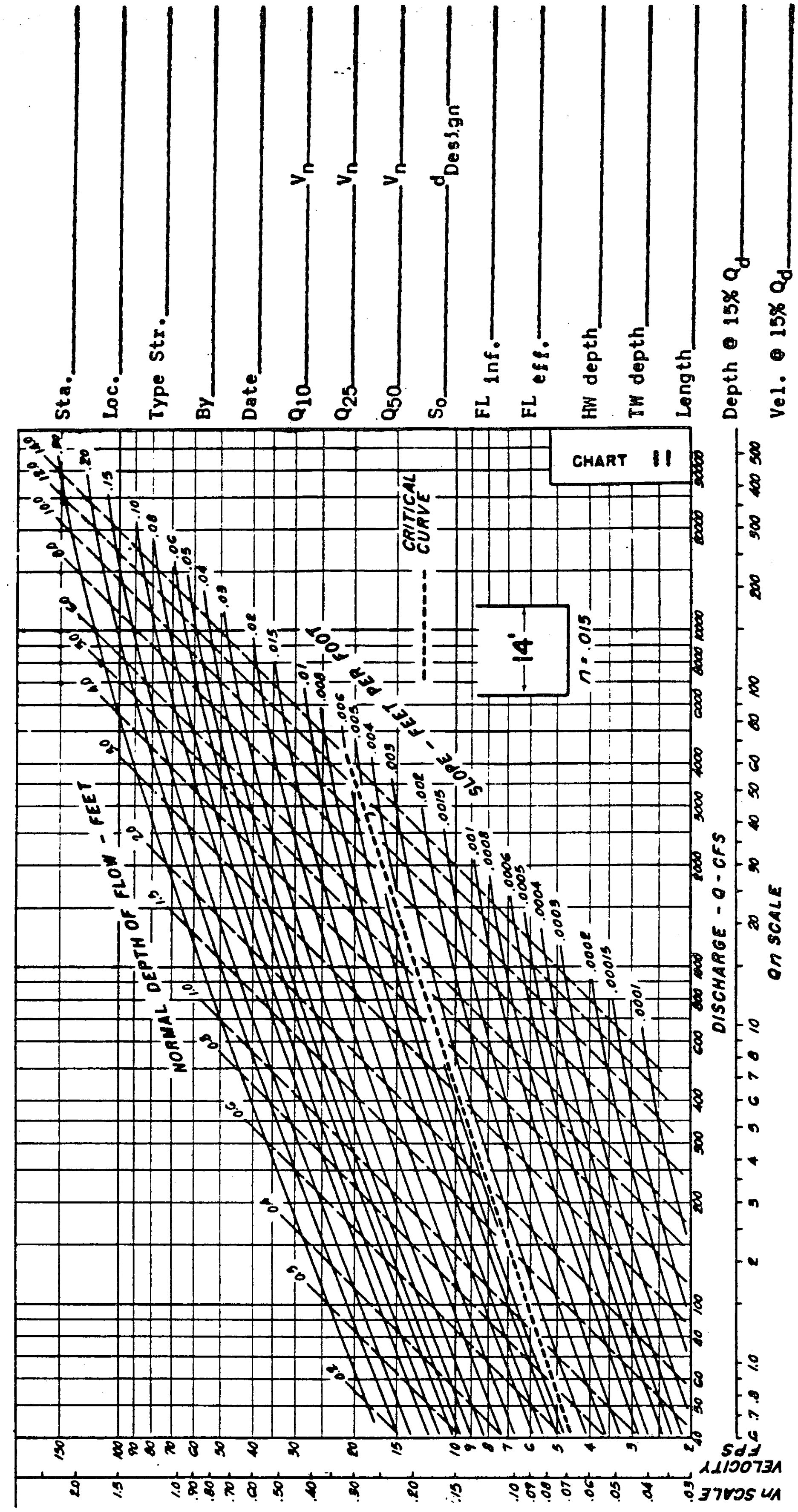
FIGURE 36



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 37

CHANNEL CHART
VERTICAL b = 14 FT.

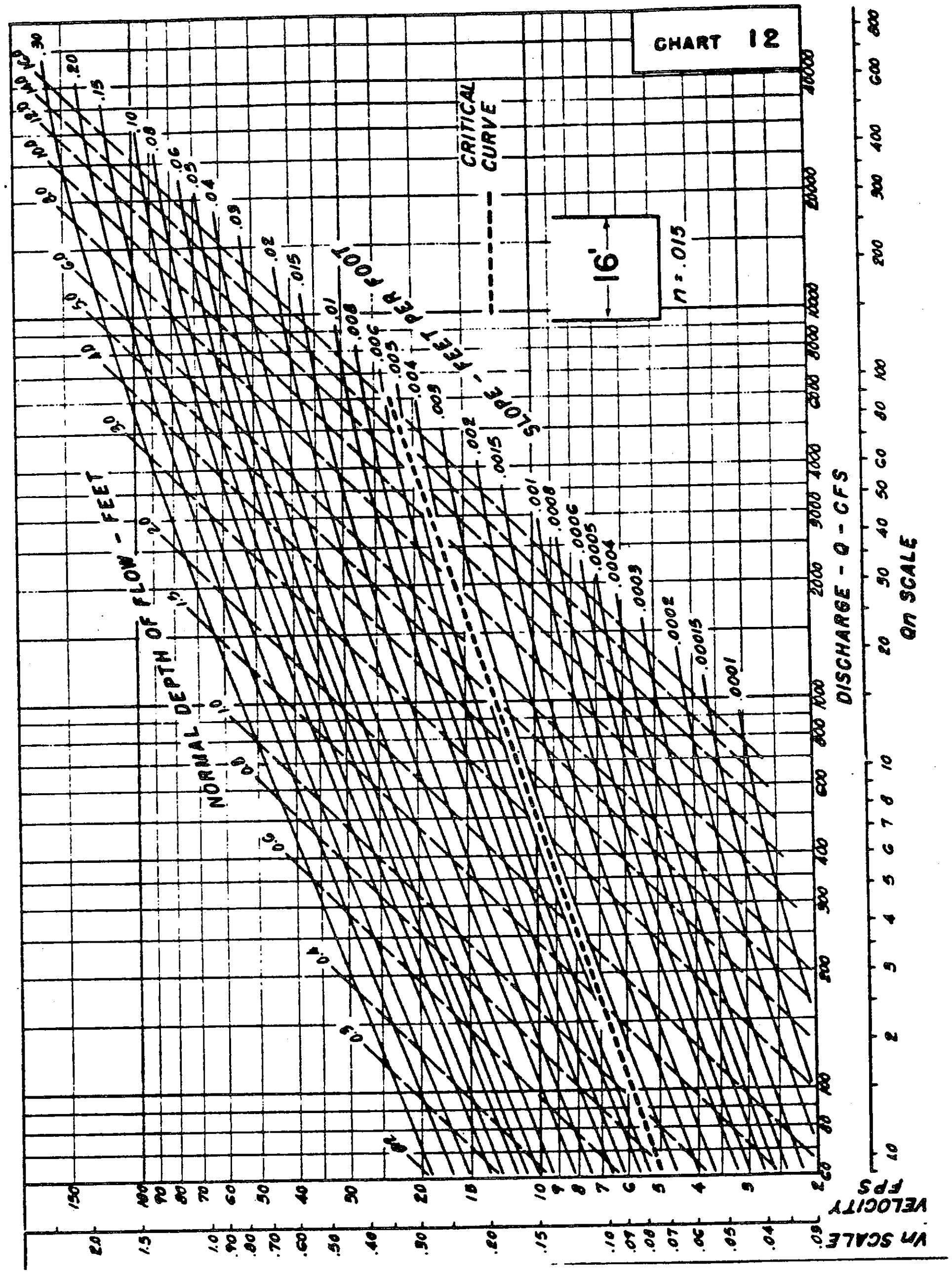


METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 38

FIG. 39

CHANNEL CHART
VERTICAL b = 16 FT.



Sta.____

Loc.____

Type Str.____

By____

Date____

Q_{10}____ V_n____

Q_{25}____ V_n____

Q_{50}____ V_n____

So____ d_{Design}____

FL inf.____

FL eff.____

HW depth____

TW depth____

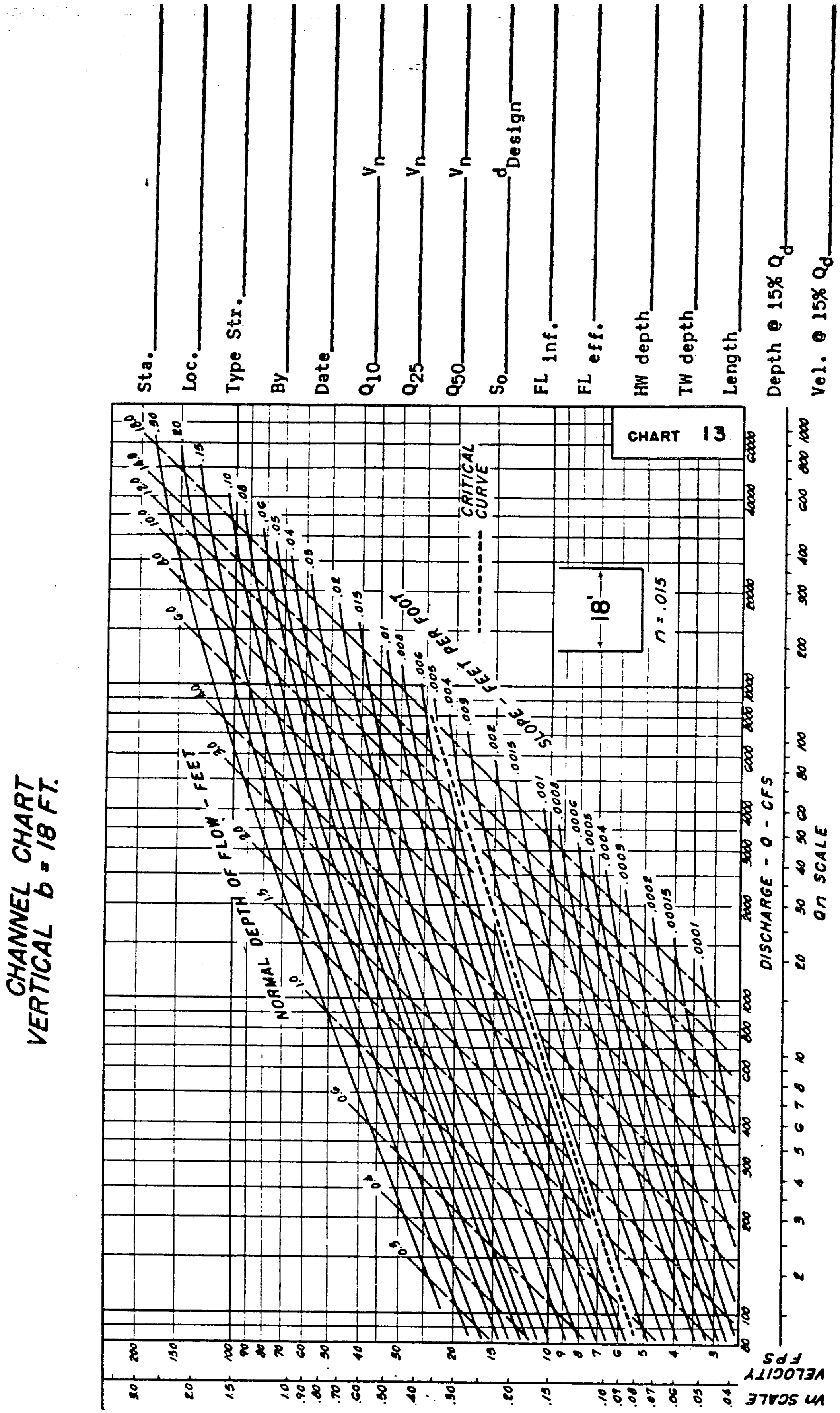
Length____

Depth @ 15% Q_d____

Vel. @ 15% Q_d____

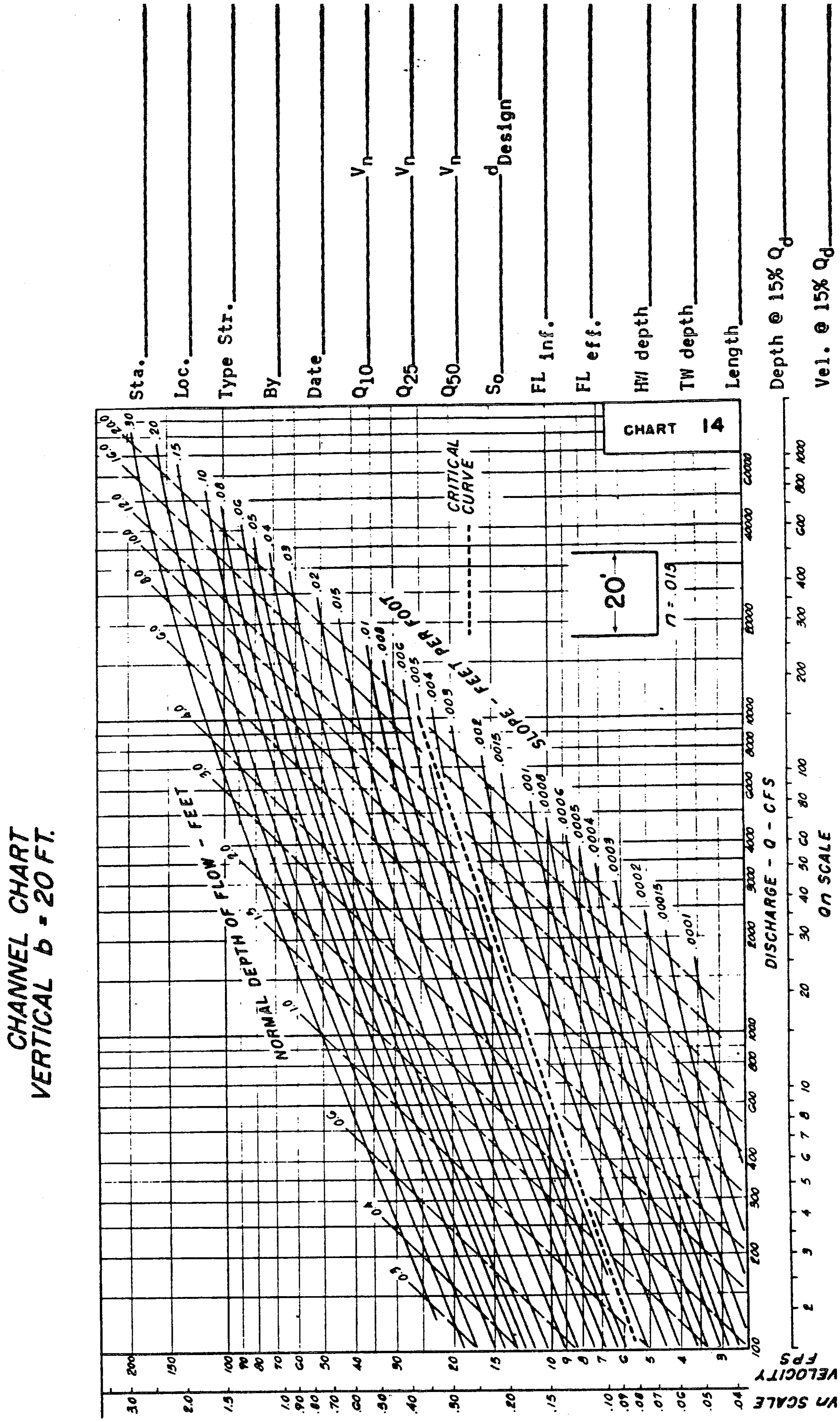
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 39



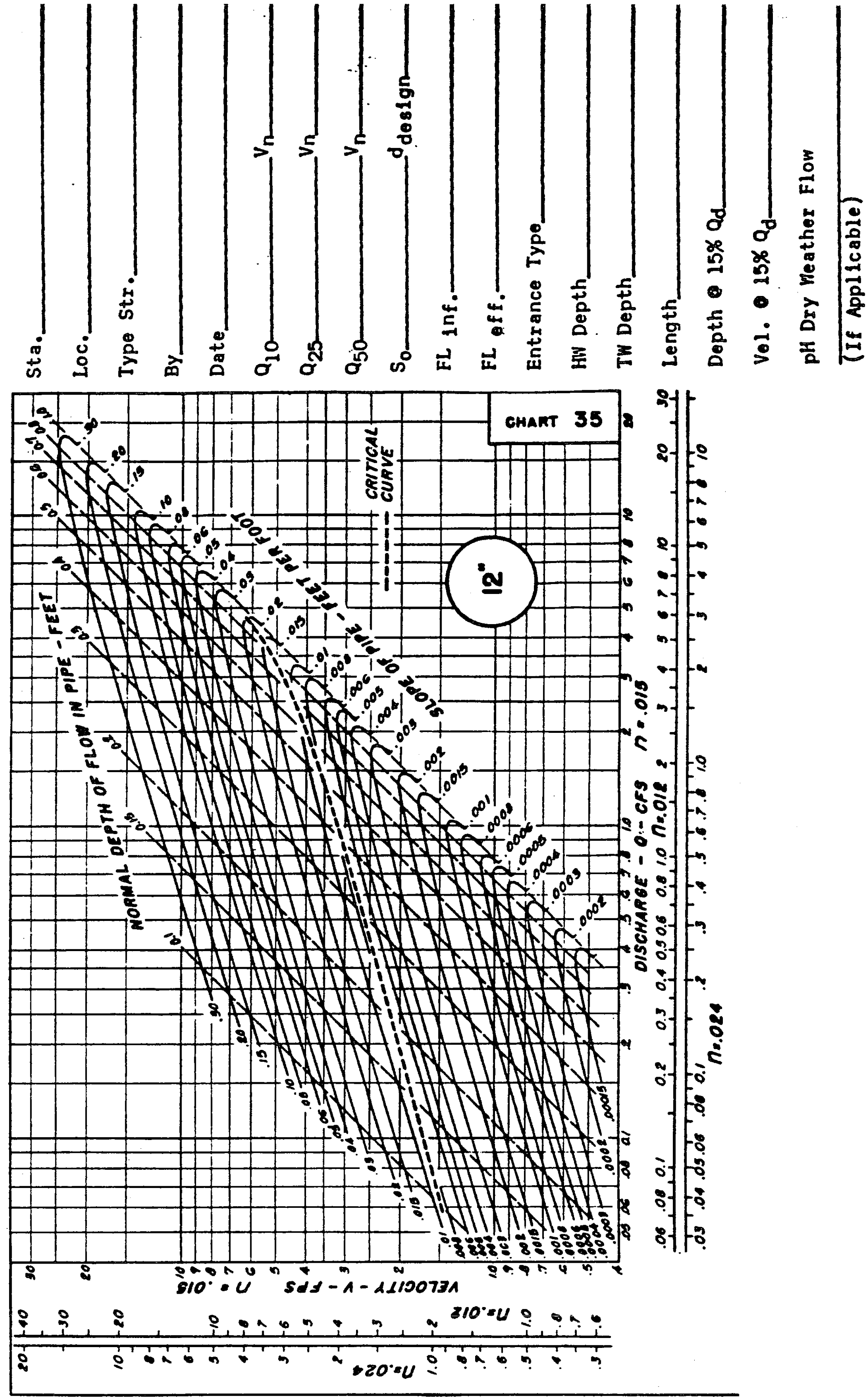
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 40



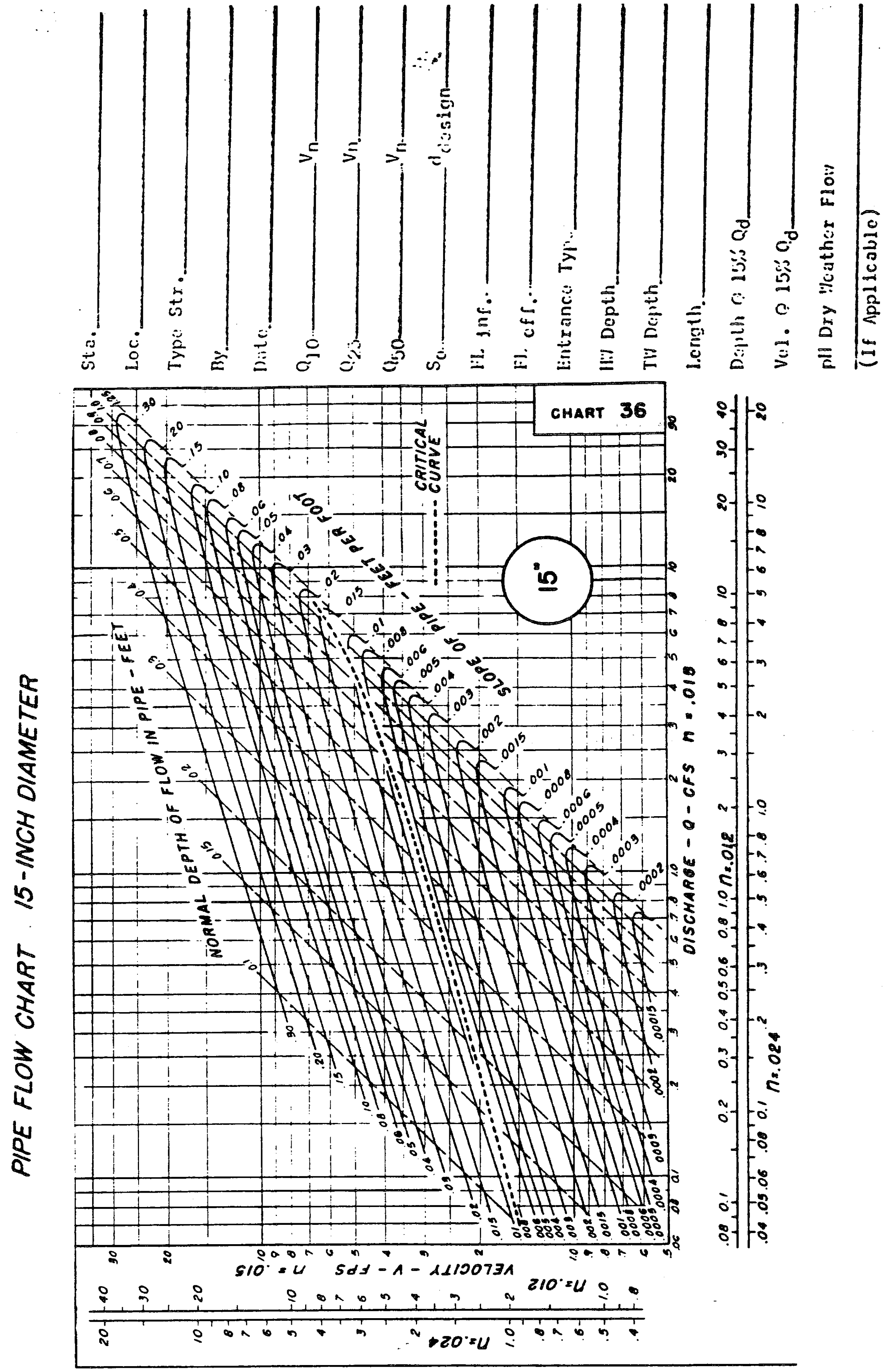
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 41



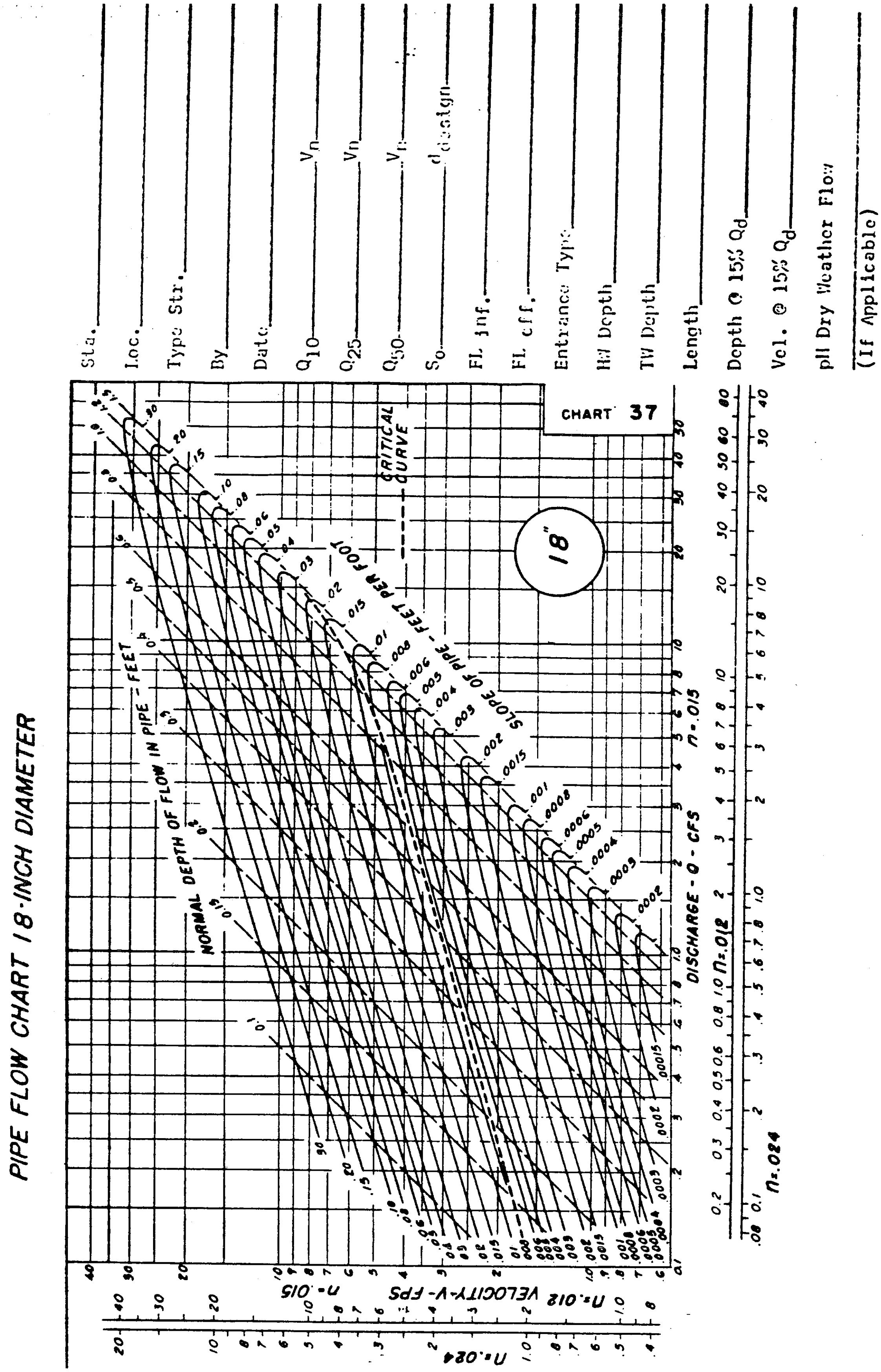
METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 42



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

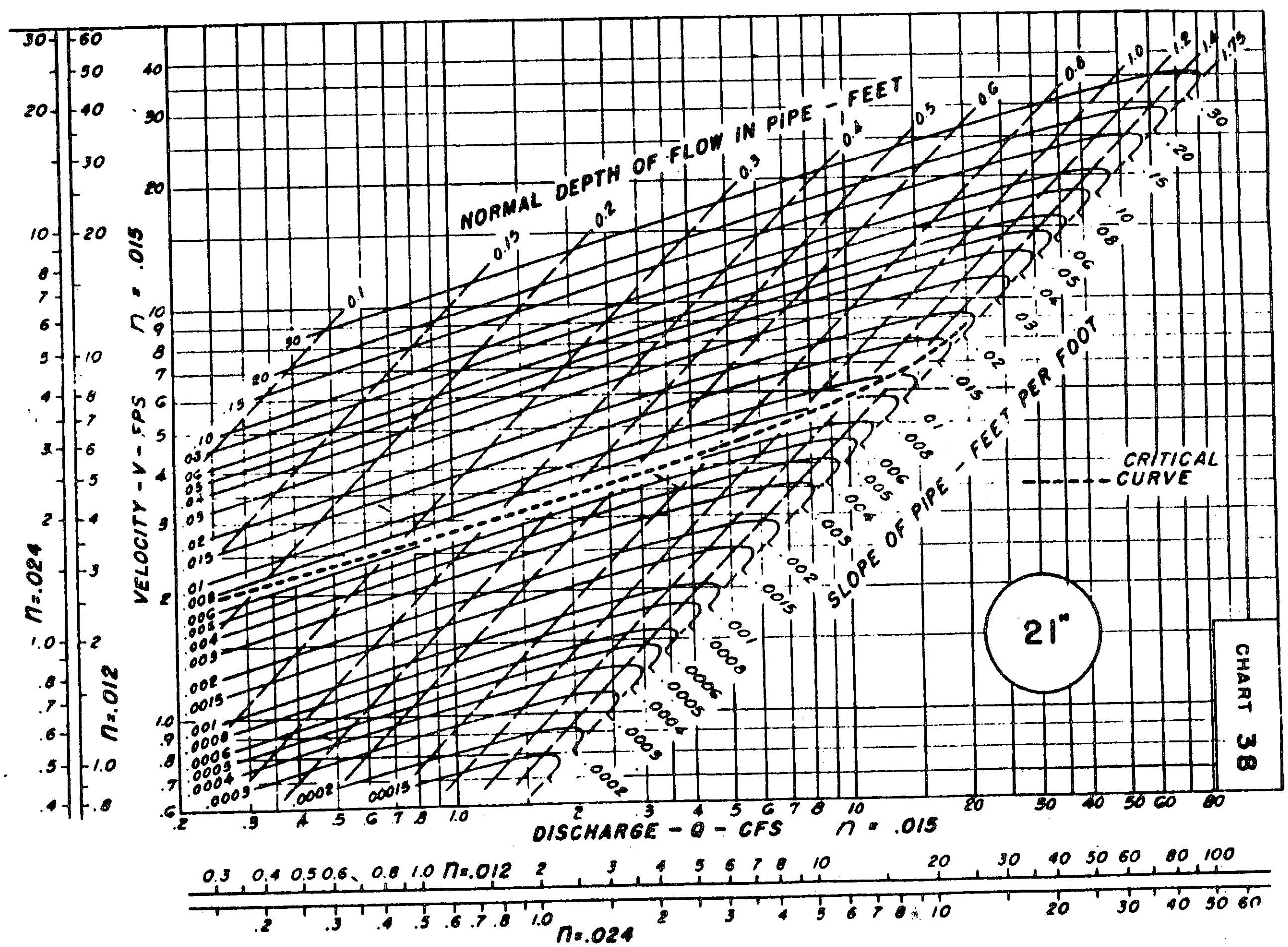
FIGURE 43



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 44

PIPE FLOW CHART 21-INCH DIAMETER



Sta.________

Loc.________

Type Str.________

By________

Date________

Q_{10}________ V_n________

Q_{25}________ V_n________

Q_{50}________ V_n________

S_o________ d_{design}________

Fl. inf.________

Fl. eff.________

Entrance Type________

HW Depth________

TW Depth________

Length________

Depth @ 15% Q_d________

Vel. @ 15% Q_d________

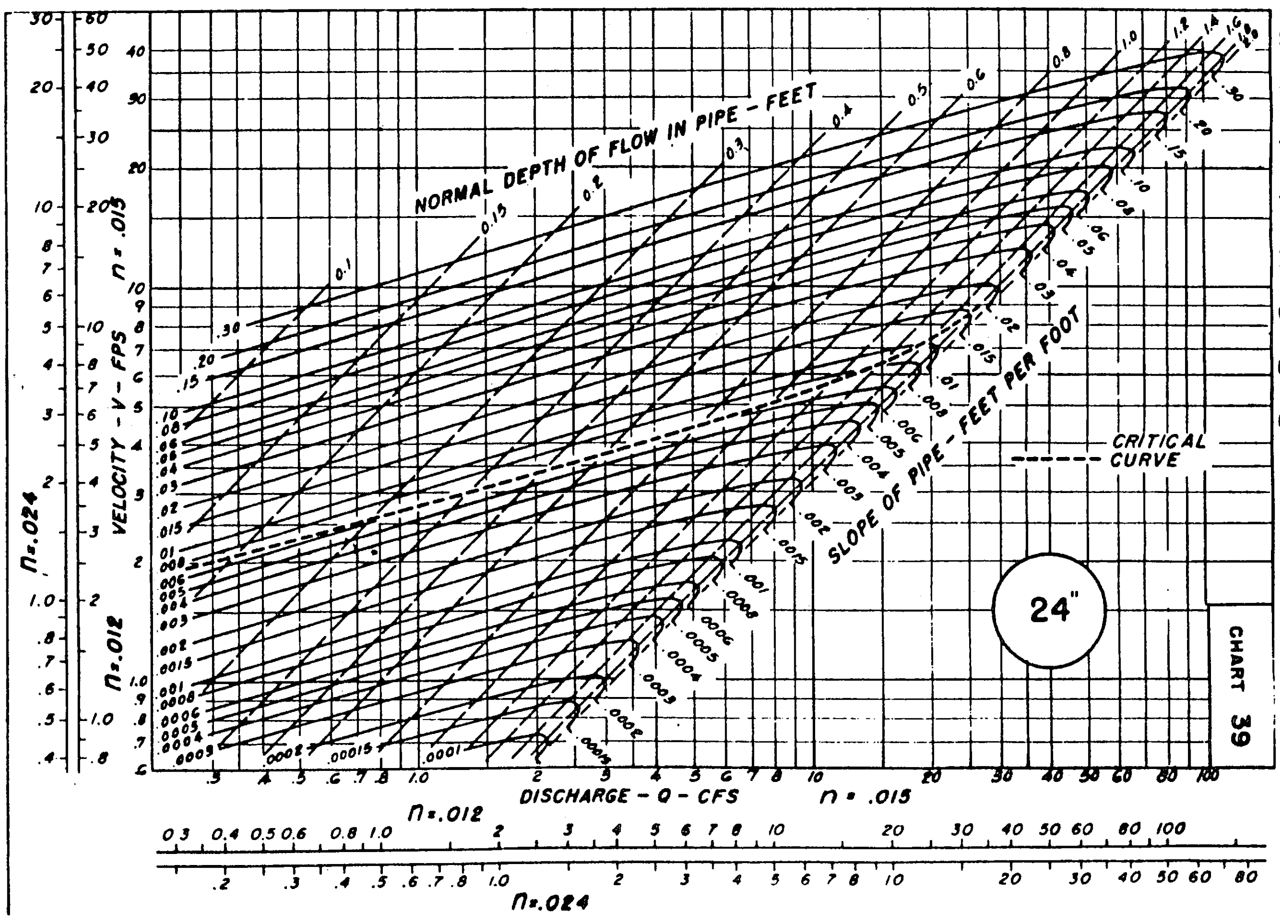
pH Dry Weather Flow________

(If Applicable)

METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 45

PIPE FLOW CHART 24-INCH DIAMETER



Sta.__________

Loc.__________

Type Str.__________

By__________

Date__________

Q_{10}__________ V_n__________

Q_{25}__________ V_n__________

Q_{50}__________ V_n__________

S_o__________ d_{design}__________

FL inf.__________

FL eff.__________

Entrance Type__________

HW Depth__________

TW Depth__________

Length__________

Depth @ 15% Q_d__________

Vel. @ 15% Q_d__________

pH Dry Weather Flow

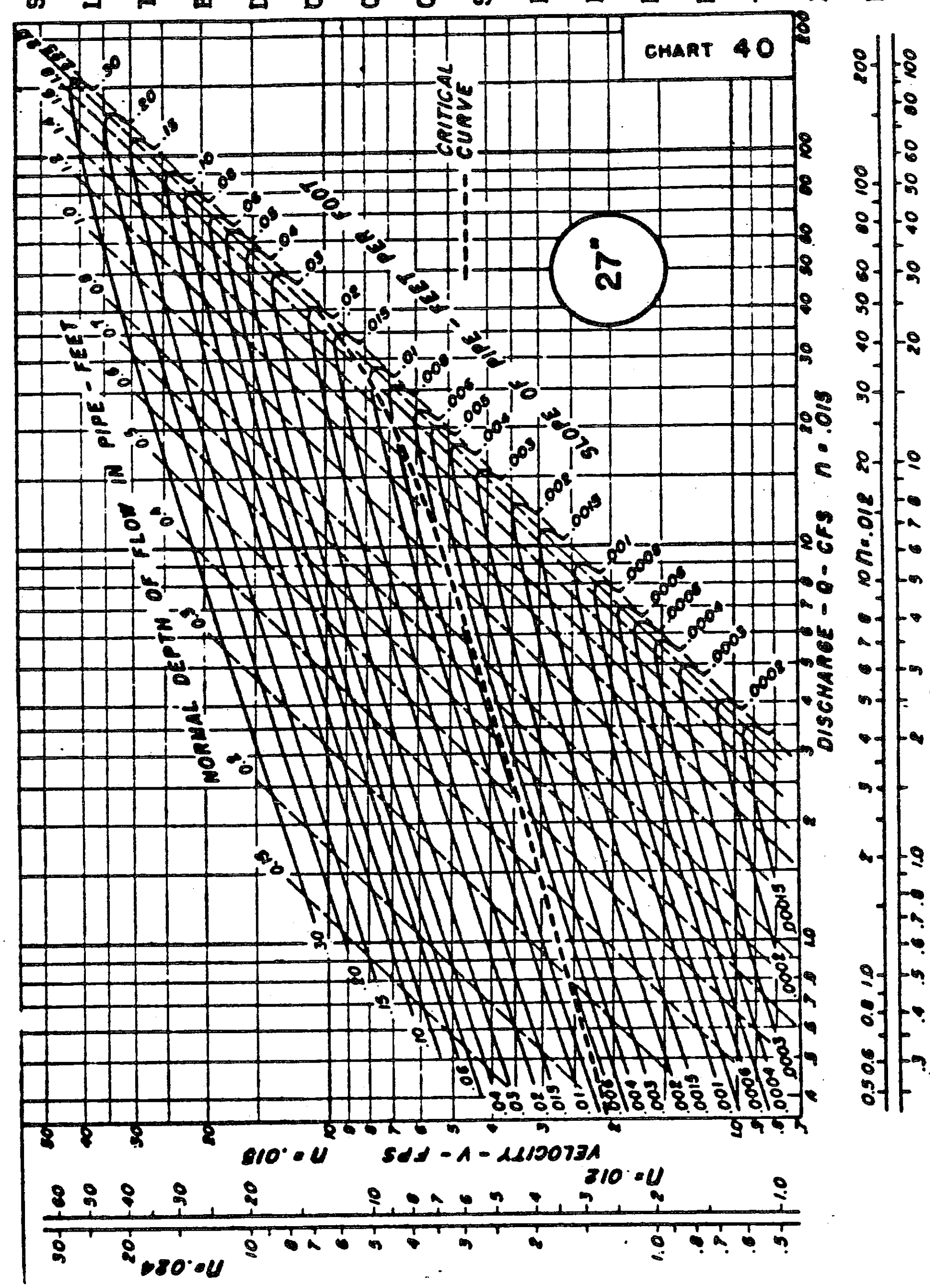
____

(If Applicable)

METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 46

PIPE FLOW CHART 27-INCH DIAMETER

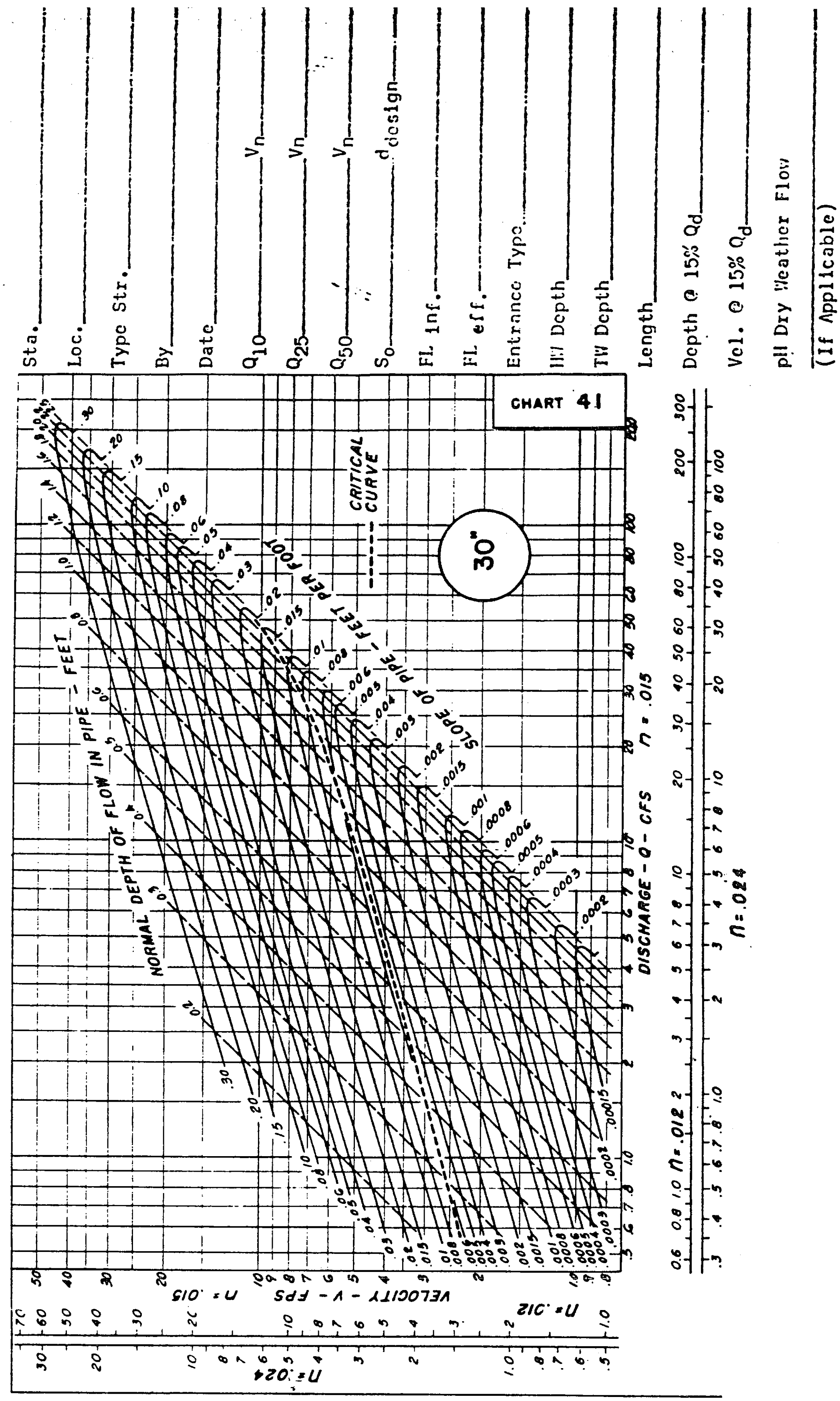


Sta.
Loc.
Type Str.
By
Date
Q_{10} V_n
Q_{25} V_n
Q_{50} V_n
S_o d_{design}
FL Inf.
FL eff.
Entrance Type
HW Depth
TW Depth
Length
Depth @ 15% Q_d
Vel. @ 15% Q_d
pH Dry Weather Flow
(If Applicable)

METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

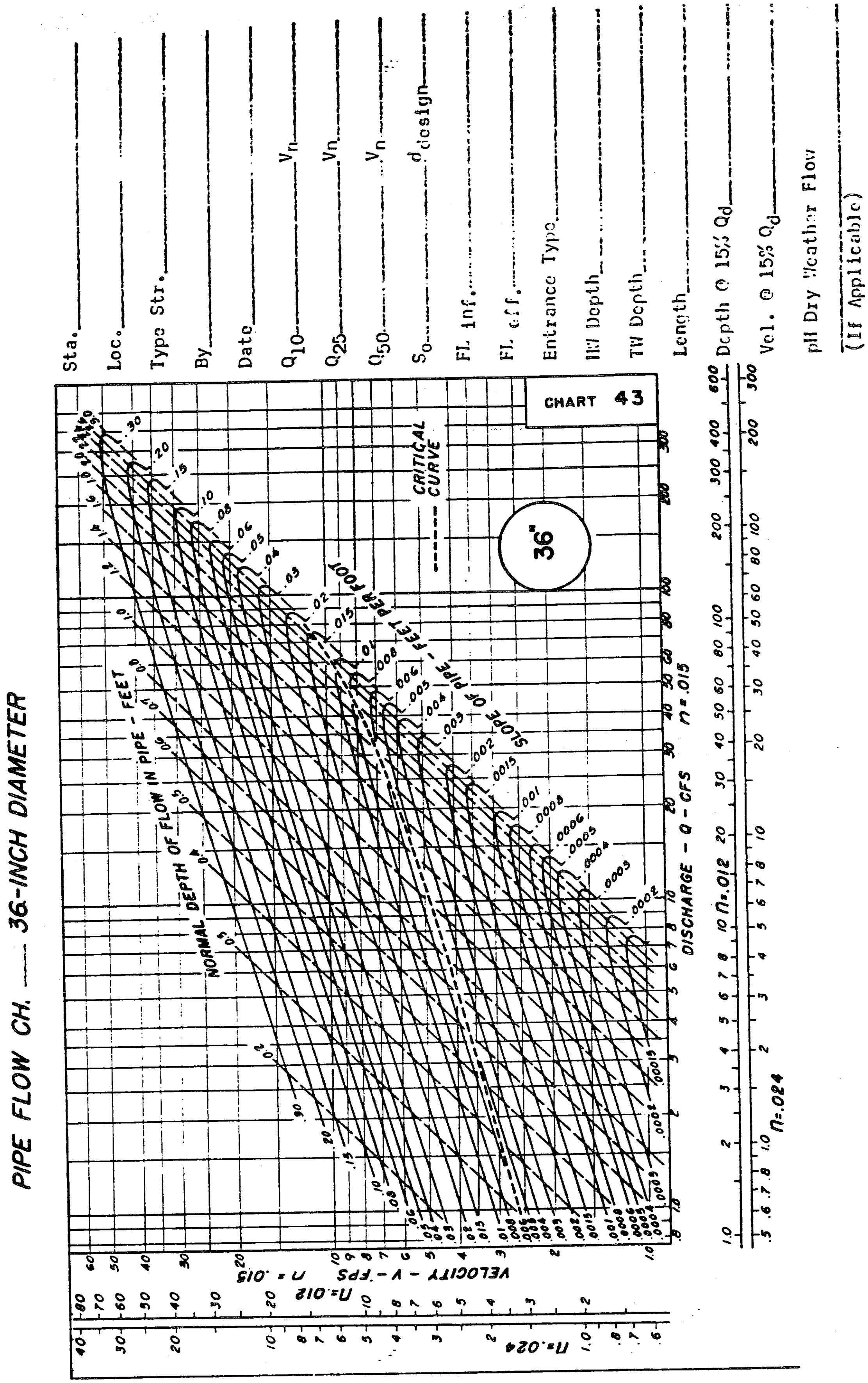
FIGURE 47

PIPE FLOW CHART 30-INCH DIAMETER



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

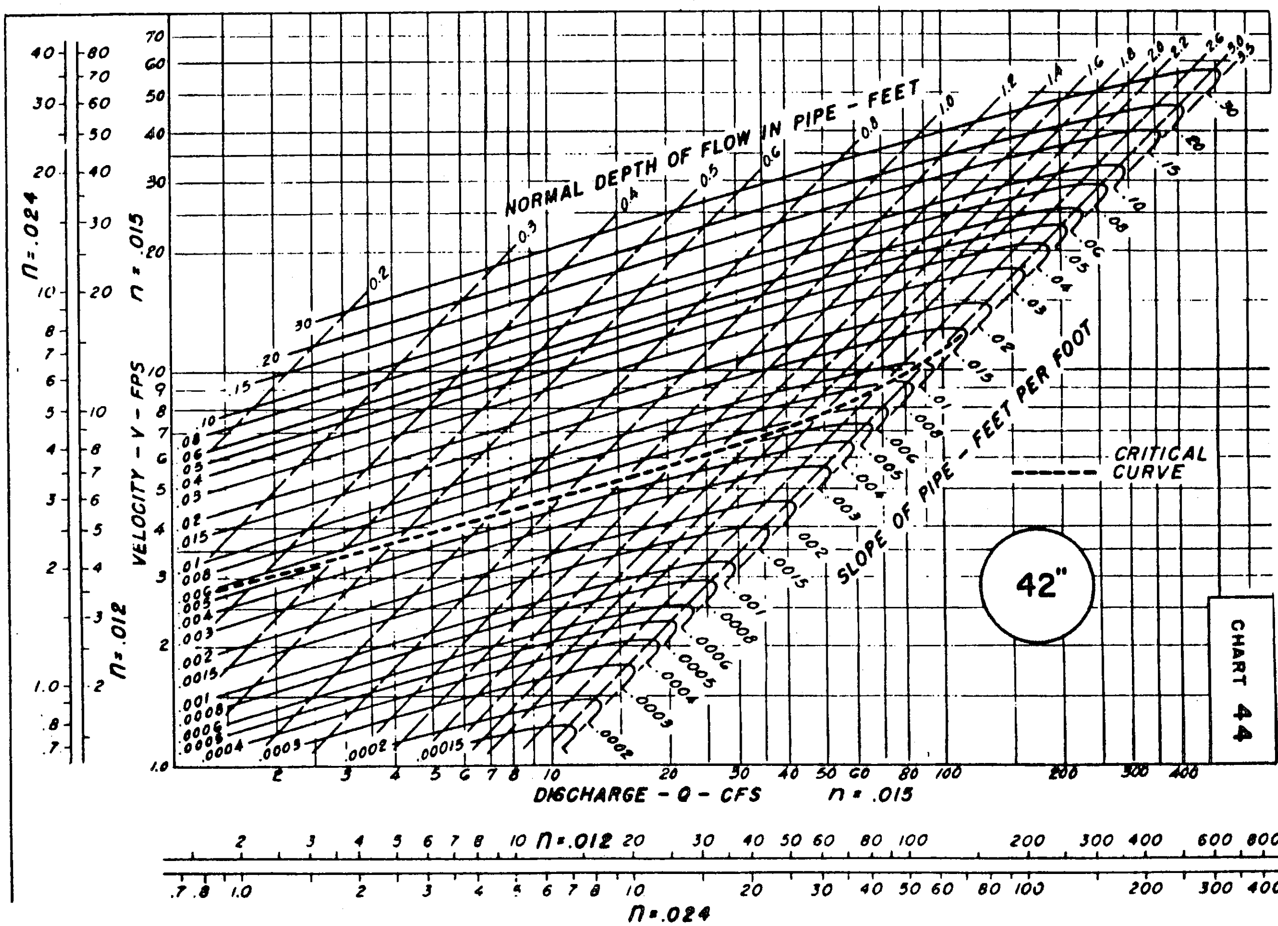
FIGURE 48



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 49

PIPE FLOW CHART 42-INCH DIAMETER



Sta.________

Loc.________

Type Str.________

By________

Date________

Q_{10}________ V_n________

Q_{25}________ V_n________

Q_{50}________ V_n________

S_o________ d_{design}________

FL inf.________

FL eff.________

Entrance Type________

HW Depth________

TW Depth________

Length________

Depth @ 15% Q_d________

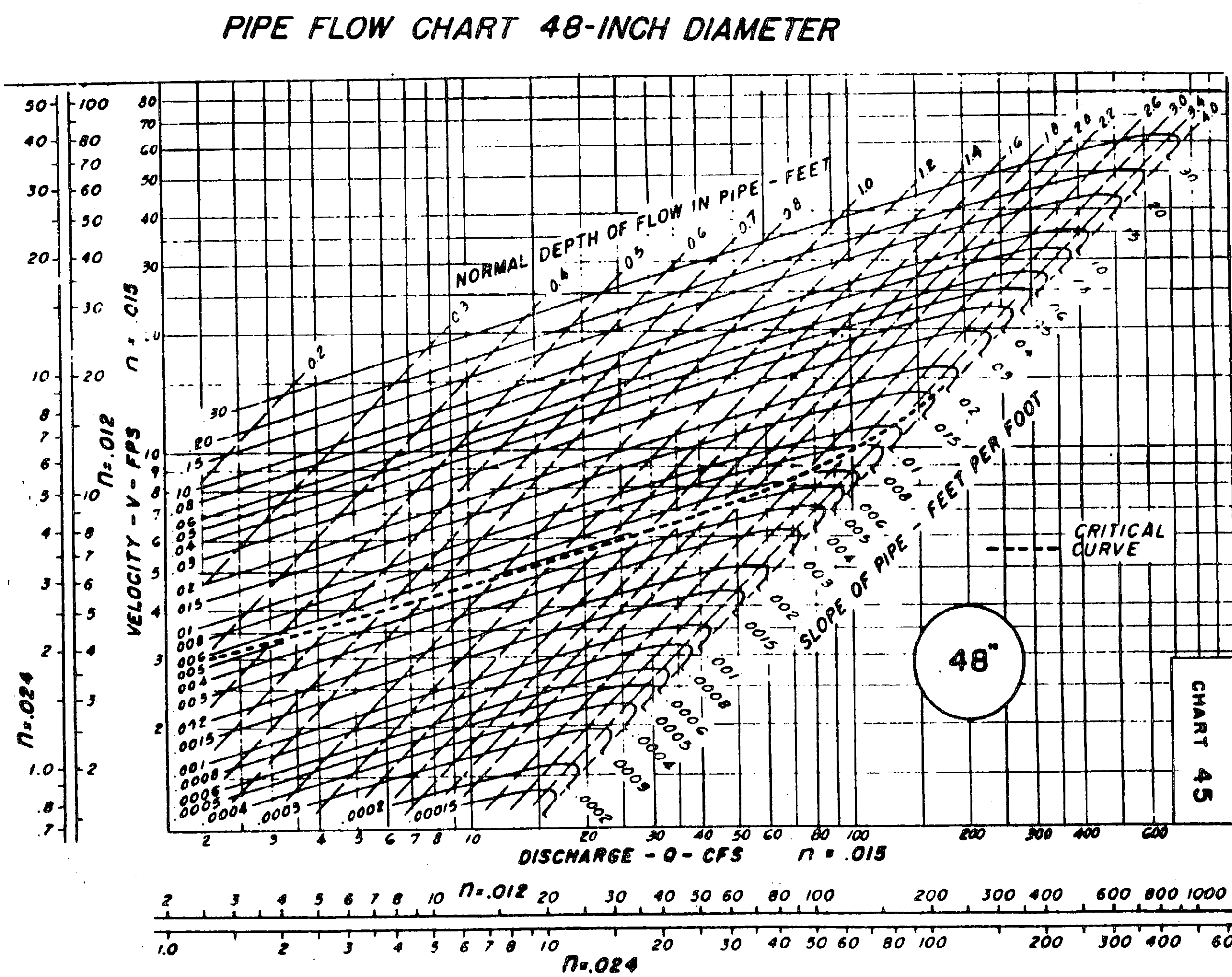
Vel. @ 15% Q_d________

pH Dry Weather Flow________

(If Applicable)

METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 50



Sta.__________

Loc.__________

Type Str.__________

By__________

Date__________

Q_{10}__________ V_n__________

Q_{25}__________ V_n__________

Q_{50}__________ V_n__________

S_o__________ d_{design}__________

FL inf.__________

FL eff.__________

Entrance Type__________

HW Depth__________

TW Depth__________

Length__________

Depth @ 15% Q_d__________

Vel. @ 15% Q_d__________

pH Dry Weather Flow

____

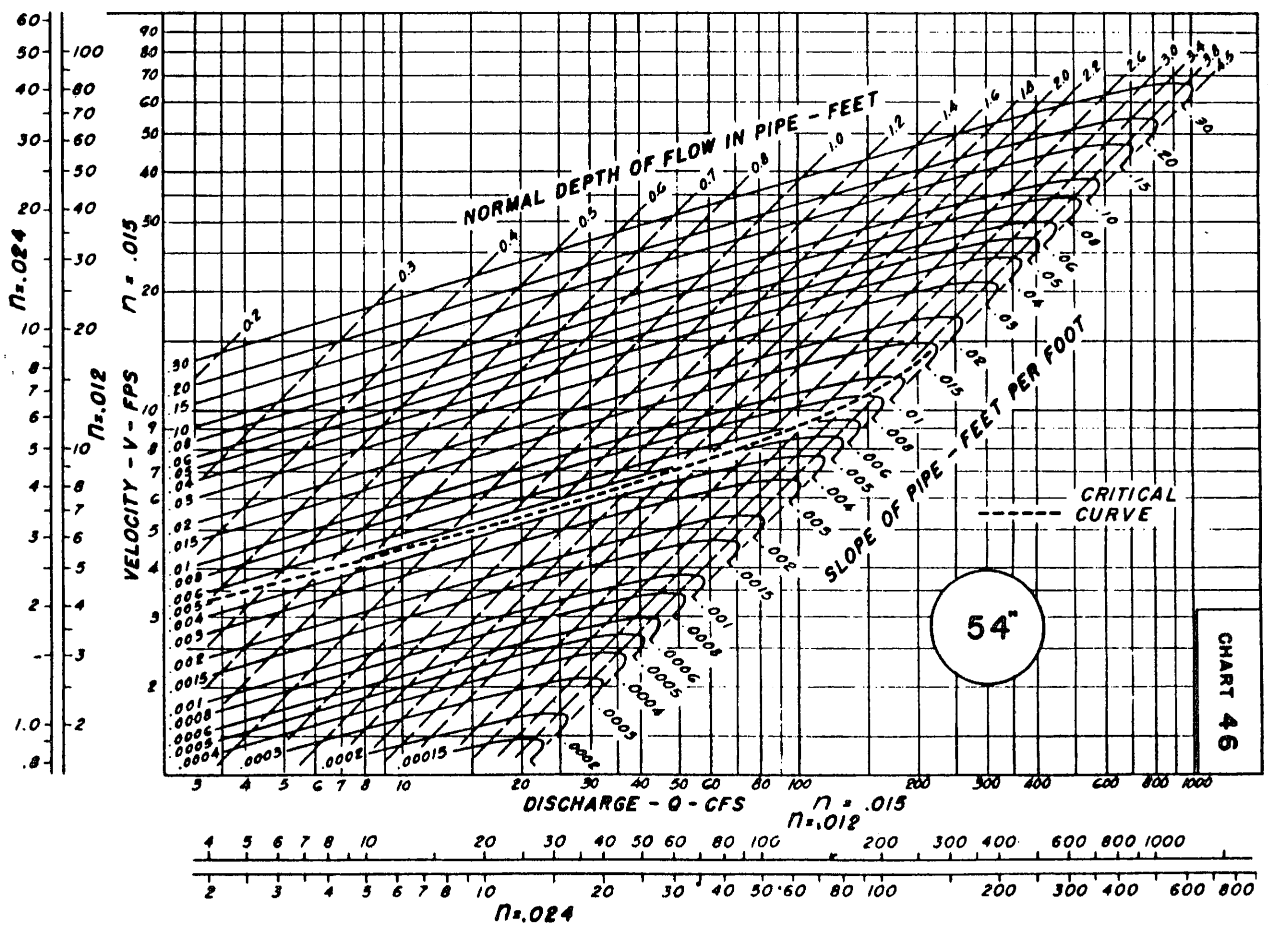
(If Applicable)

METRIC CONVERSION: 1 in = 25.4 mm

1 ft = 0.3048 m

FIGURE 51

PIPE FLOW CHART 54-INCH DIAMETER



Sta.________

Loc.________

Type Str.________

By________

Date________

Q_{10}________ Vn________

Q_{25}________ Vn________

Q_{50}________ Vn________

S_o________ d_{design}________

$FL_{inf.}$________

$FL_{eff.}$________

Entrance Type________

HW Depth________

TW Depth________

Length________

Depth @ 15% Q_d________

Vel. @ 15% Q_d________

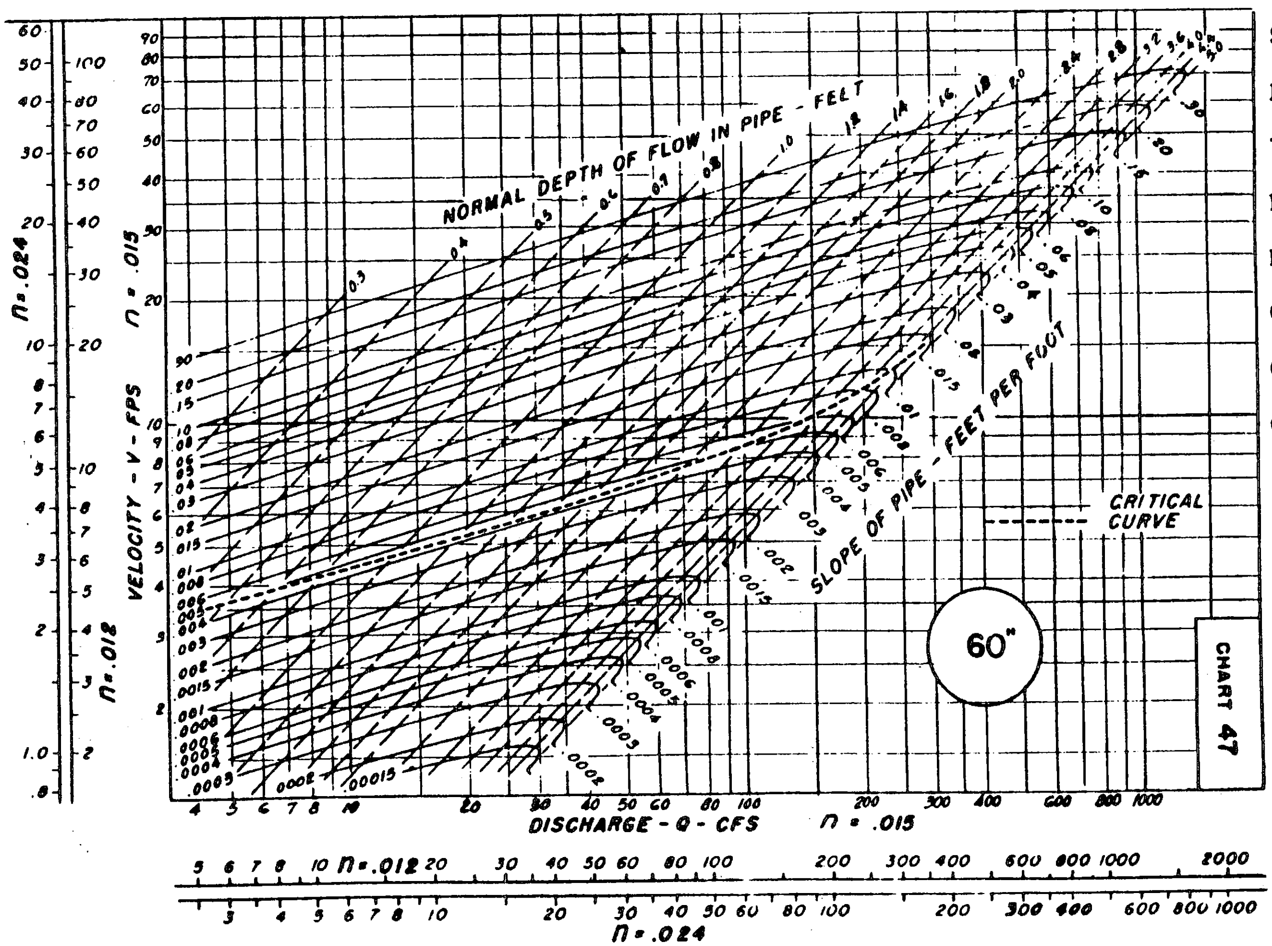
pH Dry Weather Flow________

(If Applicable)

METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 52

PIPE FLOW CHART 60-INCH DIAMETER



Sta.________________

Loc.________________

Type Str.________________

By________________

Date________________

Q_{10}________ V_n________

Q_{25}________ V_n________

Q_{50}________ V_n________

S_o________ d_{design}________

FL inf.________________

FL eff.________________

Entrance Type________________

HW Depth________________

TW Depth________________

Length________________

Depth @ 15% Q_d________________

Vel. @ 15% Q_d________________

pH Dry Weather Flow________________

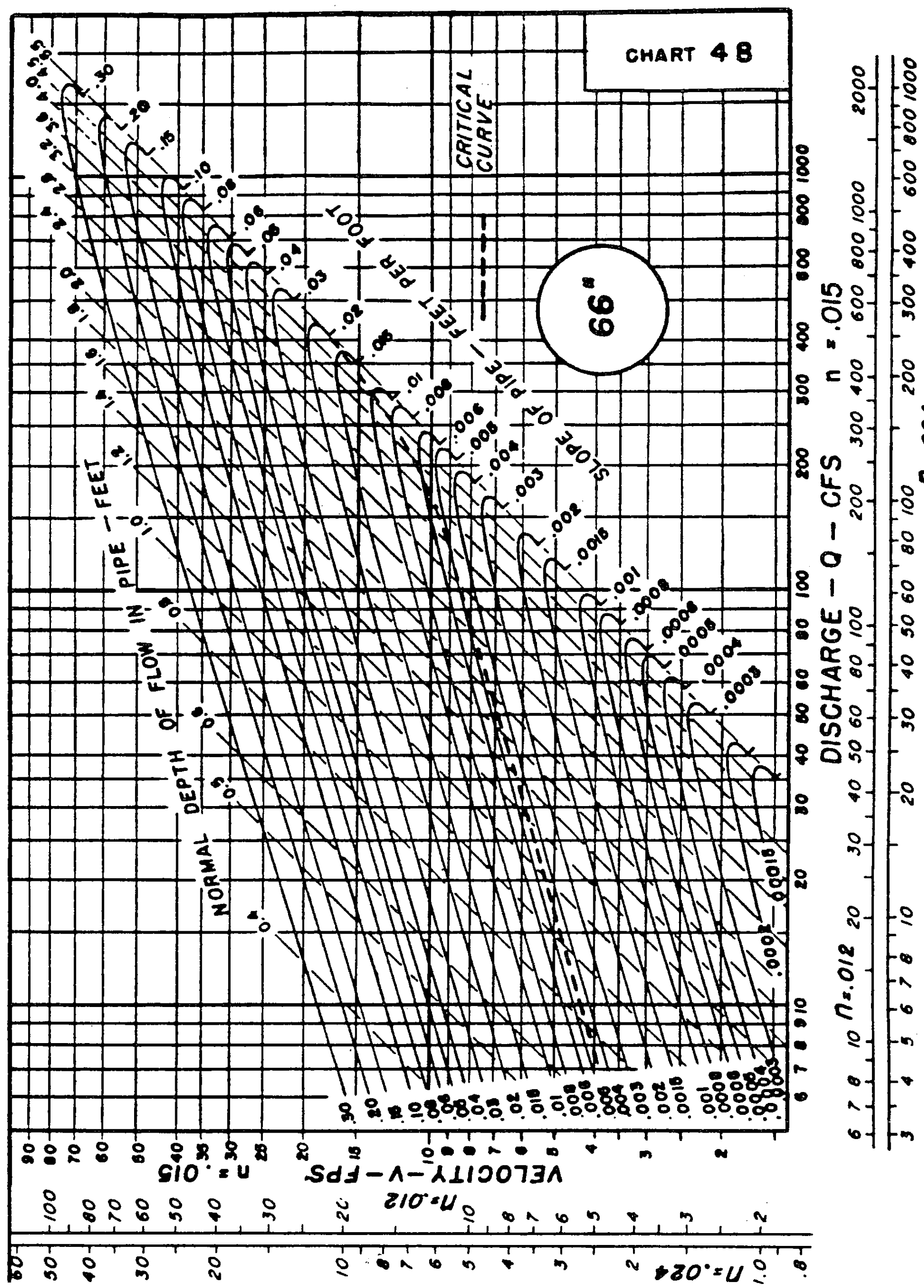
(If Applicable)

METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 53

Sta. ____
Loc. ____
Type Str. ____
By ____
Date ____
Q_{10} ____ Vn ____
Q_{25} ____ Vn ____
Q_{50} ____ Vn ____
So ____ d_{design} ____
$FL_{inf.}$ ____
$FL_{eff.}$ ____
Entrance Type ____
HW Depth ____
TW Depth ____
Length ____
Depth @ 15% Q_d ____
Vel. @ 15% Q_d ____
pH Dry Weather Flow ____
(If Applicable)

PIPE FLOW CHART 66-INCH DIAMETER



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 54

Sta.

Loc.

Type Str.

By

Date

Q_{10} ____ Vn

Q_{25} ____ Vn

Q_{50} ____ Vn

S_o ____ d_{design}

$FL_{Inf.}$

$FL_{eff.}$

Entrance Type

HW Depth

TW Depth

Length

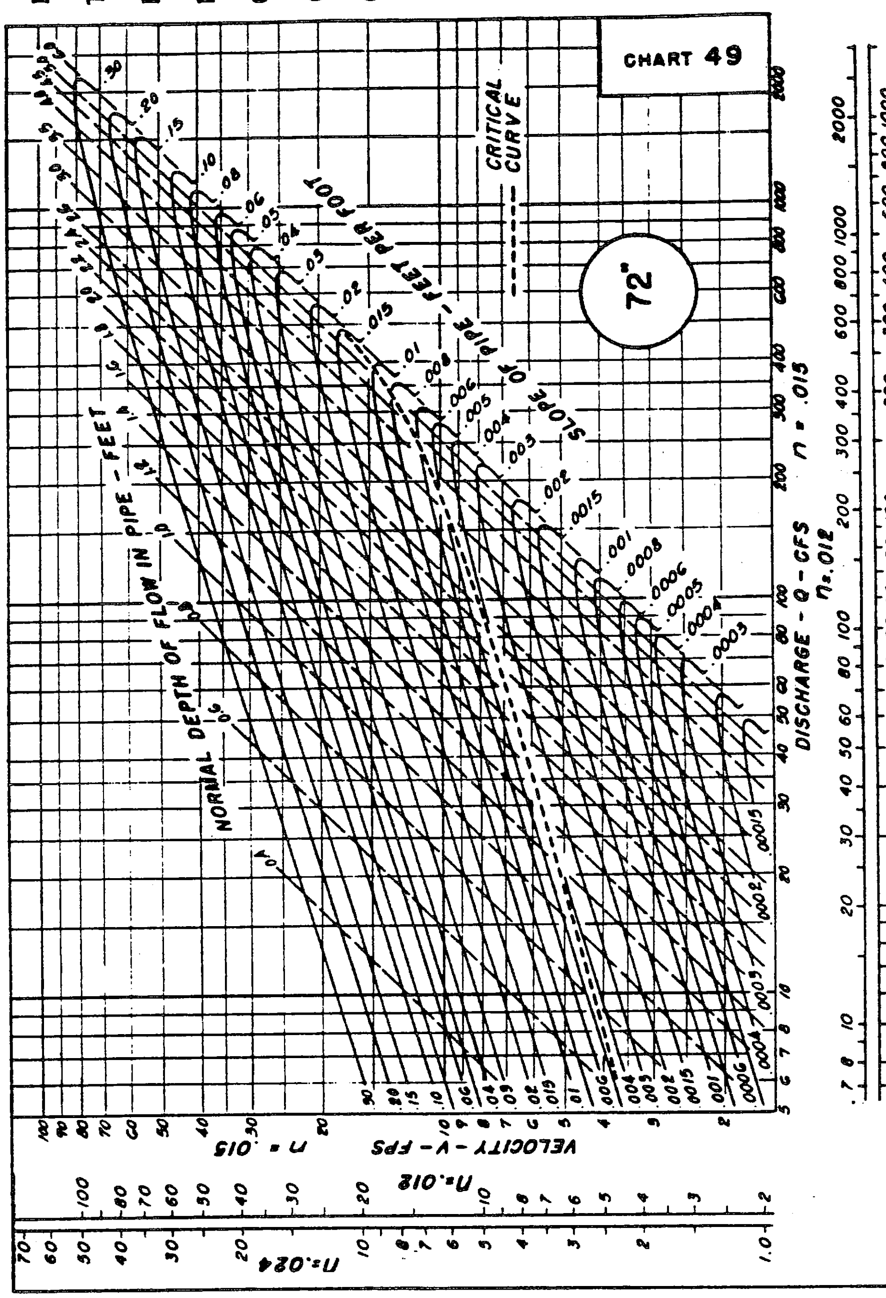
Depth @ 15% Q_d

Vel. @ 15% Q_d

pH Dry Weather Flow

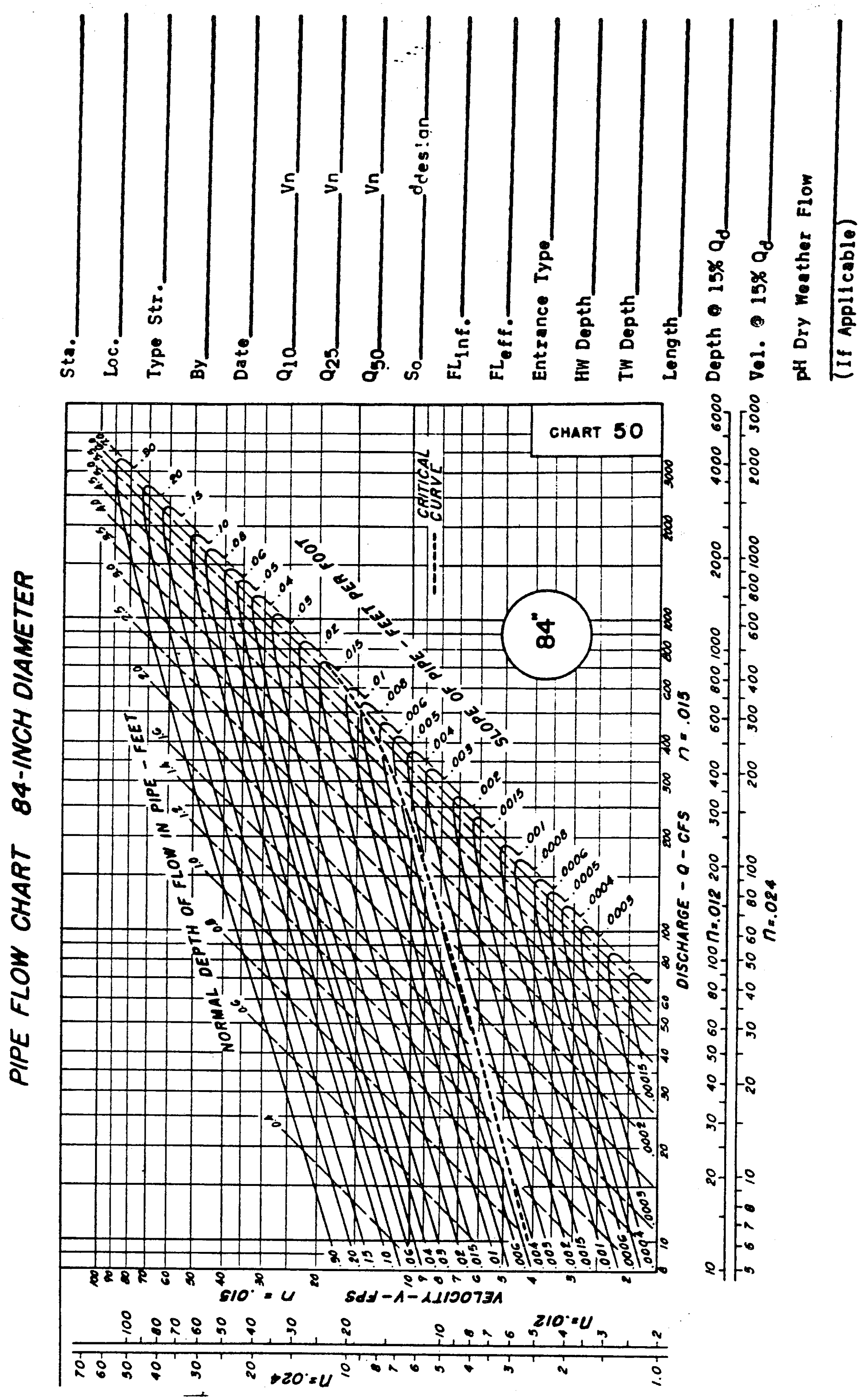
(If Applicable)

PIPE FLOW CHART 72-INCH DIAMETER



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 55

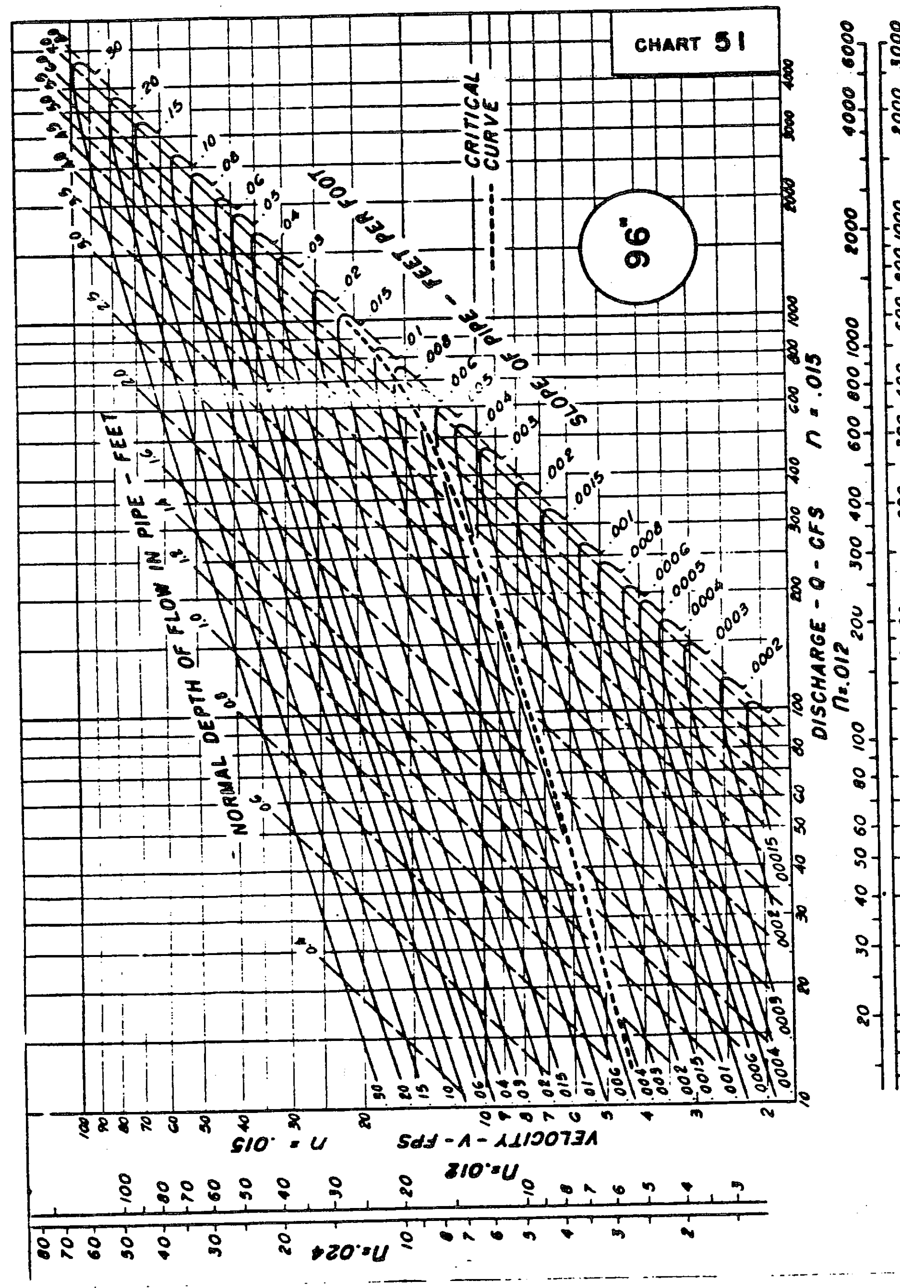


METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 56

Sta.
Loc.
Type Str.
By
Date
Q_{10} Vn
Q_{25} Vn
Q_{50} Vn
So d_{design}
$FL_{Inf.}$
$FL_{Off.}$
Entrance Type
HW Depth
TW Depth
Length
Depth @ 15% Q_d
Vel. @ 15% Q_d
pH Dry Weather Flow
(If Applicable)

PIPE FLOW CHART 96-INCH DIAMETER



METRIC CONVERSION: 1 in = 25.4 mm
1 ft = 0.3048 m

FIGURE 57

APPENDIX A - BLANK WORKSHEETS

SCS TR-55 Worksheets:

- Runoff Curve Number
- Time of Concentration
- Graphical Peak Discharge

Closed Drainage Design Sheet (Form SS-1)

Culvert Design Sheets:

- Site Data and Rainfall
- Culvert Hydraulics

SCS TR-55 WORKSHEETS

RUNOFF CURVE NUMBER AND RUNOFF — WORKSHEET 1

Project________________ By________ Date____________

Location________________ Checked________ Date____________

Circle one: Present Developed ________________________

1. Runoff curve number (RCN)

Soil name and hydrologic group	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	RCN			Area ☐ acres ☐ mi^2 ☐ %	Product of RCN x area
		Table B	Fig. C	Fig. D		
	Totals =					

RCN (weighted) = total product / total area = ________ = ________; Use RCN = ☐

2. Runoff

Frequency------------------------------------yr.
Rainfall, P (24-hour)--------------------------in.
Runoff, Q---------------------------------------in.
(Use P and RCN with Table E)

Storm #1	Storm #2	Storm #3

Adapted from USDA Soil Conservation Service

TIME OF CONCENTRATION (T_C) OR TRAVEL TIME (T_T) WORKSHEET 2

Project__________________________ By__________ Date __________

Location__________________________ Checked__________ Date __________

Circle one: Present Developed ____________________

Circle one: T_C T_t through subarea ____________________

NOTES: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

Sheet Flow (Applicable to T_C only)		Segment ID			
1.	Surface description (Table F)				
2.	Manning's roughness coeff., n (Table F)				
3.	Flow length, L (total L ≤ 300 ft.)	ft			
4.	Two-yr. 24-hour rainfall, P_2	in			
5.	Land slope, s	ft/ft			
6.	$T_t = \frac{0.007(nL)^{0.8}}{P_2^{0.5} s^{0.4}}$ Compute T_t	hr		+	=

Shallow concentrated flow		Segment ID			
7.	Surface description (paved or unpaved)				
8.	Flow length, L	ft			
9.	Watercourse slope, s	ft/ft			
10.	Average velocity, V (Figure G)	ft/s			
11.	$T_t = L/(3600V)$ Compute T_t	hr		+	=

Channel flow		Segment ID			
12.	Cross sectional flow area, a	ft^2			
13.	Wetted perimeter, P_W	ft			
14.	Hydraulic radius, $r = a/P_w$ Compute r	ft			
15.	Channel slope, s	ft/ft			
16.	Manning's roughness coeff., n				
17.	$V = \frac{1.49 r^{2/3} s^{1/2}}{n}$ Compute V	ft/s			
18.	Flow length, L	ft			
19.	$T_t = L/(3600V)$ Compute T_t	hr		+	=
20.	Watershed or subarea T_C or T_t (add T_t in steps 6, 11, and 19)	hr			

GRAPHICAL PEAK DISCHARGE METHOD

WORKSHEET 3

Project______________________________ By__________ Date___________

Location_____________________________ Checked_________ Date___________

Circle one: Present Developed ____________________________________

1. Data:

Drainage area	A_m =	______________	mi^2 (acres/640)
Runoff curve number	RCN =	______________	(From Worksheet 1)
Time of concentration	T_c =	______________	hr (From Worksheet 2)
Rainfall distribution type	=	______________	(II, III)
Pond and swamp areas spread throughout watershed	=	______________	percent of A_m (____ acres of mi^2 covered)

		Storm #1	Storm 2	Storm #3
2. Frequency	yr			
3. Rainfall, P (24-hour)	in			
4. Initial abstraction, I (Use RCN with Table H)	in			
5. Compute I_a/P				
6. Unit peak discharge, q_u (Use T_c and I_a/P with Figure J-1 or J-2)	csm/in			
7. Runoff, Q (From Worksheet 1)	in			
8. Pond and swamp adjustment factor, F_p (Use percent pond and swamp area with Table I. Factor is 1.0 for zero percent pond and swamp area.)				
9. Peak discharge, q_p (Where $q_p = q_u A_m Q F_p$)	cfs			

FORM SS-1

Calculations for runoff determination									Comp. by			Date			Page	
Project No.			Location:						Chekd. by			Date			of	
From Pipe Inlet		To Pipe Outlet		Through Reach				(CA_d) (i) / 360 =		Q	Through Reach					
	Flowline		Flowline	A_d	C	CA_d	CA_d	t_c	i	Q	Pipe Size	Q_{full}	S_o	V_n	L	Flow
Location	Elevation	Location	Elevation	hectares		hectares	hectares	min	mm/hr	cms	n = ( )	cms	m/m	m/sec	m	min

PROJECT NO.__________________ DESIGN FREQUENCY______yr. DESIGNED BY____________________ DATE__________

PROJECT NAME____ CHECKED BY____________________ DATE__________

	CHECK MAPS USED	NUMBER OF TITLE	SCALE	CONVERSION FACTORS
_____	USGS Topog Quad	__________	1:24000	1 cm^2 = 0.0576 km^2 = 5.76 ha
_____	USGS Topog Quad	__________	1:25000	1 cm^2 = 0.0625 km^2 = 6.25 ha
_____	USGS Topog Quad	__________	1:62500	1 cm^2 = 0.3906 km^2 = 39.06 ha
_____	Forest Service Quad	__________	1:31680	1 cm^2 = 0.0999 km^2 = 9.99 ha
_____	National Forest	__________	1:126720	1 cm^2 = 1.6052 km^2 = 160.53 ha
_____	BLM or Soil Conservation Quad	__________	1:31680	1 cm^2 = 0.0999 km^2 = 9.99 ha
_____	BLM Grazing	__________	1:63360	1 cm^2 = 0.4007 km^2 = 40.07 ha
_____	USGS Army Topog	__________	1:250000	1 cm^2 = 6.25 km^2 = 625.00 ha
_____	County	__________	1:126720	1 cm^2 = 1.606 km^2 = 160.58 ha
_____	Other	__________	______	1 cm^2 = ______ km^2 = ______ ha

DESCRIPTION OF TERRAIN AND LAND FACTOR(S) USED: ______________________________

DESCRIPTION OF TOPOGRAPHY AND K-FACTOR(S) USED: ______________________________

RAINFALL INTENSITIES

I_2 = ________ in/hr
I_{100}/I_2 = ________
I_{100} = ________ in/hr
I_f = ________ in/hr
I_{25} = ________ in/hr
F.F. = ________
1 in = 2.54 cm

COMMENTS AND EXPLANATIONS: ______________________________

PROJECT ____________ STATION__________ CA. or CLASS___ _________ DESIGNED BY______________________ DATE___ ____

PROJECT NAME______________________________ CROSSING ANGLE__________CHECKED BY______________________ DATE________

HYDROLOGIC & CHANNEL INFORMATION

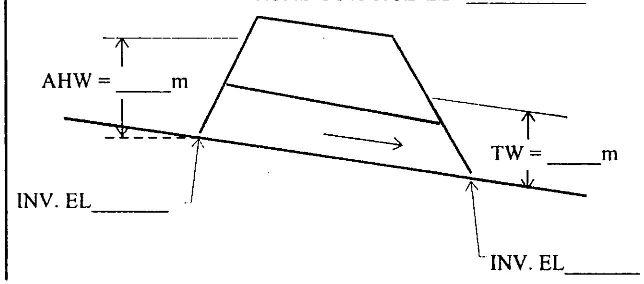
Q_{10}=

Q_{25}=

Q_{50}=

CROSS SECTIONAL DIAGRAM

ROAD SURFACE EL. ___________



Allowable outlet velocity = ______________m/s

Length of pipe = ____________m

Slope of pipe = ____________ m/m

Culvert Type	Size	Q_d	Inlet Control		Outlet Control							Controlling HW	Outlet V_o	Cost. Comments
					For h_o use>value of (1) or (2)				HW= $H+h_o-LS_o$					
			$\frac{HW}{D}$	HW	K_e	d_c	$d_c+D/2$	h_o(T.W.)	H	LS_o	HW			

APPENDIX B - COMPUTER PROGRAMS AND APPLICATIONS

APPLICATION	AVAILABLE SOFTWARE
Drainage Areas (Rational Method)	HYDRAIN (Vol. II) - HYDRO
Drainage Areas (SCS TR-55 Method)	HYDROCAD
Frequency Flood Elevations	HEC-RAS (River Analysis System)
Gutter Flow	HYDRAIN (Vol. III) - HYDRA
Grate Capacity	HYDRAIN (Vol. III) - HYDRA
Sizing Pipes (closed system)	HYDRAIN (Vol. III) - HYDRA
Hydraulic Gradeline Calculations	HYDRAIN (Vol. III) - HYDRA
Culvert Design	HYDRAIN (Vol. V) - HY8
Channel Linings	HYDRAIN (Vol. VI) - HYCHL
Energy Dissipator Design	HYDRAIN (Vol. V) - HY8
Detention Pond Design	HYDROCAD