DEPARTMENT OF PUBLIC WORKS

COUNTY OF HAWAII

Storm Drainage Standard

OCTOBER 1970

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APPROVED:

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INTRODUCTION:

These standards have been prepared to guide County engineers and personnel, engineers for subdivision developers, consultants employed by the Department of Public Works and other interested parties in the general features required for the design of storm drainage facilities in the County of Hawaii.

These standards are not intended to limit the initiative and resourcefulness of the engineer in developing drainage plans, or be viewed as maximum limits in design criteria. Higher criteria should be used where such is indicated. Lower criteria for specific basins may be permitted where substantiated by detailed studies.

The Storm Drainage Standards of the City and County of Honolulu dated March, 1969 has been used as the basis for these standards.

Part I
Hydrologic Criteria

Hydrologic Criteria

1. RECURRENCE INTERVAL

A For drainage areas of 100 acres or less, Tm (recurrence interval) = 10 years, unless otherwise specified.

B For drainage areas of 100 acres or less with sump, or tailwater effect and for the design of roadway culverts and bridges utilizing static head at entrance, Tm (recurrence interval) = 50 years.

C For drainage areas greater than 100 acres and all streams, design curves based upon maximum recorded flood peaks.

2. RUNOFF QUANTITY

A For drainage areas of 100 acres or less, the rational method shall be used.

B For drainage areas greater than 100 acres and all streams, refer to Plate 6 on page 19 titled "Design Curves for Peak Discharge vs. Drainage Area". The rational method and flood frequency analysis may be used as a check.

3. RATIONAL METHOD

The formula Q = CIA shall be used to determine quantities of flow rate, in which

Q = flow rate in cubic feet per second;
C = runoff coefficient;
I = rainfall intensity in inches per hour for a duration equal to the time of concentration; and
A = drainage area in acres.

A RUNOFF COEFFICIENT

The runoff coefficient shall be determined from Table 1 (page 14). It shall be based on the ultimate use of the drainage area. For distinctive composite drainage areas, a weighted value of runoff coefficient shall be used.

B TIME OF CONCENTRATION

1. Determine overland flow time from Plate 3 (page 17) generally for paved, bare soil and grassed areas.
2. Determine flow time over small agricultural areas with well-defined divides and drainage channels from Plate 5 (page 18)
   a. Use upper curve for well-forested areas, representing
      $$Tc = 0.0136\ K^{0.77}$$
   b. Use lower curve for areas with little or no cover, representing
      $$Tc = 0.0078\ K^{0.77}$$
3. In case of uncertainty, check the time of concentration by dividing the estimated longest route of runoff by the appropriate runoff velocity from Table 2 (page 14).

C RAINFALL INTENSITY

The design rainfall intensity of a drainage area shall be determined by the following procedure:

1. Select the appropriate 1-hour rainfall value from Plate 1 and 2 (pages 15 & 16) for the design recurrence interval.
2. Enter Plate 4 (page 17) with the one hour rainfall value and the required time of concentration. Obtain the design rainfall intensity in inches per hour.

4. FURTHER HYDROLOGIC STUDIES

Although hydrologic data in Hawaii have greatly increased since 1959, they are still insufficient for advanced hydrologic analysis to develop better criteria for the design of drainage facilities. For example, only scanty coordinated rainfall and runoff data are available that are useful for deriving unit hydrographs. In most cases, rainfall and runoff data are collected separately by different agencies. Thus, the rain gauge and stream flow gauge networks are designed and maintained often for uncoordinated purposes. The data so obtained are usually unsuitable for use in the analysis of rainfall runoff relationships because they do not adequately match geographically and are unsynchronized on many occasions.

The hydrologic condition on the Island of Hawaii is extremely heterogeneous because of irregular orographic characteristics and soil types. Each drainage basin often has its own unique hydrologic condition, even within one basin, the hydrologic condition varies radically. On the windward side of the island, for example, there is a radical change in rainfall amounts from sea level to the peak of Mauna Kea. The upper slopes have little or no runoff because of light rainfalls and extremely permeable soils and the lower slopes are subject to high intensity storms and runoff. Thus, a unit hydrograph developed from runoff data measured at a downstream gauging station cannot be reliably applied to the upstream areas. In view of the lack of hydrological homogeneity, a regional analysis of rainfall and runoff can only be considered approximate.

Since the hydrologic condition is heterogeneous, dense hydrologic networks should be developed and more hydrologic data must be collected. As the drainage basins show individual characteristics, further analyses of the accumulated data must be made for all individual basins and at various places inside the basins. The results of such analyses would be most useful and reliable for the design of drainage facilities at a given place in a given drainage basin.

Extensive hydrologic analysis of individual drainage basins would be too costly and time consuming, therefore, an approximate regional analysis such as the type of envelope curve shown in Plate 6, must be further improved and refined. When more peak discharges of record are measured, a frequency analysis will be incorporated in it to determine envelope curves for various recurrence intervals. Also, synthetic peak discharges may be computed from available unit hydrograph information and may be used to develop peak discharge frequency envelope curves separately for a number of regions on the Island of Hawaii.

Part II
Design Standards

Design Standard

1. GENERAL CONDITIONS

The design and capacity of a drainage system shall be predicated on the following conditions:

A On the basis of the runoff resulting from the selected design storm, the system shall dispose of surface runoff and subsurface water without damage to street facilities, structures or ground and cause no serious interruption of normal vehicular traffic.

B Runoff exceeding the design storm must be disposed of with the least amount of interruption to normal traffic and minimum amount of damage to surrounding property.

C System must have maximum reliability of operation with minimum maintenance and upkeep requirements.

D System must be adaptable to future expansion, if necessary, with minimum additional cost.

E Where sump conditions exist; a safety measure such as an overflow swale shall be provided to prevent flooding of adjacent lots in the event the design capacity of the closed conduit is exceeded. Floor levels of homes adjoining sumps shall be a minimum of 3 feet above low point on roadway.

F Floor levels of homes abutting streams and open channels shall be a minimum of 3 feet above freeboard elevation computed at design flow.

G In general, natural gullies, waterways, streams and tributaries shall not be replaced with a closed system except at roadway crossings.

H Roadway culverts and bridges shall be designed to pass the design flow under open channel hydraulic analysis with a minimum freeboard as specified in the attached freeboard chart. Multiple span road crossings shall have minimum clear spans of 30-feet, unless otherwise permitted by the Chief Engineer. Where possible, the roadway shall be designed to form a sag vertical curve with a low point at the waterway crossing with minimum grades to confine and control overflow at the crossing. Roadway culverts and bridges shall be designed to include only deck and roadway. Fill material shall not be used to meet roadway elevations above the deck.

I Outlets for enclosed drains emptying into open channels shall be designed to point downstream at an angle of 45°.

J Subsurface drains shall be installed wherever recommended by the Design Engineer, or Chief Engineer, where ground water is encountered, or may be present during wet weather.

K Lots abutting streams and open channels with a drainage area greater than 100 acres shall be graded to drain towards the waterway.

2. DESIGN COMPUTATIONS

The following data shall be submitted to the Chief Engineer by the Design Engineer:

A HYDRAULIC DESIGN DATA.

1. Computations for runoff, conduit and channel sizes, slopes, losses, hydraulic gradient and other hydraulic characteristics and information pertinent to the system. Computations shall be properly arranged and presented in such a manner that they may be readily checked.
2. The following data shall be shown on the construction plans.
   a. Design flow (Q), watershed area (A), roughness coefficient (n), and velocity (v), for all conduits and channels.
   b. Hydraulic grade lines.
   c. Water surface elevation at each manhole and catch basin.
   d. Building setback lines, where required.

B STRUCTURAL DESIGN DATA.

1. Structural design computations for all drainage structures other than pipes used within the limits of current loading tables and structures shown in the "Standard Details" of the County of Hawaii Department of Public Works.
2. Information pertinent to the design, such as boring data, soils report, etc.
3. Upon the completion of construction of major structures, submit pertinent data such as pile driving logs, pile tip elevations, etc.

3. CLOSED CONDUITS

A SIZES AND GRADIENTS

1. The size and gradient will be determined by the Manning formula:

$$Q = A \frac{1.486}{n} R^{2/3} S^{1/2}$$

Q = flow, in cfs　　R = hyd. rad. in ft.
A = area, in sq. ft.　　S = slope, in ft./ft.
n = roughness coefficient (Manning's)

A direct solution of this formula for pipes are found on pages 23 to 31.

2. The following limitations apply —
   a. Minimum size pipe: 18 inches inside diameter
   b. Minimum velocity: 2½ feet per second
   c. In general, pipe sizes shall not decrease in the direction of the flow.

B MATERIALS AND "n" VALUES

The following pipes are acceptable for storm drain construction together with the roughness coefficient to be used in the solution of the Manning Formula.

Materials	n
Concrete	.013
Cast iron	.013
*Corrugated metal pipe (CMP)	
Unpaved	.024
25% paved invert	.021
Lower 50% paved	.018
100% paved	.013

*Use of CMP shall be permitted only when specifically approved for an installation by the Chief Engineer in writing.

C LOADING

1. Reinforced Concrete Pipes: Reinforced concrete pipes shall be constructed to ASTM Specifications and currently classified as Class I, Class II, Class III, Class IV, and Class V pipes.
   a. Minimum pipe cover in roadways, driveways and other areas with vehicular traffic (based on the current, "Standard Specifications for Highway Bridges," AASHO) shall be as follows:

ASTM Classification	Pipe Diameter	Minimum Pipe Cover
Class III Pipe	Less than 48"	2'–6"
Class III Pipe	48" and above	2'–0"
Class IV Pipe	All sizes	2'–0"

Should there be a need for a pipe cover of less than 2'–0", the design engineer shall submit a structural design for review and approval. The decision to allow such design will be made by the Chief Engineer.

b. Minimum pipe cover in easement areas without vehicular traffic shall be 1'–0".

c. Maximum permissible depth will be determined from current loading tables in pipe handbooks for the respective pipes, using 110 lbs. per cu. ft. as the weight of earth.

d. All pipes shall be installed using a first class bedding trench condition. Proper foundations shall be provided for pipes. Pipes on unstable ground or fresh fill shall be supported by a method acceptable to the Chief Engineer.

e. Drain pipes installed along the longitudinal axis of the roadway shall be located in the pavement area between curbs.

1. **Other Closed Conduits.** There shall be no minimum cover or maximum permissible depth requirements for closed conduits other than pipes except that such structures shall be designed to support all loads that it shall be subjected to.

D MANHOLES AND INLETS

1. **Manholes:**

   a. Location. Manholes shall be located at all changes in pipe size and changes in alignment or grade and at all junction points.

   b. Spacing. Maximum manhole spacing shall be 250 feet for pipes 36 inches or less in diameter, or box drains with the smallest dimension less than 36 inches. Maximum manhole spacing for larger pipes and box drains shall be 500 feet.

   c. Special Details. Bottoms of manholes and inlets serving as manholes shall be shaped to channelize flow and sloped with slope of pipe as shown in "Design Details."

2. **Inlets (Catch Basins)**

   a. Location. Inlets shall be located at the upstream side of intersections, in sumps and where required by quantity of flow.

   b. Spacing. Maximum spacing shall be 500 feet.

   c. Types. For gutter grades up to 4%, standard 10-foot curb inlets with a depressed gutter shall be used. For grades 4% and greater, 10-foot long deflector inlets shall be used.

   d. Capacity. Inlet capacities as follows are acceptable:

Type	Gutter Grade	cfs
(1) Std. depressed gutter inlet	0.4%	6
	4.0%	4
	sump	10
(2) Deflector Inlet	4.0%	4.5
	12.0%	5.5
Greater than	12.0%	6 max.

   e. Gutter Flow. The gutter flow shall not exceed a width of 8 feet.

E PIPE SYSTEM ANALYSIS

Generally speaking, the pipe system shall be analyzed by sections, that is, outlet to manhole, manhole to manhole or manhole to inlet. The analysis shall start at the lowest point of flow and continued upstream. The design flow shall be used in determining whether the pipe will flow full or partially full. Full consideration of the tailwater, entrance and critical flow conditions shall be made.

1. **Pipe Flowing Full.** If the conditions show that the pipe section will flow full, the principles of flow of water in closed conduits shall be used. The water surface elevation of the upstream manhole is determined by adding the pipe friction and manhole losses to the water surface elevation of the downstream manhole or the beginning elevation as previously stated.

2. **Pipe Flowing Partially Full.** If the conditions show that the pipe section will flow partially-full, the principles of flow of water in open channels shall be used. The pipe partially-full condition may be determined from the *Pipe Flow Charts* on pages 23 to 31. The tailwater condition must also be considered in this determination.

3. **Manhole Losses**

   a. For junction conditions such as drop manholes, or where the outflow line deflects *more* than 10° with any inflow line, the hydraulic grade shall be determined by applying the *Entrance Control loss* and *C & D losses* (where applicable), or *A, B, C & D losses*, whichever is greater.

   b. For junction conditions where the outflow line deflects 10° or *less* with the inflow line, the hydraulic grade shall be determined by applying the *A, B, C & D losses*.

F HYDRAULIC GRADIENT COMPUTATIONS

The hydraulic gradient is (1) a line connecting points to which water will rise in manholes and inlets throughout the system during the design flow or (2) the level of flowing water at any point along an open channel.

It shall be determined starting at the downstream end of the proposed drainage system and proceeding upstream by adding the friction losses and manhole losses of the system.

The hydraulic gradient for the design flow shall be at least one foot below the top of the manhole cover, or 1 foot below the invert of catch basin inlet opening.

1. **Beginning Elevation**

   The elevation of the hydraulic gradient at the downstream end shall be selected according to the following conditions:

   a. Connection to existing drainage system — determined from the hydraulic gradient computations of the existing drain;

   b. Discharge into a stream — determined from the flow conditions of the stream;

   c. Submerged tailwater condition — begin at the tailwater elevation; and

   d. Freefall condition (conduit) — begin at the crown of the proposed drain.

2. **Friction Loss**

$$h_f = S_f(L), \text{ where:}$$

h_f = head loss due to friction

S_f = friction slope from Manning's formula, $\dfrac{(nv)^2}{2.208\ R^{4/3}}$

L = length of pipe or channel

   The friction loss shall be calculated for the condition of the design flow, that is, pipe flowing full or partially full.

3. **Manhole Losses**

   Manhole losses shall be as shown on the charts, "Head Losses in Manholes", (Plate 17 & 18, pg. 32). The losses shall be evaluated with pipes flowing full in the vicinity of the manholes; and therefore the velocity shall be for the pipe flowing full. The curves on the charts show the various losses:

   a. A curve — loss due to entrance and exit

   b. B curve — velocity head

      (1) Where the downstream velocity exceeds the upstream velocity, the head loss shall be difference in velocity heads.

      (2) Where the downstream velocity is less than the upstream velocity, the velocity head loss shall be zero.

   c. C curve — loss due to change in direction, taking the worst case for branches at a manhole.

   d. D curve — loss due to incoming volume.

G SPECIAL DETAILS

The following structures shall be installed where required:

1. Headwalls, aprons and cut-off walls at drain inlets and outlets.
2. Energy dissipators at outlets.
3. Debris — control structures.
4. Guard rails at headwalls and inlets, where they present a hazard to vehicular traffic or pedestrians.

4. OPEN CHANNELS

A CHANNEL SIZE

Use the Manning's Formula to determine the required waterway areas where uniform flow can be assumed.

$$Q = AV \text{ and } V = \frac{1.486\ R^{2/3}\ S^{1/2}}{n}$$

A = area of flow, in square feet

V = velocity, in feet per second

n = roughness coefficient (Manning's)

R = hydraulic radius, in feet

S = slope of the energy gradient, in feet per feet

The channel depth shall include design water depth and minimum freeboard allowances. Design water depth shall include rise in water surface caused by curves and junctions.

B CHANNEL RIGHT-OF-WAY

The channel width shall be sufficient to provide the required waterway area for the design storm as determined by these standards. The total right-of-way shall include a 15-foot wide maintenance road along both banks where the top width of channel exceeds 50 feet, and along one bank where the top width is 50 feet or less. The maintenance road along the channel shall be topped with 6 inches of crushed coral or base course and treated with bituminous material. In lieu of a maintenance road, for normally dry channels, access ramps or other suitable alternative measures to facilitate maintenance may be provided.

C PERMISSIBLE VELOCITIES AND "n" VALUES

Following is a list of "n" values for open channels and maximum permissible velocities. Maximum velocities shall be based upon design flow quantities.

Unlined Channels	Manning "n"	Maximum Velocity (fps)	Maximum Side Slopes
Rock, smooth and uniform	.035	15	½:1
Rock, jagged and irregular	.040	15	½:1
Ledge coral or lime stone	.025	10	1:1
Earth, no vegetation	.025	5	1½:1
Earth, grass, some weeds	.030	5	1½:1
Earth, dense weeds	.035	5	1½:1
Lined Channels			
Conc., trowel finish	.013	No limitation	
Conc., smooth wood forms	.015	No limitation	
Gunite	.020	20	
Grouted Rip-rap & CRM (Cement Rubble Masonry)	.025	20	
Asphaltic Concrete	.015	20	
Corrugated Metal Flumes Part-circle Sections	.021	25	

1. Maximum design velocity for channels cut in earth shall not exceed 5 feet per second. The velocity shall be determined by using the natural existing slope of the waterway without utilizing grade transition structures to control the maximum slope for a given unlined channel cross-section and design flow.
2. Velocities between 5 feet per second and 15 feet per second will be permitted in materials such as cemented gravel, hard pan, or mud rock depending upon its hardness and resistance to scouring. Borings and samples shall be submitted for evaluation before velocities exceeding 5 feet per second will be permitted.

D CHANNEL LINING

1. Earth channels shall be fully lined when velocities exceed 5 feet per second, unless otherwise permitted as noted in Section C-2 above.
2. All fill sections shall be lined. This lining shall be a complete lining including side slopes and invert with appropriate cut-off walls. If the invert of the channel is in a cut section the invert slab may be omitted and appropriate cut-off walls provided at the toe of the side slope lining.
3. Where linings are required or used, the linings shall be continuous. Lining of fill sections without continuing the lining out through cut sections in a channel will not be allowed unless adequate provisions are made to reduce the velocity from the lined section to meet the allowable velocity for the unlined section.
4. Total depth of channel lining will include design water depth and freeboard.
5. Attention shall be given to construction details of linings such as thickness, reinforcement, expansion and construction joints, cut-off walls, water-tight joints, placement of reinforcement, etc. Where the channel discharges into streams or other channels outside of the limits of a development, velocity reducing and transition structures shall be constructed to minimize erosion and overtopping of banks and subsequent flooding of downstream areas.
6. Where velocities are supercritical, rectangular channels shall be used, unless otherwise permitted by the Chief Engineer.
7. Earth channels shall be planted with vegetation, such as grass of a species not susceptible to rank growth.

E FREEBOARD

In designing open channels, freeboard must be provided to allow for surface roughness, wave action, air bulking, and splash and spray. These phenomena depend on the energy content of the flow. For water flowing at velocity **v** and depth **d**, the energy per foot of width per second is equal to $(wvd)(v^2/2g) = wdv^3/2g$, where **w** is the unit weight of water.

Thus, this kinetic energy can be converted to potential energy to lift the water surface when flow is stopped or changing direction as a function of depth and velocity of flow. The U.S. Bureau of Reclamation has developed an empirical expression to express a reasonable indication of desirable free board in terms of depth and velocity as follows:

$$\text{Freeboard in feet} = 2.0 + 0.025v\sqrt[3]{d}$$

where **v** is the velocity in feet per second and **d** is the depth of flow in feet. The velocity of flow can be computed by dividing the design discharge by the cross-sectional area of flow. For convenience of application, the above expression is shown graphically in Plate 7 (pg. no. 21). For discharges less than 30 cubic feet per second use channel size design of 100% greater capacity than the design capacity.

F JUNCTIONS

Junctions shall be designed to channel both flows as nearly parallel as possible to reduce velocity and momentum components, deposition of debris and erosion of banks.

G BENDS AND SUPERELEVATIONS

Changes in the direction of flow shall be made with smoothly curved channel walls allowing for superelevation in water surface. Curves will nearly always require additional depth. Trapezoidal channels for supercritical velocities are not recommended. Curve radii should be sufficiently great to limit superelevation of the water surface to one foot above computed depth of flow or 10% of water surface width, whichever is the least. The amount of superelevation for simple curves may be determined as follows:

1. **Trapezoidal Channels:**

   Subcritical velocity:

$$e = \frac{V^2(b + 2\,zd)}{(gR - 2zV^2)}$$

2. **Rectangular Channel:**

   Subcritical velocity:

$$e = \frac{V^2 b}{gR}$$

   Supercritical velocity:

$$e = \frac{2V^2 b}{gR}$$

   Supercritical velocity — compound curve:

$$e = \frac{V^2 b}{gR}$$

The compound curve is a simple curve of radius R preceeded and followed by a section of simple curve with radius of 2R, and length of $\frac{b}{\tan B}$, where $\sin B = \frac{\sqrt{gd_m}}{V}$.

Where: e = maximum difference in elevation of water surface between channel sides (ft)

z = Co-tangent of bank slope

d = normal depth (ft)

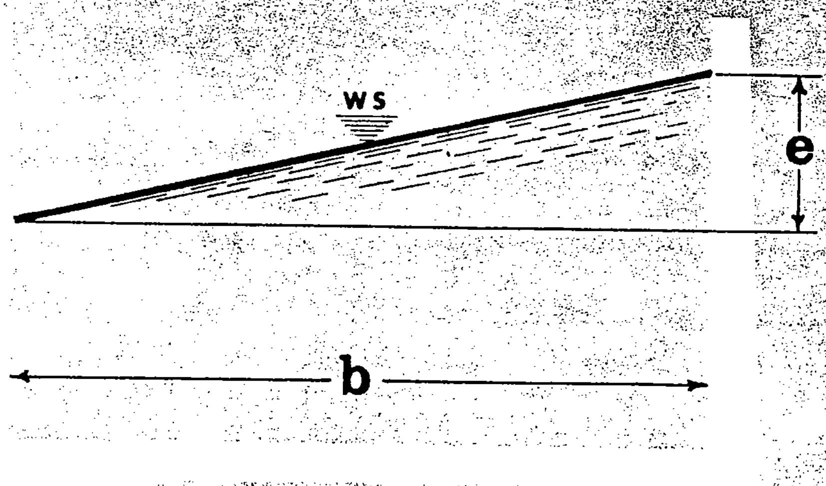
b = channel bottom width (ft)

R = radius of curve to centerline (ft)

g = acceleration due to gravity (fps^2)

V = normal velocity (fps)

dm = mean depth



Water Surface Superelevation Showing, "e"

H TRANSITIONS

1. The maximum angle between channel centerline and transition walls should be 12.5°.
2. Sharp angles in alignment of transition structures should be avoided.

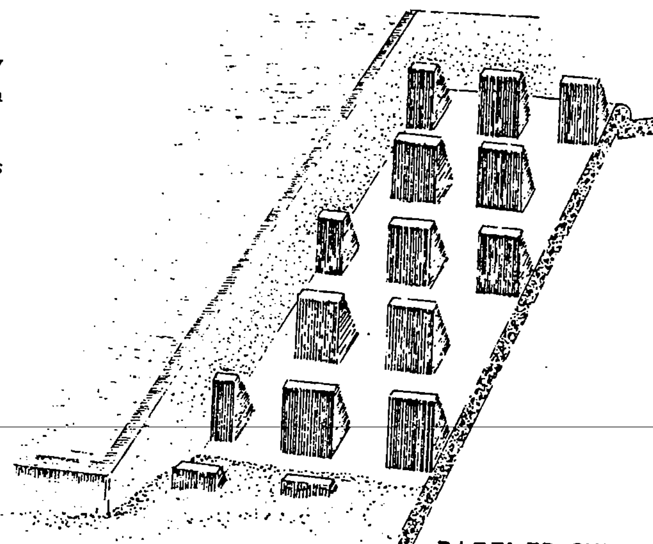
I DEBRIS BARRIERS

Debris barriers should be provided upstream of the intake to prevent clogging. Where required, boulder basins shall be provided upstream of the debris barrier.

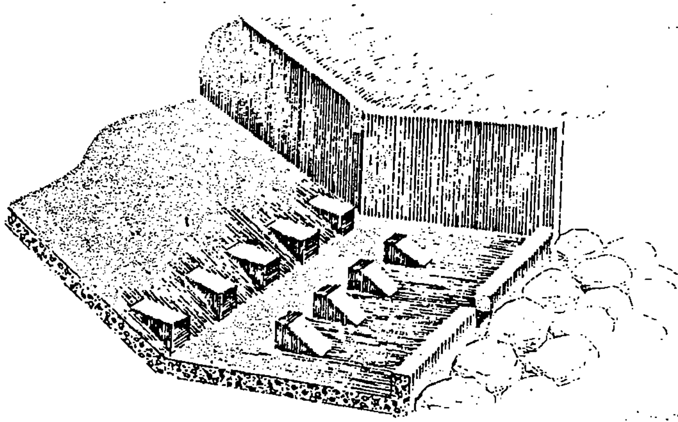
J ENERGY DISSIPATORS

Energy dissipators shall be used to dissipate energy where necessary, and to transition the flow from a lined channel to a normal flow in a unlined channel.

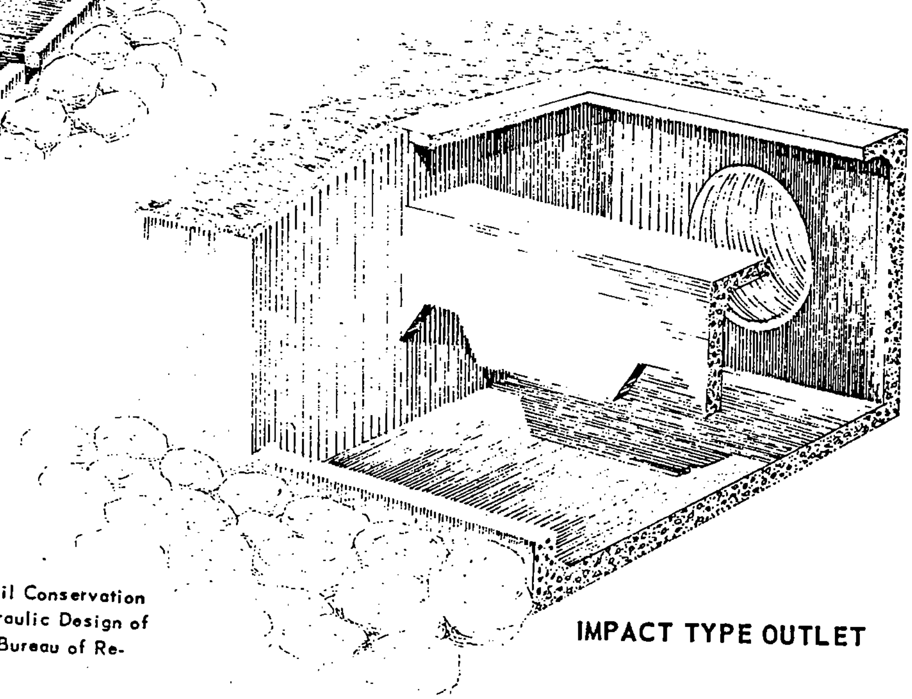
Energy dissipators may be any of the following types such as the SAF basin, baffled chute, dentated sills, buckets, impact, hydraulic jump, or other approved designs.



BAFFLED CHUTE



SAF BASIN



IMPACT TYPE OUTLET

Reference: "The SAF Stilling Basin" U. S. Soil Conservation Service Report SCS-TP-79, May 1949 and "Hydraulic Design of Stilling Basins and Energy Dissipators" U. S. Bureau of Reclamation, Engineering Monograph No. 25.

Design Charts

Table 1

GUIDE FOR THE DETERMINATION OF RUNOFF COEFFICIENTS FOR BUILT-UP AREAS*

WATERSHED CHARACTERISTICS	EXTREME	HIGH	MODERATE	LOW
INFILTRATION	NEGLIGIBLE 0.20	SLOW 0.14	MEDIUM 0.07	HIGH 0.0
RELIEF	STEEP (> 25%) 0.08	HILLY (15 - 25%) 0.06	ROLLING (5 - 15%) 0.03	FLAT (0-5%) 0.0
VEGETAL COVER	NONE 0.07	POOR (< 10%) 0.05	GOOD (10 - 50%) 0.03	HIGH (50 - 90%) 0.0
DEVELOPMENT TYPE	INDUSTRIAL & BUSINESS 0.55	HOTEL - APARTMENT 0.45	RESIDENTIAL 0.40	AGRICULTURAL 0.15

**NOTE*: *The design coefficient "c" must result from a total of the values for all four watershed characteristics of the site.*

Table 2

APPROXIMATE AVERAGE VELOCITIES OF RUNOFF FOR CALCULATING TIME OF CONCENTRATION

TYPE OF FLOW	VELOCITY IN FPS FOR SLOPES (in percent) INDICATED			
OVERLAND FLOW:	0–3%	4–7%	8–11%	12–15%
Woodlands	1.0	2.0	3.0	3.5
Pastures	1.5	3.0	4.0	4.5
Cultivated	2.0	4.0	5.0	6.0
Pavements	5.0	12.0	15.0	18.0
OPEN CHANNEL FLOW:				
Improved Channels	Determine Velocity by Manning's Formula			
Natural Channel* (not well defined)	1.0	3.0	5.0	8.0

**These values vary with the channel size and other conditions so that the ones given are the averages of a wide range. Whereever possible, more accurate determinations should be made for particular conditions by Manning's formula.*

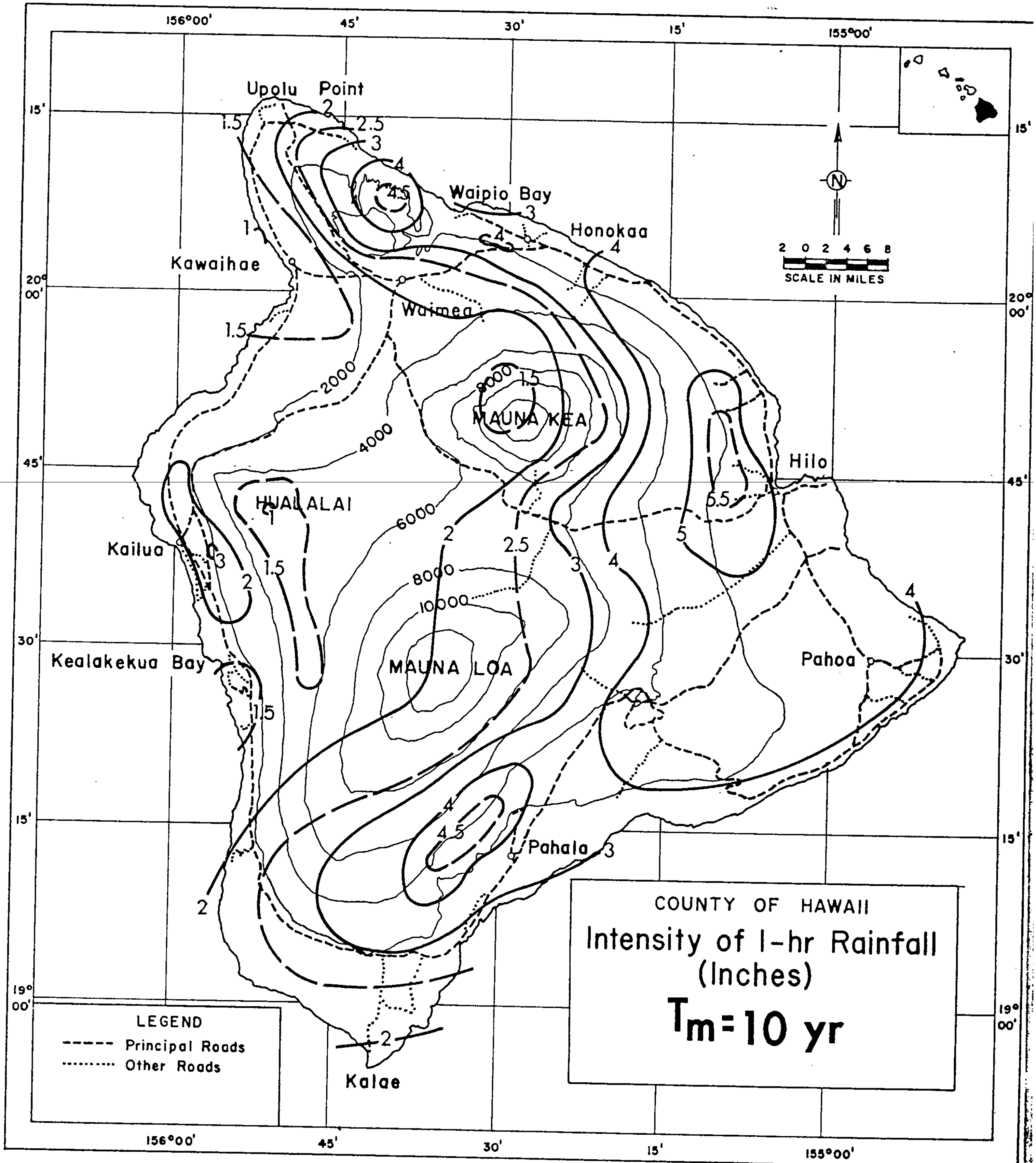

COUNTY OF HAWAII
Intensity of 1-hr Rainfall
(Inches)
Tm=10 yr
LEGEND
Principal Roads
Other Roads
SCALE IN MILES
2 0 2 4 6 8
N
156°00'
45'
30'
15'
155°00'
15'
20° 00'
45'
30'
15'
19° 00'
Upolu Point
Waipio Bay
Honokaa
Kawaihae
Waimea
MAUNA KEA
Hilo
HUALALAI
Kailua
MAUNA LOA
Kealakekua Bay
Pahoa
Pahala
Kalae
2000
4000
6000
8000
10000
1
1.5
2
2.5
3
4
4.5
5
5.5

Plate 1

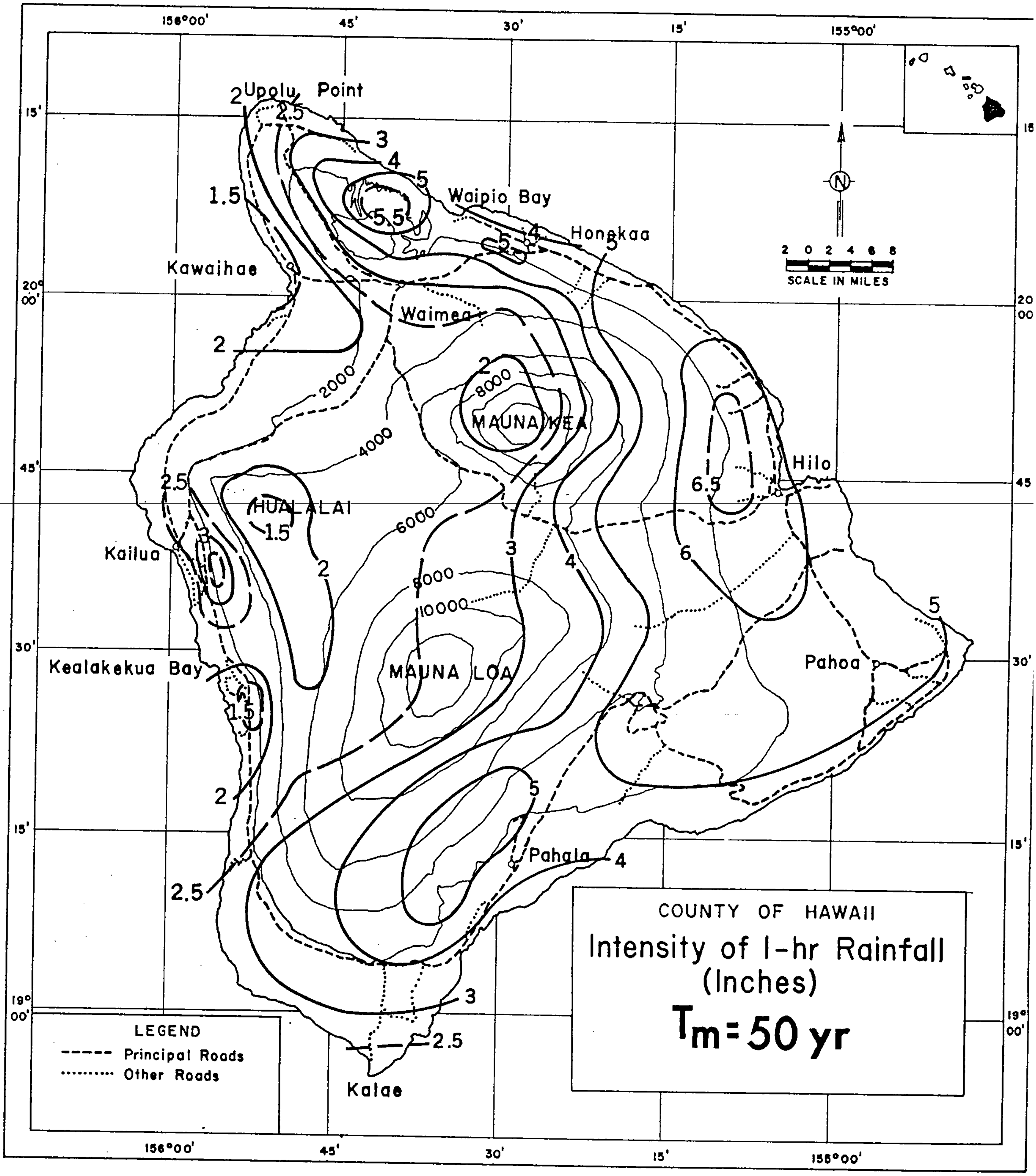

COUNTY OF HAWAII
Intensity of I-hr Rainfall (Inches)
Tm = 50 yr
LEGEND
Principal Roads
Other Roads
SCALE IN MILES
Upolu Point
Waipio Bay
Honokaa
Kawaihae
Waimea
MAUNA KEA
HUALALAI
Kailua
Kealakekua Bay
MAUNA LOA
Hilo
Pahoa
Pahala
Kalae
156°00'
45'
30'
15'
155°00'
20° 00'
19° 00'

Plate 2

Plate 3

Overland Flow Chart

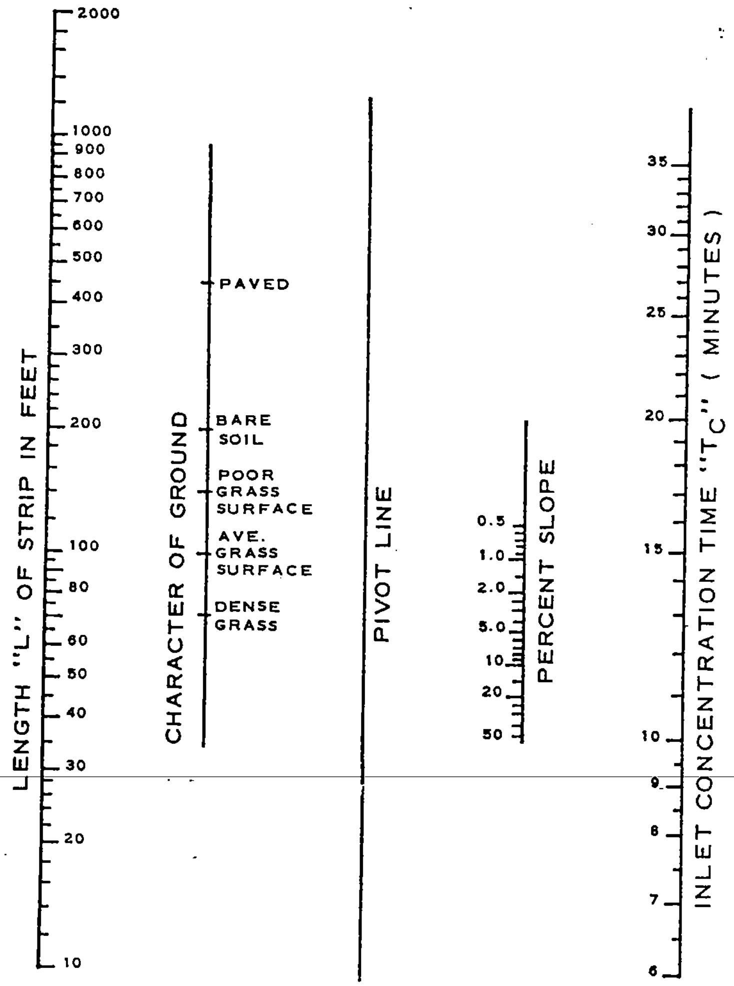
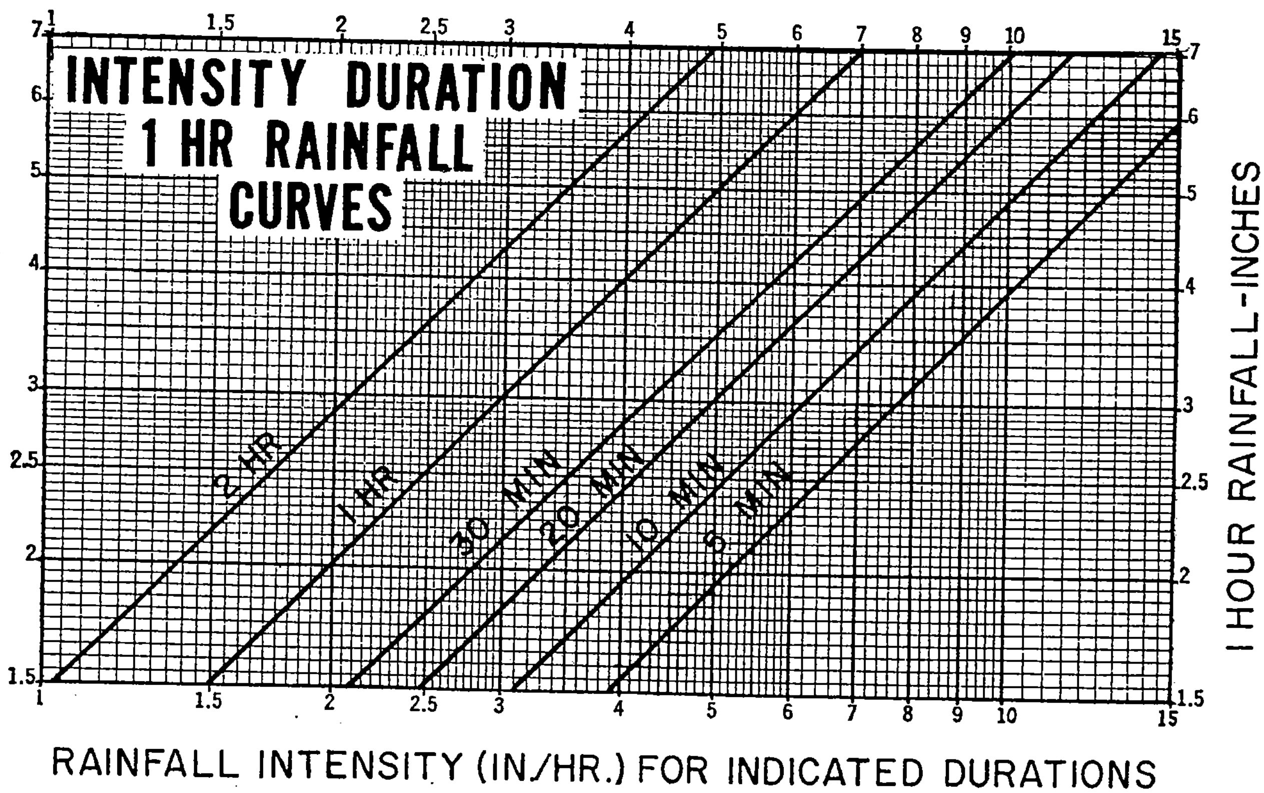
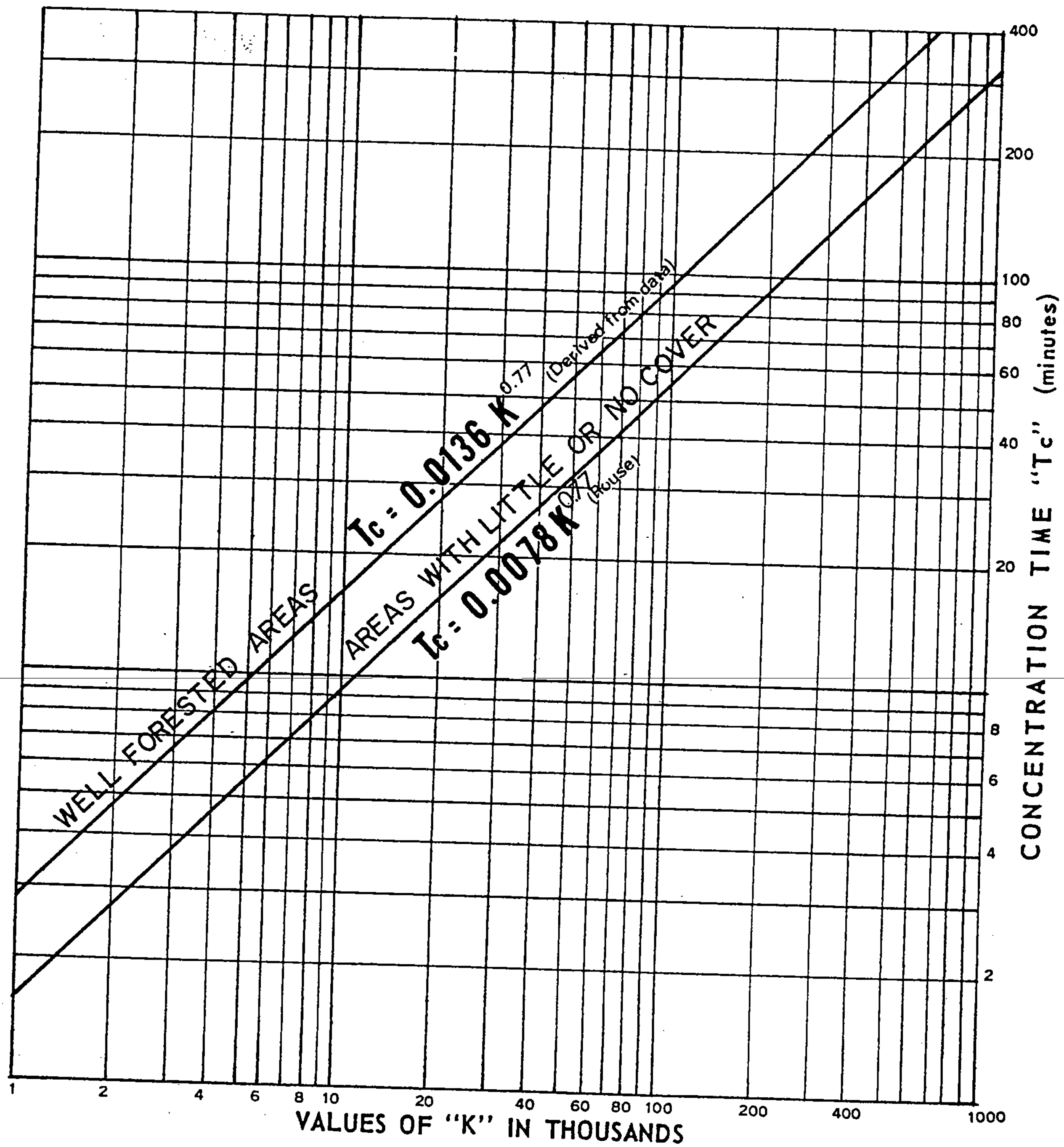


Plate 4





L = Maximum length of travel in feet
H = Difference in elevation between most remote point and outlet in feet.
S = Slope H/L

$$K = \frac{L}{\sqrt{S}} = \sqrt{\frac{L^3}{H}}$$

graph from Hunter Rouse "Engineering Hydraulics."

Plate 5

Time of Concentration

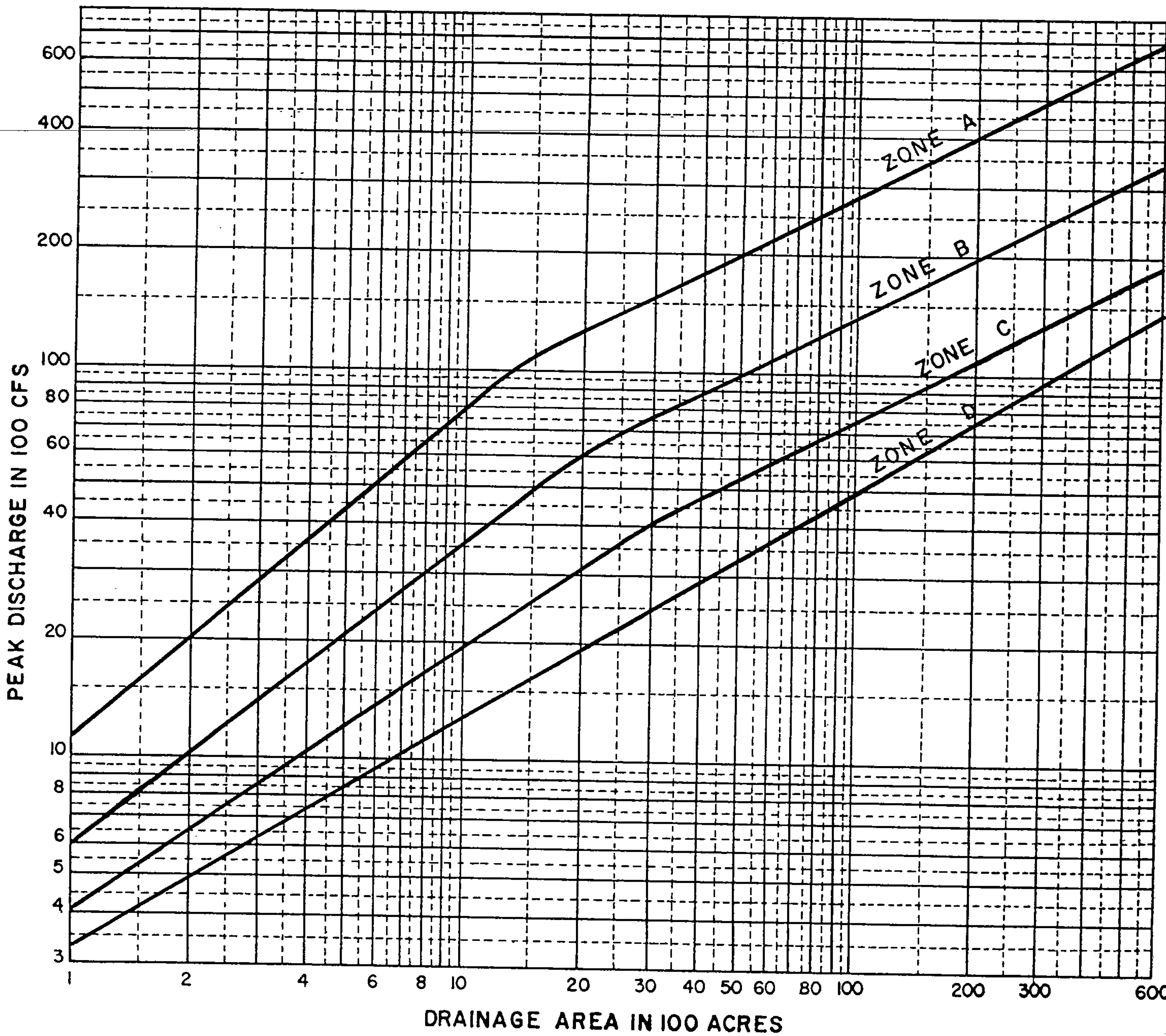
(OF SMALL AGRICULTURAL DRAINAGE BASIN)

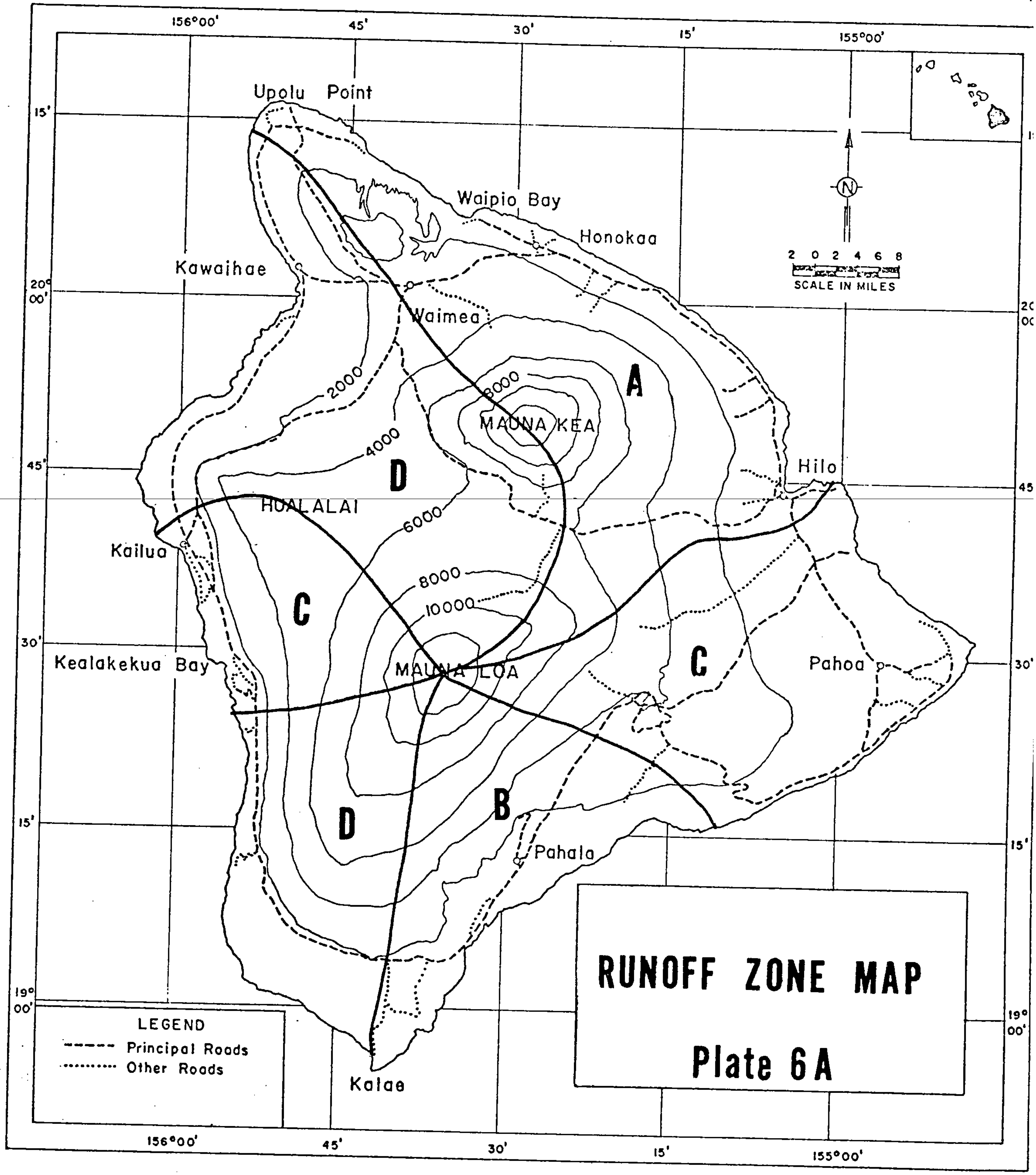
Plate 6

DESIGN CURVE FOR PEAK DISCHARGE VS. DRAINAGE AREA
(more than 100 acres)

CURVES ARE FOR STREAM CHANNELS AND DRAINAGE STRUCTURES

APPROXIMATE 100 YEAR RECURRENCE INTERVAL




156°00'
45'
30'
15'
155°00'
Upolu Point
Waipio Bay
Honokaa
Kawaihae
Waimea
2 0 2 4 6 8
SCALE IN MILES
2000
4000
6000
8000
10000
A
MAUNA KEA
D
HUALALAI
Hilo
Kailua
C
Kealakekua Bay
MAUNA LOA
C
Pahoa
B
D
Pahala
Kalae
LEGEND
Principal Roads
Other Roads
RUNOFF ZONE MAP
Plate 6A

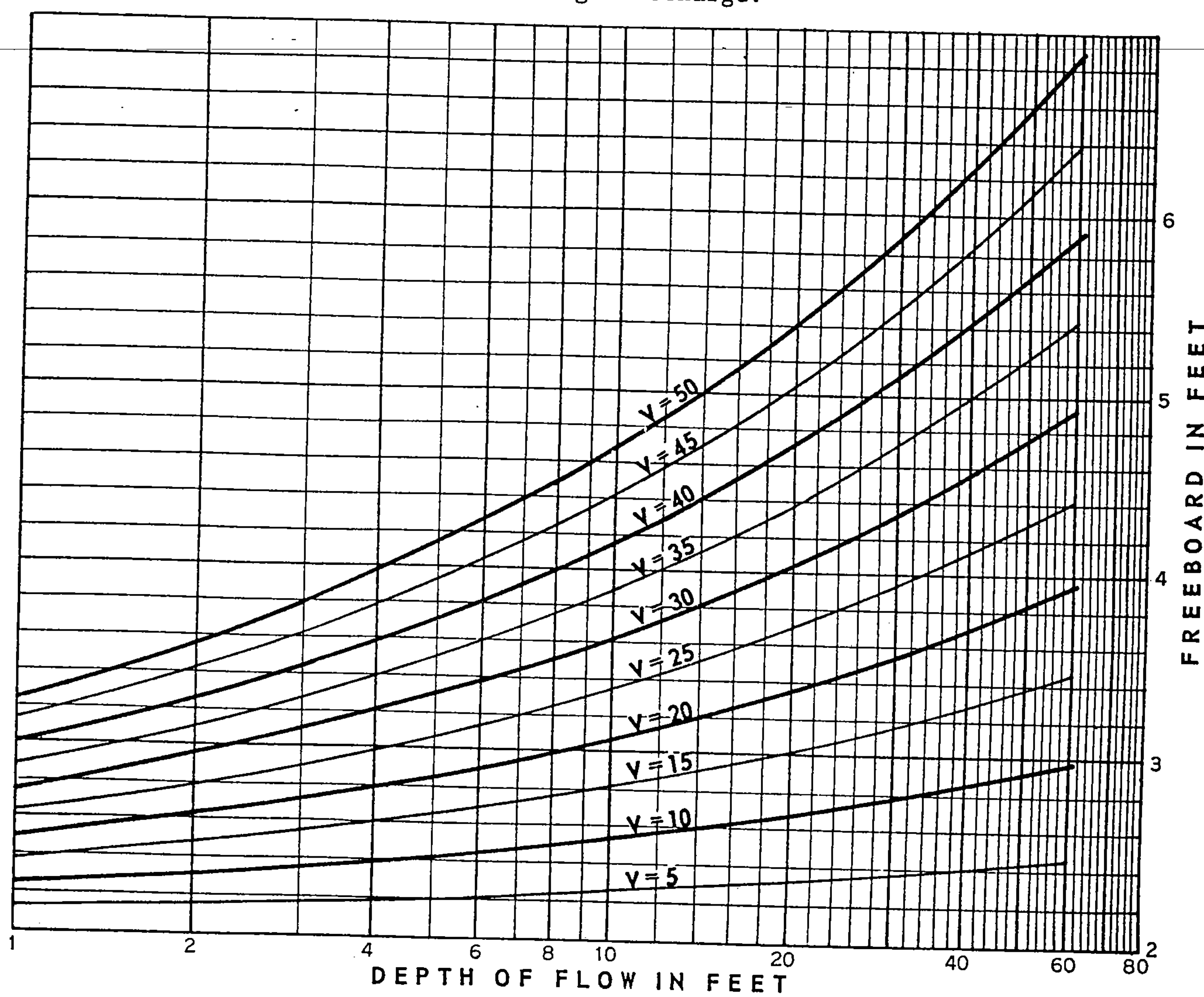
FREEBOARD ALLOWANCES Plate 7

FREEBOARD IN FEET :(For Q>30 cfs)

$$2.0 + 0.025\, V \sqrt[3]{d}$$

Where V = Velocity, in feet per second
d = Depth of flow, in feet

NOTE: For discharges less than 30 cfs, channel shall be designed for 100% greater capacity than the design discharge.



Pipe Flow Charts

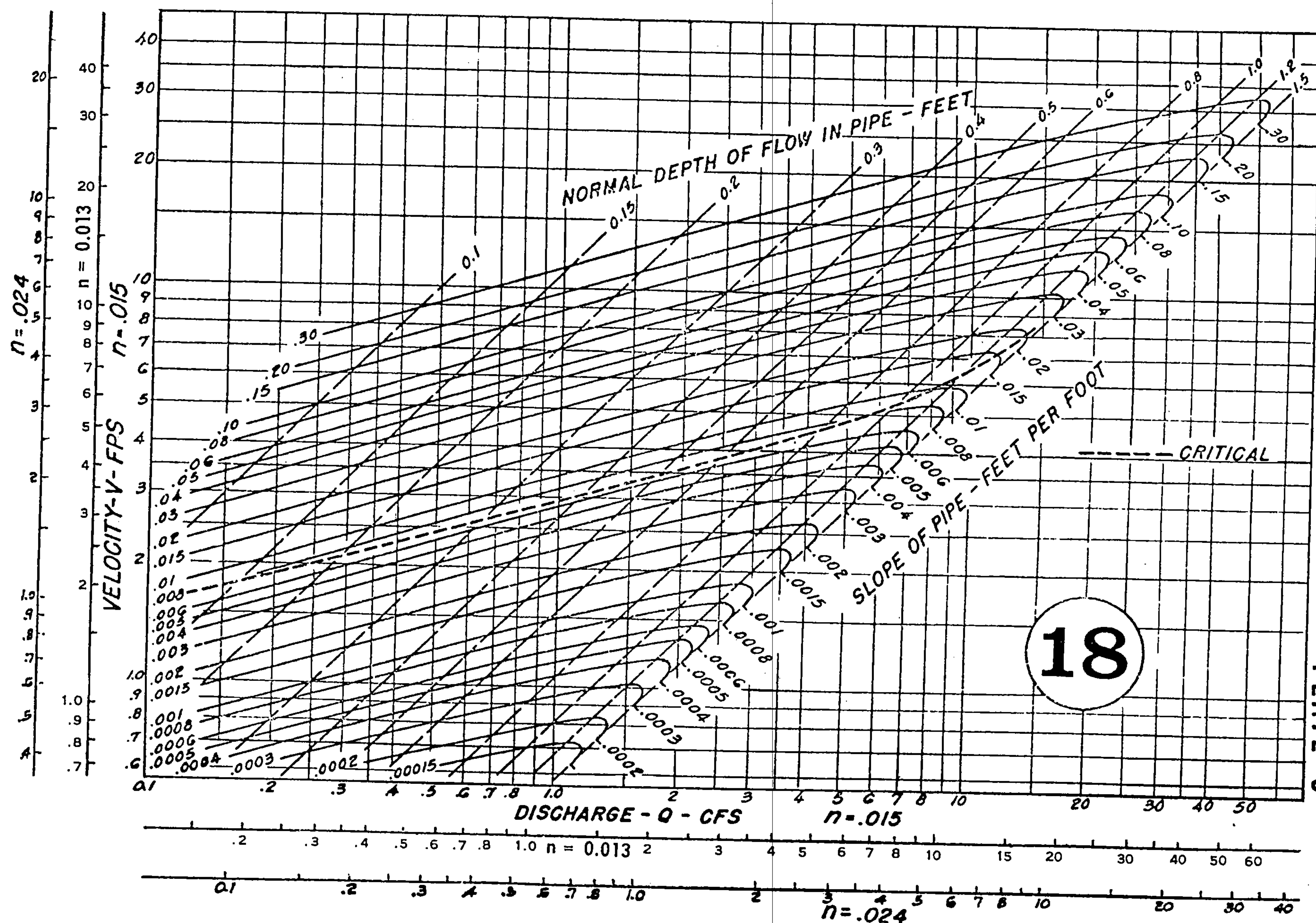
The following pipe flow charts have been derived by the *U. S. Public Roads Administration, Division Two, Washington, D. C.* These charts are designed to enable direct solution of the Manning formula for circular pipes flowing full and for uniform part-full flow in circular pipes. The "n" scales of 0.013 and 0.024 have been inserted to facilitate the use of these charts for storm drainage systems in Hawaii. The following examples will help to explain the use of the pipe flow charts.

EXAMPLES

A. Determine the depth and velocity of flow in a long 30-inch pipe, n 0.013, on a 0.5-percent slope (S_o = 0.005) discharging 25 cfs. Enter the 30-inch diameter chart at Q = 25 on n = 0.013 scale, follow up to intersection with line for slope S_o = 0.005, and read normal depth d_n = 1.8 feet and normal velocity V = 6.6 fps. To find critical depth, enter chart at Q = 25 on n = 0.013 scale, and read critical depth d_c = 1.6 feet at intersection with dotted critical curve. Also critical velocity V_c = 7.6 fps. (Note: Critical depth and velocity would be the same, regardless of pipe roughness.)

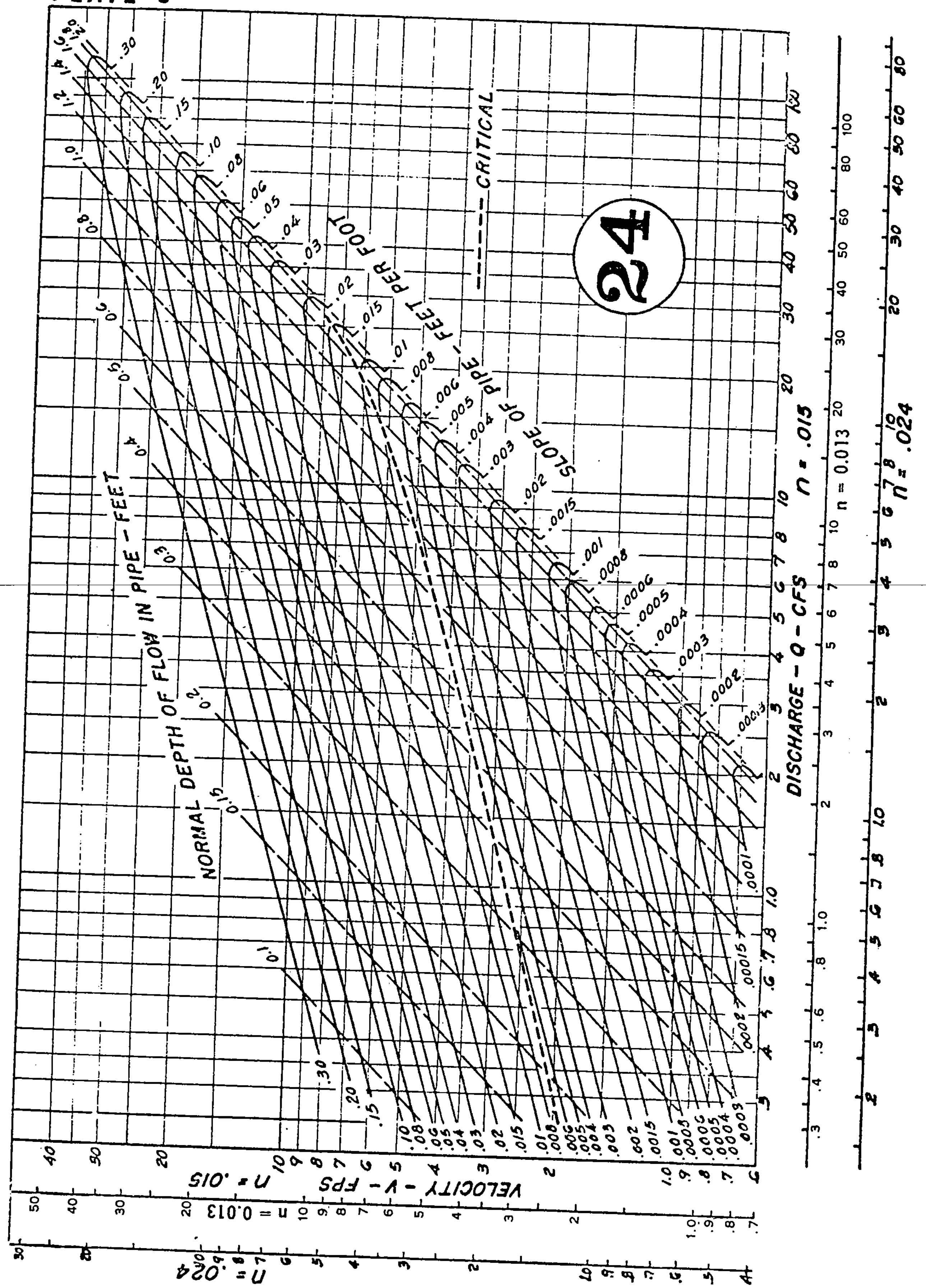
B. Determine friction slope for a 30-inch corrugated metal pipe, n = 0.024, on a slope S_o = 0.008 ft/ft with a discharge Q = 25 cfs. Enter the 30-inch diameter chart at Q = 25 on n = 0.024 scale. Note that this ordinate falls to the right of the 0.008 slope line, therefore, the pipe will flow full. Read friction slope S_f = 0.012 at the line for depth equal to pipe diameter.

(Note: $Q = 25 \times \frac{0.024}{0.015} = 40$ cfs on the Q-scale for n = 0.015.)

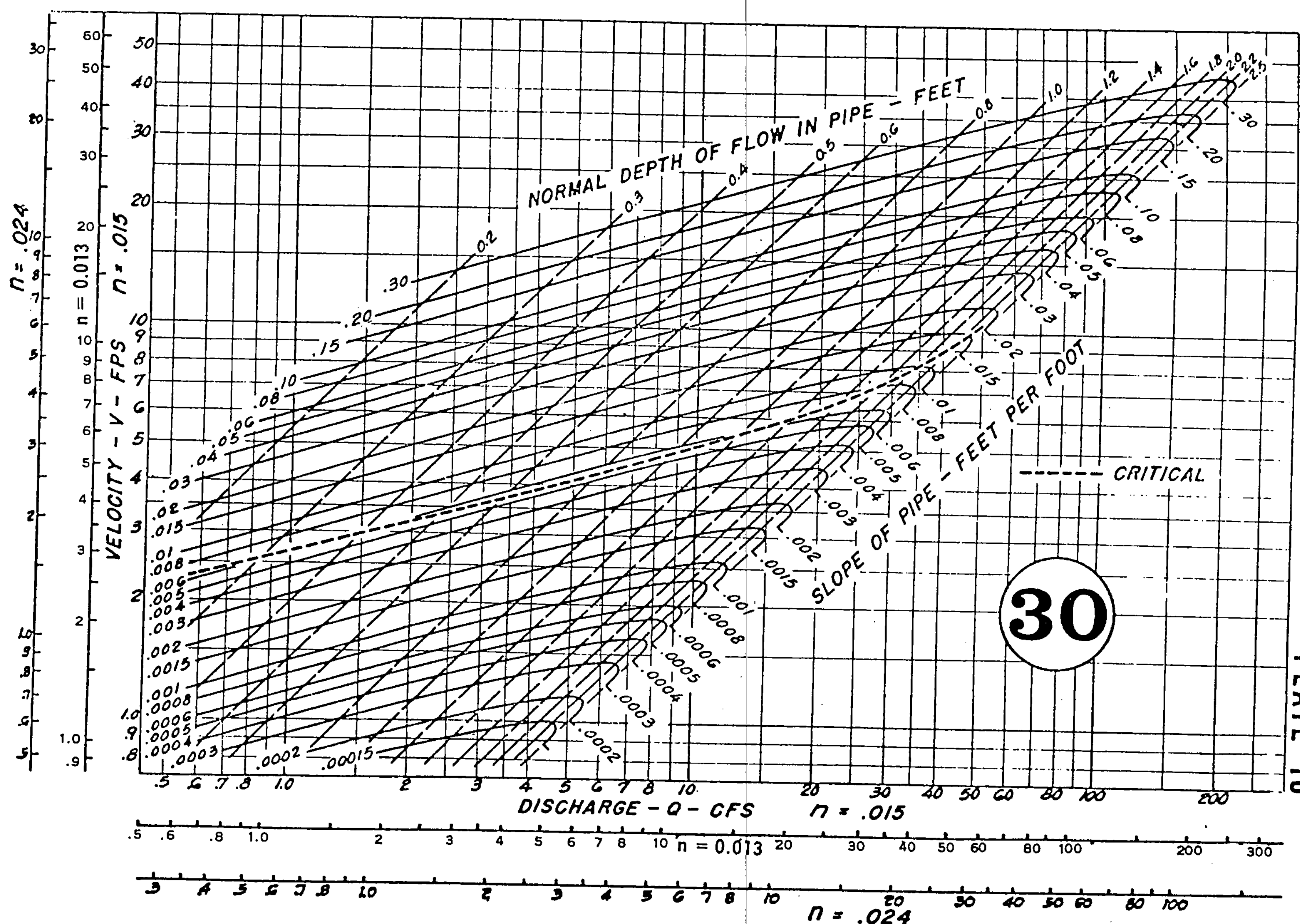


Pipe Flow Chart 18 inch Diameter

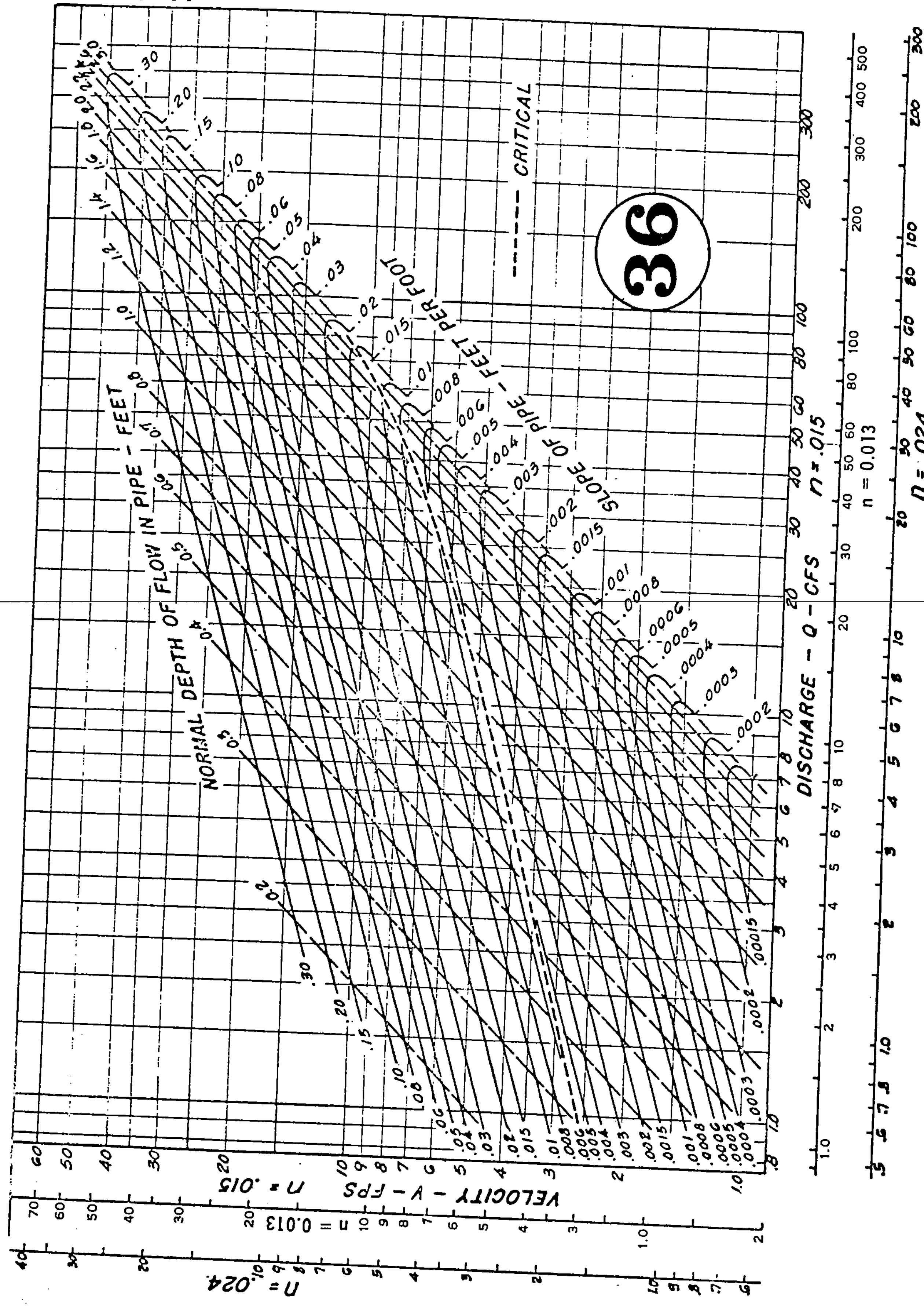
PLATE 9



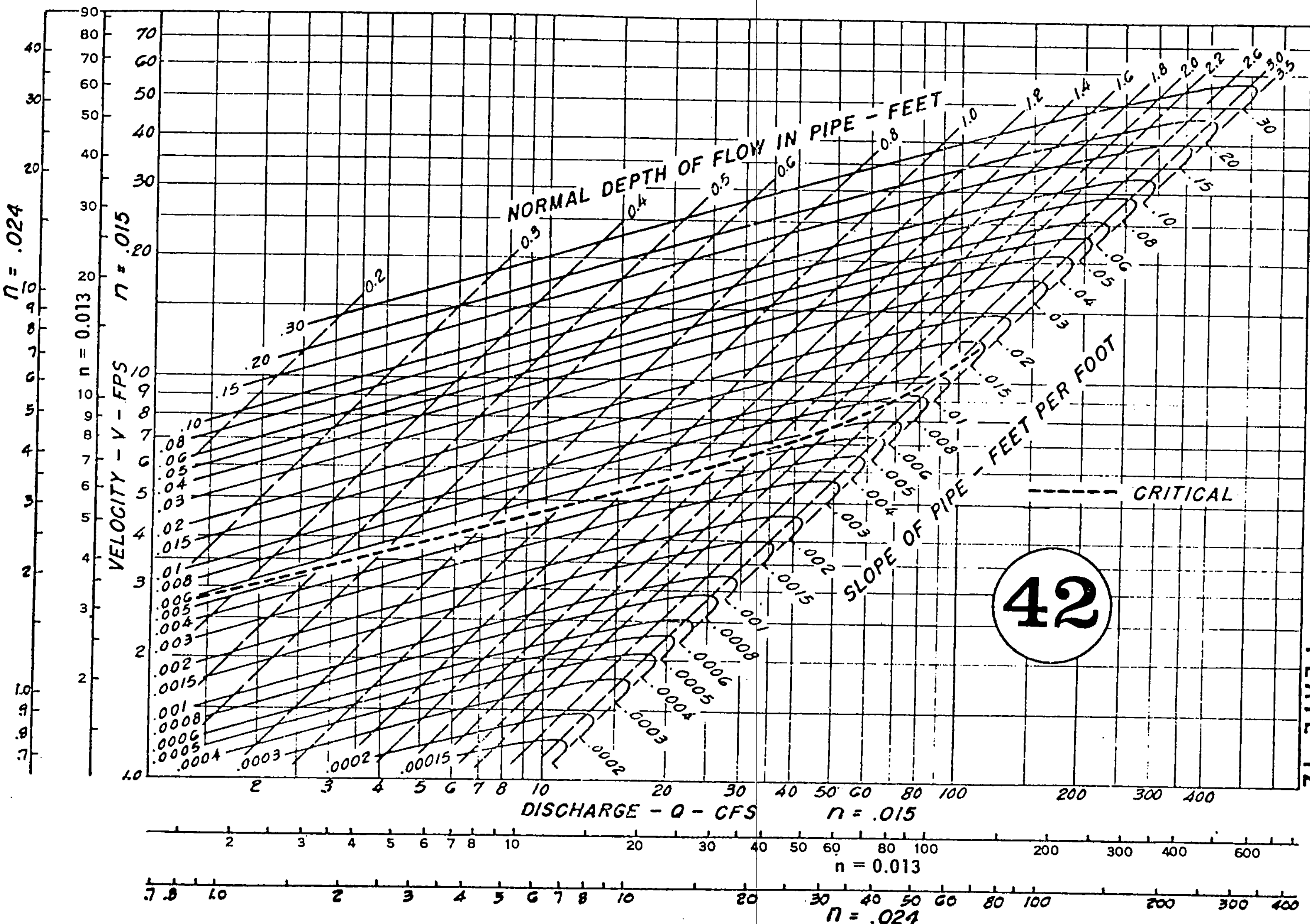
Pipe Flow Chart 24 inch Diameter



Pipe Flow Chart 30 inch Diameter

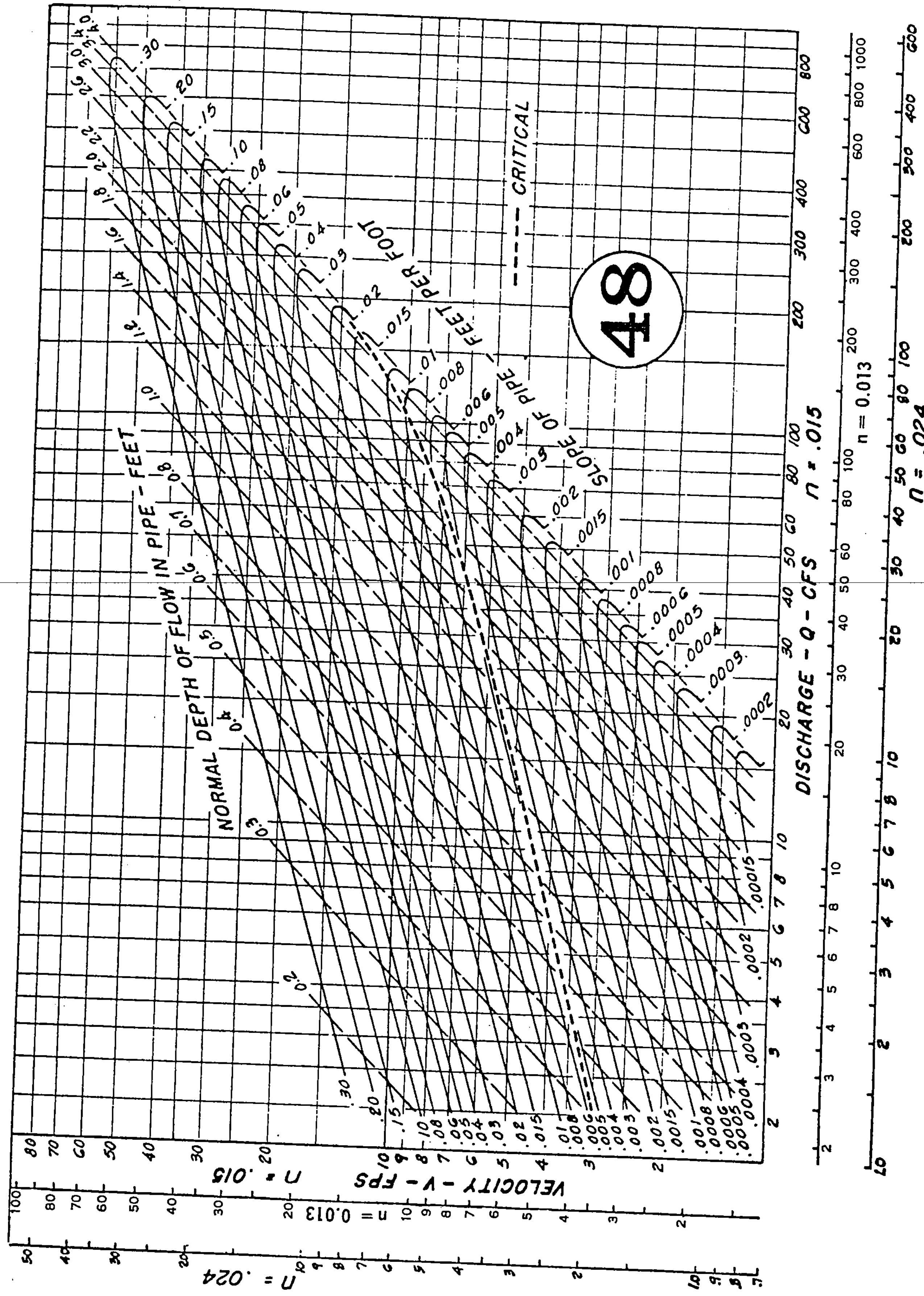


Pipe Flow Chart 36 inch Diameter



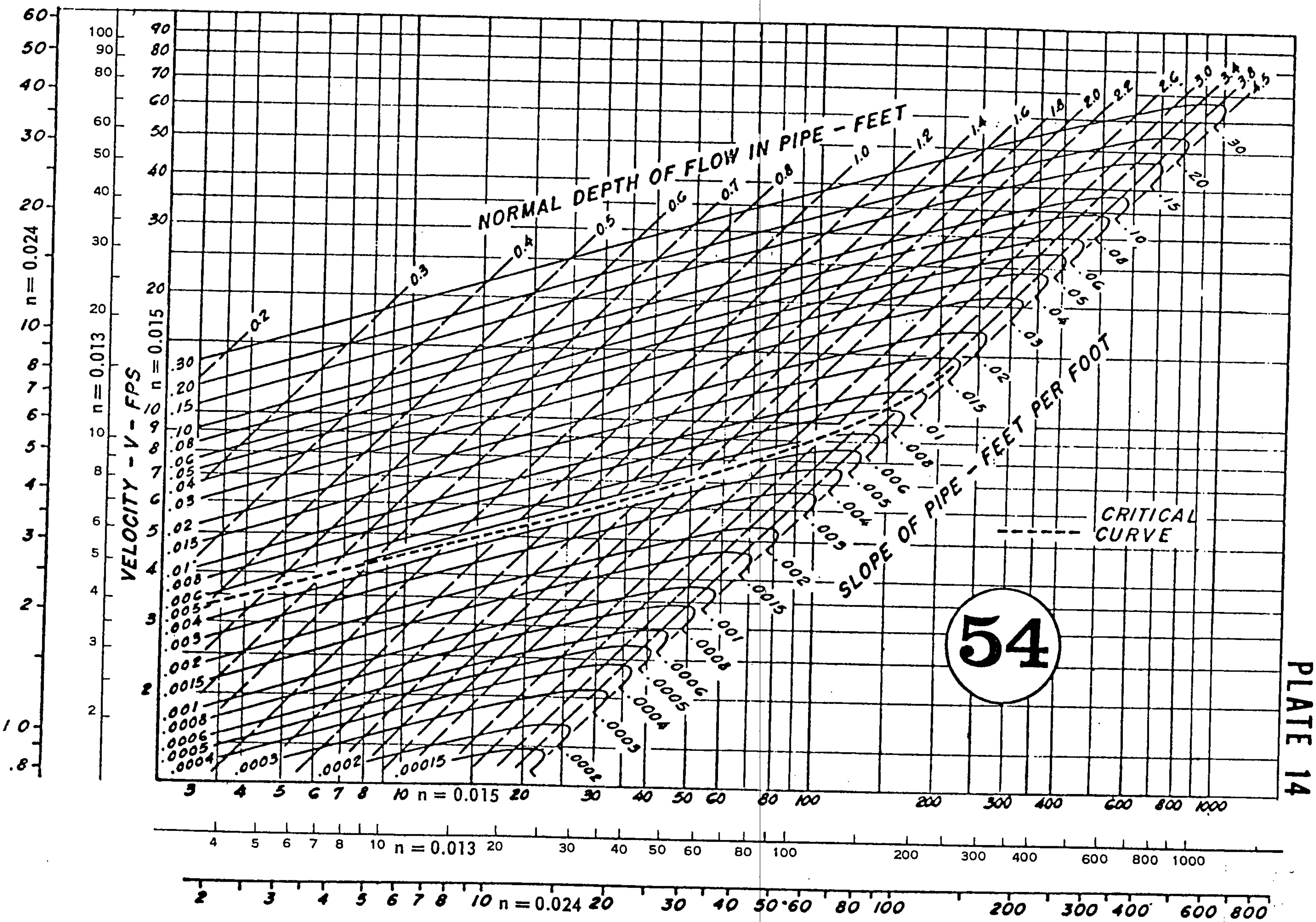
Pipe Flow Chart 42 inch Diameter

PLATE 13



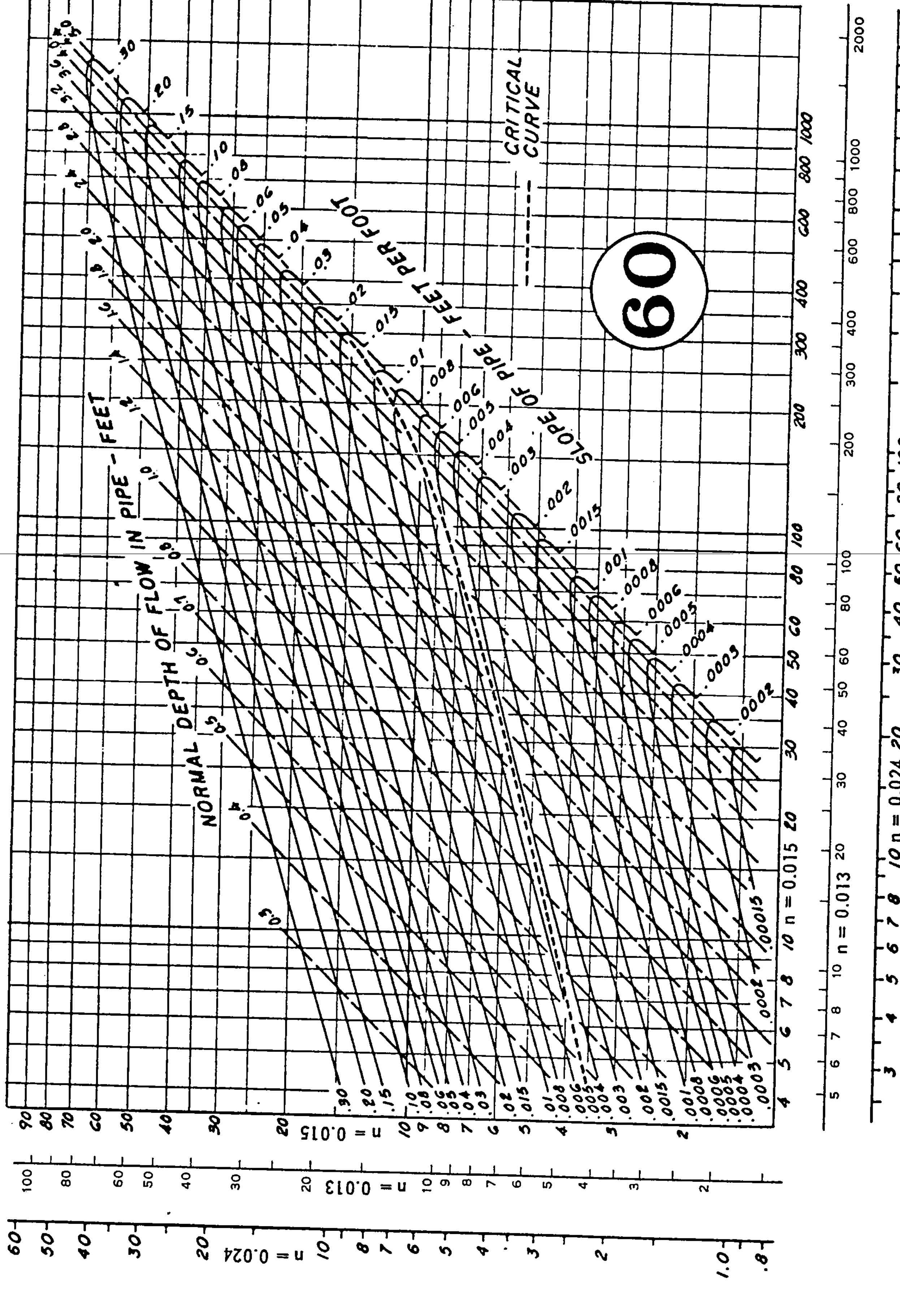
Pipe Flow Chart 48 inch Diameter

PLATE 14



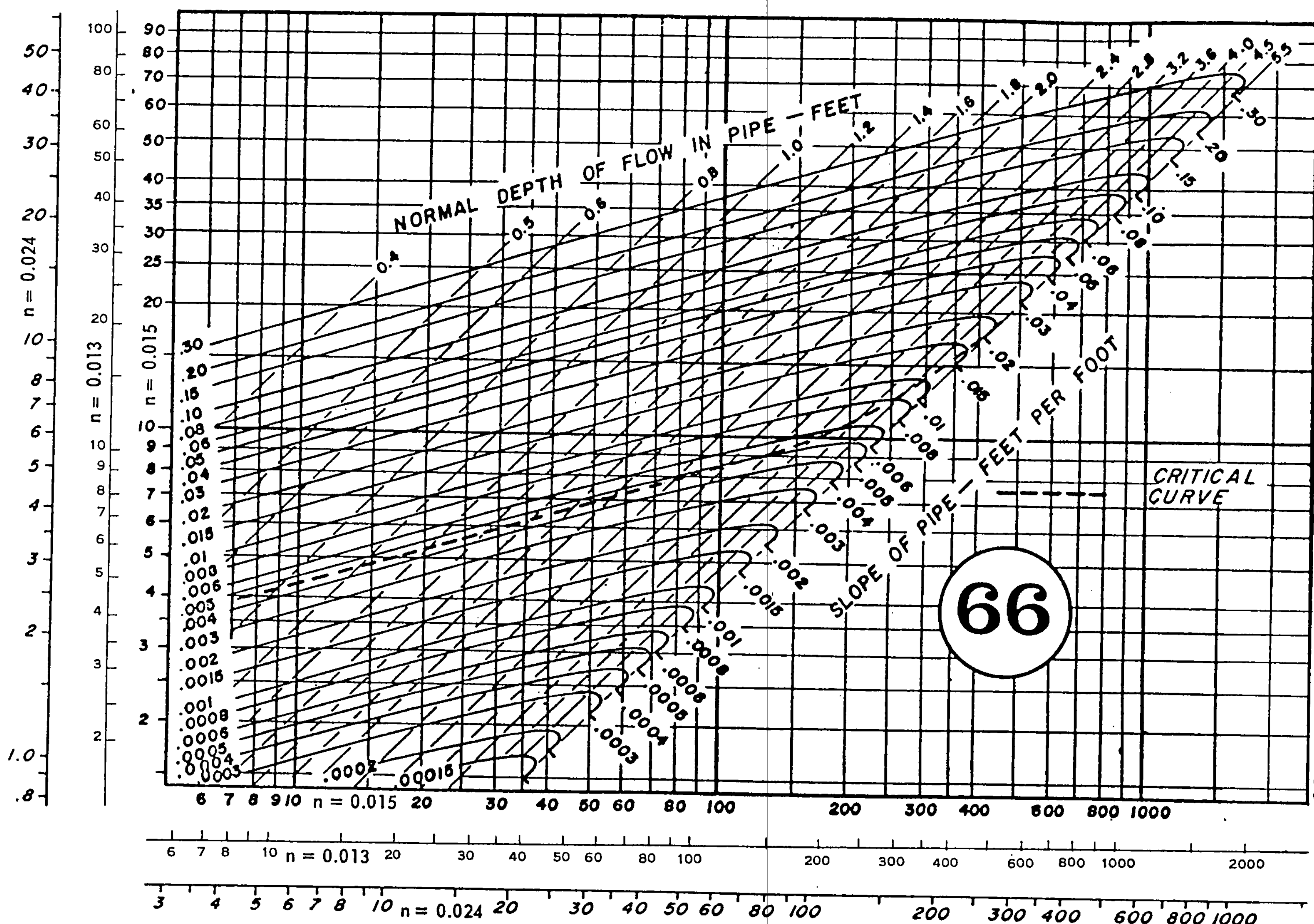
Pipe Flow Chart 54 inch Diameter

PLATE 15

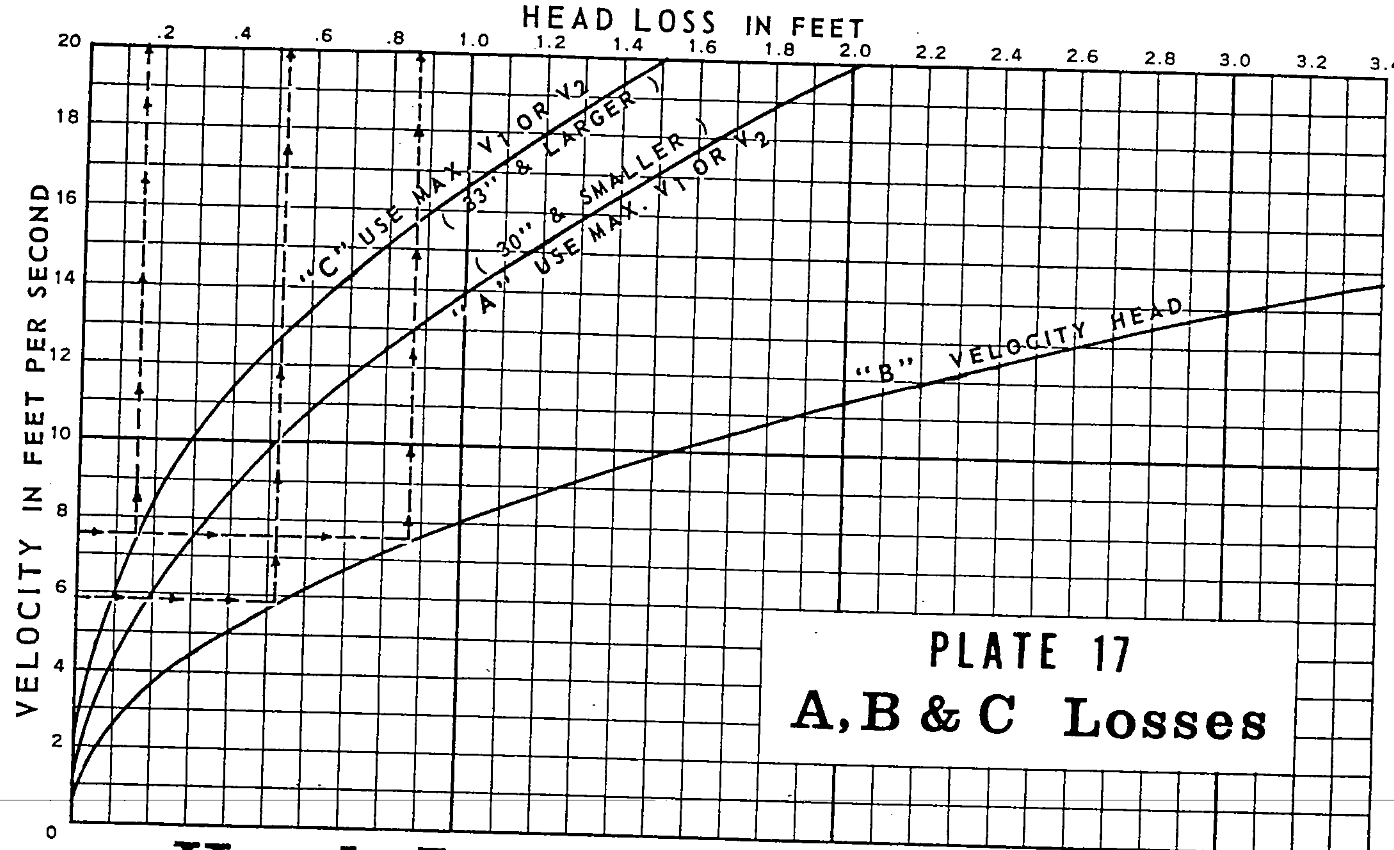


Pipe Flow Chart 60 inch Diameter

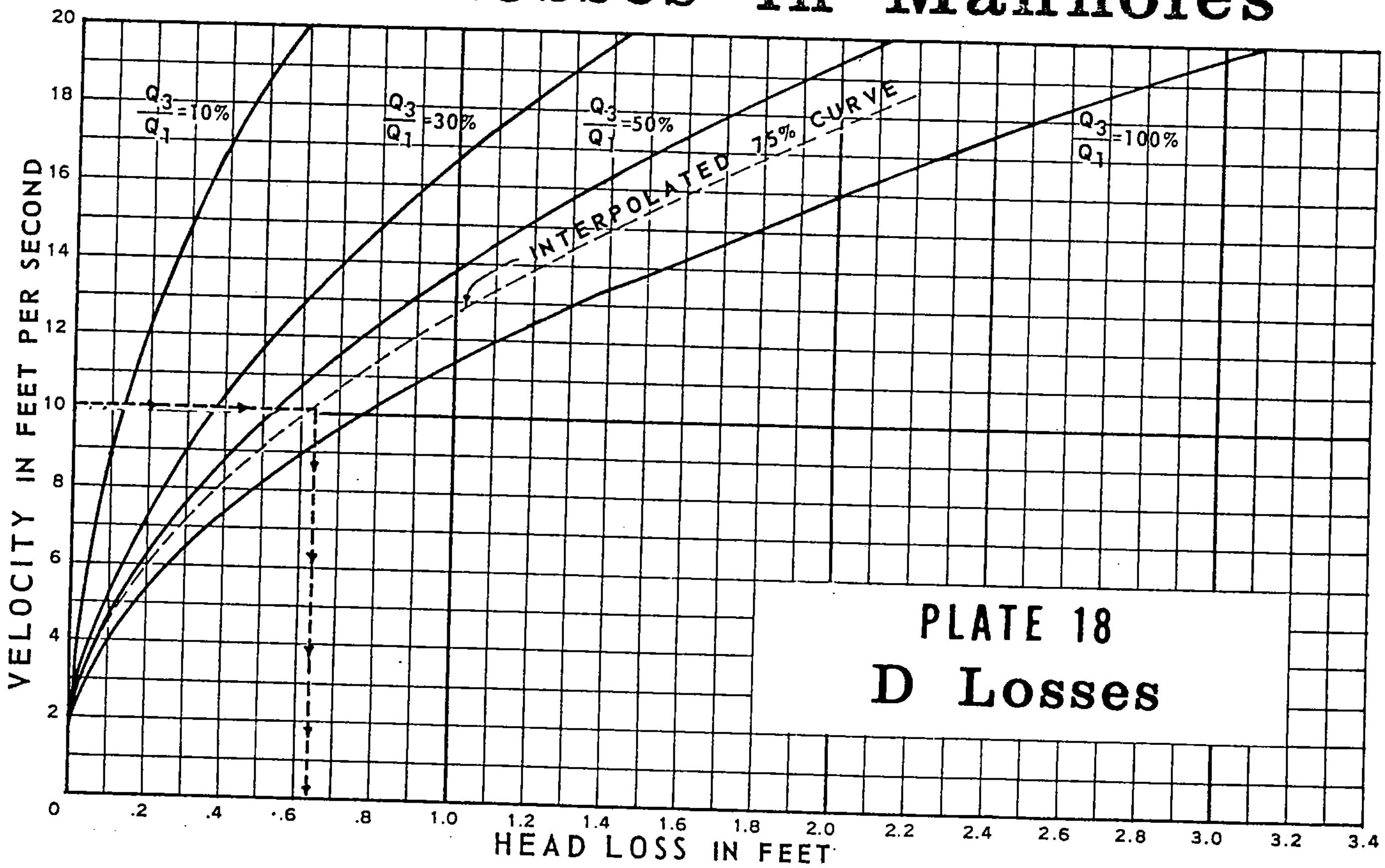
PLATE 16



Pipe Flow Chart 66 inch Diameter



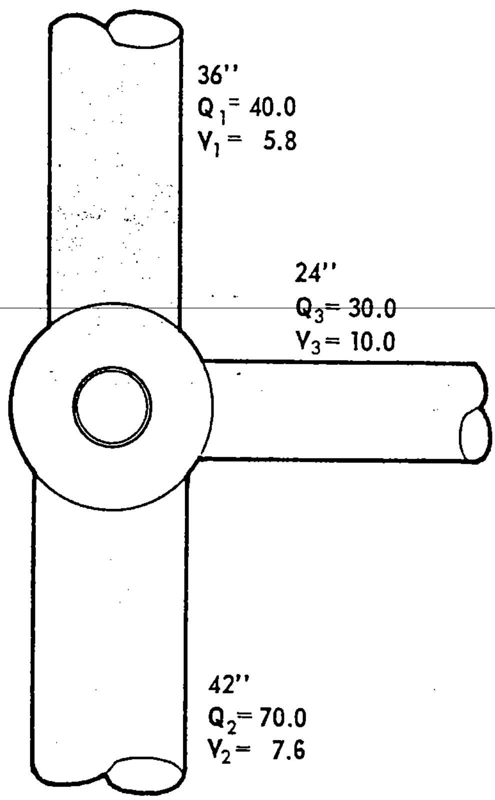
Head Losses in Manholes



SOURCE: BALTIMORE COUNTY DEPARTMENT OF PUBLIC WORKS

Example: Analysis & Solution

GIVEN: Pipe size, Q, Velocity and Direction of flow.



LEGEND

Q_1 = Upstream Volume, cfs

Q_2 = Downstream Volume, cfs

Q_3 = Incoming Volume, cfs

V_1 = Upstream Velocity, fps

V_2 = Downstream Velocity, fps

V_3 = Upstream Branch Velocity, fps

h_v = Head Loss, in ft.

SOLUTION

"A" LOSS (ENTRANCE & EXIT LOSS)

1. Determine higher velocity between V_1 and V_2
2. Use Curve "A" or "C" depending on pipe size and determine h_A (Ex. Prob. $h_A = 0.16$)

"B" LOSS (VELOCITY HEAD LOSS)

1. Use Curve "B" and determine h_V for V_1 and V_2
   a. If V_2 is lower than V_1, then h_B shall be 0.
   b. If V_2 is higher than V_1, then h_B shall be $h_{B_2} - h_{B_1}$
   (Ex. Prob. $h_{B_2} = 0.87$ and $h_{B_1} = 0.53$
   $h_B = 0.87 - 0.53 = 0.34$)

"C" LOSS (DIRECTIONAL CHANGE LOSS)

1. Use worst case and determine degree of bend.
2. With higher V_1 or V_2, use Curve "C" and determine head loss (h)
   a. For 0° to 22½° bends, h_c shall be 0.67 times h.
   b. For 22½° to 45° bends, h_c shall be 1.00 times h.
   c. For 45° to 90° bends, h_c shall be 2.00 times h
   (Ex. Prob. $h = 0.16$
   $h_c = 2 \times 0.16 = 0.32$)

"D" LOSS (LOSS DUE TO INCOMING VOLUME)

1. Add total branch volume and determine ratio of branch volume to upstream volume.
2. Use appropriate curve and determine h_D with higher V_1 or V_3
   (Ex. Prob. $\frac{Q3}{Q1} = \frac{30}{40} = 75\%$
   $h_D = 0.64$)

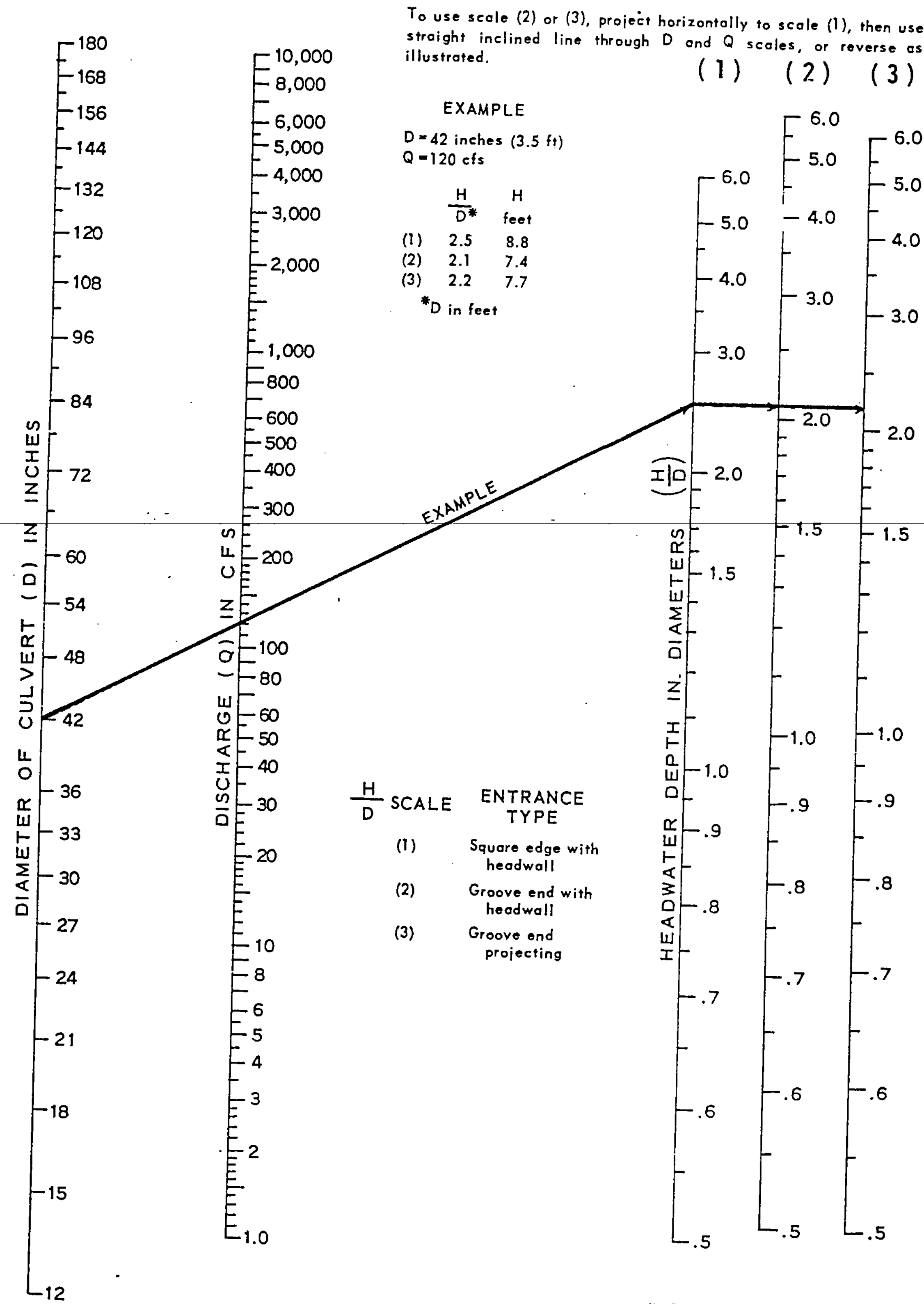
TOTAL LOSS:

1. Add h_A, h_B, h_C and h_D
   (Ex. Prob. $h_T = 0.16 + 0.34 + 0.32 + 0.64$
   $h_T = 1.46$)

Losses		
A		= 0.16
B	0.87 – 0.53	= 0.34
C	2 (0.16)	= 0.32
D		= 0.64
Total Loss		1.46 ft.

NOMOGRAPH FOR CONCRETE PIPE CULVERTS WITH ENTRANCE CONTROL

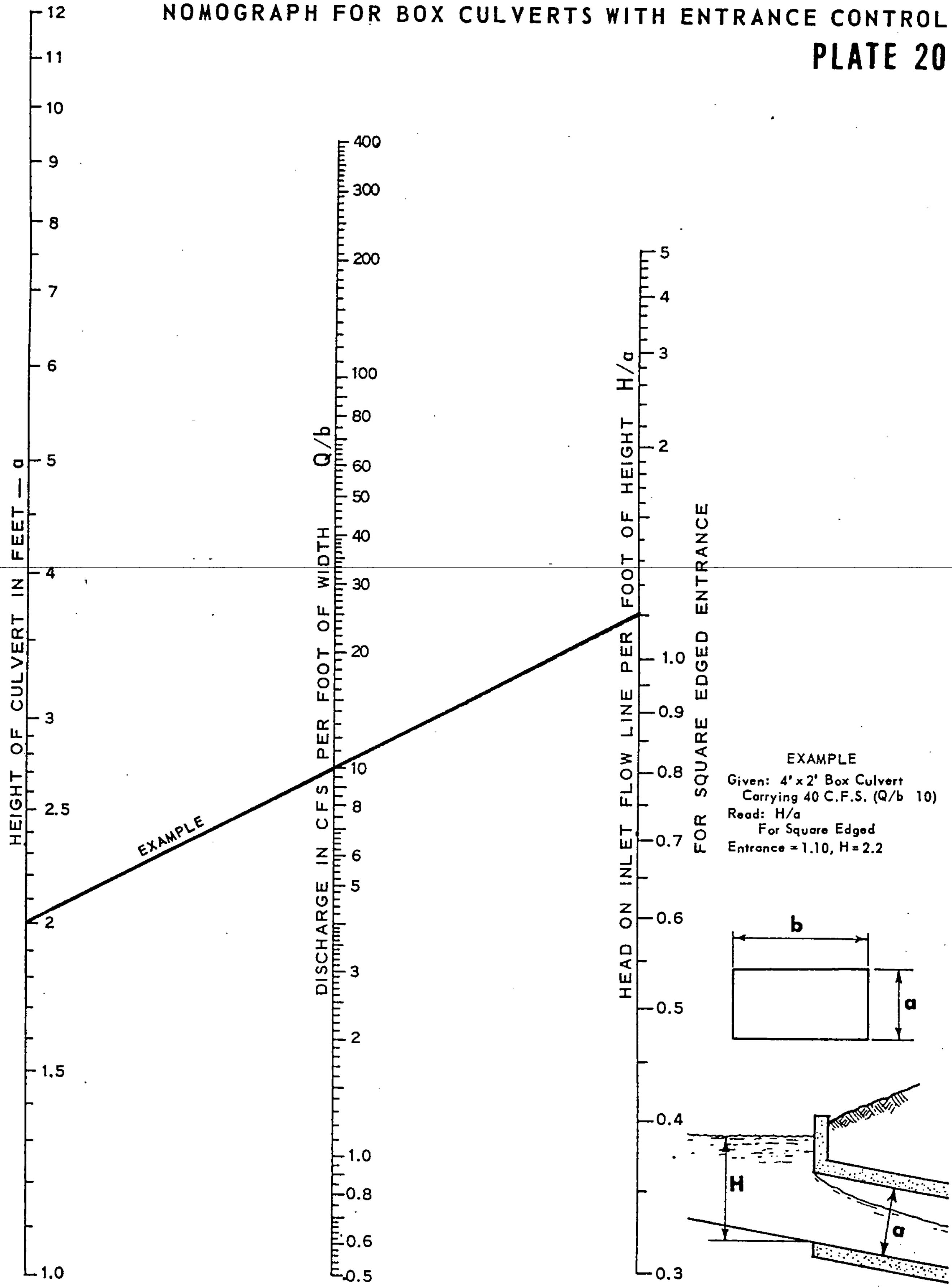
PLATE 19



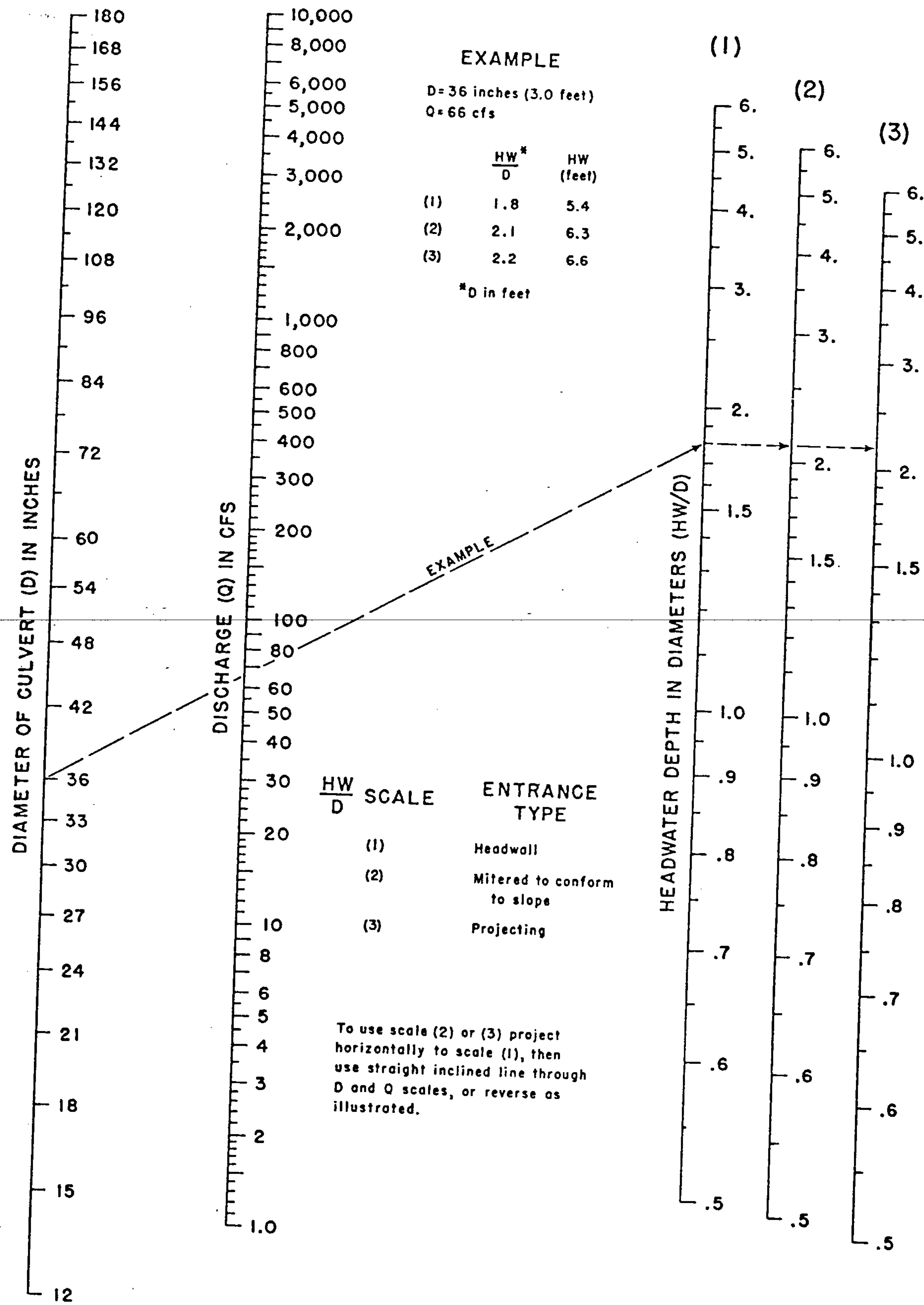
(U.S. Bureau of Public Roads.)

NOMOGRAPH FOR BOX CULVERTS WITH ENTRANCE CONTROL

PLATE 20



NOMOGRAPH FOR C. M. PIPE CULVERTS WITH ENTRANCE CONTROL

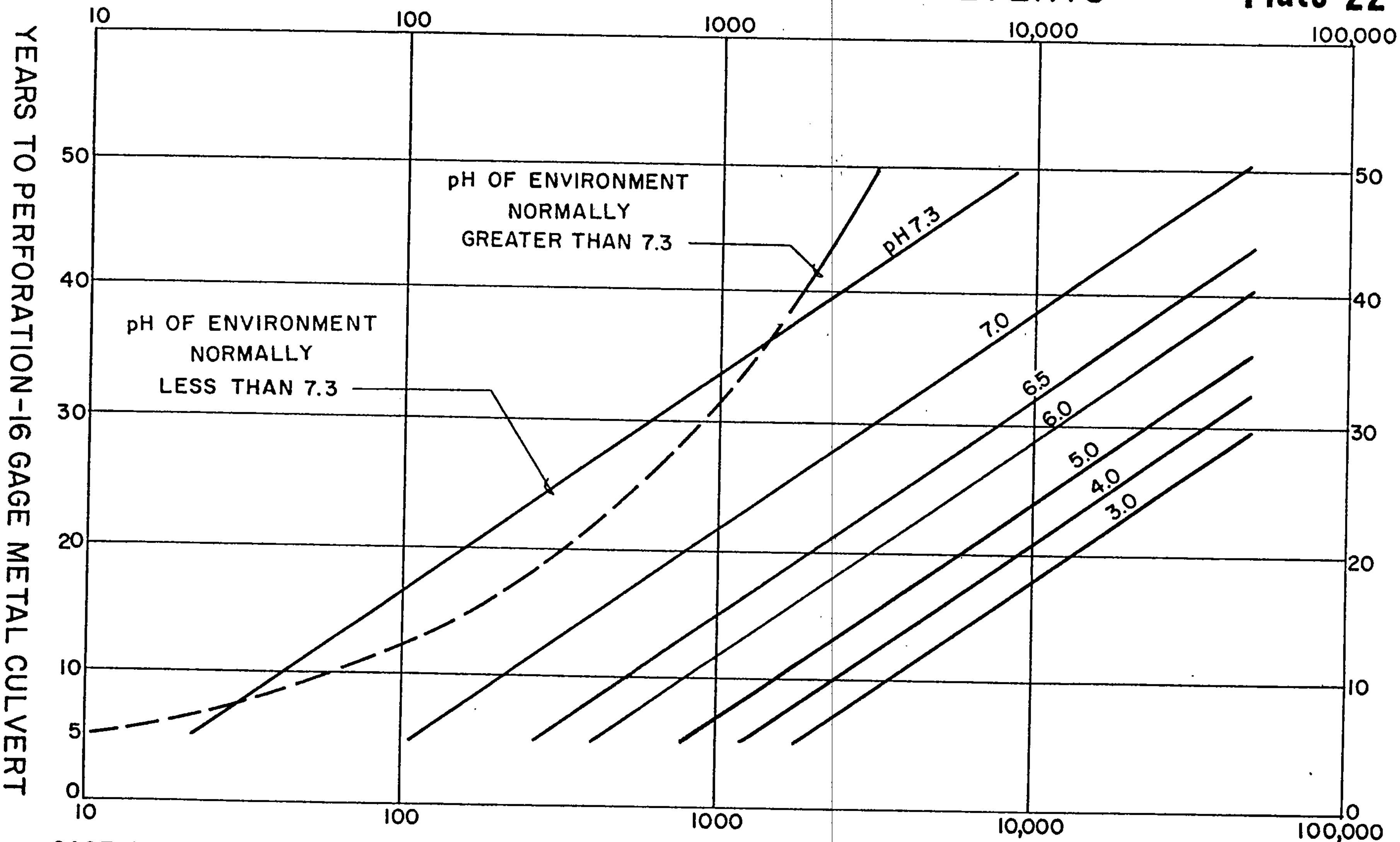


BUREAU OF PUBLIC ROADS

Plate 21

Plate 22

CHART FOR ESTIMATING YEARS TO PERFORATION OF METAL CULVERTS



GAGE	14	12	10	8
FACTOR	1.3	1.8	2.3	2.8

MULTIPLY YEARS TO PERFORATION BY FACTOR FOR INCREASE IN METAL GAGE.

NOTE: MINIMUM CULVERT LIFE EXPECTANCY SHALL BE 40 YEARS

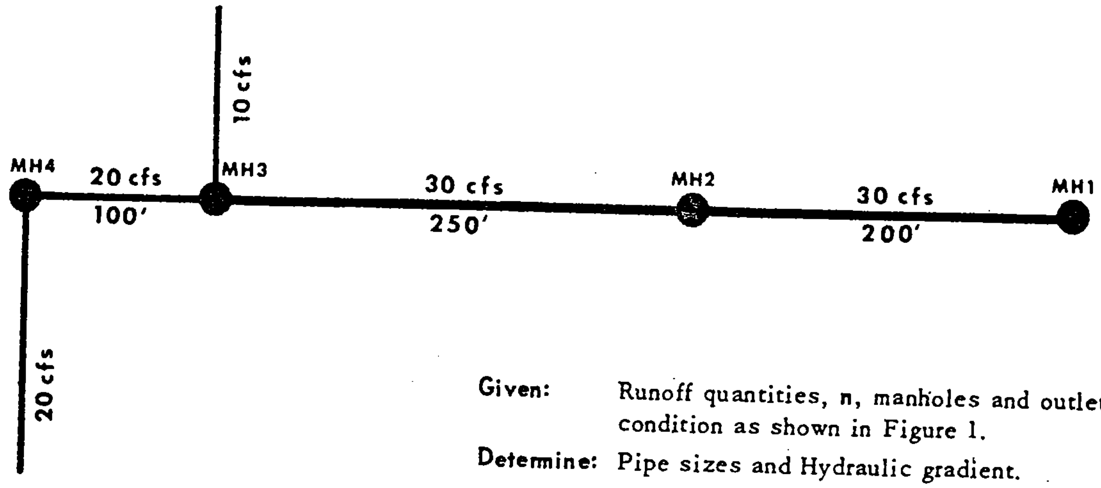
SOURCE: CALIFORNIA DEPARTMENT OF PUBLIC WORKS 1963

Appendix

PIPE SYSTEM ANALYSIS

Figure 1

EXAMPLE OF COMPUTATION



Given: Runoff quantities, n, manholes and outlet condition as shown in Figure 1.

Determine: Pipe sizes and Hydraulic gradient.

SOLUTION

USE PLATES 8 TO 20 (pg. 23 to 35) AS AID TO ANALYSIS.

Make preliminary determination of pipe sizes for the data given using pipe flow charts. This is shown in Figure 2.

Using the pipe sizes and slopes of pipes as determined above, compute hydraulic gradient for the system. This is shown in Figure 3.

1. Controlling grade at M.H. #1 is 100.00 as shown in Figure 1.

   Study conditions of flow between manholes or inlets to determine if entrance control or losses govern hydraulic gradient.

2. With the selected pipe size between M.H. #1 and M.H. #2, 24" diameter pipe at $S = 0.010$, compute the head loss in the pipe by the formula $h = SL$ or $h_f = S_fL$, whichever controls.

   h = elevation head loss
   h_f = friction head loss
   S = slope of the pipe
   S_f = friction slope (used when pipe flowing full)
   L = length of the pipe or channel

   Since the pipe is flowing full, as determined by the pipe flow chart using 24" diameter, the friction slope 0.018 must be used. The head loss in the pipe is:

   $$h_f = S_f L$$
   $$h_f = (0.018)\ (200) = 3.60 \text{ feet}$$

   The downstream hydraulic gradient at M.H. #2 is equal to the controlling grade at M.H. #1 plus the head loss or

   $$100.00 + 3.60 = 103.60$$

3. Since the pipe is flowing full, and there are no bends or drops, compute the upstream hydraulic gradient at M.H. #2 by adding the manhole losses to the downstream hydraulic gradient at M.H. #2. These values are obtained from charts on manhole losses. From the charts:

   A = 0.47
   B = 0.00 (since the velocities are equal)
   C = 0.00
   D = 0.00

   0.47 ft. (Total M.H. losses)

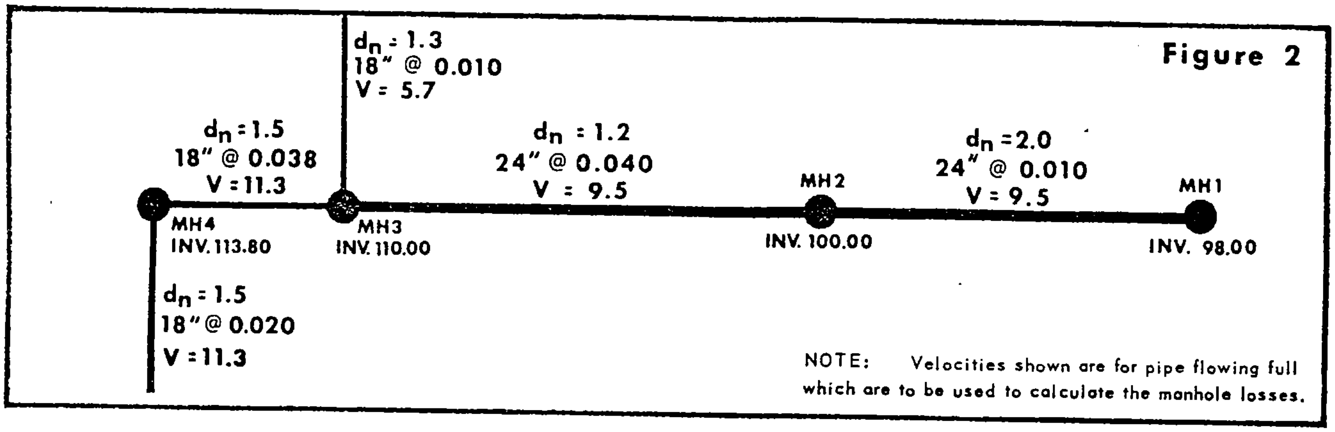


Figure 2

The upstream hydraulic gradient at M.H. #2 is:

$$103.60 + 0.47 = 104.07$$

4. With the selected pipe size between M.H. #2 and M.H. #3, 24" diameter pipe at S = 0.040, compute the head loss elevation in the pipe:

$$h = SL$$
$$h = (0..040)\ (250) = 10.00 \text{ feet}$$

Since the pipe is not flowing full as determined by the pipe flow chart, the elevation head loss and the normal depth must be added to the invert of M.H. #2. Therefore, the downstream hydraulic gradient at M.H. #3 is:

$$100.00 + 10.00 + 1.20 = 111.20$$

5. Compute the upstream hydraulic gradient at M.H. #3 by adding to the invert elevation the manhole losses and entrance control losses for open channel flow. Only manhole losses "C" and "D" need be considered.
From the charts:

$$C = 2(0.40) = 0.80 \quad (90° \text{ Bend})$$
$$D \qquad = \underline{0.69}$$
$$1.49 \text{ ft. (Total M.H. losses)}$$

Entrance control loss for Q = 30 cfs, D = 24" is:

$$H/D = 2.35$$
$$H \quad = 4.70 \text{ feet}$$

The upstream hydraulic gradient at M.H. #3 is:

$$110.0 + 1.49 + 4.70 = 116.19$$

6. With selected pipe size between M.H. #3 and M.H. #4, 18" diameter pipe at S 0.038, compute the head loss in the pipe:

$$h_f = S_f L$$
$$h_f = (0.038)\ (100) = 3.80 \text{ feet}$$

The downstream hydraulic gradient at M.H. #4 is:

$$116.19 + 3.80 = 119.99$$

since the tailwater condition of the pipe is submerged.

7. Since there is a bend greater than 10° at MH #4, compare losses and use the higher HGL.

$$A = 0.66 \qquad\qquad C = 0.80$$
$$B = 0.00 \qquad\qquad \underline{D = 0.00}$$
$$C = 0.80\ (0.40 \times 2) \qquad 0.80$$
$$\underline{D = 0.00}$$
$$1.46 \qquad\qquad H/D = 3.2 \quad H = 4.80$$

$$119.99 + 1.46 = 121.45 > 113.80 + 4.80 + 0.80 = 119.40$$

The upstream hydraulic gradient at MH #4 is 121.45

Adjust pipe sizes if warranted by the hydraulic gradient as computed above. ♦♦♦

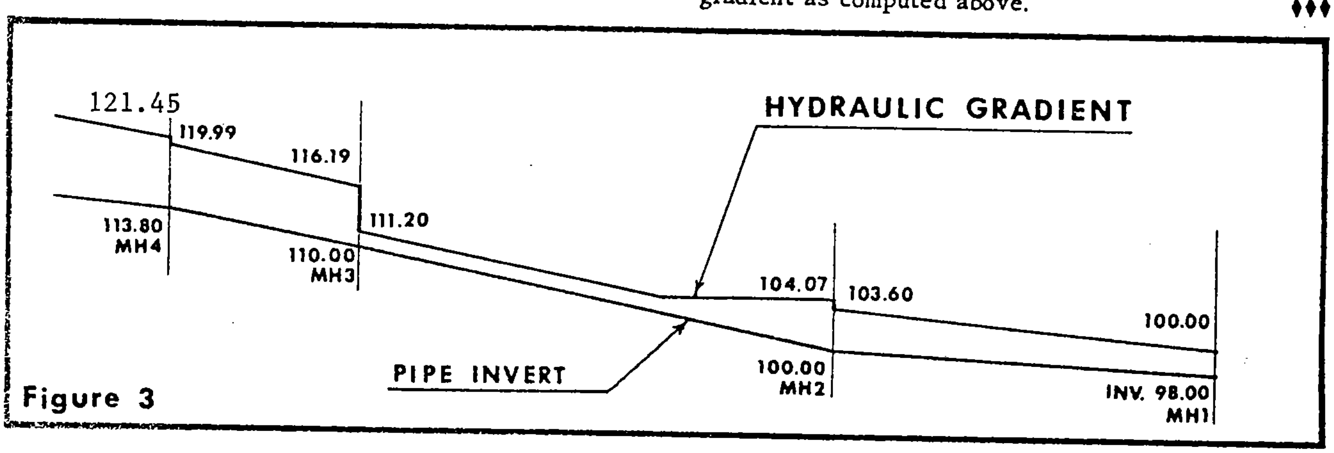


Figure 3

PROJECT
DRAIN LINE

DRAINAGE DESIGN DATA

DATE
COMPUTED BY
CHECKED BY

Inlet	"Tc"	"C"	"I" / "i"	Drain Area	"Q" Runoff / Accum. "Q"	"dn"	"Vf"	Size of Pipe	Length of Seg.	"Sf"	"S"	Invert Elevation	"Hf"	Manhole Loses				Ent. Cont.	Hydraulic Grade Elev.		Finished Grade Elevation	Remark
														A	B	C	D		Down	Up		

Suggested layout of tabulated computation form for **DRAINAGE DESIGN DATA** to be submitted for approval

CULVERT DESIGN

PROJECT: ______________________ COMPUTED BY: ______________________

DATE: ______________________ CHECKED BY: ______________________

HYDROLOGY

L =

H =

S ave. =

$$K = \frac{L}{S} = \underline{\qquad} =$$

CHARACTER OF GROUND

Tc =

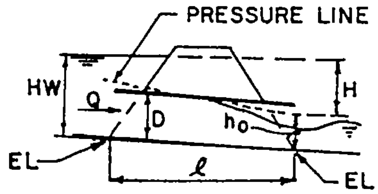
Design Frequency = Years

STORM DURATION

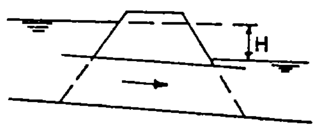
I =

C = Q=CIA= cfs

Δ =



UNSUBMERGED OUTLET



SUBMERGED OUTLET

Δ EL =

$$S_o = \frac{\Delta EL}{\ell} =$$

$$\text{Site Index No.} = \frac{\ell}{100\, S_o}$$

CULVERT IDENTIFICATION			CAPACITY CHART	INLET CONTROL		OUTLET CONTROL NOMOGRAPH							
ENTRANCE TYPE	MATERIAL	SIZE	HW (FT.)	$\frac{HW}{D}$	HW (FT.)	Ke	H	$D^{5/2}$	$\frac{Q}{D^{5/2}}$	dc	ho	$-S_o\ell$	HW

$$dc = \frac{QD}{D^{5/2}}$$

$$h_o = \frac{dc + D}{2}$$

$$HW = H + h_o - S_o\ell$$

SUGGESTED LAYOUT OF COMPUTATIONS FOR CULVERT DESIGN TO BE SUBMITTED FOR APPROVAL.

H = Head in feet

K_e or C_e = Entrance loss coefficient

D = Diameter of pipe in feet

n = Manning's roughness coefficient

ℓ = Length of culvert in feet

Q = Design discharge rate in cfs

dc = Critical depth in feet

h_o = Pressure head in feet

S_o = Slope of pipe

HW = Head water in feet

FOR USE WITH CULVERT CAPACITY CHARTS OF U.S. BUREAU OF ROADS

Drainage Plan Showing Design Data to be Submitted on Drawing