HYDROLOGIC AND HYDRAULIC TRAINING SESSION

October 16, 17, 18 - 1972
Revised December 1973

Arizona Highway Department
Structures Section
Hydraulics Division

Prepared by:

M. Sheldon
E. Jencsok
R. Jordan
W. Newcomb
J. Unten

TABLE OF CONTENTS

A. Hydrology of Nonuniform Areas

B. Effects of Storage on Peak Flows

C. Median Drainage

D. Pavement Drainage

E. Hydrology of Urban Areas

F. Storm Sewer Design

G. Scour and Bank Protection

H. Computer Applications

I. Nonuniform Flow

J. Hydrologic and Hydraulic Criteria

K. References

Nonuniform Drainage Areas.

Definition: An area having various subwatersheds of nonuniform hydrologic characteristics.

An example is shown in Figure 1.

Total D.A. = 17 sq. mis.

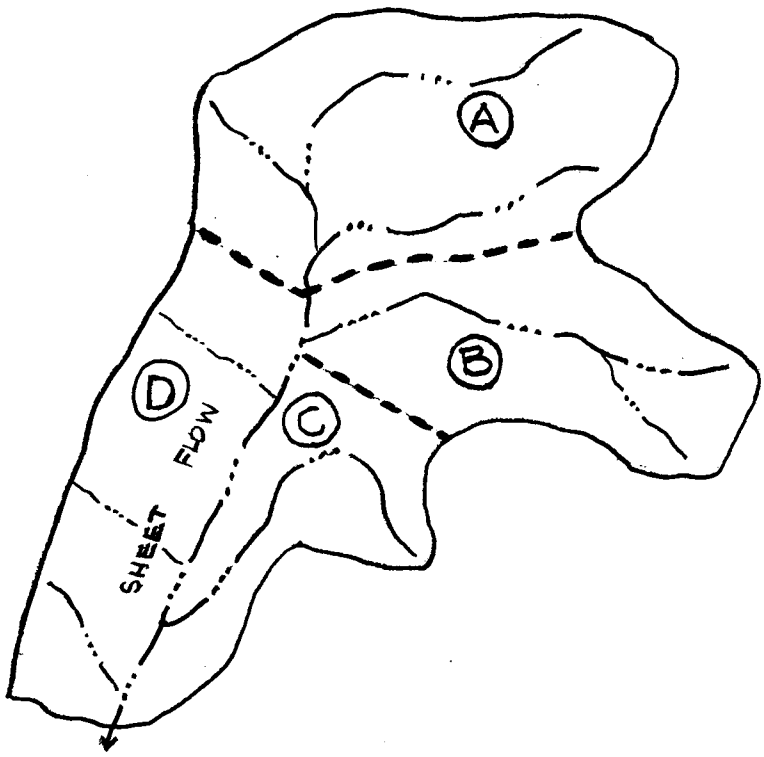


Figure 1

Following are the assumed hydrologic characteristics of each subwatershed:

		WATERSHED			
		A	B	C	D
D.A. (sq. mis.)		5.2	4.4	3.4	4.0
Top Elev.		4240	3700	2500	1700
Bottom Elev.		2240	1940	1380	1500
Length (Mis.)		3.0	4.1	2.6	1.1
Vegetation		30% Juniper Grass	50% Herbaceous	40% Herbaceous	
		70% Herbaceous	50% Desert Brush	60% Desert Brush	Herbaceous
Cover Density		20% Juniper Grass	25% Herbaceous	25% Herbaceous	
		40% Herbaceous	10% Desert Brush	10% Desert Brush	30%
Soil Type		C Juniper Grass D Herbaceous	C Herbaceous D Desert Brush	C Herbaceous B Desert Brush	C
P_{50}	1 hour	1.8	1.7	1.6	1.6
	2 hour	2.0	2.0	1.9	1.9
	3 hour	2.3	2.3	2.2	2.2
	6 hour	3.1	3.0	2.9	2.9

QUESTIONS AND ANSWERS

RELATING TO ANALYSIS OF A NONUNIFORM WATERSHED

Q. Why is this a nonuniform watershed?

A. Following are some of the reasons:

1. Variation in vegetative types and soils.
2. Variation in drainage tributary lengths and slopes.
3. Variation in the times of concentration of major tributaries.

Q. How does the analysis of a nonuniform watershed differ from a uniform watershed?

A.
1. A nonuniform area takes into account the travel time along the main channel between the outlet of the various delineated subwatersheds.
2. A nonuniform area also takes into account the various vegetative types and soils of the watershed.
3. The types of flow, whether in defined channels or sheet flow may govern the boundary delineation of each subwatershed and also affects the time of concentration.

Q. How does one decide when to subdivide a drainage area for hydrology evaluation?

A.
1. First, a drainage area must be large enough and must have a major cost drainage structure involved before it is worthwhile to make an analysis.
2. The hydrologic characteristics must be sufficiently varied in a major way before a significant difference in the peak Q_p determination is found as compared to the uniform area method.

Q. What hydrology method is used to determine the peak discharge (Q_p) of each subwatershed?

A. SCS Method Part II must be used to determine the peak discharge of each subwatershed. An additional part of the analysis is to determine a hydrograph of flow for each subwatershed based on the hydrologic characteristics. This hydrograph is based on the SCS Triangular Hydrograph as shown in NEH 4, Supplement A, Soil Conservation Service Hydrology Handbook.

Time to Peak (T_p) = $D/2 + .6T_c$

Time of hydrograph base (T_B) = $2.67T_p$

Volume under hydrograph = Volume of runoff (Q)

Q. How are the subhydrographs used to determine the peak discharge of the entire drainage area?

A. 1. A method of summating these hydrographs is shown in NEH 4, Supplement A of the SCS Hydrology Handbook. If all the subhydrograph's peak discharges occurred at the same time, the various peak Q's would be added numerically.

2. Due to the lag time between the outlet of each subwatershed to the point of outlet of the next downstream subwatershed, and also the difference in time to peaks (T_p) of the hydrographs, the peak Q_p's of the hydrographs will require summation by a graphical method which will be described later.

Q. How are the lag times between each subwatershed determined?

A. Lag Time is normally determined by knowing the main channel geometrics and thus determining the velocity of flow by Mannings equation and then converting this to hours of travel time. If the channel geometrics are not known an estimate can be determined by using Figure 2-7 (for defined channels).

Now for an example. (Use the data shown for Figure 1.)

Step 1: Determine hydrologic data necessary to fill out SCS Part II based on entire drainage area.

Step 2: Based on judgement only, delineate the subwatersheds based on your estimate of which subwatersheds may be classified as nonuniform.

Step 3: Fill out the SCS Part II hydrology work sheet for each of the four subwatersheds that was assumed for the example.

Step 4: Determine the hydrograph characteristics (T_p, Q_p, and T_B) for each duration (1, 2, 3, and 6 hours) for each of the four subwatersheds.

Step 5: Determine the lag time between the outlet points of each subwatershed.

Step 6: Plot all the triangular hydrographs for a one hour duration storm on graph paper with Q_p on the ordinate and time in hours on the abcissa. Time zero should be the point at which the most downstream watershed enters the main stream channel. The second subwatershed hydrograph starting point should be set at the lag time between the

outlets of the first and second subwatersheds. The third and fourth subwatersheds are plotted in the same manner.

Step 7: Summate the hydrographs at various time points along the abcissa. This is done by adding the ordinates of each hydrograph that is intersected at each time interval. At one of these points a maximum Q_p value will be obtained.

Step 8: Repeat Steps 6 and 7 for the 2, 3, and 6 hour durations. The maximum Q_p value obtained between the 1, 2, 3, and 6 hour durations will be the final design Q_p.

Following are the calculations for the example on Figure 1 for each Step:

Step 1: See enclosed hydrologic design data sheet. (page 6.)

Step 2: Watershed boundary delineations are shown on Figure 1.

Step 3: Fill out SCS Part II hydrology sheets as described in Arizona Hydrology Manual. (Areal reduction is assumed for each subarea as is used for entire drainage area.) (See pages 7, 8, 9, and 10.)
Following is method of determining composite curve number for subwatershed A. The same method is required for the remaining 3 subwatersheds.

D.A. = 5.2 sq. mis.
30% (or 1.56 sq. mis.) is Juniper Grass
70% (or 3.64 sq. mis.) is Herbaceous

Juniper Grass (1.56 sq. mis., C Soil, 20% cover) = CN 83
Herbaceous (3.64 sq. mis., D Soil, 40% cover) = CN 89

Composite CN = (83 x 1.56 + 89 x 3.64) / 5.2 = CN 87.2
Also, composite CN = 83 x 30% + 89 x 70% = CN 87.2

ENTIRE AREA

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway Prescott-Congress County Yavapai
Location
Project No. Station 2220+
Name of Stream No Name

DESIGN DATA:

Design Frequency	50	years
Drainage Area	17.0	square miles
Drainage Length	41,000	feet
Elevation		
Top of Drainage area	4,240	feet
At Structure	1,300	feet
Drainage Area Slope	7.1	%
Vegetative Cover Type	Herbaceous	
Vegetative Cover Density	25	%
Soil Group	C	
Precipitation		
P = 6 hour =	3.0	inches
P = 24 hour =	4.3	inches

DESIGN COMPUTATION:

Curve Number	85	
Time of Concentration	1.26	hours (9.0 f.p.s.)
Peak Design Discharge		
$Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	3,390	cfs (199 csm)

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge
1 hour	1.70	.910	1.54	.48	3,130
2 hour	2.00	.930	1.86	.69	3,230
3 hour	2.28	.955	2.18	.93	3,390
6 hour	3.00	.966	2.90	1.51	3,300

Computed by: MS Date 9-27-72

AHD 20-049 11-68

SUBWATERSHED A

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway _____ County _____
Location _____
Project No. _____ Station _____
Name of Stream _____

DESIGN DATA:

Design Frequency	50	years
Drainage Area	5.2	square miles
Drainage Length	15,800	feet
Elevation		
Top of Drainage area	4240	feet
At Structure	2240	feet
Drainage Area Slope	12.6	%
Vegetative Cover Type	30% JUNIPER GRASS 70% HERBACEOUS	
Vegetative Cover Density	20% " " 40% "	
Soil Group	C " " D "	
Precipitation		
P = 6 hour =	3.1	inches
P = 24 hour =	4.4	inches

DESIGN COMPUTATION:

Curve Number	87	
Time of Concentration	.49	hours (Fig. 2-7) (8.95 fps)
Peak Design Discharge		
$Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	2030	cfs (390 CSM)

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge
1 hour	1.80	.910	1.64	.64	2030
2 hour	2.10	.930	1.95	.87	1690
3 hour	2.38	.955	2.27	1.13	1590
6 hour	3.10	.966	2.99	1.73	1320

Computed by: ML Date 9/27/72

AHD 20-049 11-68

SUBWATERSHED B

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway _____ County _____
Location _____
Project No. _____ Station _____
Name of Stream _____

DESIGN DATA:

Design Frequency	50	years
Drainage Area	4.4	square miles
Drainage Length	21,600	feet
Elevation		
Top of Drainage area	3700	feet
At Structure	1940	feet
Drainage Area Slope	8.1	%
Vegetative Cover Type	50% HERBACEOUS	50% DES. BRUSH
Vegetative Cover Density	25% "	10% "
Soil Group	C "	D "
Precipitation		
P = 6 hour =	3.0	inches
P = 24 hour =	4.3	inches

DESIGN COMPUTATION:

Curve Number	88.7	
Time of Concentration	.73	hours (Fig. 2-7) (8.2 fps)
Peak Design Discharge $Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	1500	cfs (341 CSM)

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge
1 hour	1.70	.910	1.55	.66	1500
2 hour	2.00	.930	1.86	.90	1330
3 hour	2.28	.955	2.18	1.15	1260
6 hour	3.00	.966	2.90	1.79	1110

Computed by: MA Date 9/27/72

AHD 20-049 11-68

SUBWATERSHED C

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway _____ County _____
Location _____
Project No. _____ Station _____
Name of Stream _____

DESIGN DATA:

Design Frequency	50	years
Drainage Area	3.4	square miles
Drainage Length	13,700	feet
Elevation		
Top of Drainage area	2500	feet
At Structure	1380	feet
Drainage Area Slope	8.2	%
Vegetative Cover Type	40% HERB. 60%	DES. BRUSH
Vegetative Cover Density	25% " 10%	" "
Soil Group	C " B	" "
Precipitation		
P = 6 hour =	2.9	inches
P = 24 hour =	4.2	inches

DESIGN COMPUTATION:

Curve Number	84	
Time of Concentration	.52	hours (Fig. 2-7) (7.3 fps)
Peak Design Discharge $Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	810	cfs (238 CSM)

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge
1 hour	1.60	.910	1.46	.40	810
2 hour	1.90	.930	1.77	.59	740
3 hour	2.18	.955	2.08	.81	736
6 hour	2.90	.966	2.80	1.36	676

Computed by: MA Date 9/27/72

AHD 20-049 11-68

SUBWATERSHED D

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway _____ County _____
Location _____
Project No. _____ Station _____
Name of Stream _____

DESIGN DATA:

Design Frequency	50	years
Drainage Area	4.0	square miles
Drainage Length	5800	feet
Elevation		
Top of Drainage area	1700	feet
At Structure	1500	feet
Drainage Area Slope	3.46	%
Vegetative Cover Type	HERBACEOUS	
Vegetative Cover Density	30	%
Soil Group	C	
Precipitation		
P = 6 hour =	2.9	inches
P = 24 hour =	4.2	inches

DESIGN COMPUTATION:

Curve Number	84	
Time of Concentration	.92	hours (Fig. 2-5) overland (1.8 fps)
Peak Design Discharge $Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	764	cfs (191 CSM)

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge
1 hour	1.60	.910	1.46	.40	736
2 hour	1.90	.930	1.77	.59	736
3 hour	2.18	.955	2.08	.81	764
6 hour	2.90	.966	2.80	1.36	741

Computed by: [illegible] Date 9/27/72

AHD 20-049 11-68

Step 4: Hydrograph Characteristics

1 Hour Duration

Subwatershed	Q_p cfs	T_p (Hours) $D/2 + .6T_c$	T_B (Hours) $2.67\ T_p$
A	2030	.79	2.11
B	1500	.94	2.51
C	810	.81	2.16
D	736	1.05	2.80

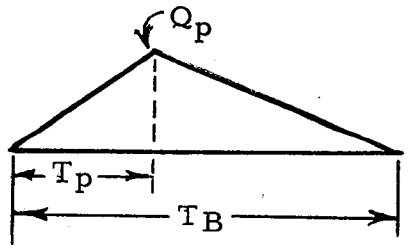


Figure 2

2 Hour Duration

	Q_p	T_p	T_B
A	1690	1.29	3.44
B	1330	1.44	3.84
C	740	1.31	3.50
D	736	1.55	4.14

3 Hour Duration

A	1590	1.79	4.78
B	1260	1.94	5.18
C	736	1.55	4.14
D	764	2.05	5.47

6 Hour Duration

A	1320	3.29	8.78
B	1110	3.44	9.18
C	676	2.05	5.47
D	741	3.55	9.48

Step 5:

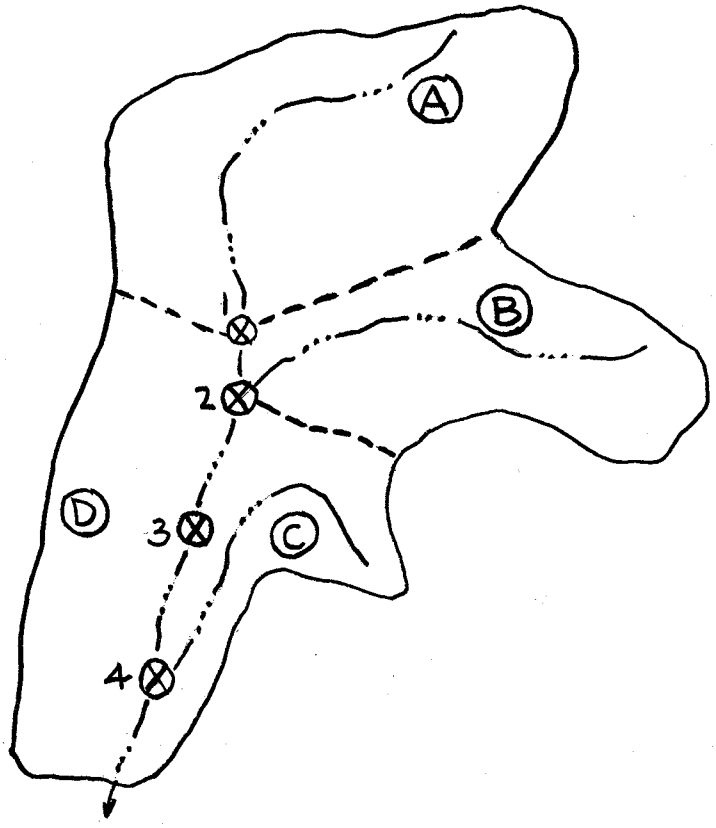


Figure 3

Location of outlet points of each subwatershed.

1 is outlet of watershed	A
2 is outlet of watershed	B
3 is outlet of watershed	D (sheet flow-assumed midpoint)
4 is outlet of watershed	C

Assuming no channel X-sections are available to determine stream velocity, use Figure 2-7.

Point	to Point	L(ft.)	H(ft.)	lag time(hours)
1	2	4000	300	.21
2	3	8000	440	.40
3	4	8000	120	.65

Step 6: (See Figures 5, 6, 7, and 8.)

The hydrographs will be plotted by using the following coordinates: (Q_p, ordinate)
(Σ Lag, ΣT_p, ΣT_B, abscissa)

Hydrograph	Q_p (cfs)	lag (hrs.)	T_p (hrs.)	T_B (hrs.)	Σ Lag	ΣT_p	ΣT_B
			1 Hour*				
C	810	.65	.81	2.16	0	.81	2.16
D	736	.40	1.05	2.80	.65	1.70	3.45
B	1500	.21	.94	2.51	1.05	1.99	3.56
A	2030		.79	2.11	1.26	2.05	3.37
			2 Hour				
C	740	.65	1.31	3.50	0	1.31	3.50
D	736	.40	1.55	4.14	.65	2.20	4.79
B	1330	.21	1.44	3.84	1.05	2.49	4.89
A	1690		1.29	3.44	1.26	2.55	4.70
			3 Hour				
C	736	.65	1.55	4.14	0	1.55	4.14
D	764	.40	2.05	5.47	.65	2.70	6.12
B	1260	.21	1.94	5.18	1.05	2.99	6.23
A	1590		1.79	4.78	1.26	3.05	6.04
			6 Hour				
C	676	.65	2.05	5.47	0	2.05	5.47
D	741	.40	3.55	9.48	.65	4.20	10.13
B	1110	.21	3.44	9.18	1.05	4.49	10.23
A	1320		3.29	8.78	1.26	4.55	10.04

1 Hour Example

* Σ Lag: 0 + .65 = .65
.65+ .40 = 1.05
1.05+ .21 = 1.26

ΣT_p : 0 + .81 = .81
.65+ 1.05 = 1.70
.94+ 1.05 = 1.99
.79+ 1.26=2.05

$T_p + \Sigma$ Lag

ΣT_B: $T_B + \Sigma$ Lag 2.16 + 0 = 2.16
2.80 + .65 = 3.45
2.11 + 1.26 = 3.37 2.51 + 1.05 = 3.56

Step 7: (See Figures 5, 6, 7, and 8 for summation.)
The ordinates can be summated mathematically or graphically. The graphic determination is much simpler than the mathematical determination, however, the Hydraulics Branch has a computer program in basic language to determine the summation of the ordinates of all subhydrographs at various time intervals.

The summation results from the computer program are as follows:

Time Hours	cfs 1 hr.	2 hr.	3 hr.	6 hr.
.65	650	367	309	214
.81	922			
1.05	946	783	648	430
1.26	1303	1195	962	611
1.31		1359		
1.55			1654	
1.70	3180			
1.99	4092			
2.05	4128			1608
2.16	3739			
2.20		3468		
2.49		3935		
2.55		3943		
2.70			3523	
2.99			3822	
3.05			3821	
3.37	217			
3.45	102			
3.50		2090		
3.55	0			
4.13			2270	
4.20				3188
4.49				3304
4.55				3297
4.70		129		
4.78		59		
4.89		0		
5.47				2602
6.03			93	
6.11			42	
6.22			0	
10.03				47
10.12				21
10.22				0

Step 8: Repeat instructions of Steps 6 and 7 to obtain 2, 3, and 6 hour durations. See Figures 5, 6, 7, and 8 for results of graphical solutions.

Discussion and Summary

The results show that the 1 hour duration storm gives the maximum discharge of 4130 cfs. This gives a higher figure than the 3390 cfs determined on the basis of assuming only one drainage area. (See page 6 .) This would indicate that some error would be expected if a drainage would be analyzed as a whole rather than take into consideration the various subareas. No definite recommendations can be made in relation to when or when not to analyze a drainage area for subareas; the only thing that can be said is that if a drainage installation is relatively expensive and crucial in relation to the total drainage concept, a decision may be made to utilize this method of hydrologic analysis for design.

A designer must also give consideration to the fact that although the 2, 3, and 6 hour duration peak discharges are less than the 1 hour design discharge, the volumes associated with the longer duration storms are much larger. This may be an important consideration if a decision is made to flood route the design hydrograph.

ONE HOUR DURATION

Time to Peak = 2.05 Hours

Peak Discharge = 4130 cfs

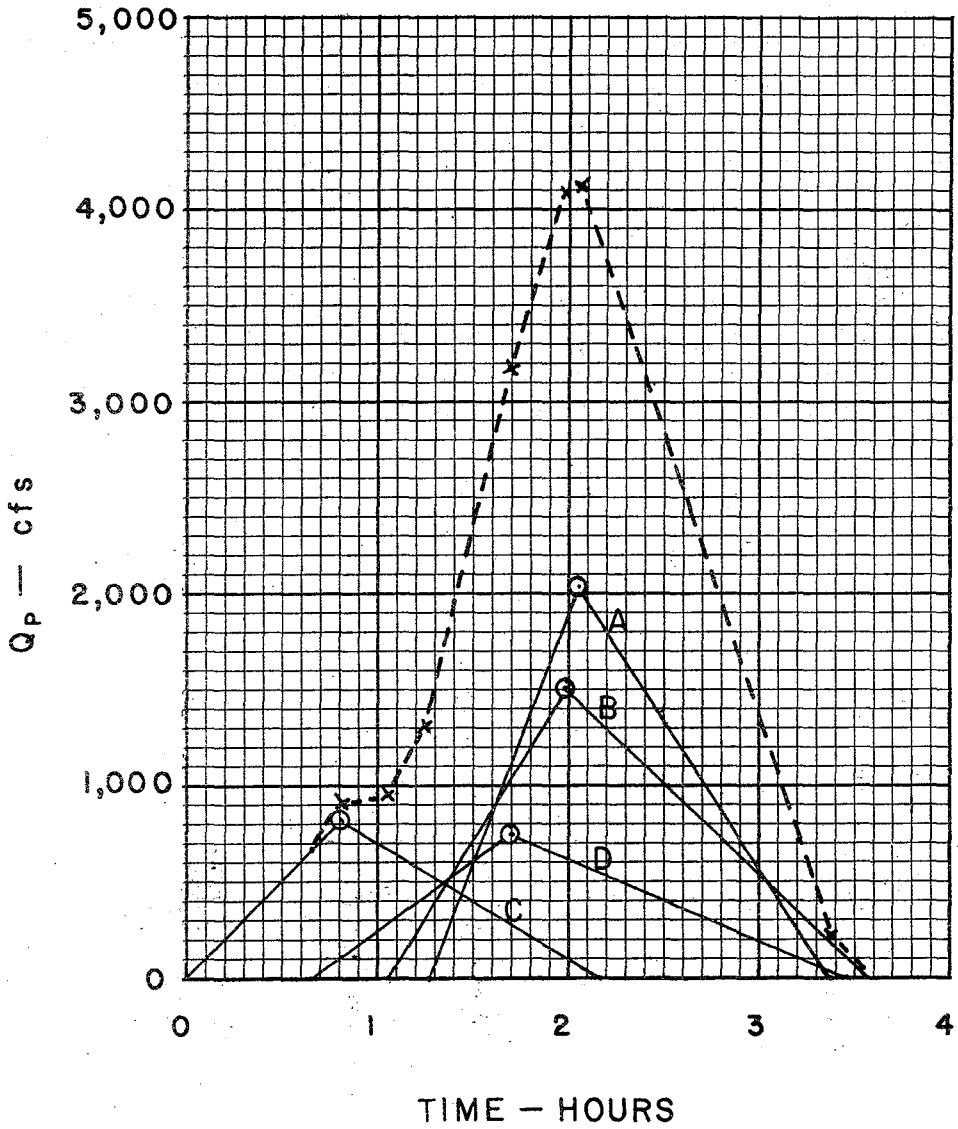


Fig. 5

TWO HOUR DURATION

Time to Peak = 2.55 Hours
Peak Q_P = 3,940 cfs

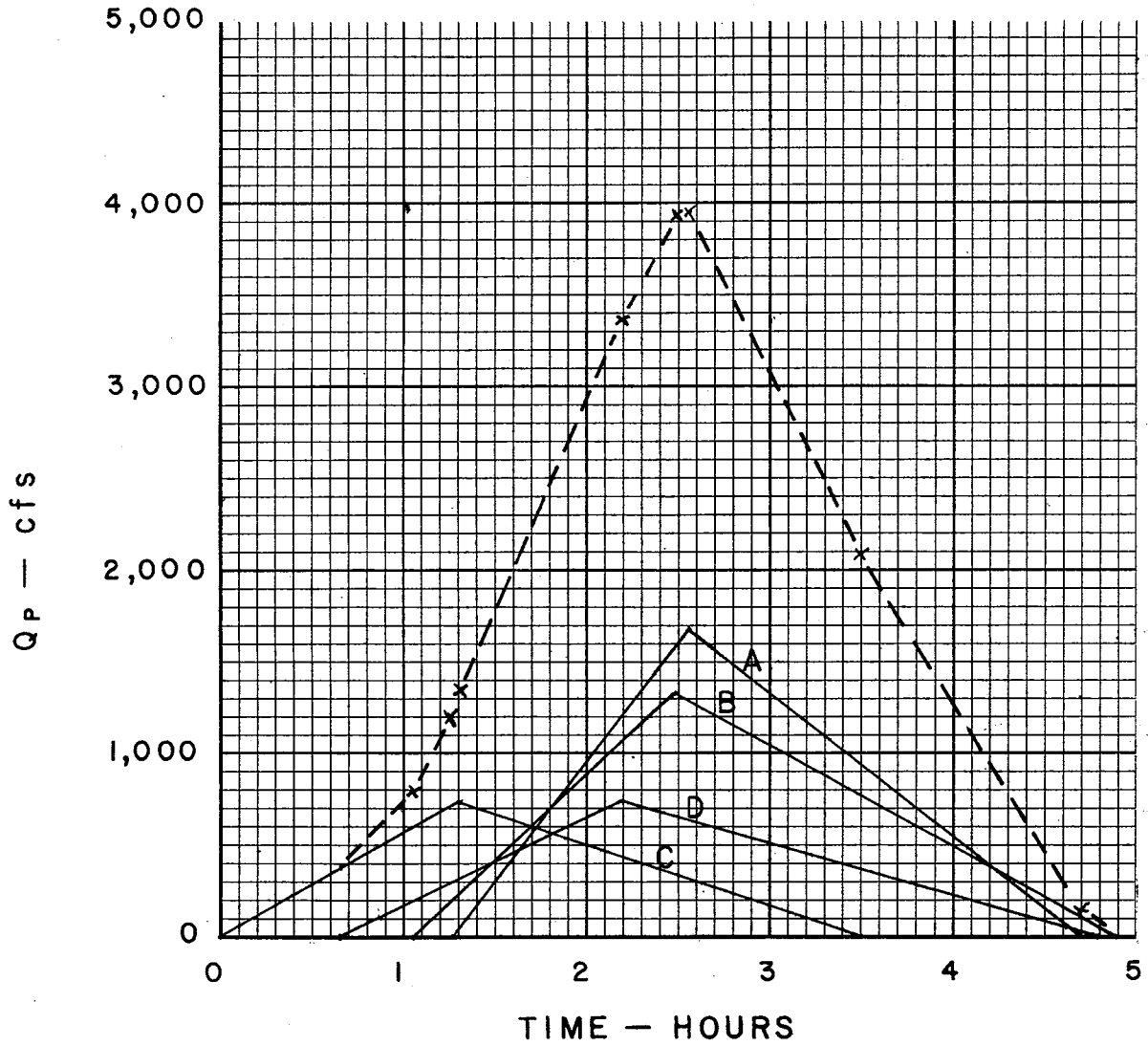


Fig. 6

THREE HOUR DURATION

TIME TO PEAK = 2.99

PEAK Q_P = 3820 cfs

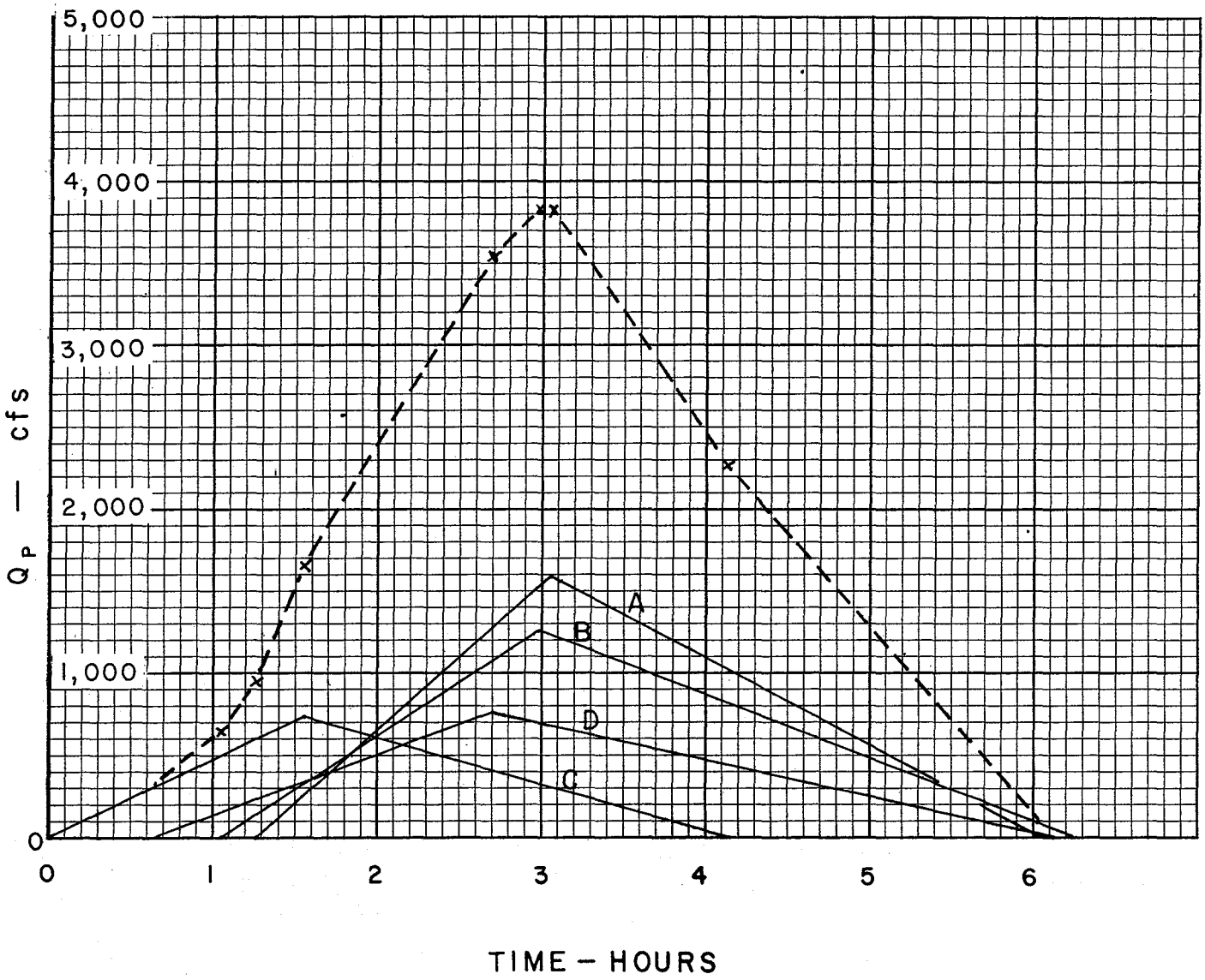


Fig. 7

SIX HOUR DURATION

Time to Peak = 4.5 Hours

Peak Q_P = 3,300 cfs

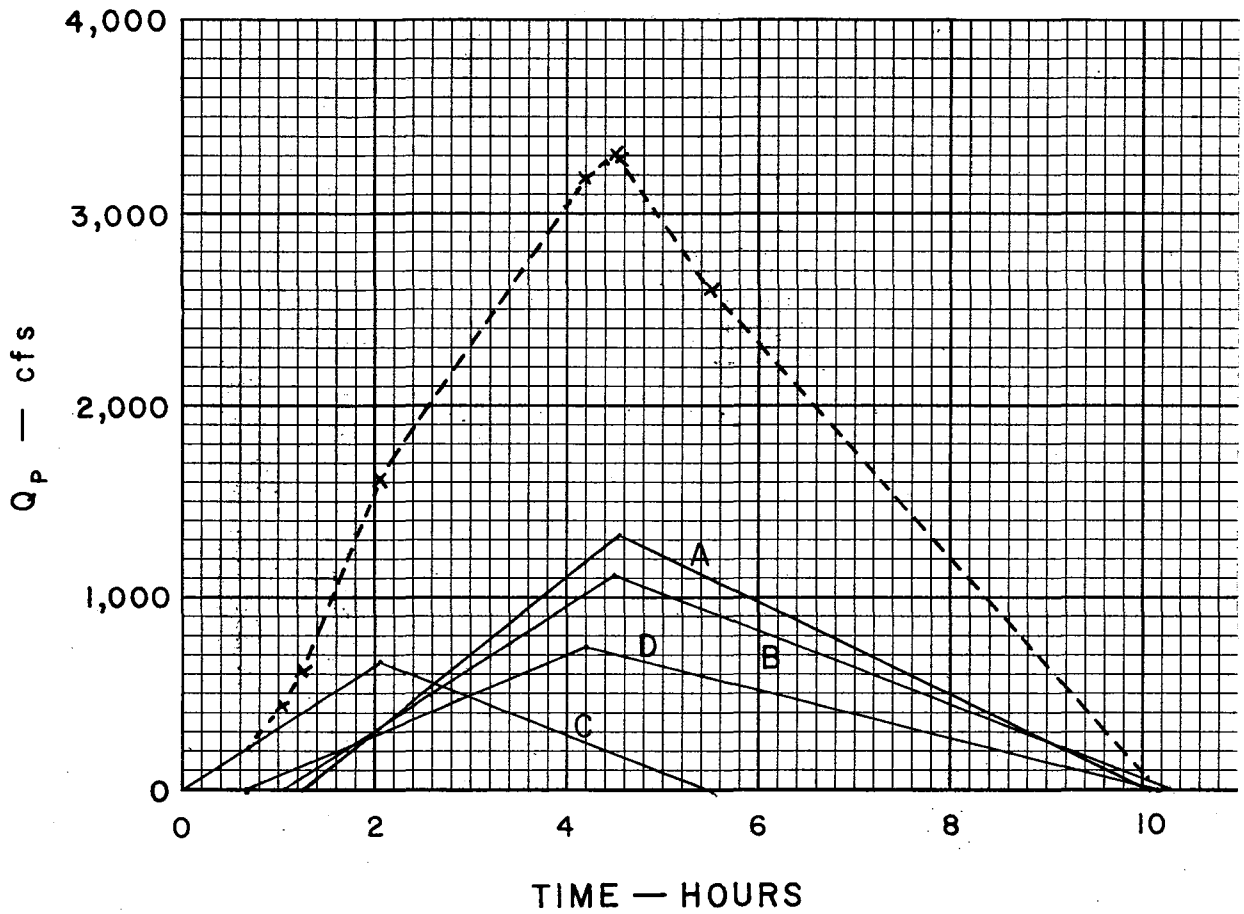


Fig. 8

THE EFFECTS OF STORAGE ON PEAK FLOWS

Flood routing is the solution to the storage equation (1), which is an expression of the continuity equation.

$$I = O + ds/dt \qquad (1)$$

Where
- I = Inflow rate
- O = Outflow rate
- S = Storage
- t = Time

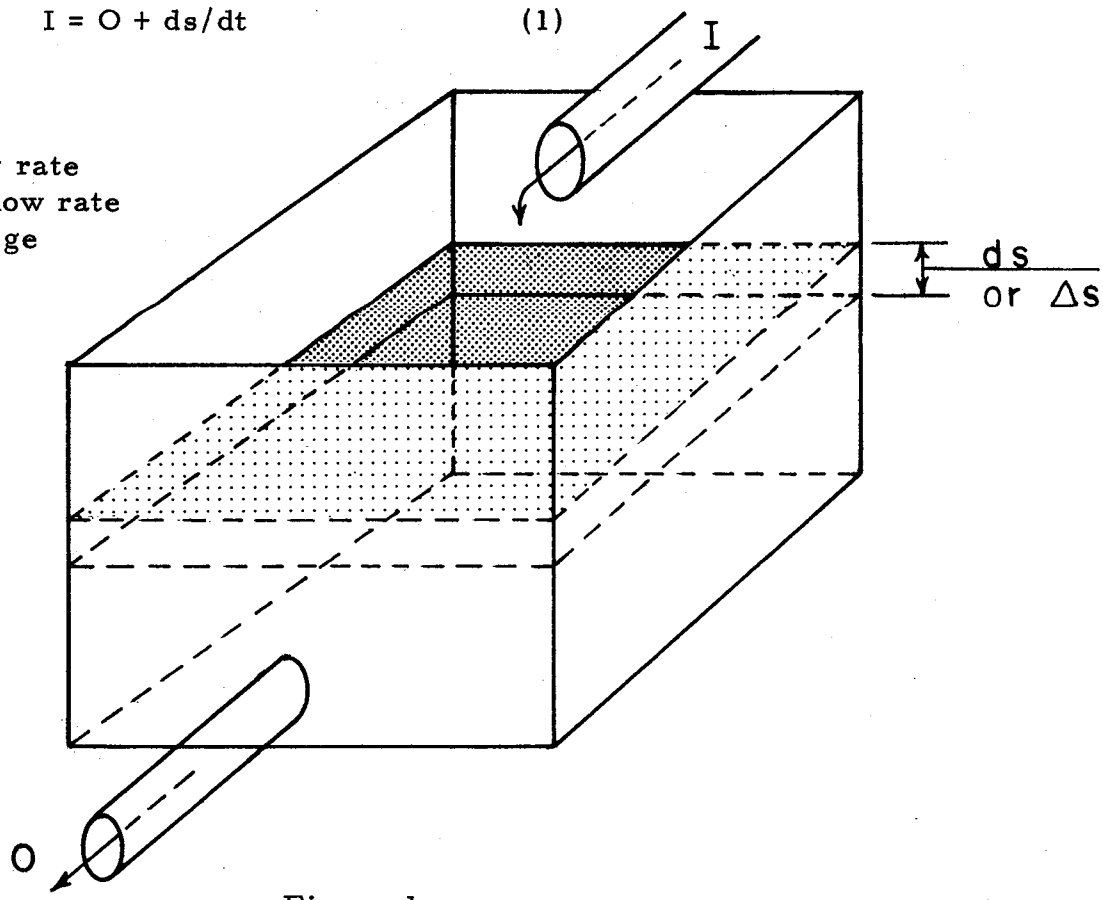


Figure 1

Figure 1 provides a crude graphical representation of the flood routing problem. During a short time interval dt or Δt, inflow and outflow occur at specified rates. The difficulty arises in determining the peak outflow rate for a given set of design conditions.

$$ds = Idt - Odt$$

for any time t

$$S \Big|_{o}^{t} = \int_{o}^{t} Idt - \int_{o}^{t} Odt$$

The equations presented will not be derived. They are provided for those who may be more interested in the theoretical aspects of the flood routing problem. A more or less empirical approach will be presented to flood routing. It should be noted that the problem is somewhat simplified by the fact that a relationship exists between storage and outflow. That is, for a given volume of storage a rate of outflow is defined.

THE AIRPORT DRAINAGE METHOD 1/

The first method to be covered is the "Airport Drainage Method" which is a graphical solution to equation (1).

for any time t $$S = I_a - O_a \qquad (2)$$

where

S = storage (ac-ft or ft^3)

I_a = Accumulated Inflow (ac-ft or ft^3)

O_a = Accumulated Outflow (ac-ft or ft^3)

What information is required to use the "Airport Drainage Method"?

1. Conventional culvert design
2. Stage storage curve
3. Stage discharge curve
4. Accumulative inflow curve

To obtain the maximum Storage, plot I_a and O_a on the same graph vs time. The average outlfow rate must be determined by a trial and error procedure. The maximum Storage occurs when the difference between the two curves is the greatest. As a rule of thunb assume the average outflow rate is equal to .8 of the maximum outflow rate.

The first example provides a typical calculation using the "Airport Drainage Method" as a Flood Routing tool.

THE STORAGE INDICATION METHOD 2/

The second method presented is the "Storage Indication Method". With this procedure we divide the inflow hydrograph into very small time increments, and use a rather complex counting method to determine when the peak rate of outflow occurs and what its magnitude is.

1/ Airport Drainage, Federal Aviation Agency, 1965.
2/ S.C.S. National Engineering Handbook, Section 4, Chapter 17, 1971.

$\overline{I}$ = the average rate of Inflow during the time interval Δt.

$\overline{O}$ = the average rate of Outflow during the time interval Δt.

ΔS = the change in Storage during the time interval Δt (Δt must be much smaller than T_p i.e. $\Delta t < .1\ T_p$)

$$I = O + ds/dt \qquad (1)$$

$$\overline{I} = \overline{O} + \Delta s/\Delta t$$

$$\Delta t(\overline{I} - \overline{O}) = \Delta s$$

$$\overline{I} = \frac{I_1 + I_2}{2} \qquad \overline{O} = \frac{O_1 + O_2}{2} \qquad \Delta S = S_2 - S_1$$

$$\Delta t\,(\overline{I}) - \frac{\Delta t}{2}\,(O_1 + O_2) = S_2 - S_1$$

$$\therefore\ \overline{I} + \left[\frac{S_1}{\Delta t} - \frac{O_1}{2}\right] = \left[\frac{S_2}{\Delta t} + \frac{O_2}{2}\right] \qquad (3)$$

What information is required to use the "Storage Indication Method"?

1. Conventional culvert design
2. Stage storage curve
3. Stage discharge curve
4. Inflow hydrograph (S. C. S., or from a unit hydrograph)

Example two provides a semi-graphical solution for equation (3) which is the basis of the Storage-Indication Method.

WHEN SHOULD STORAGE BE CONSIDERED IN DESIGN?

The available storage may have a pronounced affect upon the size of culvert required. If the ratio of maximum available storage to the one hour runoff volume is greater than about .20, the sizing of the culvert from B. P. R. Charts may result in unnecessary over design. It is recommended that this rule of thumb be used to determine when storage need be considered in design.

WHICH METHOD OF FLOOD ROUTING SHOULD BE USED?

It is recommended that the "Airport Drainage Method" be used when the Rational Method is used for hydrologic design. However, the drainage area should be small (i. e. less than 1. 0 sq. mi.) and the Time of Concentration short (i. e. much less than 60 min.).

The Storage Indication Method should be used when the S. C. S. Method is used for hydrologic design. There are no other limitations except that an inflow hydrograph must be available.

Rather than attempting to provide a cookbook type of format, two detailed design examples which illustrate both methods of flood routing are presented. These examples are to be used as a guide when flood routing calculations are required.

Example No. 1. Flood Routing by the "Airport Drainage Method"

Assume it is your responsibility to provide the drainage recommendations for a project on Highway 60 near Seneca. Assume the existing highway is to be improved and the existing drainage structures are to be extended. Using standard culvert design procedures you determine that the existing CMP is not adequate. High fills make the construction of an additional culvert relatively expensive. The field conditions allow high ponding. The effect of flood routing is to be investigated.

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway Hwy. 60 County Gila
Location Near Seneca
Project No. F-***-*** Station 800±
Name of Stream Unknown

DESIGN DATA

Design Frequency		50	years
Drainage Area	A_1	48.0	acres
	A_2		acres
	A_3		acres
Drainage Length		1500	feet
Elevation			
Top of Drainage Area		4800	feet
At Structure		4620	feet
Drainage Area Slope		12.0	%
Precipitation			
P = 6-hour		2.9	inches
P = 24-hour		4.0	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		1.8	inches
			use 10 as min.
Time of Concentration	Tc	5.0	minutes
Rainfall Intensity	i	4.90	inches/hour
Runoff Coefficient	C_1	.55	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	.55	

Peak Discharge Qp = CiA = (.55)(4.90)(48.0) = 129 cfs use 130 cfs

Computed by R.C.J. Date 9-14-72

Checked by C.D.

AHD 20-047 R5-71

CULVERT COMPUTATIONS (FOR EXTENSIONS TO EXISTING CULVERTS ONLY)

ARIZONA HIGHWAY DEPARTMENT

PROJECT: F-XXX-XXX

STATION: 800 ±

DESIGNER: R.C.J.

DATE: 9-14-72

HYDROLOGIC AND CHANNEL INFORMATION

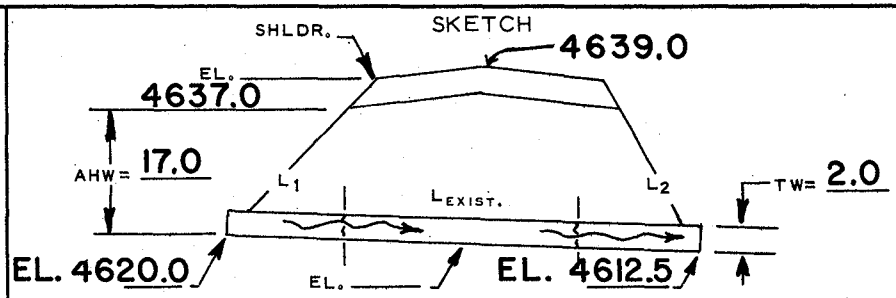
DRAINAGE AREA: 48 Ac.

Q_1 = 130 cfs
Q_2 =

TW_1 =
TW_2 = 2.0 ft.

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

SKETCH



S_{O_1} = .06 L_1 = 15

$S_{O_{EXIST.}}$ = .06 $L_{EXIST.}$ = 85

S_{O_2} = .06 L_2 = 25

L/100 S_O ≈

MEAN STREAM VELOCITY =

CULVERT DESCRIPTION		Q	CAP. CHART HW	HEADWATER COMPUTATION										CONTROL HW	OUTLET VELOCITY	COST	DHW ELEV.	COMMENTS
				INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$												
SIZE	ENTR. TYPE			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c + D}{2}$	TW	H_o	LS_o	HW					
36"	end sect.	130		5.0	15.0	.5	24.1*	3.0	3.0	2.0	3.0	7.5	19.6	19.6		Too high, Exist. C.M.P. is not adequate.		
2-36"	end sect.	65		1.75	5.2	.5	6.3	2.6	2.8	2.0	2.8	7.5	1.6	5.2	13.7	Add new barrel.		

*off chart use formula $H = \left[1 + .5 + \frac{29\,(.024)^2\,125}{(.75)^{1.33}}\right]\frac{18.4^2}{64.3}$

BPR No. 5 Page 5-6

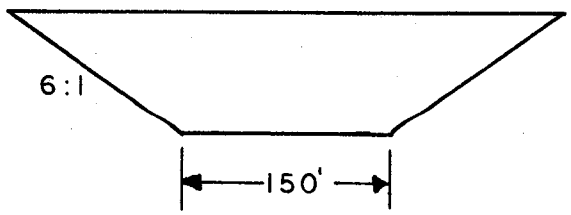
SUMMARY AND RECOMMENDATIONS New barrel is required, because of high fill new barrel will be expensive and will cause traffic delay. Check to see if enough storage is available to justify a flood routing calculation.

AHD 20-0447 3-72

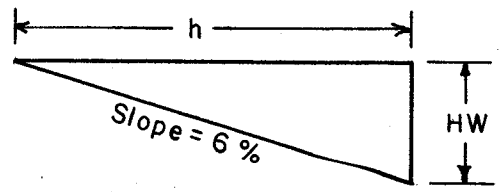
Is there enough storage available to justify a flood routing calculation?

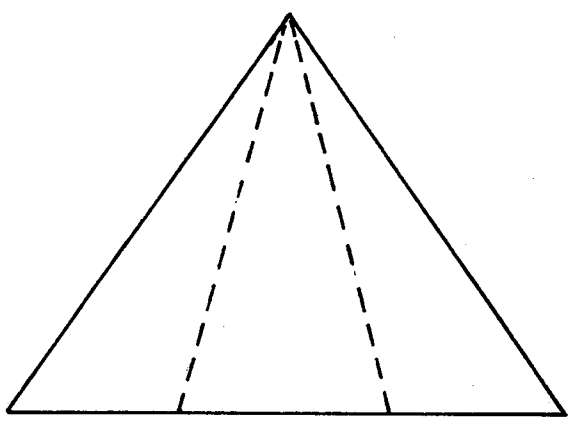
Assume the cross section can be generalized by the following:

(Cross section at Roadway)



(Profile)





(Plan)

therefore the storage is in a pyramid shape

Vol = 1/3 Ah

Max. HW = 17 ft.

h = 17/.06 = 283.

A = (150)(17) + $6(17)^2$ = 4284 ft.^{2}

V = 1/3 (283)(4284) = 404,000 ft.^{3}

Volume of Runoff for a one hour storm.

= (C)(P_1)(A)(43560)/12

= (.55)(1.8)(48)(43560)/12

= 172,500 ft.^{3}

Maximum Storage /one hour Runoff Volume

= 404,000 /172,500 = 2.3 = S/R

Since S/R >> .2 flood routing is justified.

In this example the entire runoff event can be stored; flood routing is not only justified it is essential.

From Contour map, or field survey or from generalized shape of basin:

Development of Stage-Storage Curve

Stage Elevation	Volume of Storage (ft.^{3})
4620	0
21	870
22	3,600
23	8,400
4625	25,000
27.5	61,000
4630	117,000
32.5	195,000
4635	300,000
37	404,000

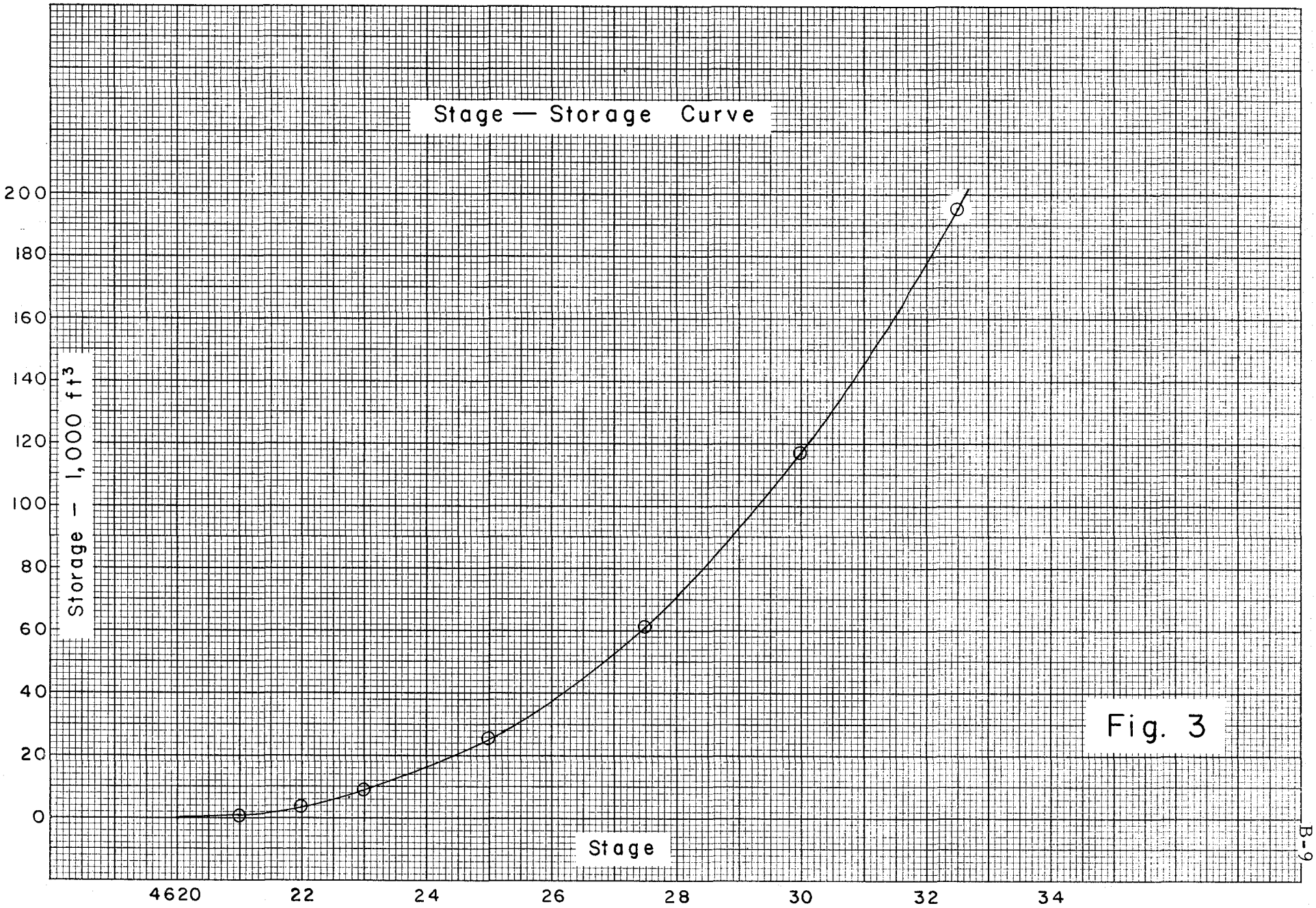

Stage — Storage Curve
Storage — 1,000 ft³
200
180
160
140
120
100
80
60
40
20
0
Stage
4620
22
24
26
28
30
32
34

Fig. 3

ARIZONA HIGHWAY DEPARTMENT — Developement of Stage - Discharge Curve — CULVERT COMPUTATION SHEET

PROJECT: F-***-***

STATION: 800±

DESIGNER: R.C.J.

DATE: 9-15-72

HYDROLOGIC AND CHANNEL INFORMATION

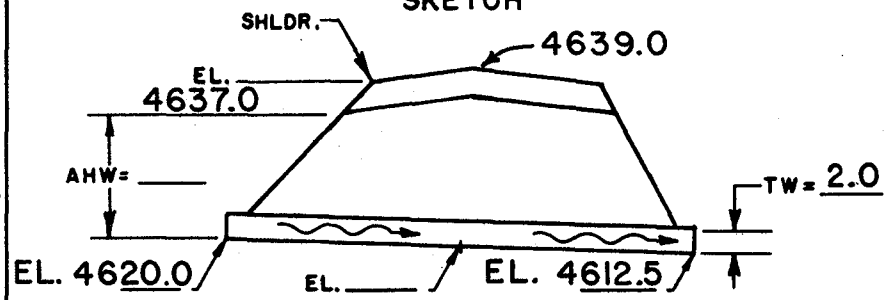
DRAINAGE AREA 48 Ac.

Q_1 = ____
Q_2 = 130 cfs

TW_1 = ____
TW_2 = 2.0 for 130 cfs

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

SKETCH



S_0 = .06 L = 125 L/100 S_0 = ____

MEAN STREAM VELOCITY = ____

CULVERT DESCRIPTION		Q	CAP. CHART HW	HEADWATER COMPUTATION										CONTROL HW	OUTLET VELOCITY	COST	DHW ELEV.	COMMENTS
				INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$												
SIZE	ENTR. TYPE			HW/D	HW	K_e	H	d_c	$\frac{d_c + D}{2}$	TW	h_o	LS_o	HW					
36"	h/w	20		.7	2.1	.5	.6	1.4	2.2		2.2	7.5	—	2.1			4622.1	
36"	h/w	40		1.10	3.3	.5	2.4	2.1	2.6		2.6	7.5	—	3.3			4623.3	
36"	h/w	60		1.60	4.8	.5	5.3	2.5	2.8		2.8	7.5	.6	4.8			4624.8	
36"	h/w	80		2.35	7.1	.5	9.5	2.8	2.9		2.9	7.5	4.9	7.1			4627.1	Inlet Control
36"	h/w	100		3.35	10.0	.5	15.0	3.0	3.0		3.0	7.5	10.5				4630.5	Outlet Control

SUMMARY & RECOMMENDATIONS

AHD 20-050 (12-69)

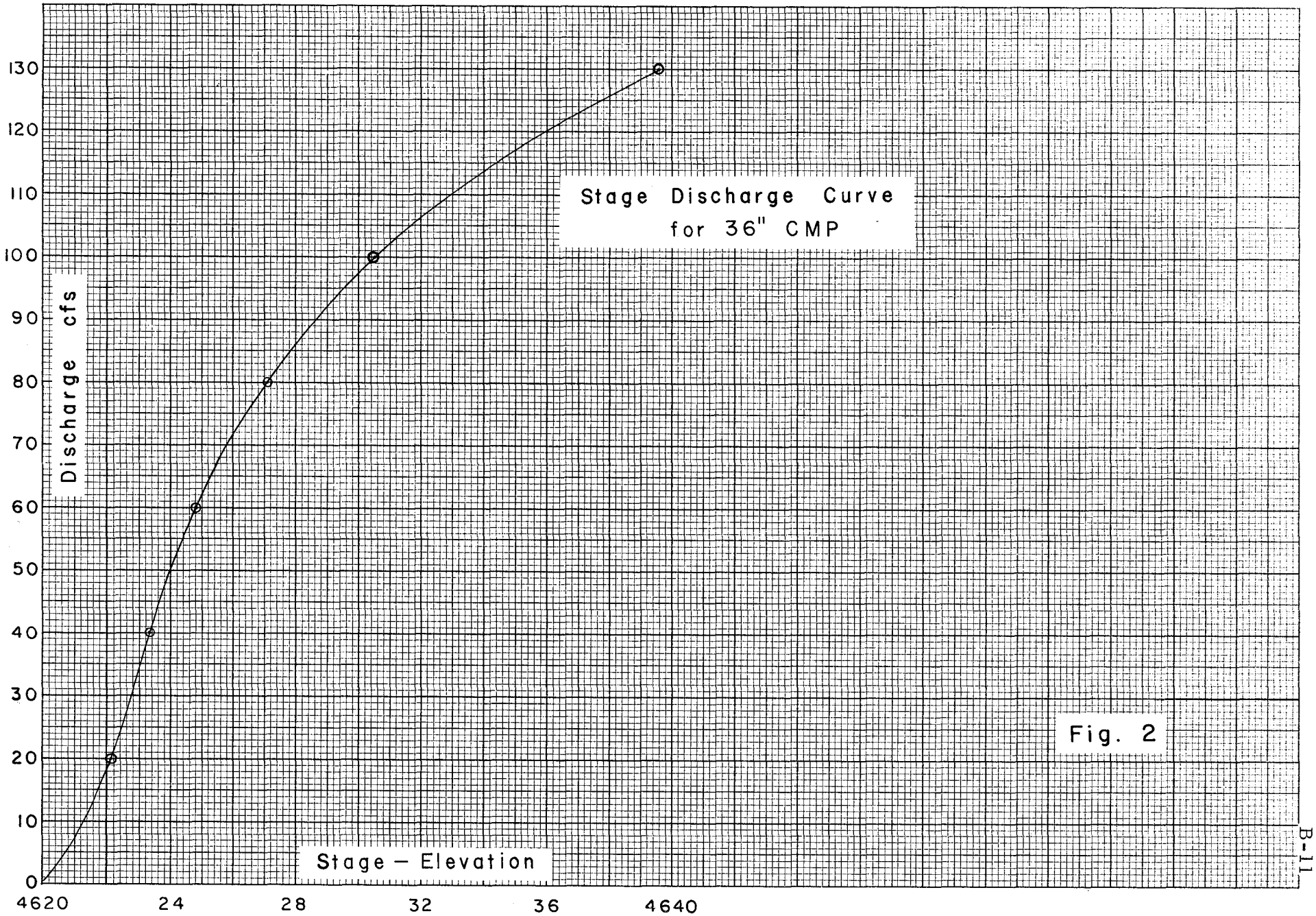

Stage Discharge Curve
for 36" CMP
Discharge cfs
0
10
20
30
40
50
60
70
80
90
100
110
120
130
Stage — Elevation
4620
24
28
32
36
4640

Fig. 2

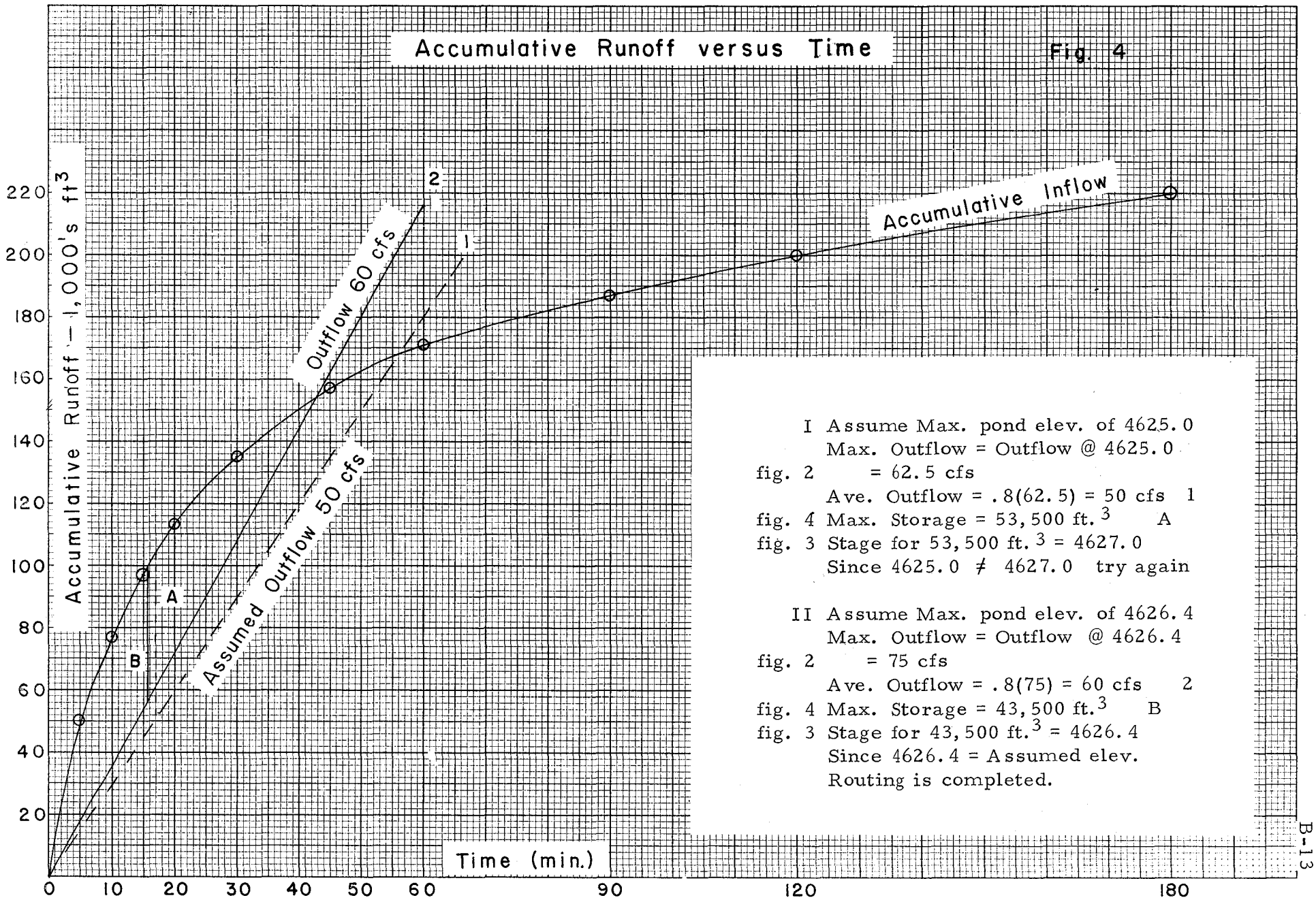
Intensities for Storm Durations 3/
(5m - 180m)

Duration	Intensity
5	6.26
10	4.86
15	4.10
20	3.55
30	2.84
45	2.20
60	1.80
90	1.31
120	1.05
180	.77

Development of Accumulative Runoff v. s. Time

Minutes	CA	I (in/hr)	Time (sec)	Accumulative Runoff Vol. (ft. ^{3})
5	26.4	6.26	300	50,000
10	26.4	4.86	600	77,000
15	26.4	4.10	900	97,000
20	26.4	3.56	1200	113,000
30	26.4	2.84	1800	135,000
45	26.4	2.20	2700	157,000
60	26.4	1.80	3600	171,000
90	26.4	1.31	5400	187,000
120	26.4	1.05	7200	200,000
180	26.4	.77	10800	220,000

3/ For a 50 yr. frequency



I Assume Max. pond elev. of 4625.0
Max. Outflow = Outflow @ 4625.0
fig. 2 = 62.5 cfs
Ave. Outflow = .8(62.5) = 50 cfs 1
fig. 4 Max. Storage = 53,500 ft.^{3} A
fig. 3 Stage for 53,500 ft.^{3} = 4627.0
Since 4625.0 $\neq$ 4627.0 try again

II Assume Max. pond elev. of 4626.4
Max. Outflow = Outflow @ 4626.4
fig. 2 = 75 cfs
Ave. Outflow = .8(75) = 60 cfs 2
fig. 4 Max. Storage = 43,500 ft.^{3} B
fig. 3 Stage for 43,500 ft.^{3} = 4626.4
Since 4626.4 = Assumed elev.
Routing is completed.

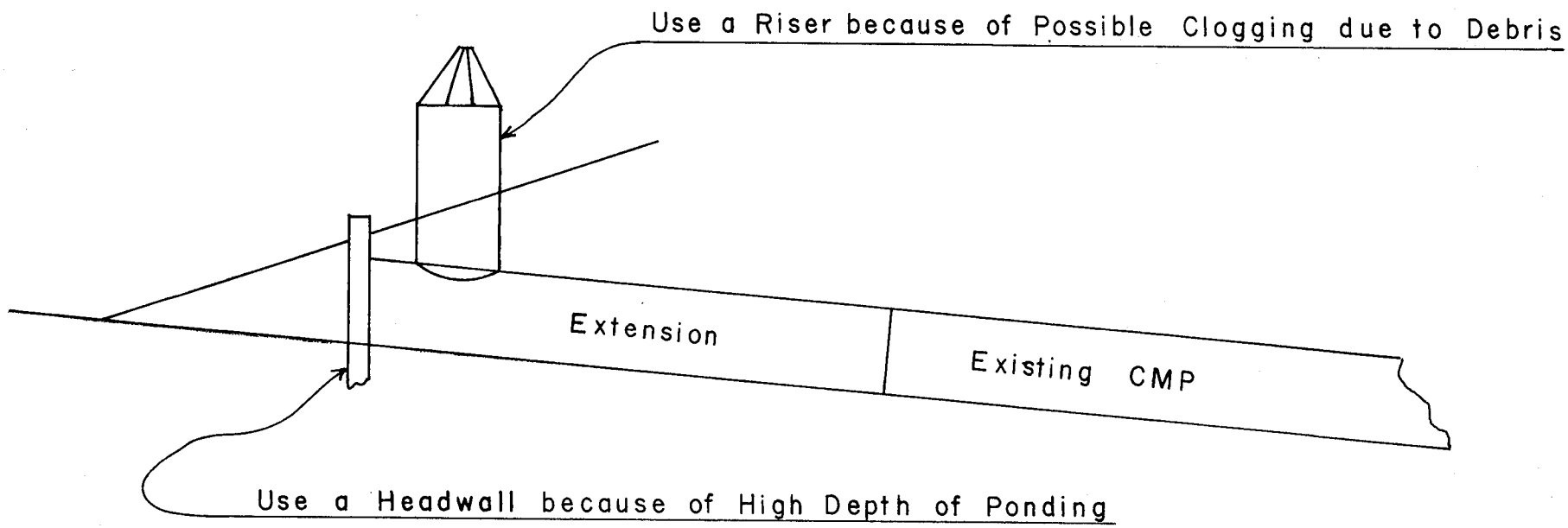
Results of Routing

Maximum pond elevation	=	4626.4
Max. rate of outflow	=	75
Size of Culvert	=	36 cmp
Outlet Velocity	=	14 fps 4/

This flood routing technique has shown the existing culvert to be adequate. The inlet treatment on the following sheet should be considered.

4/ Suggest rip-rap outlet protection be considered

Inlet Treatment

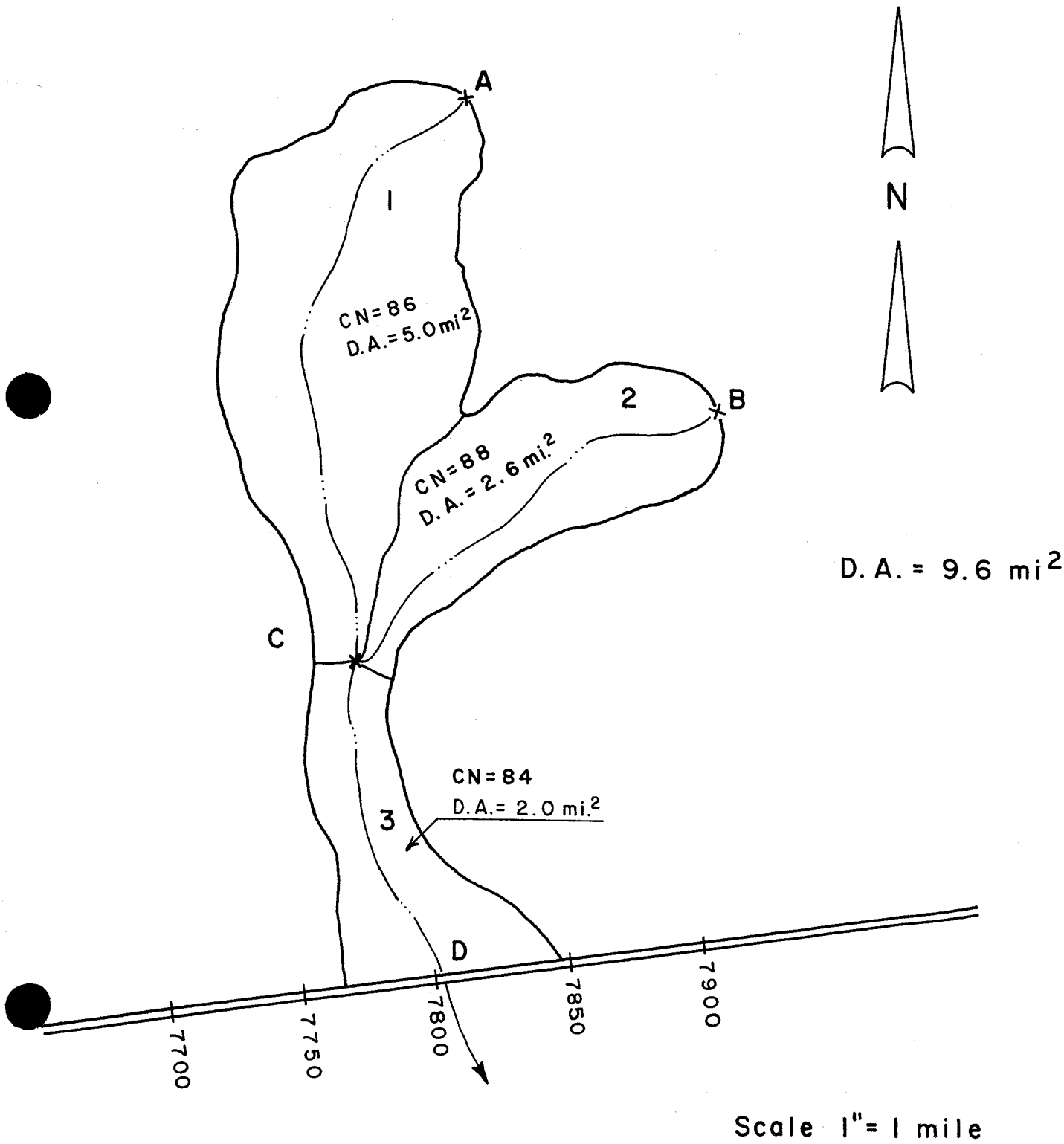


For details see BPR H.E.C. No. 9, Plate 10.

Example #2 Flood Routing by the "Storage Indication Method"

Assume you are responsible for the drainage design for a CBC on Interstate ** at Sta 7802±. Standard culvert design calculations indicate that a 7 span 10' x 5' x 226' CBC is required. Appreciable storage appears to be available. Since this structure would be expensive, the possibility of utilizing Flood Routing techniques to reduce the peak rate of discharge should be investigated. The following example provides the design calculations required by the Storage Indication Method.

Drainage Area Map



ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway Phoenix - Ehrenberg County Maricopa
Location West of ******
Project No. I-10-***(**) Station 7802+
Name of Stream Unknown D.A. #1

DESIGN DATA:

Design Frequency	50	years
Drainage Area	5.0	square miles
Drainage Length	23 500	feet A-C
Elevation		
Top of Drainage area	2,500	feet
At Structure	2,145	feet
Drainage Area Slope	1.5	%
Vegetative Cover Type	Desert Brush	
Vegetative Cover Density	15-20	%
Soil Group	B-C	
Precipitation		
P = 6 hour =	2.9	inches
P = 24 hour =	3.7	inches

DESIGN COMPUTATION:

Curve Number	86	
Time of Concentration	1.48	hours
Peak Design Discharge		
$Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	1600	cfs

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge	T_p
1 hour	2.10		2.10	.92	1600	1.39
2 hour	2.28		2.28	1.06	1360	1.89
3 hour	2.46		2.46	1.21	1225	2.39
6 hour	2.90		2.90	1.58	985	3.89

Computed by: R.C.J. Date 9-19-72

Checked by: C.D.

AHD 20-049 11-68

Lag = .6(1.38)

WATERSHED LAG

PRESENT CONDITION

23500.	
1.5	
86.	
2.65	LAG
4.42	TC

ADOT HYDROLOGY

3.7	P24
2.9	P6
2.46	P3
2.284	P2
2.1	P1
5.	DA
23500.	LONG
5931.574468	WIDE
2500.	HIGH
2145.	LOW
1.510638298	SLOP
86.	CN
.9254144876	RO1
1.069452102	RO2
1.210884779	RO3
1.48	TC
1.3172	TP
1700.199711	Q1M1
1613.474827	Q1M2
1370.801953	Q2M2
1227.111041	Q3M2

JRF

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway Phoenix - Ehrenberg County Maricopa
Location West of ******
Project No. I-10-***(**) Station 7802±
Name of Stream Unknown D.A. #2

DESIGN DATA:

Design Frequency	50	years
Drainage Area	2.6	square miles
Drainage Length	17,500	feet B-C
Elevation		
Top of Drainage area	3,895	feet
At Structure	2,145	feet
Drainage Area Slope	10.0	%
Vegetative Cover Type	Desert Brush	
Vegetative Cover Density	20	%
Soil Group	C-D	
Precipitation		
P = 6 hour =	2.9	inches
P = 24 hour =	3.7	inches

DESIGN COMPUTATION:

Curve Number	89.5	
Time of Concentration	.58	hours
Peak Design Discharge		
$Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	1690	cfs

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge	T_p
1 hour	2.10		2.10	1.14	1690	.85
2 hour	2.28		2.28	1.30	1220	1.35
3 hour	2.46		2.46	1.45	990	1.85
6 hour	2.90		2.90	1.85	695	3.35

Computed by: R.C.J. Date 9-19-72
Checked by: C.D.

AHD 20-049 11-68

Lag = .6(1.38)

#2

ADOT HYDROLOGY

3.7	P24
2.9	P6
2.46	P3
2.284	P2
2.1	P1
2.6	DA
17500.	LONG
4141.933714	WIDE
3895.	HIGH
2145.	LOW
10.	SLOP
89.5	CN
1.145145704	RO1
1.303282341	RO2
1.457164001	RO3
0.58	TC
0.5162	TP
2791.653147	Q1M1
1699.353012	Q1M2
1216.654672	Q2M2
992.2592962	Q3M2

JRW

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway Ehrenberg - Phoenix County Maricopa
Location West of *********
Project No. I-10-***(**) Station 7802±
Name of Stream Unknown D.A. #3

DESIGN DATA:

Design Frequency	50	years
Drainage Area	2.0	square miles
Drainage Length	12,500	feet C-D
Elevation		
Top of Drainage area	2,145	feet
At Structure	2,080	feet
Drainage Area Slope	.50	%
Vegetative Cover Type	Desert Brush	
Vegetative Cover Density	5	%
Soil Group	B	
Precipitation		
P = 6 hour =	2.9	inches
P = 24 hour =	3.7	inches

DESIGN COMPUTATION:

Curve Number	84	
Time of Concentration	1.38	hours
Peak Design Discharge		
$Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	600	cfs

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge	T_p
1 hour	2.10		2.10	.82	600	1.33
2 hour	2.28		2.28	.95	500	1.83
3 hour	2.46		2.46	1.09	450	2.33
6 hour	2.90		2.90	1.44	365	3.83

Computed by: R.C.J. Date 9-19-72
Checked by: C.D.

AHD 20-049 11-68

Lag = 0

#3

ADOT HYDROLOGY

3.7	P24
2.9	P6
2.46	P3
2.284	P2
2.1	P1
2.	DA
12500.	LONG
4460.544	WIDE
2145.	HIGH
2080.	LOW
0.52	SLOP
84.	CN
.8154746261	RO1
0.951095431	RO2
1.085001423	RO3
1.38	TC
1.2282	TP
642.7124557	Q1M1
594.4122275	Q1M2
503.6435324	Q2M2
451.1517944	Q3M2

JRW

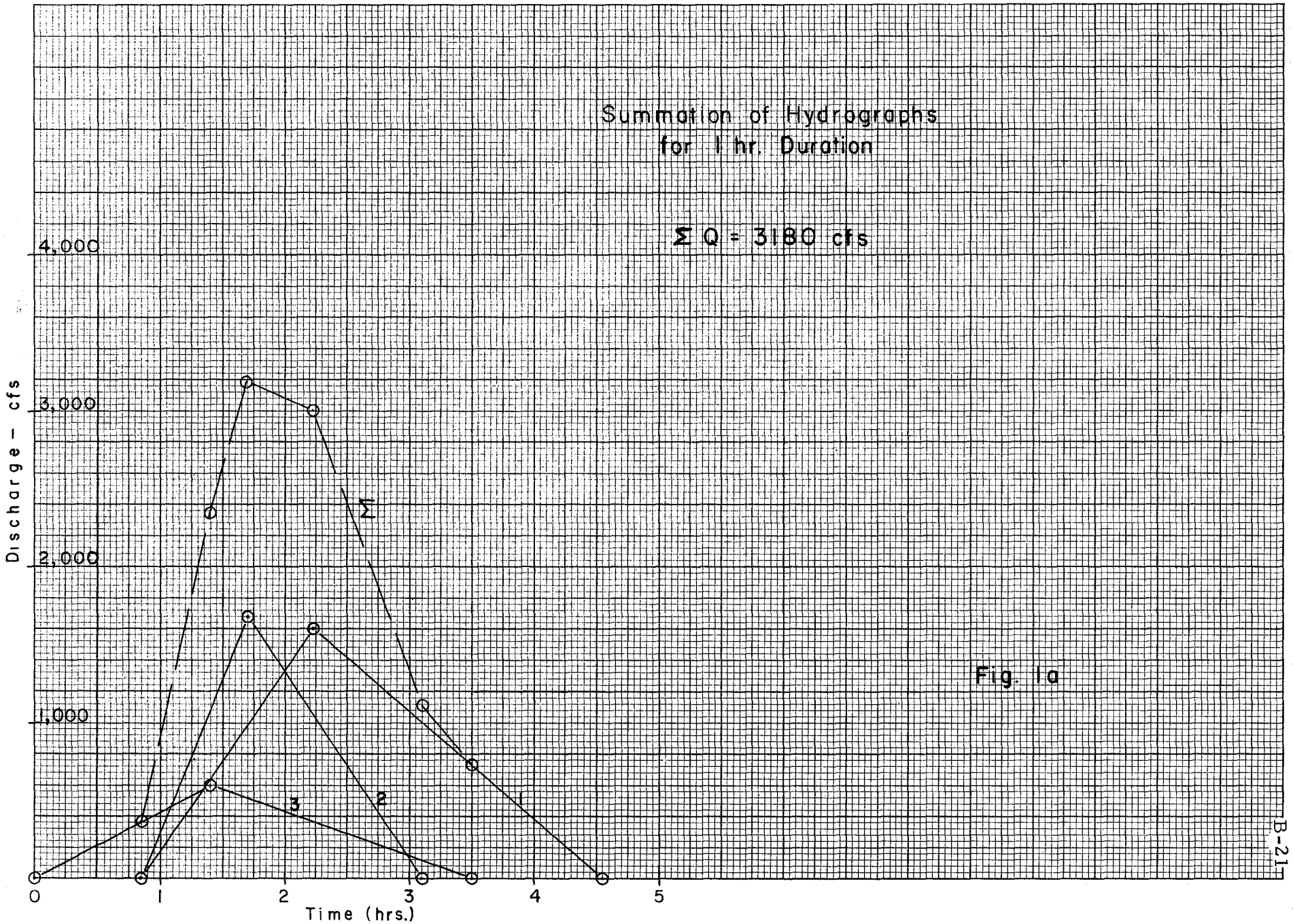

Summation of Hydrographs
for 1 hr. Duration
Σ Q = 3180 cfs
4,000
3,000
2,000
1,000
Discharge – cfs
Σ
1
2
3
0
1
2
3
4
5
Time (hrs.)

Fig. 1a

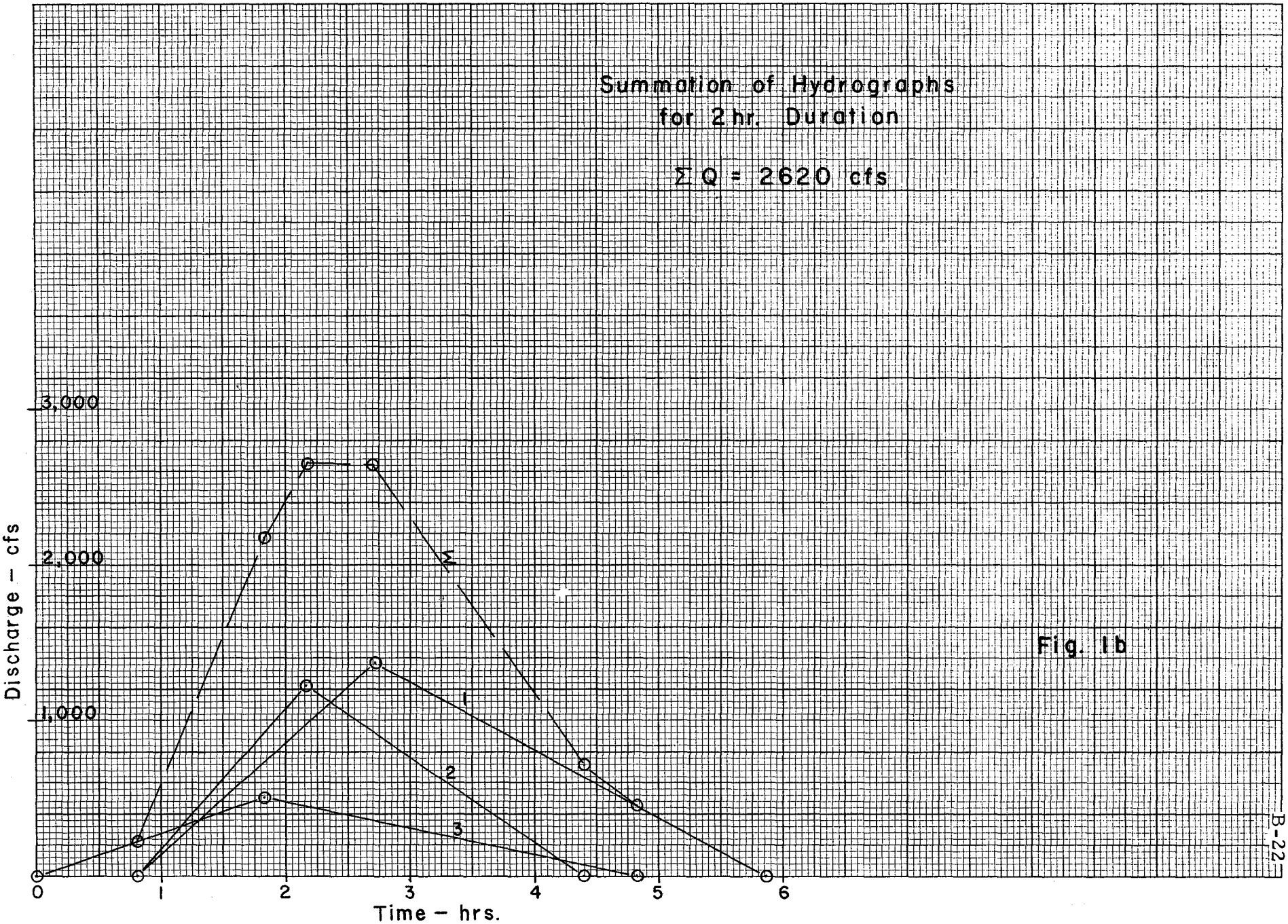

Summation of Hydrographs
for 2 hr. Duration
ΣQ = 2620 cfs
3,000
2,000
1,000
Discharge – cfs
0
1
2
3
4
5
6
Time – hrs.
Σ
1
2
3

Fig. 1b

ARIZONA HIGHWAY DEPARTMENT

CULVERT COMPUTATION SHEET

PROJECT: I-10-***(**)

STATION: 7802±

DESIGNER: R.C.J.

DATE: 9-19-72

HYDROLOGIC AND CHANNEL INFORMATION

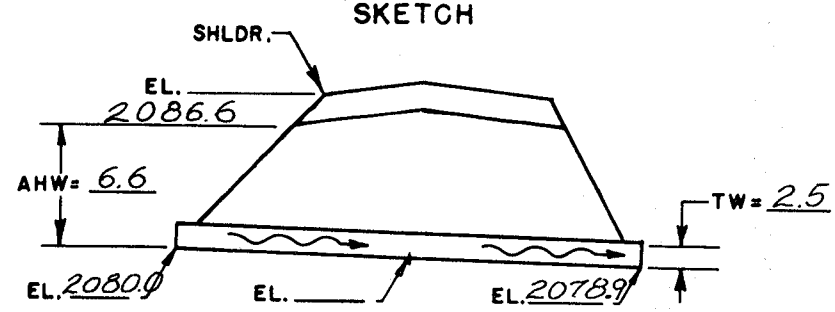
DRAINAGE AREA 9.6 mi²

Q_1 = ____ TW_1 = ____

Q_2 = 3180 cfs. TW_2 = 2.5

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

SKETCH



S_0 = .005 L = 226 L/100 S_0 = ____

MEAN STREAM VELOCITY = ____

CULVERT DESCRIPTION		Q	CAP. CHART HW	HEADWATER COMPUTATION										CONTROL HW	OUTLET VELOCITY	COST	DHW ELEV.	COMMENTS
				INLET CONT.		OUTLET CONTROL $HW = H + h_0 - LS_0$												
SIZE	ENTR. TYPE			HW/D	HW	K_e	H	d_c	$\frac{d_c + D}{2}$	TW	h_0	LS_0	HW					
6 10 x 5	CBC @ H/W	530		1.55	7.68	.4					Too	High						
7 10x5	CBC @ H/W	454		1.32	6.60	.4	2.5	4.0	4.5	2.5	4.5	1.13	5.9	6.6	13.8		2086.6	OK

SUMMARY & RECOMMENDATIONS Check to see if flood routing can be used to reduce the peak rate of discharge and the size of the structure.

AHD 20-050 12-69

<u>Is there enough storage available to justify the use of a flood routing procedure?</u>

Storage Available to elev. 2086.6 = 180 Af.

Vol. of runoff for 1 hr. storm

D.A. #1

(5.0)(640)(.92)/12

= 245

D.A. #2

(2.6)(640)(1.14)/12

= 158

D.A. #3

(2.0)(640)(.82)/12

= 87

Total = 490 Af

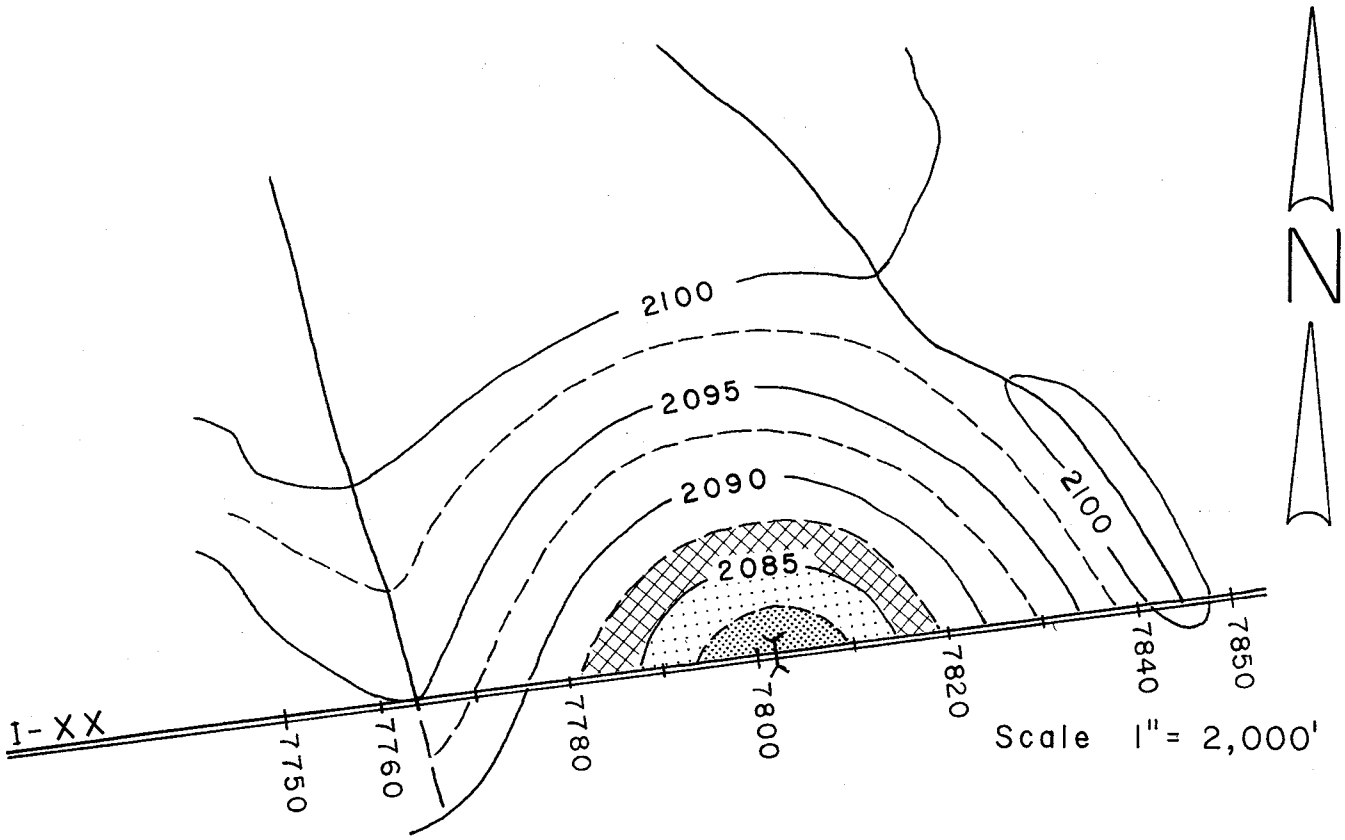
Storage / Runoff Vol. = 180/490

= .36

∴ Flood Routing is justified (i.e. S/R > .20)
Use Storage Indication Method

For a first try assume a four span 10 x 5 x 226 C.B.C.

Ponding Area Map



Stage-Storage Relationship from Ponding Area Map

Contour		Ponding Area	Volume for Pond Elevations 2082.5	2085.0	2087.5
2080.0					
		12 Ac.	15 ac-ft	45 ac-ft	75 ac-ft
2082.5					
		32 Ac.		40 ac-ft	120 ac-ft
2085.0					
		45 Ac.			56 ac-ft
2087.5					
	Totals		15 ac-ft	85 ac-ft	251 ac-ft

Stage – Storage Curve

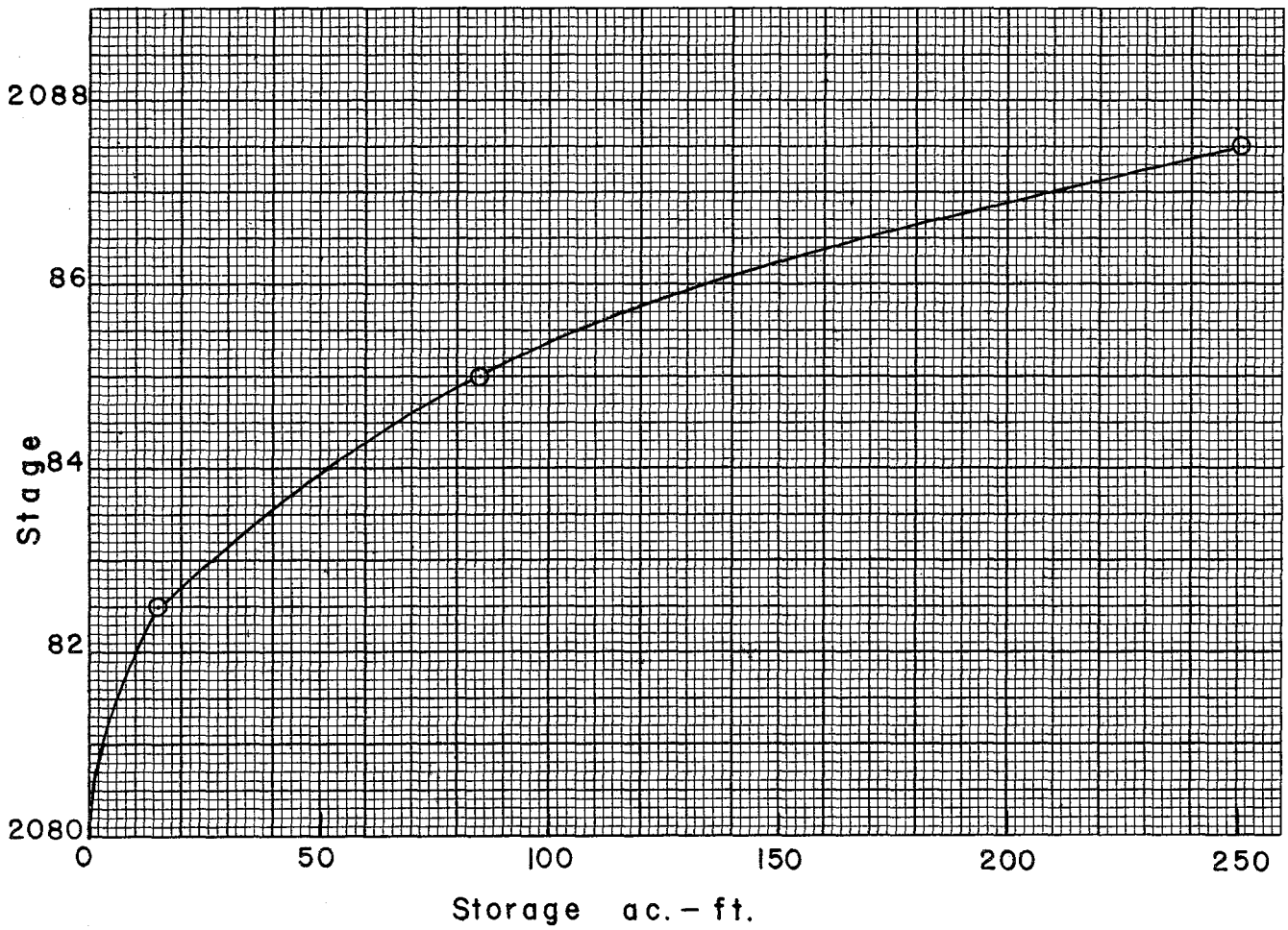


Fig. 2

ARIZONA HIGHWAY DEPARTMENT

CULVERT COMPUTATION SHEET

PROJECT: I-10-*** (**) Development of Stage-Discharge Curve DESIGNER: R.C.J.

STATION: 7802 ± DATE: 9-19-72

HYDROLOGIC AND CHANNEL INFORMATION

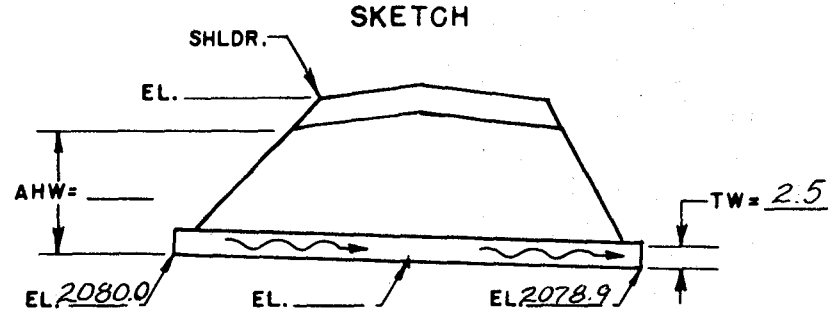
DRAINAGE AREA 9.6 mi^2

Q_1 = _____ TW_1 = _____

Q_2 = _____ TW_2 = 2.5 max.

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

SKETCH



S_0 = .005 L = 226 L/100 S_0 = _____

MEAN STREAM VELOCITY = _____

CULVERT DESCRIPTION		Q/barrel	CAP. CHART HW	HEADWATER COMPUTATION										CONTROL HW	OUTLET VELOCITY	COST	DHW ELEV.	COMMENTS
				INLET CONT.		OUTLET CONTROL $HW = H + h_0 - LS_0$												
SIZE	ENTR. TYPE			HW/D	HW	K_e	H	d_c	$\frac{d_c + D}{2}$	TW	h_0	LS_0	HW					
10x5	CBC @ H/W & W/W	50		.28	1.40	.4								1.4			2081.4	
		100		.44	2.20	.4								2.2			2082.2	
		150		.58	2.80	.4								2.95			2082.95	
		250		.83	4.15	.4								4.2			2084.2	
		350		1.05	5.25	.4								5.3			2085.3	
		400		1.19	5.95	.4								5.9			2085.9	
		500		1.45	7.25	.4	3.0	4.27	4.64	2.5	4.64	1.13	6.3	7.2	14.3		2087.2	

SUMMARY & RECOMMENDATIONS

AHD 20-050 12-69

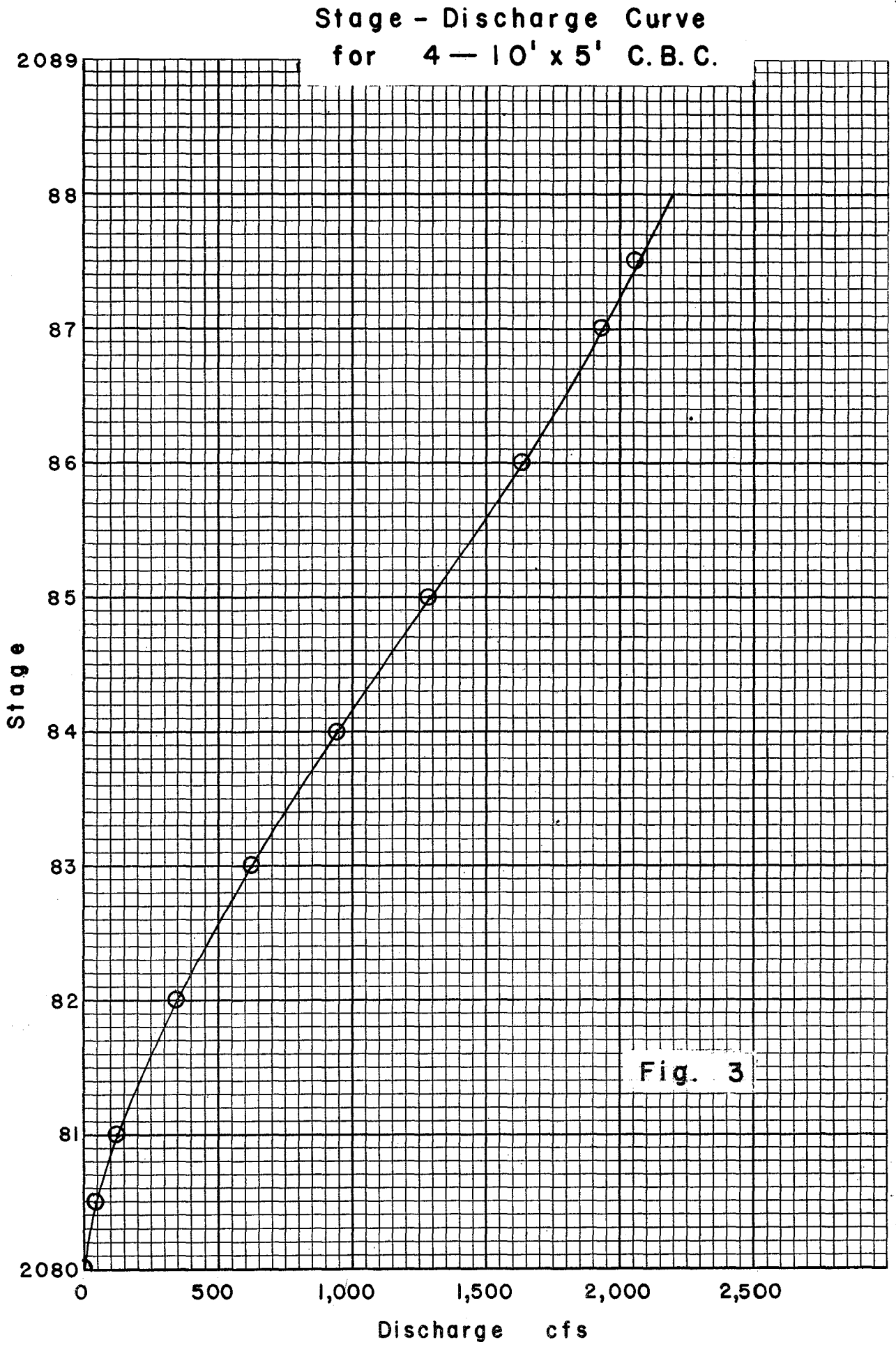

Stage - Discharge Curve
for 4 — 10' x 5' C.B.C.
2089
88
87
86
85
84
83
82
81
2080
Stage
0
500
1,000
1,500
2,000
2,500
Discharge cfs
Fig. 3

Table 1

Working Table with $\Delta t = .2$ hrs.

	From Fig. 3		From Fig. 2			
Stage elev.	O_2 cfs	$O_2/2$ cfs	S_2 ac-ft	S_2 cfs-hrs*	$S_2/\Delta t$ cfs	$S_2/\Delta t + O_2/2$ cfs
2080	0	0	0	0	0	0
2080.5	50	25	0.5	6	30	55
2081	120	60	3	36	180	240
2082	350	175	10	121	605	780
2083	620	310	27	327	1635	1945
2084	945	473	52	629	3145	3618
2085	1290	645	85	1029	5145	5790
2086	1640	820	134	1621	8105	8925
2087	1930	965	210	2541	12705	13670
2087.5	2060	1030	251	3037	15185	16215

* cfs hrs. = 12.1 x ac-ft

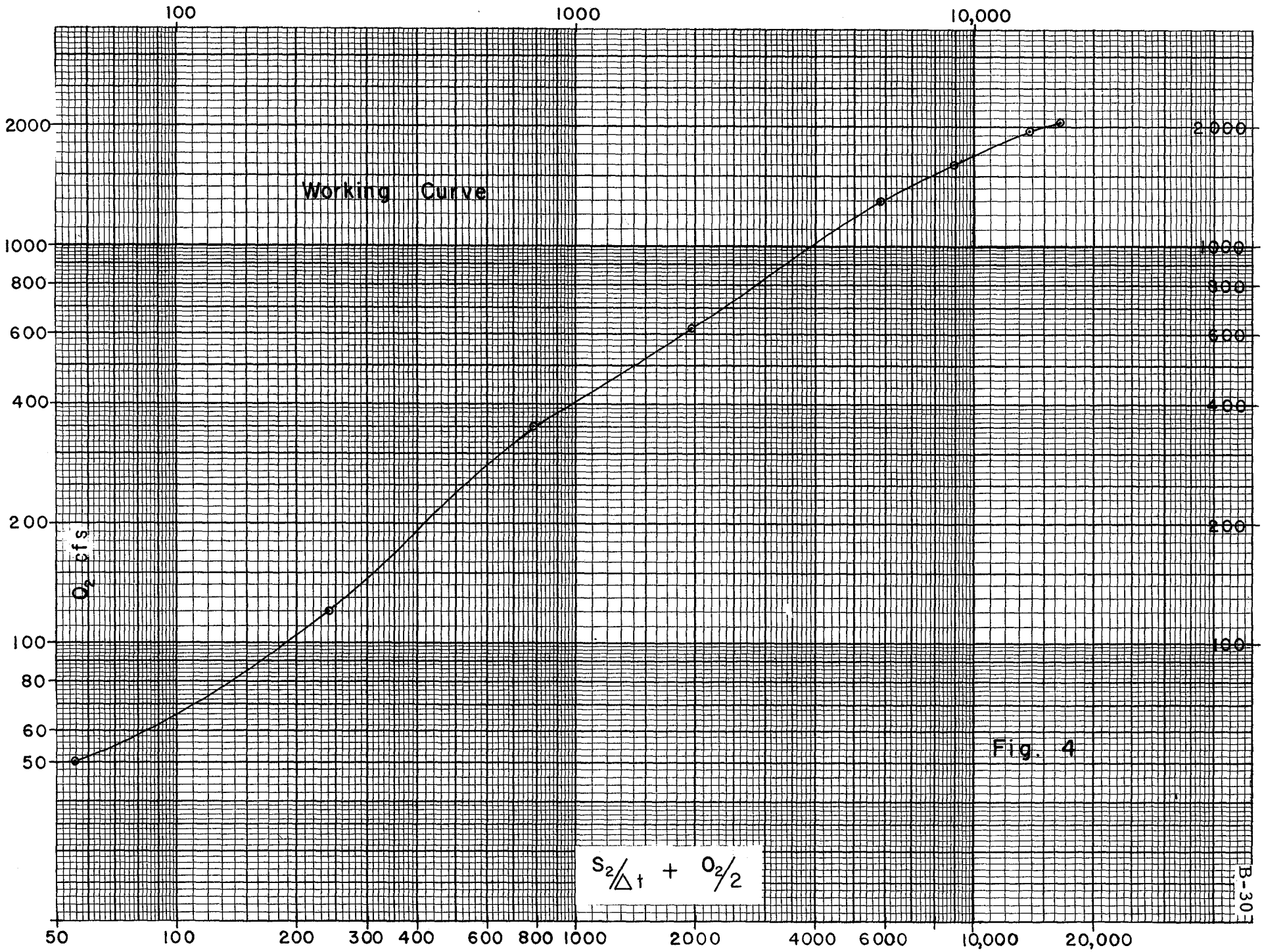

Working Curve
O_2 cfs
$S_2/\Delta t + O_2/2$
2000
1000
800
600
400
200
100
80
60
50
100
1000
10,000
50
100
200
300
400
600
800
1000
2000
4000
6000
10,000
20,000

Fig. 4

Table 2

Routing Table for Storage-Indication
Method for the one-hour storm duration
Routing Interval (Δt) = .2 hrs.

Routing Interval	time hrs.	Inflow (I) 1/ cfs	$S_2/\Delta t + O_2/2$ 2/ cfs	Outflow (O_2) 3/ cfs
0	0	0	0	0
1	.2	85	→ 85	60
2	.4	170	(170 + 85 - 60) 195	102
3	.6	260	(260 + 195 - 102) 353	168
4	.8	350	(350 + 353 - 168) 535	255
5	1.0	940	(940 + 535 - 255) 1220	465
6	1.2	1660	(1660 + 1220 - 465) 2415	725
7	1.4	2360	(2360 + 2415 - 725) 4050	1010
8	1.6	2920	(2920 + 4050 - 1010) 5960	1300
9	1.8	3140	(3140 + 5960 - 1300) 7800	1510
10	2.0	3070	(3070 + 7800 - 1510) 9360	1650
11	2.2	3020	(3020 + 9360 - 1650) 10730	1760
12	2.4	2600	(2600 + 10690 - 1760) 11570	1810
13	2.6	2180	(2180 + 11570 - 1810) 11940	1820

1/ from fig. 1a. 3/ from fig. 4
2/ call this expression Working Storage

Routing Interval	time hrs.	Inflow (I) cfs	$S_2/\Delta t + O_2/2$ cfs	Outflow (O2) cfs
14	2.8	1740	(1740 + 11940 - 1820) 11860	1810
15	3.0	1310	(1310 + 11860 - 1810) 11360	1800

∴ 1820 cfs is the peak rate of outflow
DHW = 2086.6 (fig. 3)
Max. Storage = 180 Af.

Will another storm duration produce a higher peak rate of outflow?

To Answer this question the routing procedure would have to be repeated for durations greater than one hour.

For this example we will assume that the routing of the one-hour storm will produce the highest peak rate of discharge. Therefore the four span 10 x 5 C.B.C. is recommended.

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

MEDIAN DRAINAGE DESIGN

October, 1972

MEDIAN DRAINAGE

Introduction

Storm runoff from medians is removed at intervals by median inlets or standard culverts to reduce the traffic hazard, to prevent saturation of the embankment, or to reduce median erosion due to high velocity flow.

Design Data

The following design data will be required for median drainage design:

Design Frequency: The design frequency to be used for median drainage design is the same as used for cross-culvert design which is described in a/.

Design Discharge: The design discharge for median drainage will be calculated by the Rational Equation which is described in a/.

Allowable Ponding Depth: The depth of ponding in a median should not exceed the elevation of the subbase, (depth may exceed subbase if median is paved) and the depth should not present a hazard to traffic.

Median Dikes: Median dikes are required at the downstream side of all median drains except where the natural ground acts as a dike or where the inlet is located in a sag. The elevation of the median dike should be at least .5' higher than the design high-water elevation of the inlet.

Median Flow Velocity: Where median flow velocity is excessive and erosion is a problem, median paving will be required.

Hydraulic Capacity Charts: Hydraulic capacity charts for the following standard median inlets have been developed:

C - 15.08 Catch Basin Type 4 (Off roadway location)
C - 15.09 Median Catch Basin
C - 15.10 Median Dike Catch Basin

---

a/ Hydrologic Design for Highway Drainage in Arizona

The capacity of standard culverts should be determined from FHWA HEC #5 or 10.

Clogging Factors: The capacity of median inlets is subject to appreciable reduction due to debris that tends to clog the waterway opening. The Hydraulic Capacity Charts for the standard inlets have the clogging factors incorporated and no additional reduction should be made.

Median Drainage Design Sheet: To standardize median drainage calculations a design data sheet has been prepared. This design form should be used for all median drainage calculations and submitted with each design.

Design Procedure

Location Data:

1. The required input is self-explanatory.

Design Data:

1. Select the recommended design frequency from a/.
2. Obtain 6-hour and 24-hour precipitation values for the project area from Precipitation Maps 1-6 in a/.
3. Calculate 1-hour precipitation value as described in a/.
4. Determine the allowable headwater ponding at the inlets.

Runoff Calculations:

1. Select Station to Station limits of median area to be drained.
2. Estimate width and length of the median and calculate the area.
3. Select the appropriate runoff coefficient C from Fig. 3-3 of a/.
4. Calculate the time of concentration t_c as outlined in a/. For drainage areas shorter than 1,000 ft. the minimum time of concentration of 10 min. should be used.
5. Calculate rainfall intensity I, as outlined in a/.
6. Calculate design discharge Q by the Rational Equation.
7. Calculate the depth of flow and the velocity for the given channel geometry when appropriate.
8. If the depth of flow in the median exceeds the allowable depth, the discharge will have to be reduced by reducing the length of the drainage area.
   If the median flow velocity exceeds the permissible velocity, the median should be paved or the storm runoff must be removed at more frequent intervals.

---

a/ Hydrologic Design for Highway Drainage in Arizona

Inlet Calculation:

1. Indicate inlet location station.
2. Select type of inlet to be used.
3. Calculate highwater elevation HW from appropriate capacity charts. If the capacity of the inlet at the allowable highwater is less than the design discharge, inlets at more frequent intervals must be used.
4. Show design highwater elevation DHW and Dike Elev.

Example

The capacity of Standard Inlets C-15.08 and C-15.09 is generally controlled by the grate. The capacity of inlet C-15.10 may however be controlled by either the grate, the median culvert, or the cross-culvert.

The following calculations will be required to determine the DHW elevation with a C-15.10 inlet.

Given: Median drainage area data

Find: DHW elevation for a C-15.10 Catch Basin

Solution:

1. Calculate design discharge, the depth of flow and the velocity in the median from the given data. Calculations are not shown. $Q = 6.8$ cfs
2. Calculate DHW for grate, median culvert, and cross-culvert.
   a. from Chart 3 for grate control
      HW = 1.4'

   b. for median culvert control calculate

$$\frac{L}{100\,S_o} = \frac{15}{100(.008)} = 20$$

where L = median culvert length
S_o = median culvert slope

The $\frac{L}{100\,S_o}$ value is close to inlet control therefore use the inlet control line to determine the headwater

HW = 1.4 ft.

The grate still controls the headwater.

   c. Calculate the normal depth of flow in the cross-

culvert. Hydrologic and hydraulic design data should be available for the cross-culvert. In this example the depth of flow is 1.1 ft (Calculations are not shown).

If the highwater elevation for the grate is .5± ft. higher than the elevation of normal depth in the cross-culvert at the junction of the median and cross-culvert,then the grate highwater will control.

If the normal depth in the cross-culvert is greater than the grate highwater elevation, water will back into the median. High tailwater from the cross culvert may also back into the median. If these conditions exist, consideration should be given to providing a separate culvert for the median flow.

HYDRAULIC CAPACITY CHART
MEDIAN CATCH BASIN

Standard C-15.08 CB Type 4
(Off roadway location)
50% Clogging

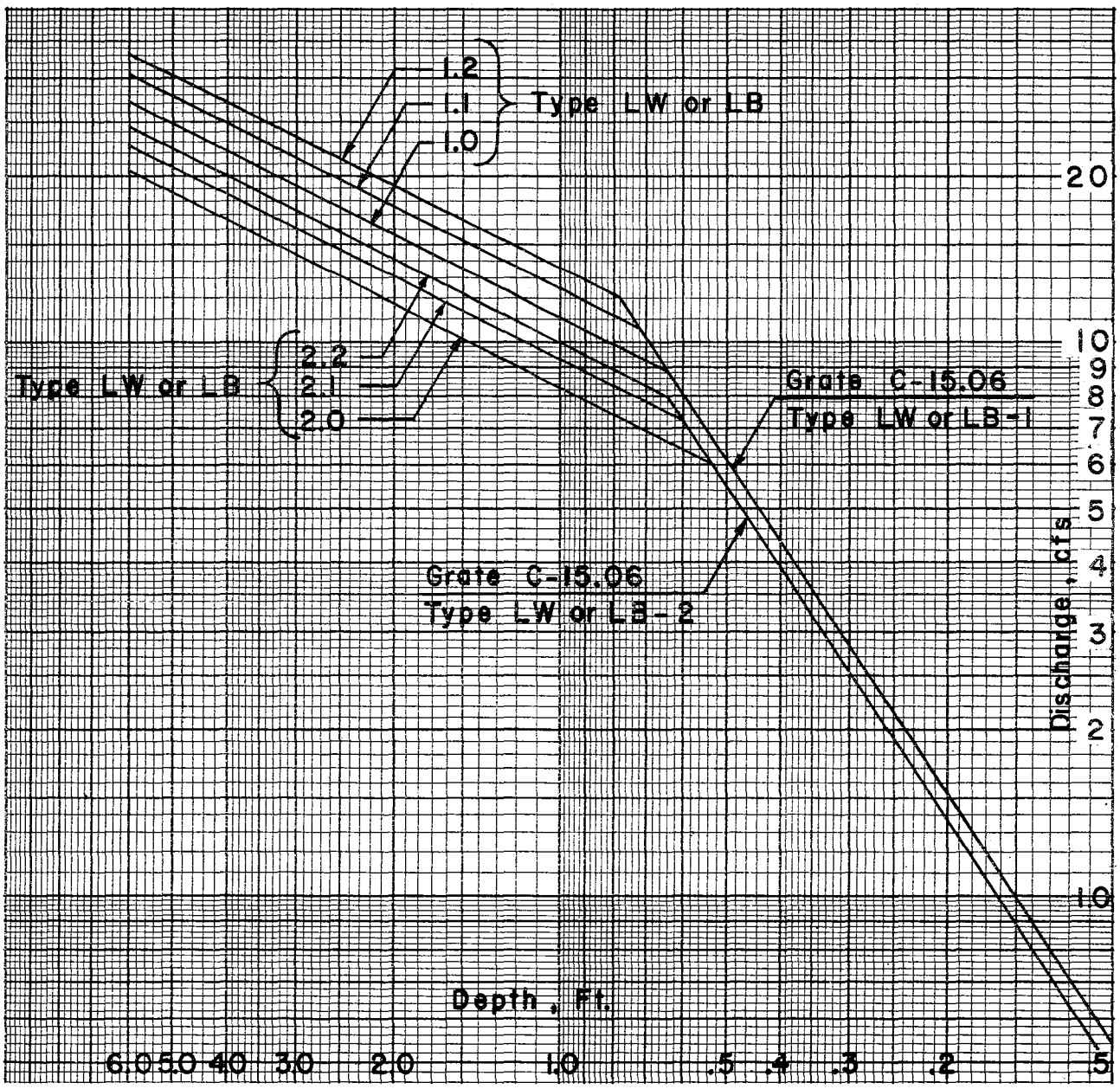


Chart 1

AHD
Structures Section
Hydraulics Branch
10-15-72

FHWA
HEC #12

HYDRAULIC CAPACITY CHART
MEDIAN CATCH BASIN

Standard C-15.09
50% Clogging

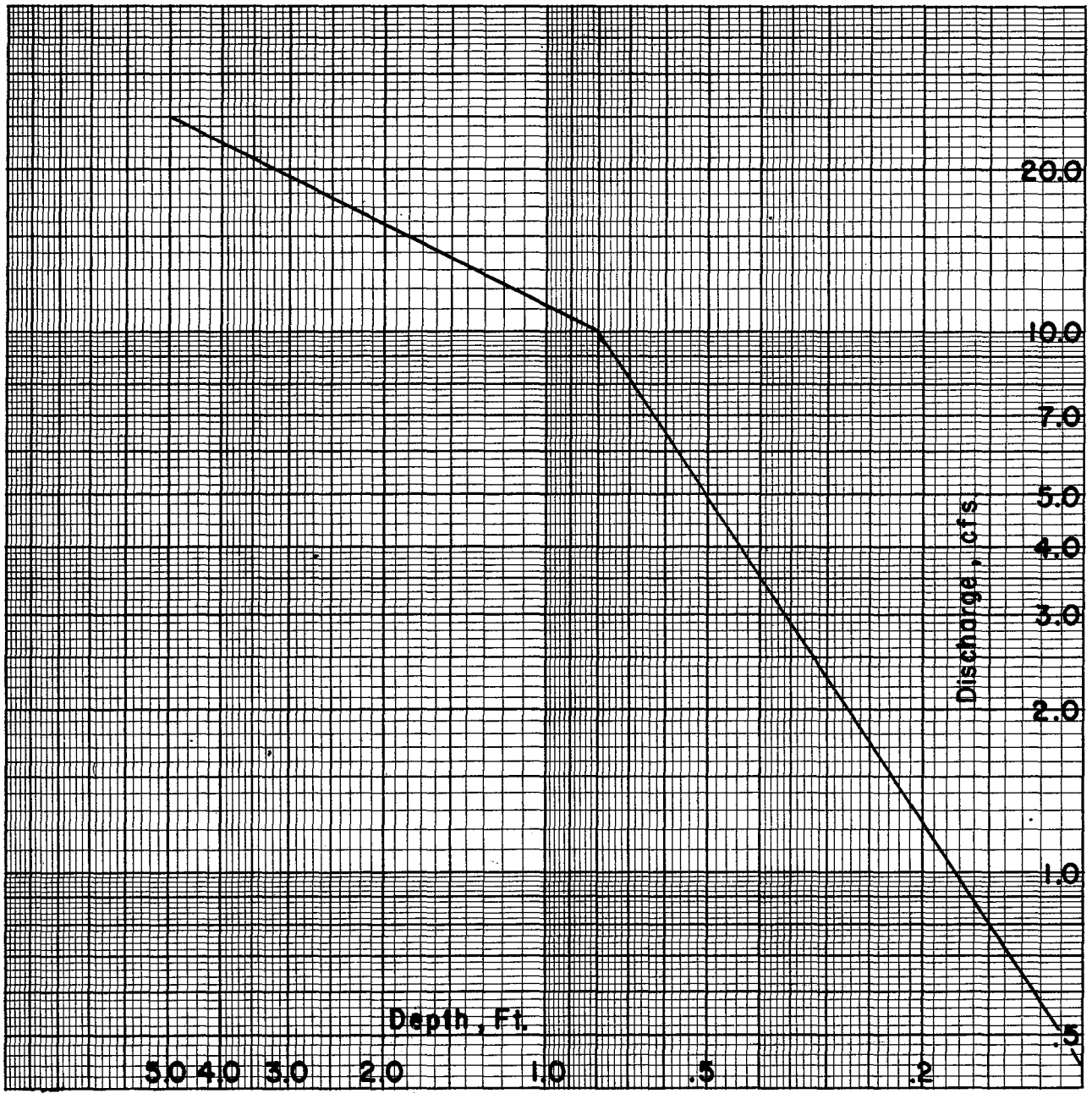


Chart 2

AHD
Structures Section
Hydraulics Branch
10-15-72

FHWA
HEC #12

HYDRAULIC CAPACITY CHART
MEDIAN CATCH BASIN

Standard C-15.10
50% Clogging

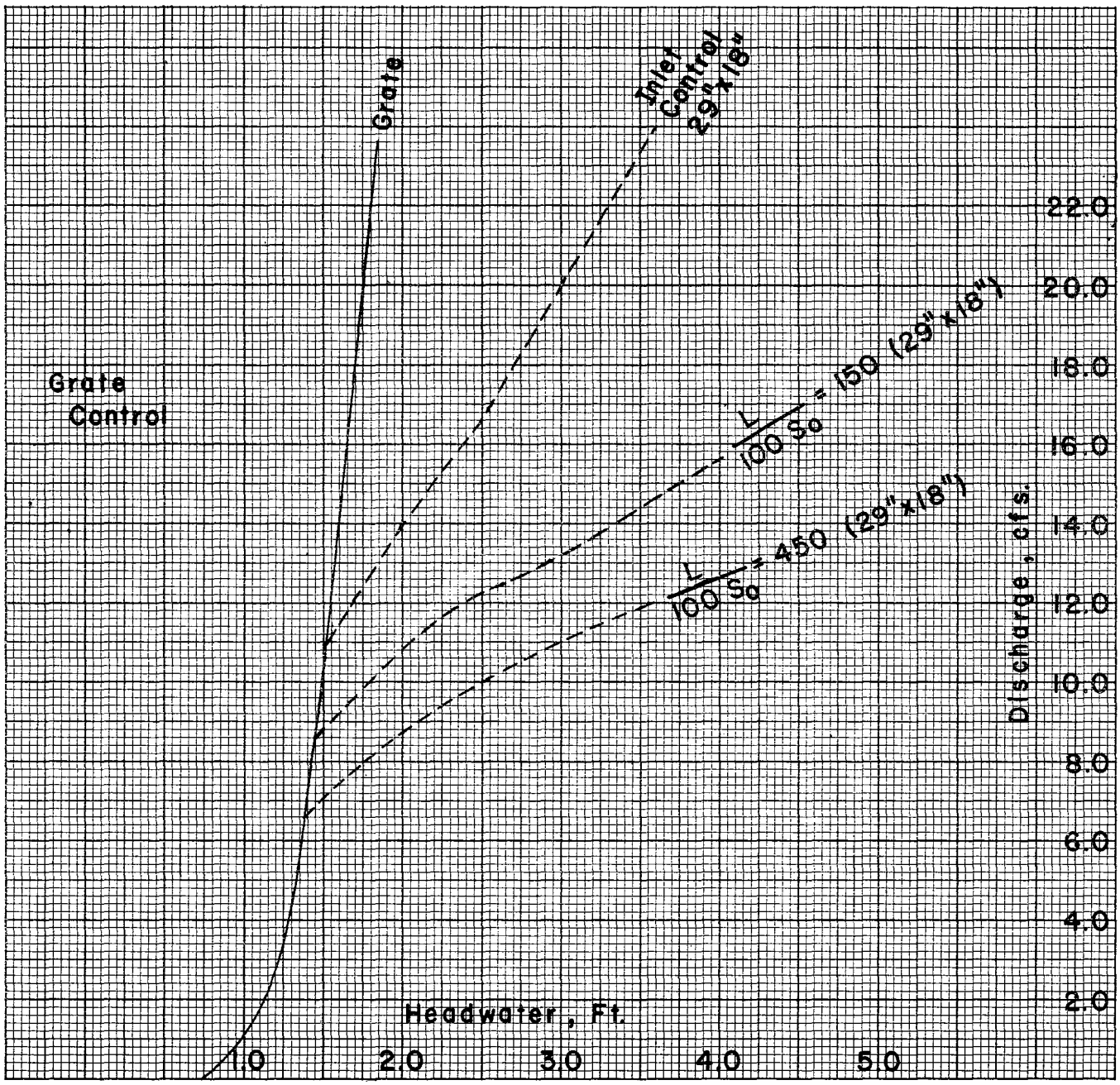


Chart 3

AHD
Structures Section
Hydraulics Branch
10-15-72

FHWA
HEC #12

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

Example 1

MEDIAN DRAINAGE WORKSHEET

LOCATION DATA
Highway: Ehrenberg - Phoenix
Location:
Project No.: I-10-?

DESIGN DATA
Frequency: 50 years
P_6 = 2.95 in. P_{24} = 3.70 in. P_1 = 2.2 in.
AHW =

RUNOFF CALCULATIONS

From Station	To Station	L ft.	W ft.	D.A. acr.	C	T_c min.	I in/hr	Q cfs	Channel Type	S_o ft/ft	Z	n	D ft.	V fps
7275+	7287	1,200	76	2.09	.55	10	5.95	6.8	Triang	.008	10	.025	.52	2.5
									check T_c					
									travel time = 1200/60 (2.5) = 8.0 min					
									∴ 10 min. is OK					

INLET CALCULATIONS

Station	Inlet Type	Q cfs	HW ft.	DHW Elev.	Dike Elev.
7287+00	C 15.10 29"x18" median culvert	6.8 1/	1.4	1021.4 2/	1021.9
		Check cross culvert to determine it's affect on the C 15.10 inlet			
		Assume $L/(100S_o) \approx 15/(100)(.008) = 20$			
			Assume Inlet control		

Channel Type: Trapezoidal, (Triangular)

Inlet Type: C-15.08, C-15.09, (C-15.10) Pipe Culvert

1/ From chart
2/ Elev. of median is 1020.0

Computed by: RCJ Checked by: E.I.J. Date: 10-15-72

C-8

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

MEDIAN DRAINAGE WORKSHEET

LOCATION DATA
Highway _____
Location _____
Project No. _____

DESIGN DATA
Frequency _____ years
P_6= _____ in. P_{24}= _____ in. P_1= _____ in.
AHW = _____

RUNOFF CALCULATIONS

From Station	To Station	L ft.	W ft.	D.A. acr.	C	T_c min.	I in/hr	Q cfs	Channel Type	S_o ft/ft	Z	n	D ft.	V fps

INLET CALCULATIONS

Station	Inlet Type	Q cfs	HW ft.	DHW Elev.	Dike Elev.

Channel Type: Trapezoidal
Triangular

Inlet Type: C-15.08
C-15.09
C-15.10
Pipe Culvert

Computed by: _____ Checked by: _____ Date: _____

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

PAVEMENT DRAINAGE DESIGN

October, 1972

PAVEMENT DRAINAGE

Introduction

The primary purpose of pavement drainage is to remove storm water from the traveled way, thereby reducing the hazard to roadway traffic and to prevent erosion of the roadway embankment by uncontrolled runoff from the paved roadway.

The design procedure described herein is limited to drainage of pavements where an underground storm sewer system is not required and where pavement runoff must be concentrated at the roadway shoulder to prevent erosion of the embankment face.

Design Data

The following design data will be required in pavement drainage:

Design Frequency: The design frequency to be used for pavement drainage is described in a/.

Design Discharge: The design discharge for pavement drainage will be calculated by the Rational Equation which is described in a/.

Embankment Curb: Embankment curb will be required at the edge of pavedroadways on fills where storm runoff cannot be permitted to run down the embankment slope and erode the embankment face.

Ponding Width: The allowable spread of water T on the pavement should be limited to a width that will generally not interfere with or present a hazard to traffic flow. The pavement spread criteria to be used is described in b/.

Hydraulic Capacity Charts: Hydraulic capacity charts for the following standard embankment inlets have been developed for the design conditions normally encountered.

C - 4.01 Concrete Spillway
C - 4.02 Spillway Inlet, CMP Downdrain

---

a/ Hydrologic Design for Highway Drainage in Arizona, 12-1-68
b/ Office Memo: Allowable Flooded Width Used in the Design of Curbed Roadway Drainage, 8-24-70

Clogging Factors: The capacity of pavement inlets is subject to appreciable reduction due to debris that tends to clog the waterway opening. The Hydraulic Capacity Charts for the standard inlets have the clogging factors incorporated and no additional reduction should be made.

Inlets on Grade: Normally, it is not necessary or economical to intercept all of the flow past an inlet. A portion of the flow may be permitted to bypass an inlet to be picked up at succeeding inlets downstream.

Inlets in Sags: Inlets in Sags must intercept all of the flow reaching it. If the inlet capacity is less than the approach flow, water will overtop the curb and may erode the embankment face. It may therefore be necessary to place inlets a short distance upstream from the sag inlet to reduce the total flow Q_T reaching the sag.

Pavement Drainage Design Sheet: To standardize pavement drainage calculations a design data sheet has been prepared. This design form should be used for all pavement drainage calculations and submitted with each design.

Design Procedure

Location Data:

1. The required input is self-explanatory.

Design Data:

1. Select the recommended design frequency from a/.
2. Obtain 6-hour and 24-hour precipitation values for the project area from Precipitation Maps 1-6 in a/.
3. Calculate 1-hour precipitation value as described in a/.
4. Determine allowable pavement spread $T_{all.}$ from b/ and typical roadway section.
5. Determine curb height h from the roadway construction standards.

Runoff Calculation:

1. Select Station to Station limits of pavement area to be drained.
2. Determine the length and width of the pavement area and calculate the area.
3. The runoff coefficient to be used for pavement drainage is .95.
4. Calculate time of concentration as outlined in a/. For reaches shorter than about 1,000', the minimum time of concentration of 10 min. may be used.
5. Calculate rainfall intensity I as outlined in a/.

---

a/ Hydrologic Design for Highway Drainage in Arizona, 12-1-68

b/ Office Memo: Allowable Flooded Width Used in the Design of Curbed Roadway Drainage, 8-24-70

6. Calculate the pavement discharge Q by the Rational Equation.
7. Calculate the depth of flow D and the pavement spread T from the nomograph for triangular channel flow. If the depth of flow exceeds the curb depth or the pavement spread exceeds the allowable spread, the design discharge is too great and the drainage area length should be reduced.

Inlet Calculation:

1. Indicate inlet location Station.
2. Select type of inlet to be used.
3. With the previously calculated depth of flow D determine the discharge intercepted Q_i by the inlet from the appropriate capacity chart.
4. Calculate the carryover discharge Q_c.

Continue with calculations for the next reach. For the second and all subsequent reaches, the pavement discharge Q plus the carryover discharge Q_c equals the total discharge Q_T. The last inlet in the reach must intercept and carry all the pavement drainage off.

Examples

Example 1: Inlet spacing calculations for a 1750 ft. long roadway section on a 2% slope terminating in a sag are shown. Two inlets at intervals of 750 ft. were selected for the on-grade section. The sag inlet is located 250 ft. beyond the last on-grade inlet. Design calculations are shown in the example.

Example 2: Inlet spacing calculations for a long continuous roadway section on a 1.0% slope terminating in a sag are shown. In this example the inlets are located at their maximum spacing. The following design procedure should be used.

1. Calculate the discharge for the maximum depth of pavement flow.
2. Calculate the length of roadway required to generate this discharge. This will be the distance for the first inlet.
3. Calculate the inlet capacity Q_i for the maximum depth of pavement flow and discharge.
4. Calculate the length of roadway required to generate the discharge intercepted by the inlet Q_i. This will be the inlet spacing for all subsequent on-grade inlets.
5. The last inlet (sag inlet) will intercept both the carryover flow Q_c and the flow generated by the last pavement section.

The design calculations are shown in the example and will be explained.

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

PAVEMENT DRAINAGE WORKSHEET

Example 1

LOCATION DATA
Highway Kingman- Ashfork
Location
Project No. I-40

DESIGN DATA
Frequency 10 years
P_6= 2.1 in. P_{24}= 2.6 in. P_1= 1.6 in.
T_{all} = 17.0
Curb Height = 3 "

RUNOFF CALCULATIONS

From Station	To Station	L ft.	W ft.	D.A. acr.	C	T_c min.	I in/hr	Q cfs	Q_c cfs	Q_T cfs	S_o ft/ft	S_x ft/ft	z/n	D ft.	T ft.
4340±	4332+50	750	38	.65	.95	10	4.4	2.73		2.73	.02	.02	3125	.19	9.5
4332+50	4325±	750	38	.65	.95	10	4.4	2.73	1.50	4.23	.02	.02	3125	.22	11.0
4325±	4322+50	250	38	.22	.95	10	4.4	.91	2.73	3.64	.005	.02	3125	.27	13.5
					.95										
					.95										
					.95										
					.95										

INLET CALCULATIONS

Station	Inlet Type	D ft.	Q_i cfs	$Q_c = Q_T - Q_i$
4332+50	C-4.01 Single	.19	1.23	1.50
4325 ±	C-4.01 Single	.22	1.50	2.73
4322+50	C-4.02 Single*	.25	4.30	–
	* Sag			

Inlet Type: C-4.01 Single
Double

C-4.02 Single
Double

Computed by: E.I. Jencsok Checked by: C. D. Date: 10-1-72

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

PAVEMENT DRAINAGE WORKSHEET

Example 2

LOCATION DATA
Highway Kingman-Ashfork
Location
Project No. I-40

DESIGN DATA
Frequency 10 years
P_6= 2.1 in. P_{24}= 2.6 in. P_1= 1.6 in.
T_{all} = 17.0
Curb Height = 3"

RUNOFF CALCULATIONS

From Station	To Station	L ft.	W ft.	D.A. acr.	C	T_c min.	I in/hr	Q cfs	Q_c cfs	Q_T cfs	S_o ft/ft	S_x ft/ft	z/n	D ft.	T ft.
4390±	4328±	1200± (1180)	38	1.05	.95	10	4.4	4.3		4.3	.01	.02	3125	.25	12.5
4328±	4322±	600± (100)	38	.52	.95	10	4.4	2.2	2.1	4.3	.01	.02	3125	.25	12.5
4322±	4316±	600±	38	.52	.95	10	4.4	2.2	2.1	4.3	.01	.02	3125	.25	12.5
4316±	4313+50	250±	38	.22	.95	10	4.4	.95	2.1	3.05	.005	.02	3125	.25	12.5
					.95										
					.95										
					.95										

INLET CALCULATIONS

Station	Inlet Type	D ft.	Q_i cfs	$Q_c = Q_T - Q_i$
4328±	C-4.02 Single	.25	2.2	2.1
4322±	C-4.02 Single	.25	2.2	2.1
4316±	C-4.02 Single	.25	2.2	2.1
4313+50	C-4.02 Sag	.25	4.3	—

Inlet Type: C-4.01 Single
Double

C-4.02 Single
Double

Computed by: E. J. Jenosek Checked by: C.D. Date: 10-1-72

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

PAVEMENT DRAINAGE WORKSHEET

LOCATION DATA
Highway _____
Location _____
Project No. _____

DESIGN DATA
Frequency _____ years
P_6= _____ in. P_{24}= _____ in. P_1= _____ in.
T_{all} = n = _____
Curb Height =

RUNOFF CALCULATIONS

From Station	To Station	L ft.	W ft.	D.A. acr.	C	T_c min.	I in/hr	Q cfs	Q_c cfs	Q_T cfs	S_o ft/ft	S_x ft/ft	z/n	D ft.	T ft.
					.95										
					.95										
					.95										
					.95										
					.95										
					.95										
					.95										

INLET CALCULATIONS

Station	Inlet Type	D ft.	Q_i cfs	$Q_c = Q_T - Q_i$

Inlet Type: C-4.01 Single
Double

C-4.02 Single
Double

Computed by: _____ Checked by: _____ Date: _____

HYDRAULIC CAPACITY CHART
Embankment Inlets

Standards C-4.01 & C-4.02
On Grade

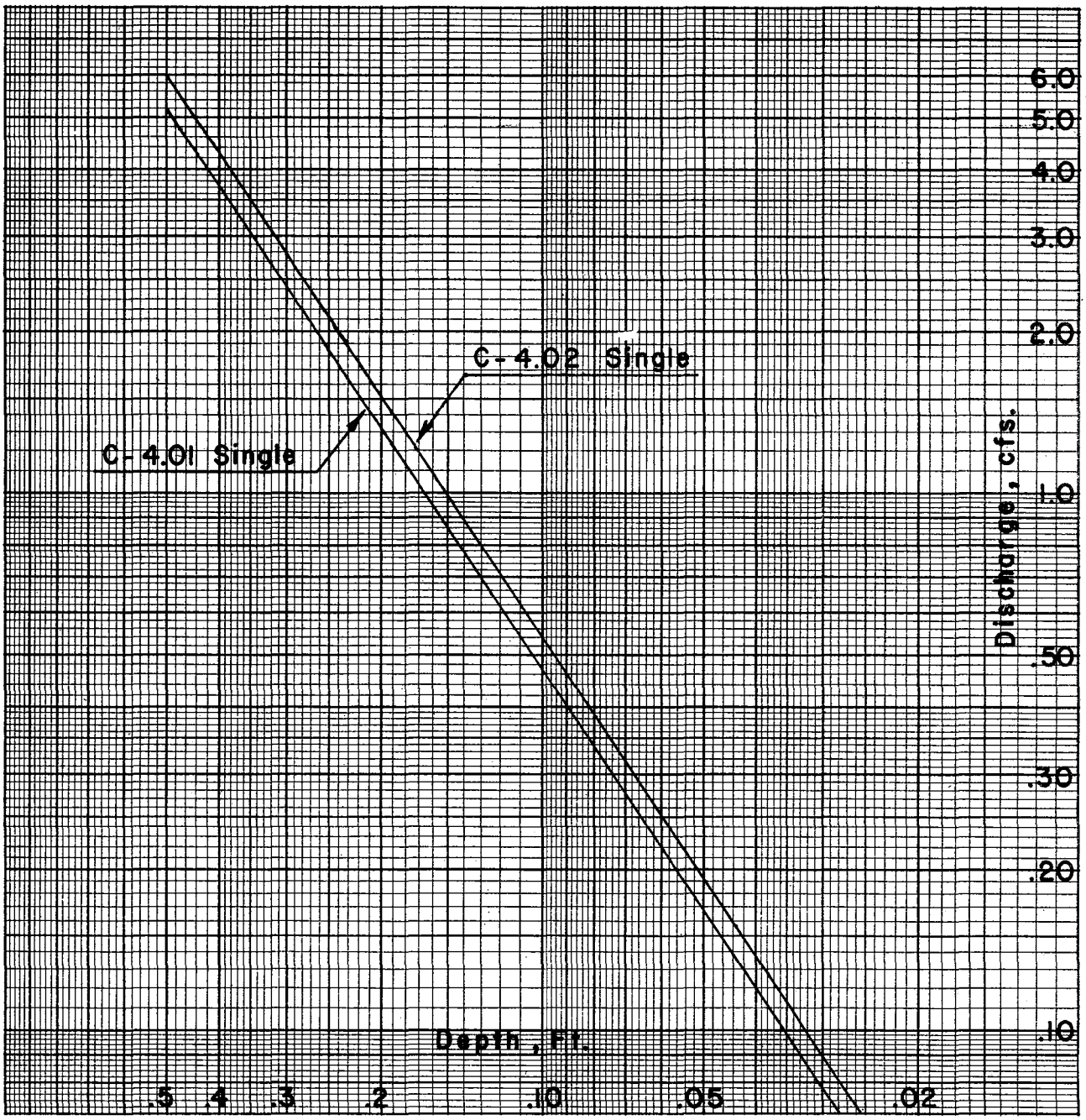


Chart 1

AHD
Structures Section
Hydraulics Branch
10-15-72

J. Hopkins University
Study

HYDRAULIC CAPACITY CHART
Embankment Inlets

Standards C-4.01 Single & Double
Sag Condition

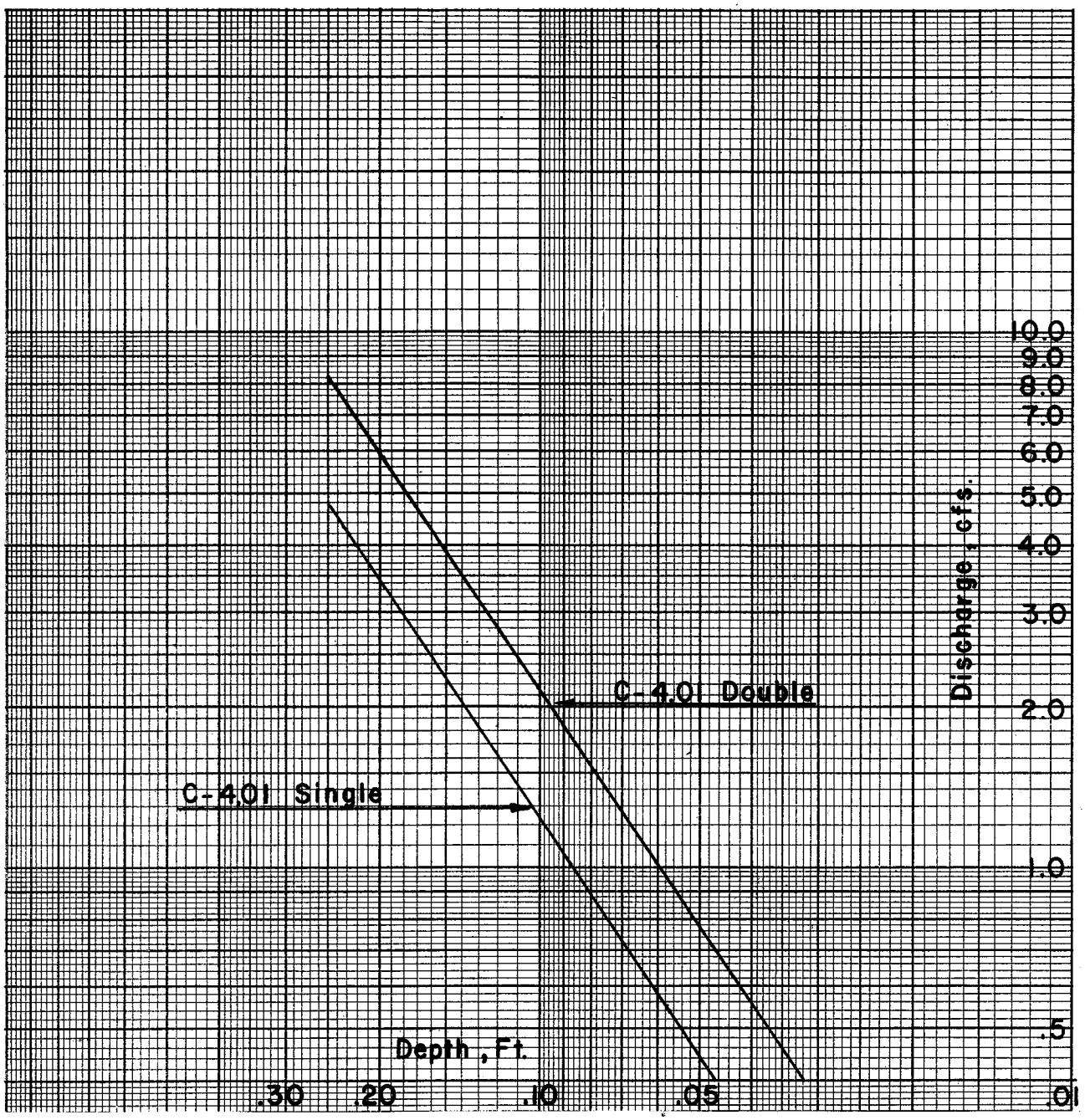


Chart 2

AHD
Structures Section
Hydraulics Branch
10-15-72

FHWA
HEC #12

HYDRAULIC CAPACITY CHART
Embankment Inlets

Standard C-4.02 Single & Double
Sag Condition

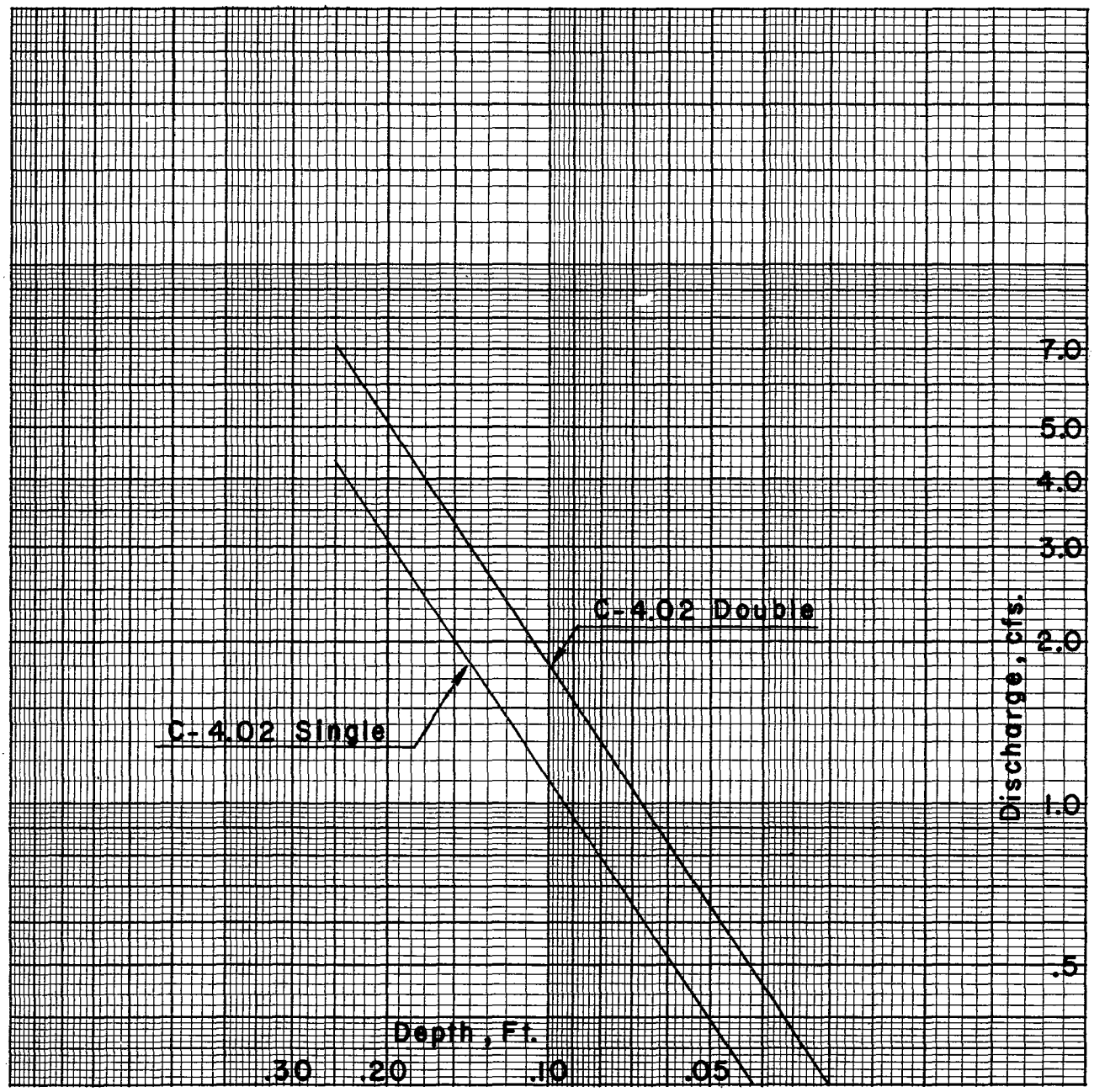


Chart 3

AHD
Structures Section
Hydraulics Branch
10-15-72

FHWA
HEC #12

URBAN HYDROLOGY

A. Recommended Procedure

The following design procedure using the rational formula is recommended for urban areas less than one square mile. Although this procedure is an approximate method, it presents a good balance between design time and construction costs, and is the most widely used method.

The rational method will be used for on-site as well as off-site drainage.

For areas greater than one square mile it is recommended that a pre-design conference be held with the Hydraulics Branch.

A pre-design conference with the Hydraulics Branch is recommended for projects where a large discharge is concentrated on the pavement from off-site drainage.

B. Rational Formula

The rational formula is discussed on pages 19-21 in the manual entitled, "Hydrologic Design for Highway Drainage in Arizona."

Section 3.32 Storm Duration - Time of Concentration, of the above manual will be discussed more fully to bring it up to date with recent research.

C. Watershed Delineation

An approximate watershed boundary may be obtained by using a U.S.G.S. 7 1/2 minute topographic map if available and the area is large enough. Air photos may be used but some method of obtaining the necessary elevations must be used in conjunction with the air photos. The exact boundaries should be verified by a field reconnaissance.

If the area is too small or no map is available, one will have to be prepared.

The next step is to locate points of concentration of water on the pavement from on-site and off-site drainage. Delineate the sub-drainage areas associated with the flow concentration points. Such points would be street intersections, commercial drives, etc.

D. Runoff Coefficient

The next step is to break the drainage area into areas with common impervious rates such as streets, roof tops, sidewalks, etc.

Using the coefficients given by Figure 3-3 (Values of Runoff Coefficient "C") in the manual "Hydrologic Design for Highway Drainage in Arizona." Multiply the sub-drainage area in acres by the corresponding coefficient. Follow this procedure for all of the sub-drainage areas within a main drainage area. Add all of the CA values and multiply by the rainfall intensity to obtain the storm runoff.

In certain cases it may be desirable to obtain a weighted "C" for the main drainage area. To obtain a weighted "C" for the main drainage area, divide the total CA value by the total acreage in the main drainage area.

E. Future Land Use in the Drainage Area

Since the proposed project will function for years in the future, projected land use maps and zoning maps should be consulted for possible land use changes if such maps are available. If projected land use and zoning maps are not available, talk to local residents in the area and city officials to get their views on the future land use in the watershed.

F. Time of Concentration T_c

The time of concentration should be calculated by the Manning equation ($V = \frac{1.49}{n} R^{2/3} S^{1/2}$) and the continuity equation ($Q = AV$) where applicable. The Manning equation, when used to calculate gutter flow, should be modified to use the form $Q = 0.56 (\frac{Z}{n}) S^{1/2} d^{8/3}$. This modification is necessary because experimental data shows that flow in triangular channels where the top width of the water surface is greater than 40 times the depth the hydraulic radius as defined in the Manning equation is not properly described (See B.P.R. Cir. No. 12, page 12-11, equation (2)). The continuity equation ($Q = AV$) can be used to obtain the velocity if the modified Manning equation is used, since the discharge and area are known. The time of concentration can be calculated by dividing the length by the velocity which will give the answer in seconds. Convert to minutes by dividing the seconds by 60.

V = velocity in ft./sec.
n = Manning's "n"
R = hydraulic radius
S = slope in ft./ft.

Q = discharge in cubic feet per second
A = area in square feet
d = depth of water at the curb in feet
Z = reciprocal of the cross slope

The depth of water at the curb will have to be calculated. We cannot always assume that the side street is running full curb to curb. This will involve getting the cross section and longitudinal slope of the streets that are acting as collectors for the storm water.

If the design discharge submerges the collector street, the velocity of flow and time of concentration are effected. However, this is beyond the scope of this seminar. If this situation occurs, it will be evaluated on an individual basis by the Hydraulics Branch.

The minimum time of concentration that should be used is 10 minutes.

G. Rainfall Intensity

To obtain the rainfall intensity refer to pages 5, 6, and 21 of the manual entitled, "Hydrologic Design for Highway Drainage in Arizona."

The equations previously given to you substituting for Figure 2-2 may be used if applicable.

The following example will illustrate the previous principles.

EXAMPLE

Calculate the storm runoff for the area shown on the air photo. The concentration points will be at the intersection of Thomas Road with the following streets:

1. N. 48th Dr.
2. N. 47th Ave.
3. N. 46th Dr.
4. N. 44th Ln.
5. Planned shopping center

Calculate the storm runoff from the drainage area associated with N. 48th Dr. (D.A. #1) by separating sub-areas with different infiltration rates and summing the product of the runoff coefficient and its sub-area.

Drainage Area No. 1
Total Drainage Area, 137 Acres
Area in Streets

Street Name	Ft. Width	Ft. Length	Area Sq. Ft.
W. Flower St.	32	1830	58,500
W. Cheery Lynn Dr.	32	1665	53,300
W. Earll Dr.	32	2220	71,000
W. Avalon Dr.	32	1850	59,100
W. Pinchot Dr.	32	2010	64,300
W. Catalina Dr.	32	3730	119,200
W. Merrell St.	32	1170	37,450
W. Verde Ln.	32	2560	82,000
N. 50th Dr.	32	870	27,850
N. 49th Ave.	32	170	5,440
N. 50th Ave.	32	150	4,800
No Name Cul-de-sac	32	130	4,160
N. 48th Ave.	32	1650	52,800
N. 47th Ave.	32	2350	75,100
W. Mulberry Dr.	32	800	25,600
N. 49th Dr.	32	520	16,650
N. 48th Ave.	32	1270	40,600
No name Cul-de-sac	32	210	6,710
No name Cul-de-sac	32	110	3,520
N. 49th Ln.	32	320	10,220
N. 49th Ave.	32	310	9,910
N. 48th Dr.	32	450	14,400
W. Thomas Rd.	42	2070	87,000

$$\frac{929,610}{43560} = 21.34 \text{ acre}$$

All streets in this drainage area are asphaltic and in good condition. Use a "C" value of 0.85.

Compute CA

21.34 acre area
x 0.85 "C"
18.14 = CA

Area in Roofs

A. Houses

1. Area of House

35'
x 70'
2450 Sq. Ft.

2. Number of Houses

430

3. Total Area of House Roofs

2450 Area of House
x 430 No. of Houses in D.A.
1,053,500 Sq. Ft. Total House Roof Area

B. Commercial Buildings

1. Area of Building

70'
x 60'
4200 Sq. Ft.

2. Number of Buildings

1

3. Total Area of Building Roof

4200 Sq. Ft. Area of Building
x 1 No. of Building in D.A.
4200 Sq. Ft.

Use a "C" value of 0.95 for all roof areas.

Compute CA

1,053,500 Sq. Ft. Total Area of House Roofs
4,200 Sq. Ft. Total Area of Commercial Building Roof
1,057,700 Sq. Ft.

1,057,700/43,560 = 24.28 acre

CA = 0.95 x 24.28 = 23.07

Area in Commercial Parking Area

19,800 Sq. Ft. Scaled from Air Photo

$$\frac{19,800 \text{ Sq. Ft.}}{43560} = 0.454 \text{ Use } 0.45 \text{ AC.}$$

The parking area is not in good condition.
Use "C" of 0.8

Compute CA

CA = 0.8 x 0.45 = 0.36

Area in Parks and Lawns

A. Public Park

705'
320'

$$\frac{225,600 \text{ Sq. Ft.}}{43,560} = 5.18 \text{ AC.}$$

Use "C" value of 0.2

Compute CA

5.18 AC. Area
0.2 Coefficient
1.036 Use 1.04 = CA

Area in Lawns

The total drainage area was obtained from the air photo and found to be 137.00 AC.

Summate the other sub-areas and subtract from the gross area to obtain the lawn area.

21.34 AC.	Street Area
24.28 AC.	Roof Area
0.45 AC.	Commercial Parking Area
5.18 AC.	Park Area
51.25 AC.	

137.00 AC.	Gross Area
51.25	
85.75 AC.	Lawn Area

Use "C" value of 0.2

Compute CA

85.75 AC.	Lawn
0.2	Coefficient
17.15	Use 17.15

Compute the Weighted "C" for the Entire Drainage Area

Summate CAs for the Sub-Areas

18.14	Streets
23.07	Roofs
0.36	Commercial Parking Area
1.04	Park
17.15	Lawns
59.76	

Divide total CA for the drainage area by the gross area to obtain the weighted "C" value for the drainage area.

$$\frac{59.76}{137.00} = 0.436$$

Calculate the capacity of the street cross section shown in Figure 1.

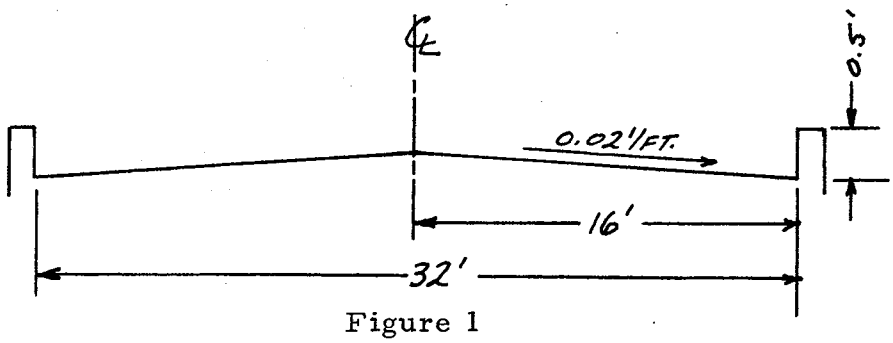


Figure 1

Typical street cross section
for N. 48th Dr., N. 47th Ave.,
N. 46th Dr., and N. 44th Ln.

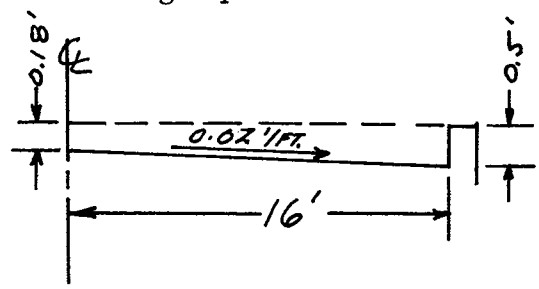
Given: Water at the top of curb
$S_o = 0.0035$ '/ft.
$n = 0.016$

Use 1/2 of street for capacity calculation

Since this cross section is not triangular, the modified Manning Equation as given on page 12-11, B.P.R. Cir. No. 12 cannot be used. Since there isn't any modified Manning Equation for trapezoidal sections, we will used the Standard form of the Manning Equation.

Use Manning's Equation

$$V = \frac{1.49}{n} (R)^{2/3} S^{1/2}$$



$\frac{1.49}{0.016} = 93.1$

$S = 0.0035\ '/ft.$

$S^{1/2} = 0.05916$

$R = \frac{A}{P} = \frac{5.6}{16.5} = 0.339$ Use 0.34

$R^{2/3} = 0.487$

$V = 93.1 \times 0.487 \times 0.05916$
$V = 2.69$ fps Use 2.7 fps

$Q = AV$
$Q = 5.6 \times 2.7 = 15.1$ cfs

$A = \frac{H}{2}(B + B^1)$

$A = \frac{16}{2}(0.2 + 0.5)$ 0.7

$A = 8.0 \times 0.7 = 5.6$ Sq. Ft.

$P = 16.0 + 0.5 = 16.5'$

<u>Capacity of total street cross section</u>

15.1 cfs
2
30.2 cfs

Assume that the street will run full, and use the velocity associated with this condition to determine the time of concentration (T_c).

<u>Calculate the time of concentration (T_c).</u>

Length of run from the intersection of Catalina Drive and West Flower Street east on West Flower Street to North 48th Drive. South on North 48th Drive to its intersection with West Thomas Road.

Total length of run = 3480 ft.

$$T_c = \frac{\text{Length}}{\text{Velocity} \times 60} = \frac{3480}{2.7 \times 60} = 21.5 \text{ Use 22 min.}$$

Calculate the intensity for a 10 year storm using Precipitation Map 1 and 2 for the 6 and 24 hour values. Obtain the 1 hour value from Fig. 2-2 Rainfall Depth-Duration Diagram, and the 22 minute value from Fig. 3-2 Standard Duration Rainfall Intensity Curves.

I_{10} yr. 24 hr. = 2.4 "/hr.
I_{10} yr. 6 hr. = 2.0 "/hr.
I_{10} yr. 1 hr. = 1.6 "/hr.
I_{10} yr. 22min. = 3.0 "/hr.

Using the same procedure as before calculate the runoff from Drainage Areas 2, 3, 4, and 5.

Drainage Area No. 2
Total Drainage Area = 48.3 AC.

Area in Streets

Street Name	Ft. Width	Ft. Length	Sq. Ft. Area
W. Osborn Rd.	32	1730	55,400
N. 47th Ave.	32	1450	46,400
N. 47th Ave.	70	1370	95,900
W. Earll Ave.	32	470	15,050
W. Thomas Rd.	42	470	19,750

$\frac{232,500}{43,560}$ = 5.34 AC.

All streets in this drainage area are asphaltic and in good condition. Use a "C" value of 0.85.

Compute CA

5.34 AC. Area
0.85 Coefficient
4.54 = CA

Area in Roofs

Area of House

65'
30'
1950 Sq. Ft.

Number of Houses

85

Total Area of House Roofs

1950 Sq. Ft. Area of House
85 No. of Houses in D.A.

$\frac{165,750}{43,560}$ = 3.8 AC.

Use a "C" value of 0.95

Compute CA

CA = 0.95 x 3.8 = 3.61

Area in Lawns

The total drainage area was obtained from the air photo and found to be 48.3 AC.

Summate the other sub-areas and subtract from the gross area to obtain the lawn area.

5.34 AC.	Street Area
3.80 AC.	Roof Area
9.14 AC.	

48.30 AC.	Gross Area
9.14 AC.	
39.16 AC.	Lawn Area

Use "C" value of 0.2

Compute CA

CA = 39.16 x 0.2 = 7.83

Compute the weighted "C" for the entire Drainage Area

Summate CAs for the sub-areas

4.54	Streets
3.61	Roofs
7.83	Lawns
15.98	Total

Divide total CA for the drainage area by the gross area to obtain the weighted "C" value for the drainage area.

$$\frac{15.98}{48.3} = 0.331$$

The average "C" value for this drainage area is low because it is not fully developed. Since this area is in a section of the city that is growing, it will be assumed that this drainage area will develop further during the life of the proposed improvements. Therefore, use a "C" value for a fully improved drainage area such as Drainage Area No. 1 which has an average watershed coefficient of 0.43

Calculate the Time of Concentration (T_c)

Use the same street cross section and assumptions as for Drainage Area No. 1.

Length of run is east on W. Osborn Rd. to N. 47th Ave. south on N. 47th Ave. to its intersection with W. Thomas Rd.

Total length of run = 4545 ft.

$$T_c = \frac{L}{V \times 60} = \frac{4545}{2.7 \times 60} = 28 \text{ min.}$$

Calculate the Intensity

Using the previous procedure I_{10} yr. 28 min. = 2.6 "/hr.

$Q = CAi$
$Q = 0.43 \times 48.3 \times 2.6 = 54.0$ cfs

Drainage Area No. 3
Total Drainage Area = 55.49 acre

Area in Streets

Street Name	Ft. Width	Ft. Length	Sq. Ft. Area
N. 46th Dr.	32	2760	88,320
W. Earll Dr.	32	1010	32,320
W. Osborn Rd.	32	930	29,760
No name Cul-de-sac	32	140	4,480
No name Cul-de-sac	32	180	5,760
No name Cul-de-sac	32	50	1,600
No name Cul-de-sac	32	200	6,400
W. Verde Ln.	32	400	12,800
N. 45th Dr.	32	500	16,000
W. Catalina Dr.	32	960	30,720
W. Thomas Rd.	42	240	10,080
Parking Lot	80	340	27,200
			265,440

$$\frac{265,440}{43,560} = 6.09 \text{ acre}$$

All streets in this drainage area are asphaltic and in good condition. Use a "C" value of 0.85.

Compute CA

6.09 acre area
0.85
5.18

Area in Roofs

A. Houses

1. Area of House
   3600 Sq. Ft.

2. Number of Houses
   103

3. Total Area of House Roofs

3600	Sq. Ft.	Area of House
103		No. of Houses
370,800	Sq. Ft.	Total

B. School

1. Area of Buildings

270'
80'
21,600 Sq. Ft.

2. Number of Buildings

3

3. Total Area of Building Roofs

21,600	Sq. Ft.	Area of Buildings
3		No. of Buildings
64,800	Sq. Ft.	Total

C. Total Roof Area of Houses and Buildings

370,800	Sq. Ft.	House Area
64,800	Sq. Ft.	Building Area
435,600	Sq. Ft.	

435,600/43,500 = 10.0 acre

Use a "C" value of 0.95

Compute CA

0.95
10
9.5

Area in Lawns

The total drainage area was obtained from the air photo and found to be 2,417,000 sq. ft.

$$\frac{2,417,000}{43,560} = 55.49 \text{ acre}$$

Summate the other sub-areas and subtract from the gross area to obtain the lawn area.

6.09 acre	Street Area
10.00 acre	Roof Area
16.09 acre	Subtotal

55.49 acre	Gross Area
16.09 acre	
39.40 acre	Lawn Area

Use "C" value of 0.2

Compute CA

CA = 39.4 x 0.2 = 7.88

Compute the weighted "C" for the entire drainage area.

Summate CAs for the Sub-Areas
5.18
9.50
7.88
22.56

Divide the total CA for the drainage area by the gross area to obtain the weighted "C" value for the drainage area.

$$\frac{22.56}{55.49} = 0.41$$

Calculate the Time of Concentration (T_c)

Use the same street cross section and assumptions as for Drainage Area No. 1.

Length of run is east on W. Osborn Rd. to N. 46th Dr. south on N. 46th Dr. to its intersection with W. Thomas Rd.

Total length of run = 3480 ft.

$$T_c = \frac{L}{V \times 60} = \frac{3480}{2.7 \times 60} = 21.5 \text{ min. use 22 min.}$$

Calculate the Intensity

Using the previous procedure I_{10}yr. 22 min. = 3.0 "/hr.

Q = 22.56 x 3.0 = 67.68 cfs Use 68 cfs

Drainage Area No. 4

Total Drainage Areas = 54.5 acre

Area in Streets

Street Name	Ft. Width	Ft. Length	Sq. Ft. Area
W. Osborn Rd.	32	350	11,200
N. 44th Ln.	32	1950	62,400
N. 45th Dr.	32	490	15,680
W. Verde Ln.	32	1490	47,680
No name Cul-de-sac	32	150	4,800
N. 43rd Ln.	32	620	19,840
W. Avalon Dr.	32	1150	36,800
W. Catalina Dr.	32	830	26,560
W. Earll Dr.	32	990	31,680
W. Thomas Rd.	42	970	40,740
			297,380 Sq. Ft.
			43,560 = 6.83 AC.

All streets in this drainage area are asphaltic and in good condition.
Use a "C" value of 0.85.

Compute CA

6.83 AC. Area
0.85
5.81

Area in Roofs

1. Area of House

35'
70'
2450 Sq. Ft.

2. Number of Houses

181

3. Total Area of House Roofs

2450 Sq. Ft. Area of House
181 No. of Houses
443,450 Sq. Ft.
43,560 = 10.18 AC.

Use a "C" value of 0.95

Compute CA

0.95
10.18
9.67

Area in Lawns

The total drainage area was obtained from the air photo and found to be 54.5 AC.

Summate the other sub-areas and subtract from the gross area to obtain the lawn area.

6.83 AC.	Street Area
10.18 AC.	Roof Area
17.01 AC.	

54.50 AC.	Gross Area
17.01 AC.	
37.49 AC.	Lawn Area

Use "C" value of 0.2

Compute CA

CA = 37.49 x 0.2 = 7.50

Compute the weighted "C" for the entire drainage area.

Summate CAs for the Sub-Areas

5.81
9.67
7.50
22.98

Divide total CA for the drainage area by the gross area to obtain the weighted "C" value for the drainage area.

$$\frac{22.98}{54.5} = 0.42$$

Calculate the Time of Concentration (T_c)

Use the same street cross section and assumptions as for Drainage Area No. 1.

Length of run is east on Osborn Rd. to N. 44th Ln. south on N. 44th Ln. to W. Earll Dr. east on W. Earll Dr. to N. 43rd Ln. south on N. 43rd Ln. to W. Verde Ln. west on W. Verde Ln. to N. 44th Ln. south on N. 44th Ln. to Thomas Rd.

Total length of run = 3955 ft.

$$T_c = \frac{L}{V \times 60} = \frac{3955}{2.7 \times 60} = 24.4 \text{ min. Use 24 min.}$$

Calculate the Intensity

Using the previous procedure $I_{10\ yr.\ 24\ min.} = 2.9$ "/hr.

Q = 22.98 x 2.9 = 66.64 Use 67 cfs

Drainage Area No. 5
Total Drainage Area = 14.72 acres

Planned Shopping Center

This watershed is not fully developed, but is planned to develop into a shopping area.

Parking Area

420'	150'
x 330'	x 270'
138,600 Sq. Ft.	40,500 Sq. Ft.
40,500 Sq. Ft.	
179,100 Sq. Ft.	

179,100/43,560 = 4.1 acre

Parking area is asphaltic and in good condition. Use a "C" value of 0.85

Compute CA

4.1 acres
0.85
3.49

Roof Area

1. Area of Building

250'
x 170'
42,500 Sq. Ft.
42,500/43,560 = 0.9756 Use 0.98 acre
Use a "C" value of 0.95

Compute CA

0.98 acre
0.95
0.93 = CA

Compute the weighted "C" for the entire drainage area.

Summate CAs for the Sub-Areas

$$\begin{array}{r} 3.49 \\ \underline{0.93} \\ 4.42 \end{array}$$

Divide total CA for the drainage area by the drainage area to obtain the weighted "C" value for the drainage area.

$$\frac{4.42}{5.08} = 0.87$$

Use a "C" value of 0.87 for the entire area.

The total drainage area was obtained from the air photo and found to be 14.72 acres.

Compute CA

$$\begin{array}{r} 14.72 \\ \underline{0.87} \\ 12.81 \end{array}$$

Calculate the Time of Concentration (T_c)

Try using Fig. 3-1 Time of Concentration for Overland Flow from the manual "Hydrologic Design for Highway Drainage in Arizona".

Assume the slope to be 0.005 '/ft.
length from air photo is 430 ft.

t_c = 9.3 min. Use 10 min.

$$V = \frac{L}{9.3 \times 60} = \frac{430}{558} = 0.77 \text{ fps}$$

Any future improvements to the parking area will tend to increase the overland velocity rather than decrease it.

Calculate the Intensity

Using the previous procedure I_{10} yr. 10 min. = 4.4 "/hr.

Q = 12.81 x 4.4 = 56.36 cfs Use 56 cfs

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: RUNOFF CALCULATION SHEET

LOCATION DATA
HIGHWAY West Thomas Road
LOCATION 43rd Avenue to 51st Avenue
PROJECT NO. XB- 930(39)

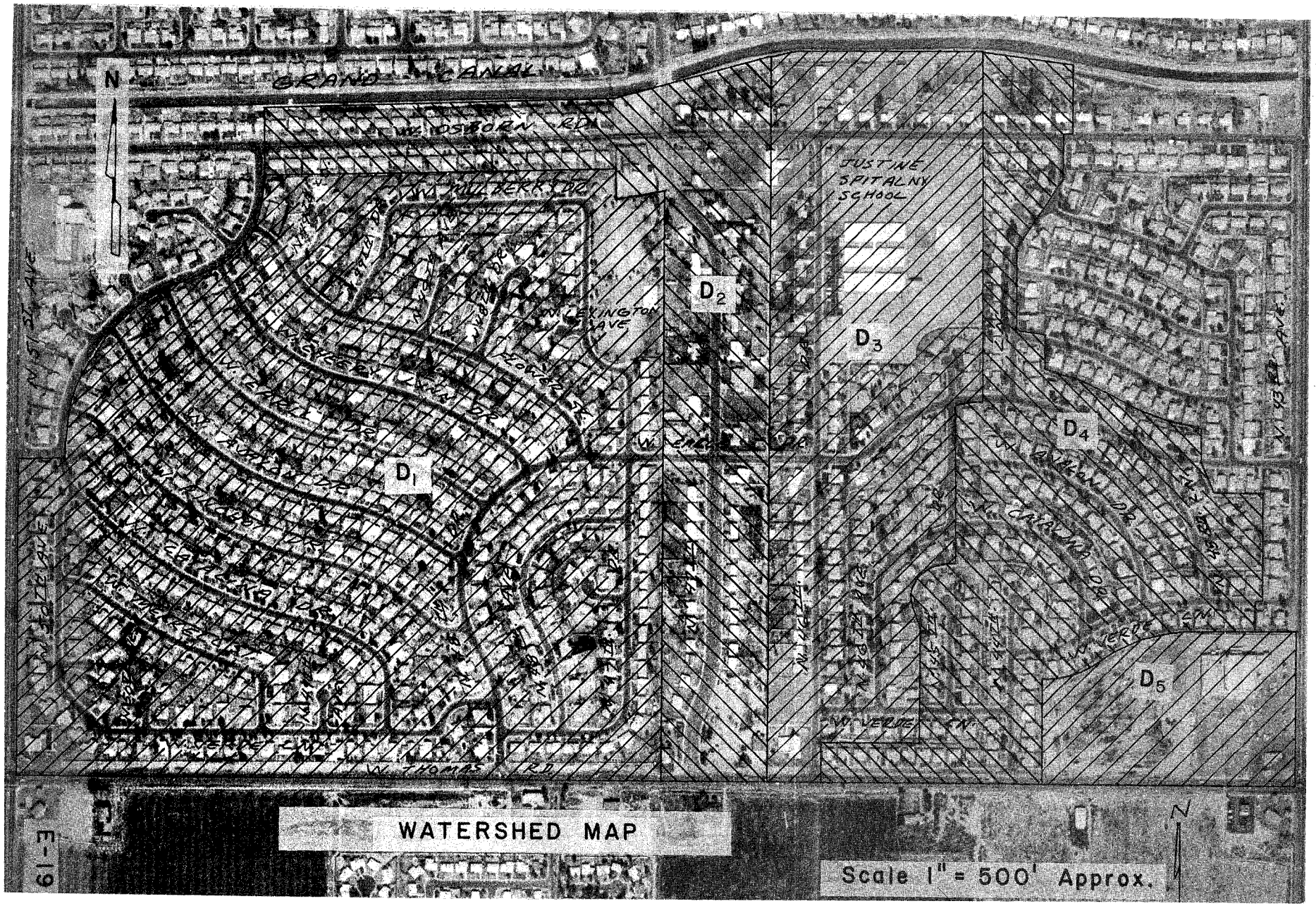
*D.A. not fully developed. Use higher weighted "C" for future development.

DESIGN DATA
FREQUENCY 10 yr.
P_6 = 2.0 "/HR. P_{24} = 2.4 "/HR. P_1 = 1.6 "/HR.

+ This watershed has not fully developed into a planned shopping center, but will in the near future. Use the weighted "C" for the entire area.

DRAINAGE AREA			STREETS			ROOFS			PARKING AREA			PARKS AND LAWNS							T_c	I	Q
No.	Sta. to Sta.	Acres	C	A	CA	C	A	CA	C	A	CA	C	A	CA	C	A	CA	ΣCA	min	in/hr	cfs
1	N. 48th Dr.	137.0	0.85	21.34	18.14	0.95	24.28	23.07	0.8	0.45	0.36	0.2	90.93	18.19				59.76	22	3.0	179
2	N. 47th Ave.	48.3	0.85	5.34	4.54	0.95	3.8	3.61	0	0	0	0.2	39.16	7.83				15.98*			
2	Modified																	20.77	28	2.6	54
3	N. 46th Dr.	55.49	0.85	6.09	5.18	0.95	10.0	9.5	0	0	0	0.2	39.4	7.88				22.56	22	3.0	68
4	N. 44th Ln.	54.50	0.85	6.83	5.81	0.95	10.18	9.67	0	0	0	0.2	37.49	7.50				22.98	24	2.9	67
5	Planned Shopping Center	14.72	0	0	0	0.95	0.98	0.93	0.85	4.1	3.49							4.42+			
5	Modified	14.72																12.81	10	4.4	56

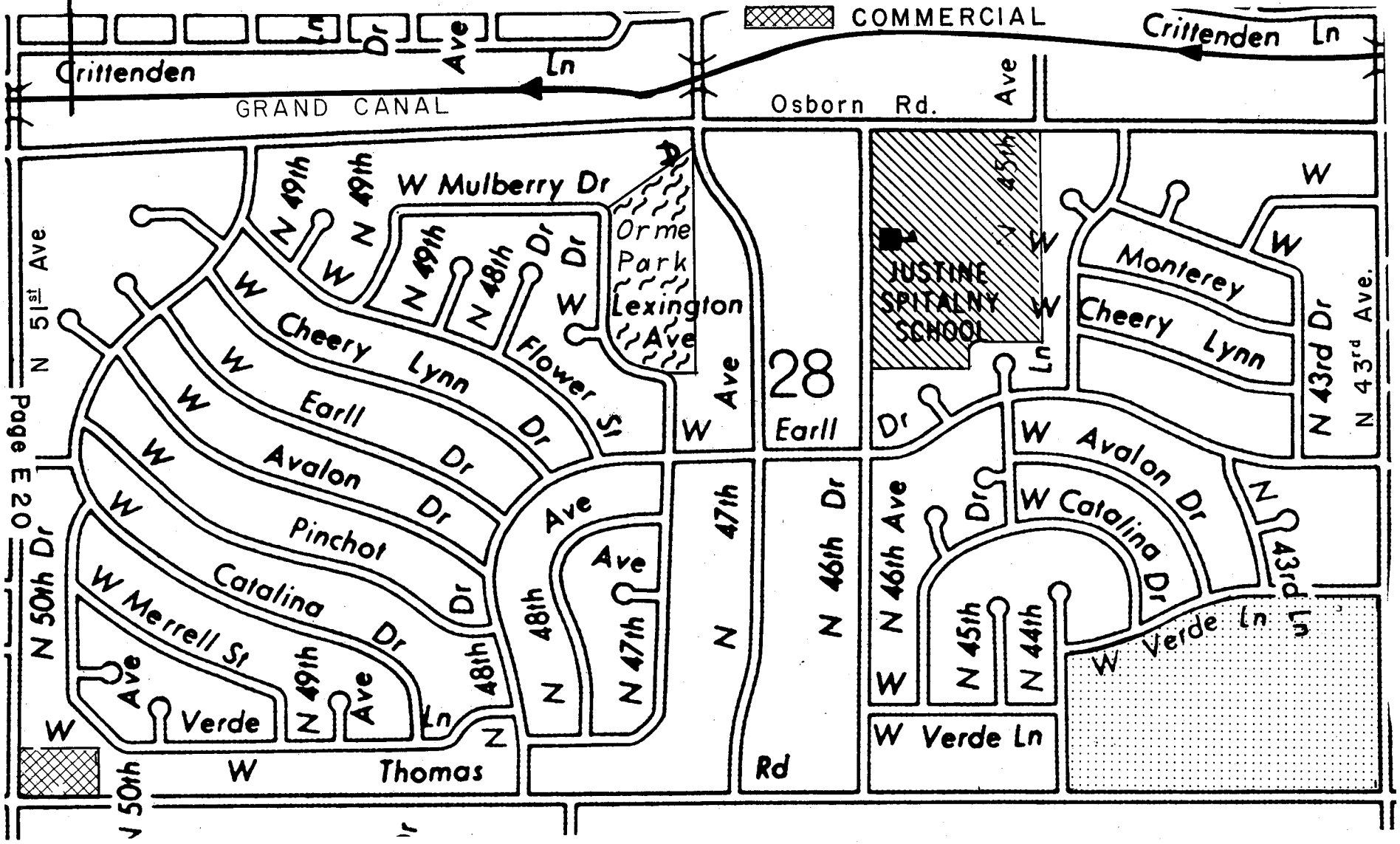
Computed by W.C.N. Checked by T.B.S. Date Oct. 5, 1972


N
GRAND CANAL
W. OSBORN RD.
W. MILDERKOD
JUSTINE SPITALNY SCHOOL
D2
D3
D4
D5
D1
N. LEXINGTON AVE
W. THOMAS RD.
N. VERDE
N. 93rd AVE.
N. 75th AVE.
N. 67th AVE.
N. 59th AVE.
N. 51st AVE.
N. 43rd AVE.
N. 35th AVE.
N. 27th AVE.
N. 19th AVE.
N. 11th AVE.
N. 3rd AVE.
N. 1st ST.
N. 1st AVE.
N. 7th ST.
N. 7th AVE.
N. 15th AVE.
N. 23rd AVE.
N. 31st AVE.
N. 39th AVE.
N. 47th AVE.
N. 55th AVE.
N. 63rd AVE.
N. 71st AVE.
N. 79th AVE.
N. 87th AVE.
N. 95th AVE.
N. 103rd AVE.
N. 111th AVE.
N. 119th AVE.
N. 127th AVE.
N. 135th AVE.
N. 143rd AVE.
N. 151st AVE.
N. 159th AVE.
N. 167th AVE.
N. 175th AVE.
N. 183rd AVE.
N. 191st AVE.
N. 199th AVE.
N. 207th AVE.
N. 215th AVE.
N. 223rd AVE.
N. 231st AVE.
N. 239th AVE.
N. 247th AVE.
N. 255th AVE.
N. 263rd AVE.
N. 271st AVE.
N. 279th AVE.
N. 287th AVE.
N. 295th AVE.
N. 303rd AVE.
N. 311th AVE.
N. 319th AVE.
N. 327th AVE.
N. 335th AVE.
N. 343rd AVE.
N. 351st AVE.
N. 359th AVE.
N. 367th AVE.
N. 375th AVE.
N. 383rd AVE.
N. 391st AVE.
N. 399th AVE.
N. 407th AVE.
N. 415th AVE.
N. 423rd AVE.
N. 431st AVE.
N. 439th AVE.
N. 447th AVE.
N. 455th AVE.
N. 463rd AVE.
N. 471st AVE.
N. 479th AVE.
N. 487th AVE.
N. 495th AVE.
N. 503rd AVE.
N. 511th AVE.
N. 519th AVE.
N. 527th AVE.
N. 535th AVE.
N. 543rd AVE.
N. 551st AVE.
N. 559th AVE.
N. 567th AVE.
N. 575th AVE.
N. 583rd AVE.
N. 591st AVE.
N. 599th AVE.
N. 607th AVE.
N. 615th AVE.
N. 623rd AVE.
N. 631st AVE.
N. 639th AVE.
N. 647th AVE.
N. 655th AVE.
N. 663rd AVE.
N. 671st AVE.
N. 679th AVE.
N. 687th AVE.
N. 695th AVE.
N. 703rd AVE.
N. 711th AVE.
N. 719th AVE.
N. 727th AVE.
N. 735th AVE.
N. 743rd AVE.
N. 751st AVE.
N. 759th AVE.
N. 767th AVE.
N. 775th AVE.
N. 783rd AVE.
N. 791st AVE.
N. 799th AVE.
N. 807th AVE.
N. 815th AVE.
N. 823rd AVE.
N. 831st AVE.
N. 839th AVE.
N. 847th AVE.
N. 855th AVE.
N. 863rd AVE.
N. 871st AVE.
N. 879th AVE.
N. 887th AVE.
N. 895th AVE.
N. 903rd AVE.
N. 911th AVE.
N. 919th AVE.
N. 927th AVE.
N. 935th AVE.
N. 943rd AVE.
N. 951st AVE.
N. 959th AVE.
N. 967th AVE.
N. 975th AVE.
N. 983rd AVE.
N. 991st AVE.
N. 999th AVE.
N. 1007th AVE.
N. 1015th AVE.
N. 1023rd AVE.
N. 1031st AVE.
N. 1039th AVE.
N. 1047th AVE.
N. 1055th AVE.
N. 1063rd AVE.
N. 1071st AVE.
N. 1079th AVE.
N. 1087th AVE.
N. 1095th AVE.
N. 1103rd AVE.
N. 1111th AVE.
N. 1119th AVE.
N. 1127th AVE.
N. 1135th AVE.
N. 1143rd AVE.
N. 1151st AVE.
N. 1159th AVE.
N. 1167th AVE.
N. 1175th AVE.
N. 1183rd AVE.
N. 1191st AVE.
N. 1199th AVE.
N. 1207th AVE.
N. 1215th AVE.
N. 1223rd AVE.
N. 1231st AVE.
N. 1239th AVE.
N. 1247th AVE.
N. 1255th AVE.
N. 1263rd AVE.
N. 1271st AVE.
N. 1279th AVE.
N. 1287th AVE.
N. 1295th AVE.
N. 1303rd AVE.
N. 1311th AVE.
N. 1319th AVE.
N. 1327th AVE.
N. 1335th AVE.
N. 1343rd AVE.
N. 1351st AVE.
N. 1359th AVE.
N. 1367th AVE.
N. 1375th AVE.
N. 1383rd AVE.
N. 1391st AVE.
N. 1399th AVE.
N. 1407th AVE.
N. 1415th AVE.
N. 1423rd AVE.
N. 1431st AVE.
N. 1439th AVE.
N. 1447th AVE.
N. 1455th AVE.
N. 1463rd AVE.
N. 1471st AVE.
N. 1479th AVE.
N. 1487th AVE.
N. 1495th AVE.
N. 1503rd AVE.
N. 1511th AVE.
N. 1519th AVE.
N. 1527th AVE.
N. 1535th AVE.
N. 1543rd AVE.
N. 1551st AVE.
N. 1559th AVE.
N. 1567th AVE.
N. 1575th AVE.
N. 1583rd AVE.
N. 1591st AVE.
N. 1599th AVE.
N. 1607th AVE.
N. 1615th AVE.
N. 1623rd AVE.
N. 1631st AVE.
N. 1639th AVE.
N. 1647th AVE.
N. 1655th AVE.
N. 1663rd AVE.
N. 1671st AVE.
N. 1679th AVE.
N. 1687th AVE.
N. 1695th AVE.
N. 1703rd AVE.
N. 1711th AVE.
N. 1719th AVE.
N. 1727th AVE.
N. 1735th AVE.
N. 1743rd AVE.
N. 1751st AVE.
N. 1759th AVE.
N. 1767th AVE.
N. 1775th AVE.
N. 1783rd AVE.
N. 1791st AVE.
N. 1799th AVE.
N. 1807th AVE.
N. 1815th AVE.
N. 1823rd AVE.
N. 1831st AVE.
N. 1839th AVE.
N. 1847th AVE.
N. 1855th AVE.
N. 1863rd AVE.
N. 1871st AVE.
N. 1879th AVE.
N. 1887th AVE.
N. 1895th AVE.
N. 1903rd AVE.
N. 1911th AVE.
N. 1919th AVE.
N. 1927th AVE.
N. 1935th AVE.
N. 1943rd AVE.
N. 1951st AVE.
N. 1959th AVE.
N. 1967th AVE.
N. 1975th AVE.
N. 1983rd AVE.
N. 1991st AVE.
N. 1999th AVE.
N. 2007th AVE.
N. 2015th AVE.
N. 2023rd AVE.
N. 2031st AVE.
N. 2039th AVE.
N. 2047th AVE.
N. 2055th AVE.
N. 2063rd AVE.
N. 2071st AVE.
N. 2079th AVE.
N. 2087th AVE.
N. 2095th AVE.
N. 2103rd AVE.
N. 2111th AVE.
N. 2119th AVE.
N. 2127th AVE.
N. 2135th AVE.
N. 2143rd AVE.
N. 2151st AVE.
N. 2159th AVE.
N. 2167th AVE.
N. 2175th AVE.
N. 2183rd AVE.
N. 2191st AVE.
N. 2199th AVE.
N. 2207th AVE.
N. 2215th AVE.
N. 2223rd AVE.
N. 2231st AVE.
N. 2239th AVE.
N. 2247th AVE.
N. 2255th AVE.
N. 2263rd AVE.
N. 2271st AVE.
N. 2279th AVE.
N. 2287th AVE.
N. 2295th AVE.
N. 2303rd AVE.
N. 2311th AVE.
N. 2319th AVE.
N. 2327th AVE.
N. 2335th AVE.
N. 2343rd AVE.
N. 2351st AVE.
N. 2359th AVE.
N. 2367th AVE.
N. 2375th AVE.
N. 2383rd AVE.
N. 2391st AVE.
N. 2399th AVE.
N. 2407th AVE.
N. 2415th AVE.
N. 2423rd AVE.
N. 2431st AVE.
N. 2439th AVE.
N. 2447th AVE.
N. 2455th AVE.
N. 2463rd AVE.
N. 2471st AVE.
N. 2479th AVE.
N. 2487th AVE.
N. 2495th AVE.
N. 2503rd AVE.
N. 2511th AVE.
N. 2519th AVE.
N. 2527th AVE.
N. 2535th AVE.
N. 2543rd AVE.
N. 2551st AVE.
N. 2559th AVE.
N. 2567th AVE.
N. 2575th AVE.
N. 2583rd AVE.
N. 2591st AVE.
N. 2599th AVE.
N. 2607th AVE.
N. 2615th AVE.
N. 2623rd AVE.
N. 2631st AVE.
N. 2639th AVE.
N. 2647th AVE.
N. 2655th AVE.
N. 2663rd AVE.
N. 2671st AVE.
N. 2679th AVE.
N. 2687th AVE.
N. 2695th AVE.
N. 2703rd AVE.
N. 2711th AVE.
N. 2719th AVE.
N. 2727th AVE.
N. 2735th AVE.
N. 2743rd AVE.
N. 2751st AVE.
N. 2759th AVE.
N. 2767th AVE.
N. 2775th AVE.
N. 2783rd AVE.
N. 2791st AVE.
N. 2799th AVE.
N. 2807th AVE.
N. 2815th AVE.
N. 2823rd AVE.
N. 2831st AVE.
N. 2839th AVE.
N. 2847th AVE.
N. 2855th AVE.
N. 2863rd AVE.
N. 2871st AVE.
N. 2879th AVE.
N. 2887th AVE.
N. 2895th AVE.
N. 2903rd AVE.
N. 2911th AVE.
N. 2919th AVE.
N. 2927th AVE.
N. 2935th AVE.
N. 2943rd AVE.
N. 2951st AVE.
N. 2959th AVE.
N. 2967th AVE.
N. 2975th AVE.
N. 2983rd AVE.
N. 2991st AVE.
N. 2999th AVE.
N. 3007th AVE.
N. 3015th AVE.
N. 3023rd AVE.
N. 3031st AVE.
N. 3039th AVE.
N. 3047th AVE.
N. 3055th AVE.
N. 3063rd AVE.
N. 3071st AVE.
N. 3079th AVE.
N. 3087th AVE.
N. 3095th AVE.
N. 3103rd AVE.
N. 3111th AVE.
N. 3119th AVE.
N. 3127th AVE.
N. 3135th AVE.
N. 3143rd AVE.
N. 3151st AVE.
N. 3159th AVE.
N. 3167th AVE.
N. 3175th AVE.
N. 3183rd AVE.
N. 3191st AVE.
N. 3199th AVE.
N. 3207th AVE.
N. 3215th AVE.
N. 3223rd AVE.
N. 3231st AVE.
N. 3239th AVE.
N. 3247th AVE.
N. 3255th AVE.
N. 3263rd AVE.
N. 3271st AVE.
N. 3279th AVE.
N. 3287th AVE.
N. 3295th AVE.
N. 3303rd AVE.
N. 3311th AVE.
N. 3319th AVE.
N. 3327th AVE.
N. 3335th AVE.
N. 3343rd AVE.
N. 3351st AVE.
N. 3359th AVE.
N. 3367th AVE.
N. 3375th AVE.
N. 3383rd AVE.
N. 3391st AVE.
N. 3399th AVE.
N. 3407th AVE.
N. 3415th AVE.
N. 3423rd AVE.
N. 3431st AVE.
N. 3439th AVE.
N. 3447th AVE.
N. 3455th AVE.
N. 3463rd AVE.
N. 3471st AVE.
N. 3479th AVE.
N. 3487th AVE.
N. 3495th AVE.
N. 3503rd AVE.
N. 3511th AVE.
N. 3519th AVE.
N. 3527th AVE.
N. 3535th AVE.
N. 3543rd AVE.
N. 3551st AVE.
N. 3559th AVE.
N. 3567th AVE.
N. 3575th AVE.
N. 3583rd AVE.
N. 3591st AVE.
N. 3599th AVE.
N. 3607th AVE.
N. 3615th AVE.
N. 3623rd AVE.
N. 3631st AVE.
N. 3639th AVE.
N. 3647th AVE.
N. 3655th AVE.
N. 3663rd AVE.
N. 3671st AVE.
N. 3679th AVE.
N. 3687th AVE.
N. 3695th AVE.
N. 3703rd AVE.
N. 3711th AVE.
N. 3719th AVE.
N. 3727th AVE.
N. 3735th AVE.
N. 3743rd AVE.
N. 3751st AVE.
N. 3759th AVE.
N. 3767th AVE.
N. 3775th AVE.
N. 3783rd AVE.
N. 3791st AVE.
N. 3799th AVE.
N. 3807th AVE.
N. 3815th AVE.
N. 3823rd AVE.
N. 3831st AVE.
N. 3839th AVE.
N. 3847th AVE.
N. 3855th AVE.
N. 3863rd AVE.
N. 3871st AVE.
N. 3879th AVE.
N. 3887th AVE.
N. 3895th AVE.
N. 3903rd AVE.
N. 3911th AVE.
N. 3919th AVE.
N. 3927th AVE.
N. 3935th AVE.
N. 3943rd AVE.
N. 3951st AVE.
N. 3959th AVE.
N. 3967th AVE.
N. 3975th AVE.
N. 3983rd AVE.
N. 3991st AVE.
N. 3999th AVE.
N. 4007th AVE.
N. 4015th AVE.
N. 4023rd AVE.
N. 4031st AVE.
N. 4039th AVE.
N. 4047th AVE.
N. 4055th AVE.
N. 4063rd AVE.
N. 4071st AVE.
N. 4079th AVE.
N. 4087th AVE.
N. 4095th AVE.
N. 4103rd AVE.
N. 4111th AVE.
N. 4119th AVE.
N. 4127th AVE.
N. 4135th AVE.
N. 4143rd AVE.
N. 4151st AVE.
N. 4159th AVE.
N. 4167th AVE.
N. 4175th AVE.
N. 4183rd AVE.
N. 4191st AVE.
N. 4199th AVE.
N. 4207th AVE.
N. 4215th AVE.
N. 4223rd AVE.
N. 4231st AVE.
N. 4239th AVE.
N. 4247th AVE.
N. 4255th AVE.
N. 4263rd AVE.
N. 4271st AVE.
N. 4279th AVE.
N. 4287th AVE.
N. 4295th AVE.
N. 4303rd AVE.
N. 4311th AVE.
N. 4319th AVE.
N. 4327th AVE.
N. 4335th AVE.
N. 4343rd AVE.
N. 4351st AVE.
N. 4359th AVE.
N. 4367th AVE.
N. 4375th AVE.
N. 4383rd AVE.
N. 4391st AVE.
N. 4399th AVE.
N. 4407th AVE.
N. 4415th AVE.
N. 4423rd AVE.
N. 4431st AVE.
N. 4439th AVE.
N. 4447th AVE.
N. 4455th AVE.
N. 4463rd AVE.
N. 4471st AVE.
N. 4479th AVE.
N. 4487th AVE.
N. 4495th AVE.
N. 4503rd AVE.
N. 4511th AVE.
N. 4519th AVE.
N. 4527th AVE.
N. 4535th AVE.
N. 4543rd AVE.
N. 4551st AVE.
N. 4559th AVE.
N. 4567th AVE.
N. 4575th AVE.
N. 4583rd AVE.
N. 4591st AVE.
N. 4599th AVE.
N. 4607th AVE.
N. 4615th AVE.
N. 4623rd AVE.
N. 4631st AVE.
N. 4639th AVE.
N. 4647th AVE.
N. 4655th AVE.
N. 4663rd AVE.
N. 4671st AVE.
N. 4679th AVE.
N. 4687th AVE.
N. 4695th AVE.
N. 4703rd AVE.
N. 4711th AVE.
N. 4719th AVE.
N. 4727th AVE.
N. 4735th AVE.
N. 4743rd AVE.
N. 4751st AVE.
N. 4759th AVE.
N. 4767th AVE.
N. 4775th AVE.
N. 4783rd AVE.
N. 4791st AVE.
N. 4799th AVE.
N. 4807th AVE.
N. 4815th AVE.
N. 4823rd AVE.
N. 4831st AVE.
N. 4839th AVE.
N. 4847th AVE.
N. 4855th AVE.
N. 4863rd AVE.
N. 4871st AVE.
N. 4879th AVE.
N. 4887th AVE.
N. 4895th AVE.
N. 4903rd AVE.
N. 4911th AVE.
N. 4919th AVE.
N. 4927th AVE.
N. 4935th AVE.
N. 4943rd AVE.
N. 4951st AVE.
N. 4959th AVE.
N. 4967th AVE.
N. 4975th AVE.
N. 4983rd AVE.
N. 4991st AVE.
N. 4999th AVE.
N. 5007th AVE.
N. 5015th AVE.
N. 5023rd AVE.
N. 5031st AVE.
N. 5039th AVE.
N. 5047th AVE.
N. 5055th AVE.
N. 5063rd AVE.
N. 5071st AVE.
N. 5079th AVE.
N. 5087th AVE.
N. 5095th AVE.
N. 5103rd AVE.
N. 5111th AVE.
N. 5119th AVE.
N. 5127th AVE.
N. 5135th AVE.
N. 5143rd AVE.
N. 5151st AVE.
N. 5159th AVE.
N. 5167th AVE.
N. 5175th AVE.
N. 5183rd AVE.
N. 5191st AVE.
N. 5199th AVE.
N. 5207th AVE.
N. 5215th AVE.
N. 5223rd AVE.
N. 5231st AVE.
N. 5239th AVE.
N. 5247th AVE.
N. 5255th AVE.
N. 5263rd AVE.
N. 5271st AVE.
N. 5279th AVE.
N. 5287th AVE.
N. 5295th AVE.
N. 5303rd AVE.
N. 5311th AVE.
N. 5319th AVE.
N. 5327th AVE.
N. 5335th AVE.
N. 5343rd AVE.
N. 5351st AVE.
N. 5359th AVE.
N. 5367th AVE.
N. 5375th AVE.
N. 5383rd AVE.
N. 5391st AVE.
N. 5399th AVE.
N. 5407th AVE.
N. 5415th AVE.
N. 5423rd AVE.
N. 5431st AVE.
N. 5439th AVE.
N. 5447th AVE.
N. 5455th AVE.
N. 5463rd AVE.
N. 5471st AVE.
N. 5479th AVE.
N. 5487th AVE.
N. 5495th AVE.
N. 5503rd AVE.
N. 5511th AVE.
N. 5519th AVE.
N. 5527th AVE.
N. 5535th AVE.
N. 5543rd AVE.
N. 5551st AVE.
N. 5559th AVE.
N. 5567th AVE.
N. 5575th AVE.
N. 5583rd AVE.
N. 5591st AVE.
N. 5599th AVE.
N. 5607th AVE.
N. 5615th AVE.
N. 5623rd AVE.
N. 5631st AVE.
N. 5639th AVE.
N. 5647th AVE.
N. 5655th AVE.
N. 5663rd AVE.
N. 5671st AVE.
N. 5679th AVE.
N. 5687th AVE.
N. 5695th AVE.
N. 5703rd AVE.
N. 5711th AVE.
N. 5719th AVE.
N. 5727th AVE.
N. 5735th AVE.
N. 5743rd AVE.
N. 5751st AVE.
N. 5759th AVE.
N. 5767th AVE.
N. 5775th AVE.
N. 5783rd AVE.
N. 5791st AVE.
N. 5799th AVE.
N. 5807th AVE.
N. 5815th AVE.
N. 5823rd AVE.
N. 5831st AVE.
N. 5839th AVE.
N. 5847th AVE.
N. 5855th AVE.
N. 5863rd AVE.
N. 5871st AVE.
N. 5879th AVE.
N. 5887th AVE.
N. 5895th AVE.
N. 5903rd AVE.
N. 5911th AVE.
N. 5919th AVE.
N. 5927th AVE.
N. 5935th AVE.
N. 5943rd AVE.
N. 5951st AVE.
N. 5959th AVE.
N. 5967th AVE.
N. 5975th AVE.
N. 5983rd AVE.
N. 5991st AVE.
N. 5999th AVE.
N. 6007th AVE.
N. 6015th AVE.
N. 6023rd AVE.
N. 6031st AVE.
N. 6039th AVE.
N. 6047th AVE.
N. 6055th AVE.
N. 6063rd AVE.
N. 6071st AVE.
N. 6079th AVE.
N. 6087th AVE.
N. 6095th AVE.
N. 6103rd AVE.
N. 6111th AVE.
N. 6119th AVE.
N. 6127th AVE.
N. 6135th AVE.
N. 6143rd AVE.
N. 6151st AVE.
N. 6159th AVE.
N. 6167th AVE.
N. 6175th AVE.
N. 6183rd AVE.
N. 6191st AVE.
N. 6199th AVE.
N. 6207th AVE.
N. 6215th AVE.
N. 6223rd AVE.
N. 6231st AVE.
N. 6239th AVE.
N. 6247th AVE.
N. 6255th AVE.
N. 6263rd AVE.
N. 6271st AVE.
N. 6279th AVE.
N. 6287th AVE.
N. 6295th AVE.
N. 6303rd AVE.
N. 6311th AVE.
N. 6319th AVE.
N. 6327th AVE.
N. 6335th AVE.
N. 6343rd AVE.
N. 6351st AVE.
N. 6359th AVE.
N. 6367th AVE.
N. 6375th AVE.
N. 6383rd AVE.
N. 6391st AVE.
N. 6399th AVE.
N. 6407th AVE.
N. 6415th AVE.
N. 6423rd AVE.
N. 6431st AVE.
N. 6439th AVE.
N. 6447th AVE.
N. 6455th AVE.
N. 6463rd AVE.
N. 6471st AVE.
N. 6479th AVE.
N. 6487th AVE.
N. 6495th AVE.
N. 6503rd AVE.
N. 6511th AVE.
N. 6519th AVE.
N. 6527th AVE.
N. 6535th AVE.
N. 6543rd AVE.
N. 6551st AVE.
N. 6559th AVE.
N. 6567th AVE.
N. 6575th AVE.
N. 6583rd AVE.
N. 6591st AVE.
N. 6599th AVE.
N. 6607th AVE.
N. 6615th AVE.
N. 6623rd AVE.
N. 6631st AVE.
N. 6639th AVE.
N. 6647th AVE.
N. 6655th AVE.
N. 6663rd AVE.
N. 6671st AVE.
N. 6679th AVE.
N. 6687th AVE.
N. 6695th AVE.
N. 6703rd AVE.
N. 6711th AVE.
N. 6719th AVE.
N. 6727th AVE.
N. 6735th AVE.
N. 6743rd AVE.
N. 6751st AVE.
N. 6759th AVE.
N. 6767th AVE.
N. 6775th AVE.
N. 6783rd AVE.
N. 6791st AVE.
N. 6799th AVE.
N. 6807th AVE.
N. 6815th AVE.
N. 6823rd AVE.
N. 6831st AVE.
N. 6839th AVE.
N. 6847th AVE.
N. 6855th AVE.
N. 6863rd AVE.
N. 6871st AVE.
N. 6879th AVE.
N. 6887th AVE.
N. 6895th AVE.
N. 6903rd AVE.
N. 6911th AVE.
N. 6919th AVE.
N. 6927th AVE.
N. 6935th AVE.
N. 6943rd AVE.
N. 6951st AVE.
N. 6959th AVE.
N. 6967th AVE.
N. 6975th AVE.
N. 6983rd AVE.
N. 6991st AVE.
N. 6999th AVE.
N. 7007th AVE.
N. 7015th AVE.
N. 7023rd AVE.
N. 7031st AVE.
N. 7039th AVE.
N. 7047th AVE.
N. 7055th AVE.
N. 7063rd AVE.
N. 7071st AVE.
N. 7079th AVE.
N. 7087th AVE.
N. 7095th AVE.
N. 7103rd AVE.
N. 7111th AVE.
N. 7119th AVE.
N. 7127th AVE.
N. 7135th AVE.
N. 7143rd AVE.
N. 7151st AVE.
N. 7159th AVE.
N. 7167th AVE.
N. 7175th AVE.
N. 7183rd AVE.
N. 7191st AVE.
N. 7199th AVE.
N. 7207th AVE.
N. 7215th AVE.
N. 7223rd AVE.
N. 7231st AVE.
N. 7239th AVE.
N. 7247th AVE.
N. 7255th AVE.
N. 7263rd AVE.
N. 7271st AVE.
N. 7279th AVE.
N. 7287th AVE.
N. 7295th AVE.
N. 7303rd AVE.
N. 7311th AVE.
N. 7319th AVE.
N. 7327th AVE.
N. 7335th AVE.
N. 7343rd AVE.
N. 7351st AVE.
N. 7359th AVE.
N. 7367th AVE.
N. 7375th AVE.
N. 7383rd AVE.
N. 7391st AVE.
N. 7399th AVE.
N. 7407th AVE.
N. 7415th AVE.
N. 7423rd AVE.
N. 7431st AVE.
N. 7439th AVE.
N. 7447th AVE.
N. 7455th AVE.
N. 7463rd AVE.
N. 7471st AVE.
N. 7479th AVE.
N. 7487th AVE.
N. 7495th AVE.
N. 7503rd AVE.
N. 7511th AVE.
N. 7519th AVE.
N. 7527th AVE.
N. 7535th AVE.
N. 7543rd AVE.
N. 7551st AVE.
N. 7559th AVE.
N. 7567th AVE.
N. 7575th AVE.
N. 7583rd AVE.
N. 7591st AVE.
N. 7599th AVE.
N. 7607th AVE.
N. 7615th AVE.
N. 7623rd AVE.
N. 7631st AVE.
N. 7639th AVE.
N. 7647th AVE.
N. 7655th AVE.
N. 7663rd AVE.
N. 7671st AVE.
N. 7679th AVE.
N. 7687th AVE.
N. 7695th AVE.
N. 7703rd AVE.
N. 7711th AVE.
N. 7719th AVE.
N. 7727th AVE.
N. 7735th AVE.
N. 7743rd AVE.
N. 7751st AVE.
N. 7759th AVE.
N. 7767th AVE.
N. 7775th AVE.
N. 7783rd AVE.
N. 7791st AVE.
N. 7799th AVE.
N. 7807th AVE.
N. 7815th AVE.
N. 7823rd AVE.
N. 7831st AVE.
N. 7839th AVE.
N. 7847th AVE.
N. 7855th AVE.
N. 7863rd AVE.
N. 7871st AVE.
N. 7879th AVE.
N. 7887th AVE.
N. 7895th AVE.
N. 7903rd AVE.
N. 7911th AVE.
N. 7919th AVE.
N. 7927th AVE.
N. 7935th AVE.
N. 7943rd AVE.
N. 7951st AVE.
N. 7959th AVE.
N. 7967th AVE.
N. 7975th AVE.
N. 7983rd AVE.
N. 7991st AVE.
N. 7999th AVE.
N. 8007th AVE.
N. 8015th AVE.
N. 8023rd AVE.
N. 8031st AVE.
N. 8039th AVE.
N. 8047th AVE.
N. 8055th AVE.
N. 8063rd AVE.
N. 8071st AVE.
N. 8079th AVE.
N. 8087th AVE.
N. 8095th AVE.
N. 8103rd AVE.
N. 8111th AVE.
N. 8119th AVE.
N. 8127th AVE.
N. 8135th AVE.
N. 8143rd AVE.
N. 8151st AVE.
N. 8159th AVE.
N. 8167th AVE.
N. 8175th AVE.
N. 8183rd AVE.
N. 8191st AVE.
N. 8199th AVE.
N. 8207th AVE.
N. 8215th AVE.
N. 8223rd AVE.
N. 8231st AVE.
N. 8239th AVE.
N. 8247th AVE.
N. 8255th AVE.
N. 8263rd AVE.
N. 8271st AVE.
N. 8279th AVE.
N. 8287th AVE.
N. 8295th AVE.
N. 8303rd AVE.
N. 8311th AVE.
N. 8319th AVE.
N. 8327th AVE.
N. 8335th AVE.
N. 8343rd AVE.
N. 8351st AVE.
N. 8359th AVE.
N. 8367th AVE.
N. 8375th AVE.
N. 8383rd AVE.
N. 8391st AVE.
N. 8399th AVE.
N. 8407th AVE.
N. 8415th AVE.
N. 8423rd AVE.
N. 8431st AVE.
N. 8439th AVE.
N. 8447th AVE.
N. 8455th AVE.
N. 8463rd AVE.
N. 8471st AVE.
N. 8479th AVE.
N. 8487th AVE.
N. 8495th AVE.
N. 8503rd AVE.
N. 8511th AVE.
N. 8519th AVE.
N. 8527th AVE.
N. 8535th AVE.
N. 8543rd AVE.
N. 8551st AVE.
N. 8559th AVE.
N. 8567th AVE.
N. 8575th AVE.
N. 8583rd AVE.
N. 8591st AVE.
N. 8599th AVE.
N. 8607th AVE.
N. 8615th AVE.
N. 8623rd AVE.
N. 8631st AVE.
N. 8639th AVE.
N. 8647th AVE.
N. 8655th AVE.
N. 8663rd AVE.
N. 8671st AVE.
N. 8679th AVE.
N. 8687th AVE.
N. 8695th AVE.
N. 8703rd AVE.
N. 8711th AVE.
N. 8719th AVE.
N. 8727th AVE.
N. 8735th AVE.
N. 8743rd AVE.
N. 8751st AVE.
N. 8759th AVE.
N. 8767th AVE.
N. 8775th AVE.
N. 8783rd AVE.
N. 8791st AVE.
N. 8799th AVE.
N. 8807th AVE.
N. 8815th AVE.
N. 8823rd AVE.
N. 8831st AVE.
N. 8839th AVE.
N. 8847th AVE.
N. 8855th AVE.
N. 8863rd AVE.
N. 8871st AVE.
N. 8879th AVE.
N. 8887th AVE.
N. 8895th AVE.
N. 8903rd AVE.
N. 8911th AVE.
N. 8919th AVE.
N. 8927th AVE.
N. 8935th AVE.
N. 8943rd AVE.
N. 8951st AVE.
N. 8959th AVE.
N. 8967th AVE.
N. 8975th AVE.
N. 8983rd AVE.
N. 8991st AVE.
N. 8999th AVE.
N. 9007th AVE.
N. 9015th AVE.
N. 9023rd AVE.
N. 9031st AVE.
N. 9039th AVE.
N. 9047th AVE.
N. 9055th AVE.
N. 9063rd AVE.
N. 9071st AVE.
N. 9079th AVE.
N. 9087th AVE.
N. 9095th AVE.
N. 9103rd AVE.
N. 9111th AVE.
N. 9119th AVE.
N. 9127th AVE.
N. 9135th AVE.
N. 9143rd AVE.
N. 9151st AVE.
N. 9159th AVE.
N. 9167th AVE.
N. 9175th AVE.
N. 9183rd AVE.
N. 9191st AVE.
N. 9199th AVE.
N. 9207th AVE.
N. 9215th AVE.
N. 9223rd AVE.
N. 9231st AVE.
N. 9239th AVE.
N. 9247th AVE.
N. 9255th AVE.
N. 9263rd AVE.
N. 9271st AVE.
N. 9279th AVE.
N. 9287th AVE.
N. 9295th AVE.
N. 9303rd AVE.
N. 9311th AVE.
N. 9319th AVE.
N. 9327th AVE.
N. 9335th AVE.
N. 9343rd AVE.
N. 9351st AVE.
N. 9359th AVE.
N. 9367th AVE.
N. 9375th AVE.
N. 9383rd AVE.
N. 9391st AVE.
N. 9399th AVE.
N. 9407th AVE.
N. 9415th AVE.
N. 9423rd AVE.
N. 9431st AVE.
N. 9439th AVE.
N. 9447th AVE.
N. 9455th AVE.
N. 9463rd AVE.
N. 9471st AVE.
N. 9479th AVE.
N. 9487th AVE.
N. 9495th AVE.
N. 9503rd AVE.
N. 9511th AVE.
N. 9519th AVE.
N. 9527th AVE.
N. 9535th AVE.
N. 9543rd AVE.
N. 9551st AVE.
N. 9559th AVE.
N. 9567th AVE.
N. 9575th AVE.
N. 9583rd AVE.
N. 9591st AVE.
N. 9599th AVE.
N. 9607th AVE.
N. 9615th AVE.
N. 9623rd AVE.
N. 9631st AVE.
N. 9639th AVE.
N. 9647th AVE.
N. 9655th AVE.
N. 9663rd AVE.
N. 9671st AVE.
N. 9679th AVE.
N. 9687th AVE.
N. 9695th AVE.
N. 9703rd AVE.
N. 9711th AVE.
N. 9719th AVE.
N. 9727th AVE.
N. 9735th AVE.
N. 9743rd AVE.
N. 9751st AVE.
N. 9759th AVE.
N. 9767th AVE.
N. 9775th AVE.
N. 9783rd AVE.
N. 9791st AVE.
N. 9799th AVE.
N. 9807th AVE.
N. 9815th AVE.
N. 9823rd AVE.
N. 9831st AVE.
N. 9839th AVE.
N. 9847th AVE.
N. 9855th AVE.
N. 9863rd AVE.
N. 9871st AVE.
N. 9879th AVE.
N. 9887th AVE.
N. 9895th AVE.
N. 9903rd AVE.
N. 9911th AVE.
N. 9919th AVE.
N. 9927th AVE.
N. 9935th AVE.
N. 9943rd AVE.
N. 9951st AVE.
N. 9959th AVE.
N. 9967th AVE.
N. 9975th AVE.
N. 9983rd AVE.
N. 9991st AVE.
N. 9999th AVE.
N. 10007th AVE.
N. 10015th AVE.
N. 10023rd AVE.
N. 10031st AVE.
N. 10039th AVE.
N. 10047th AVE.
N. 10055th AVE.
N. 10063rd AVE.
N. 10071st AVE.
N. 10079th AVE.
N. 10087th AVE.
N. 10095th AVE.
N. 10103rd AVE.
N. 10111th AVE.
N. 10119th AVE.
N. 10127th AVE.
N. 10135th AVE.
N. 10143rd AVE.
N. 10151st AVE.
N. 10159th AVE.
N. 10167th AVE.
N. 10175th AVE.
N. 10183rd AVE.
N. 10191st AVE.
N. 10199th AVE.
N. 10207th AVE.
N. 10215th AVE.
N. 10223rd AVE.
N. 10231st AVE.
N. 10239th AVE.
N. 10247th AVE.
N. 10255th AVE.
N. 10263rd AVE.
N. 10271st AVE.
N. 10279th AVE.
N. 10287th AVE.
N. 10295th AVE.
N. 10303rd AVE.
N. 10311th AVE.
N. 10319th AVE.
N. 10327th AVE.
N. 10335th AVE.
N. 10343rd AVE.
N. 10351st AVE.
N. 10359th AVE.
N. 10367th AVE.
N. 10375th AVE.
N. 10383rd AVE.
N. 10391st AVE.
N. 10399th AVE.
N. 10407th AVE.
N. 10415th AVE.
N. 10423rd AVE.
N. 10431st AVE.
N. 10439th AVE.
N. 10447th AVE.
N. 10455th AVE.
N. 10463rd AVE.
N. 10471st AVE.
N. 10479th AVE.
N. 10487th AVE.
N. 10495th AVE.
N. 10503rd AVE.
N. 10511th AVE.
N. 10519th AVE.
N. 10527th AVE.
N. 10535th AVE.
N. 10543rd AVE.
N. 10551st AVE.
N. 10559th AVE.
N. 10567th AVE.
N. 10575th AVE.
N. 10583rd AVE.
N. 10591st AVE.
N. 10599th AVE.
N. 10607th AVE.
N. 10615th AVE.
N. 10623rd AVE.
N. 10631st AVE.
N. 10639th AVE.
N. 10647th AVE.
N. 10655th AVE.
N. 10663rd AVE.
N. 10671st AVE.
N. 10679th AVE.
N. 10687th AVE.
N. 10695th AVE.
N. 10703rd AVE.
N. 10711th AVE.
N. 10719th AVE.
N. 10727th AVE.
N. 10735th AVE.
N. 10743rd AVE.
N. 10751st AVE.
N. 10759th AVE.
N. 10767th AVE.
N. 10775th AVE.
N. 10783rd AVE.
N. 10791st AVE.
N. 10799th AVE.
N. 10807th AVE.
N. 10815th AVE.
N. 10823rd AVE.
N. 10831st AVE.
N. 10839th AVE.
N. 10847th AVE.
N. 10855th AVE.
N. 10863rd AVE.
N. 10871st AVE.
N. 10879th AVE.
N. 10887th AVE.
N. 10895th AVE.
N. 10903rd AVE.
N. 10911th AVE.
N. 10919th AVE.
N. 10927th AVE.
N. 10935th AVE.
N. 10943rd AVE.
N. 10951st AVE.
N. 10959th AVE.
N. 10967th AVE.
N. 10975th AVE.
N. 10983rd AVE.
N. 10991st AVE.
N. 10999th AVE.
N. 11007th AVE.
N. 11015th AVE.
N. 11023rd AVE.
N. 11031st AVE.
N. 11039th AVE.
N. 11047th AVE.
N. 11055th AVE.
N. 11063rd AVE.
N. 11071st AVE.
N. 11079th AVE.
N. 11087th AVE.
N. 11095th AVE.
N. 11103rd AVE.
N. 11111th AVE.
N. 11119th AVE.
N. 11127th AVE.
N. 11135th AVE.
N. 11143rd AVE.
N. 11151st AVE.
N. 11159th AVE.
N. 11167th AVE.
N. 11175th AVE.
N. 11183rd AVE.
N. 11191st AVE.
N. 11199th AVE.
N. 11207th AVE.
N. 11215th AVE.
N. 11223rd AVE.
N. 11231st AVE.
N. 11239th AVE.
N. 11247th AVE.
N. 11255th AVE.
N. 11263rd AVE.
N. 11271st AVE.
N. 11279th AVE.
N. 11287th AVE.
N. 11295th AVE.
N. 11303r

PROJECTED LAND USE & ZONING MAP - 1995

PLANNED SHOPPING CENTER
PARK
SCHOOL
RESIDENTIAL
COMMERCIAL

Scale 1"=500' Approx.

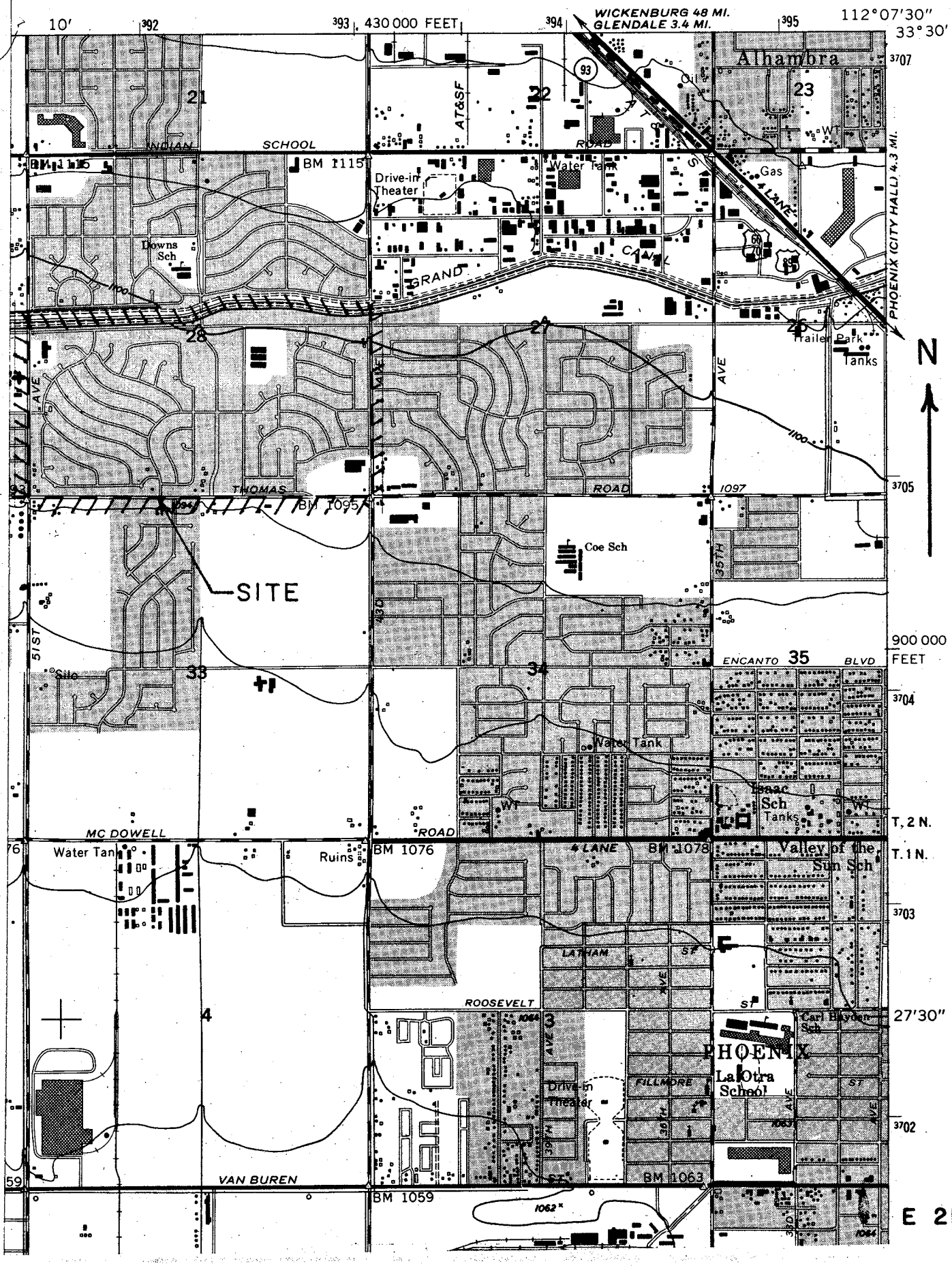


SITE MAP

APPROX. SCALE 1" = 2000'

FOWLER QUADRANGLE
ARIZONA—MARICOPA CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
NW/4 PHOENIX 15' QUADRANGLE

3551 II SE (SUNNYSLOPE)



ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN

October, 1972

STORM SEWER SYSTEM DESIGN

Introduction

The term storm sewer system as utilized in this design manual refers to the system of inlets, conduits, and other appurtenances which are designed to collect and convey storm runoff from roadways and adjacent contributory drainage areas to a convenient outlet.

The primary purpose of storm sewer system design is to limit the amount of water flowing in the streets and highways or ponding in sags to an amount that will not interfere with or present a hazard to traffic flow.

The proper storm sewer system design for highways and streets traversing urban and developed areas is generally a complex engineering problem and numerous factors are to be considered. The following design factors will have to be analyzed.

Design Criteria

The Arizona Highway Department, Hydraulics Branch has developed the following design criteria that are to be followed in storm sewer system design:

Design Frequency: The design frequency to be used for storm sewer system design is described in a/.

Design Discharge: The design discharge for storm sewer system design will be calculated by the Rational Equation which is described in a/ and in the following sections.

Ponding Width: The allowable spread of water T on the pavement should be limited to a width that will generally not interfere with or present a hazard to traffic flow. The pavement spread criteria to be used is described in b/.

---

a/ Hydrologic Design for Highway Drainage in Arizona
b/ Office Memo: Allowable Flooded Width Used in the Design of Curbed Roadway Drainage, 8-29-70

Clogging Factors: The capacity of grate, curb opening and combination inlets is subject to appreciable reduction due to debris that tends to clog the waterway opening. Clogging factors are therefore used to reduce the theoretical capacity of the inlets to the actual or street capacity. The clogging factors to be applied to grate, curb opening and combination inlets are described in c/.

Hydraulic Capacity Charts: Hydraulic capacity charts have not been developed for the standard inlets used by the AHD. The general equations for determining the capacity of the inlets are given in a following section.

Design Data Sheets

Storm Sewer System calculations should be shown on the three standard design forms and submitted with each design.

Runoff Calculation Sheet: All design data and hydrologic calculations should be shown on this form.

Inlet Calculation Sheet: All inlet design data and calculations should be shown on this form.

Storm Sewer Calculation Sheet: All storm sewer design data and calculations should be shown on this sheet.

The above forms should be adequate for all but the most complex design problems.

---

c/ Office Memo: Catch Basin Design Effective Areas 2-1-72

Hydrology

The design discharge for storm sewer system design will be calculated by the Rational Equation.

The Rational Equation relates rainfall intensity, a runoff coefficient, and drainage area size to the direct runoff from the drainage area. The relationship is expressed by the equation:

$$Q = CIA$$

where Q = the runoff in cubic feet per second (cfs)

C = a coefficient representing the ratio of rainfall to runoff

I = the rainfall intensity in inches per hour

A = the drainage area in acres

The Rational Method is described in detail in a/ and will not be discussed here again.

a/ Hydrologic Design for Highway Drainage in Arizona

Flow in Gutters

The capacity of a gutter depends on its cross-section, the longitudinal pavement grade and the roughness of the pavement. The capacity of a gutter is calculated with the modified Manning equation of the following form:

$$Q = .56 \frac{z}{n} S_o^{1/2} d^{8/3}$$

Where Q = discharge in cfs
Z = reciprical of the cross-slope ($1/S_x$)
n = Mannings roughness coefficient
S_o = Longitudinal pavement slope ft. /ft.
d = depth of flow in gutter, ft.

The spread of flow T on the pavement, which is a criteria for inlet spacing is defined as:

$$T = zd$$

The capacity or the depth of flow in a gutter with either a straight or a composite cross-section (two or more cross-slopes) can be determined from Chart 1. Roughness coefficients (Mannings n) for pavements and gutters are also shown.

The procedure used to determine the discharge in a triangular channel with uniform cross-slope is illustrated in Chart 1.

The trial and error procedure required to solve for the depth of flow in a gutter with a composite cross-section consists of assuming a depth at the curb and comparing the capacity of the composite cross-section with the design discharge. If these do not agree, a new assumption is made and the procedure is repeated. The following example illustrates the procedure used:

Given: Combined Curb and Gutter, Type A
Standard C-5.01
Pavement Cross-slope S_x = .02
Longitudinal Slope S_o = .01
Discharge Q = 5.0 cfs

Find: Depth of flow at curb and spread on pavement.

Solution:

1. From Standard C-5.01:
   Gutter cross-slope = .062 Z_a = 16.1
   Pavement cross-slope = .02 Z_b = 50
   Gutter width X = 2.0

2. From Chart 1 $n = .015$

3. $\frac{Z_a}{n} = \frac{16.1}{.015} = 1073$ $\frac{Z_b}{n} = \frac{50}{.015} = 3333$

4. Assume a depth at the curb. As a guide, use Chart 1 for a straight slope equal to the gutter cross-slope and find $d = .39$ foot. This d must be decreased slightly to allow for the greater spread on the flatter pavement or in this case, assumed $d = .39 - .06 = .33$ foot.

   Compute flow in gutter width ($X = 2.0$ feet) following instruction 3 on Chart 1.

   a. Calculate $\frac{X}{Z_a} = \frac{2.0}{16.1} = .12$ ft. which is

      the depth at the pavement edge of the gutter.

   b. The total flow in the channel at the assumed curb depth of .33 ft. and with a continuous slope of $Z_a = 16.1$ from Chart 1 is = 3.1 cfs.

   c. The flow beyond the gutter width on the assumption of a continuous slope of $Z_a = 16.1$ is computed as for the total flow using

      $$d' = d - \frac{X}{Z_a} = .33 - .12 = .21$$

      The discharge for $d' = .21$ is $Q = .9$ cfs

   d. The flow in the gutter width is then $3.1 - .9 = 2.2$ cfs at the assumed $d = .33$ ft.

5. If the assumed depth is correct, the difference in the design discharge ($Q = 5.0$ cfs) must be carried in the overflow section on the pavement. This flow is computed on Chart 1, by instruction 1, using $d' = 0.21$ foot and the $\frac{Z_b}{n}$ of the pavement section (3333). The Q in the pavement section is 2.8 cfs and the total $Q = 2.2 + 2.8 = 5.0$ which checks the design Q and also the assumed value of $d = 0.33$. Failure of the total Q to equal the design Q would require a new assumption of d and a recomputation of steps 4 and 5.

   The spread on the pavement $T = Z_b d' = 50(.21) = 10.5$ feet. The total width of flow measured from the curb is $2.0 + 10.5 = 12.5$ feet.

Pavement Inlets

The hydraulic capacity of pavement inlets depends on the inlet geometry and on the characteristics of flow in the gutter and on the pavement. The inlet capacity governs both the rate of water removal from the gutter and the amount of flow that can enter the storm sewer. Thus, inlet capacity and sewer capacity are based generally on a balanced design. In some cases the system may be unbalanced i. e. the sewer pipes may be designed for a different frequency than the inlets to provide for additional capacity at a future time.

Pavement inlets are generally divided into three major classes each with many variations. These classes are:

1. Grate inlets: These inlets consist of an opening in the gutter covered by one or more grates.

2. Curb opening inlets: These inlets consist of a vertical opening in the curb through which the gutter flow passes.

3. Combination inlets: These units consist of both a curb-opening and a grate inlet acting as a unit.

Because of the great range of conditions encountered in design and the large number of inlet types and variations used by the Arizona Highway Department, capacity charts have not been developed; rather the basic equations are given from which the designer can compute the capacity. The basic equations as well as the text describing them are taken from FHWA Hydraulic Engineering Circular 12.

Capacity of Grate Inlets in a Sag

A grate inlet in a sag operates first as a weir having a crest length roughly equal to the outside perimeter (P) along which the flow enters. Bars are disregarded and the side against the curb is not included in computing P. Weir operation continues to a depth (d) of about 0.4 foot above the top of grate and the discharge intercepted by the grate is:

$$Q_i = 3.0\ Pd^{1.5}$$

where Q_i = rate of discharge into the grate opening, in cubic feet per second

P = perimeter of grate opening, in feet, disregarding bars and neglecting the side against the curb.

d = depth of water at grate, in feet

When the depth at the grate exceeds about 1.4 feet, the grate begins to operate as an orifice and the discharge intercepted by the grate is:

$$Q_i = 0.67A\ (2gd)^{0.5} = 5.37Ad^{0.5}$$

where Q_i = rate of discharge into the grate opening, in cubic feet per second

A = clear opening of the grate, in square feet

g = acceleration of gravity, 32.2 feet per second^2

d = depth of ponded water above top of grate, in feet

Between depths over the grate of about 0.4 and about 1.4 feet the operation of the grate inlet is indefinite due to vortices and other disturbances. The capacity of the grate is somewhere between that given by the above equations. Because of the tendency of debris and trash to collect on the grate, the perimeter and the area of opening should be corrected for clogging as described in c/.

The following example will illustrate the use of the above equations.

Given: Grate Inlet in a Sag
Standard C-15.04 Type 4 Catch Basin
Standard C-15.06 Type LW-1.2 Grate

Find: depth at Q = 3.0 cfs
depth at Q = 20.0 cfs

Solution : 1. Compute perimeter of grate opening ignoring the bars and omitting any side over which water does not enter.
P = 2(2' - 1 1/2") + 3' - 7"
= 7' - 10"

2. Multiply by clogging factor to determine effective perimeter.
P_e = (P) (1/2) = 7' - 10" (1/2) = 3.92 ft.

3. Compute depth over grate by the weir equation for:
a. Q = 3.0 cfs and P = 3.92 ft.

$$d = \left[\frac{Q_i}{(3.0)\ (P)}\right]^{2/3} = \left[\frac{3.0}{(3.0)\ (3.92)}\right]^{2/3} = .40$$

The depth over the grate is at the weir control depth, therefore the weir control equation will give the correct answer.

c/ Office Memo: Catch Basin Design Effective Areas - 2-1-72

b. $Q = 20.0$ cfs and $P = 3.92$ ft.

$$d = \left[\frac{Q_i}{(3.0)\ (P)}\right]^{2/3} = \left[\frac{20.0}{(3.0)\ (3.92)}\right]^{2/3} = 1.42$$

Since the depth over the grate exceeds 1.4 ft., the orifice equation should be used to calculate the depth over the grate.

4. Determine net area of the grate from Standard C-15.06, Table 1
   $A_n = 5.01$ sq. ft.
   Reduce net area to effective area due to clogging
   $A = \frac{5.01}{2} = 2.5$ ft.^{2}

5. Calculate depth over grate by the orifice equation for
   $Q = 20.0$ cfs and $A = 2.5$ ft.^{2}

$$d = \left[\frac{Q_i}{5.37A}\right]^2 = \left[\frac{20.0}{(5.37)\ (2.5)}\right]^2 = 1.7 \text{ ft.}$$

If the grate has appreciable cross-slope so that the side away from the curb is higher than the curb side, the inflow over the side should be determined separately from that over the ends. In weir control use the depth at the middle of the grate for end inflow and the depth away from the curb for side inflow.

For depths between .4 feet and 1.4 feet the depth should be computed by both the weir and the orifice equations and the higher of the two used for design.

Capacity of Curb-Opening Inlets in a Sag

The capacity of curb-opening inlets in a sag depends upon the depth of water at the inlet and the inlet geometry. The inlet operates as a weir until the water submerges the entrance. When the water depth exceeds about 1.4 times the height (h) of the curb-opening entrance, the inlet operates as an orifice. Between weir-type operation and orifice-type operation the capacity is indeterminate. Chart 2 gives the minimum height (h_m) of opening required for weir type operation. If the opening height (h) equals or exceeds h_m (Chart 2), Charts 3-4-5 will give the depth of ponding measured at the curb, just above the depressed area. The use of these charts is explained in a following example.

Charts 3-4-5 apply only to depressed curb-opening inlets with a height of opening equal or exceeding the appropriate h_m from Chart 2. When the inlet is not depressed, the approximate capacity can be computed by the weir equation:

$$Q_i = 3.0\ L_i d_i^{1.5}$$

where

Q_i = capacity of the inlet, in cubic feet per second

d_i = depth of water above inlet lip, in feet

L_i = length of clear opening, in feet

When the depth at the opening exceeds 1.4 times the height (h) of the curb-opening entrance, the capacity may be computed by the following orifice equation:

$$Q_i = 0.67\ A \left[2g\ (d_i - \frac{h}{2}) \right]^{1/2}$$

where

A = area of opening, in square feet (hL_i)

h = height of opening, in feet

Q_i, d_i, and L_i are the same as in the previous equation.

The following example illustrates the procedure for computing the capacity of depressed curb-opening inlets:

Given: Curb-Opening Inlet in a Sag
Standard C-15.03 with 2-3 ft. wings
L_i = 10' - 1"
W = 2.0' (Gutter width)
a = 2.0" (Inlet depression)
h = 7"
S_x = .02

Find: Depth of Ponding for Q = 14 cfs

Solution: 1. Use Chart 2 to check the adequacy of the opening height to maintain free fall in the inlet.
For Q = 14 cfs
L_i = 10' - 1"
h_m = .45'

The opening height h = 7" exceeds the requirement for free fall and the charts can be used to determine the depth of ponding.

2. From Chart 4 the maximum depth of ponding at the curb, just above the depressed area and the spread of water is:
d_m = .60 ft. and
T = zd = (50)(.60) = 30 ft.

When the curb-opening inlets are not depressed, the approximate capacities should be computed with either the weir or the orifice equations. The procedure is similar to the procedure used for computing capacities of grates in a sag.

Capacity of Combined Grate and Curb-Opening Inlets in a Sag

Combination inlets are desirable in a sag because the curb-opening provides a relief opening if the grate becomes clogged. The capacity of combination inlets, when the grate and the curb-opening are approximately the same length, is determined by computing the capacities of the grate only without a clogging factor. The curb-opening provides a relief opening for the debris that would normally clog the grate. When the curb-opening can be much longer than the grate (C-15.05) the capacity of the curb-opening with the approriate clogging factor should be calculated. The capacity of the grate should not be computed. The grate will provide an additional safety factor.

Capacity of Grate Inlets on a Continuous Grade

Grate inlets are recommended where clogging due to debris and traffic interference is not a problem.

Grates with bars parallel to the direction of flow are much more efficient than grates with bars transverse to the direction of flow especially when longitudinal pavement grades exceed 1%. As the grade increases, longitudinal bar grates become increasingly superior to transverse bar grates both in regard to capacity and decrease in tendency to catch debris. Thus, longitudinal bar grates should be used whenever possible.

An efficient grate inlet on a continuous grade will intercept all the flow within a width equal to that of the grate. For grates over 3 ft. long on flat slopes ($<$1.0%) or for depressed grates the inflow along the longitudinal edge may be estimated by treating this edge as a curb-opening. The required length of grate to intercept all of the flow over it should be computed by the following equation:

$$L_b = \frac{V}{2} \quad (d + d_b)^{1/2}$$

where L_b = length of clear opening of grate, ft.

V = mean approach velocity in the width of the grate opening, fps

d = depth of flow at the curb, ft.

d_b = depth of the bar, ft.

The capacity of an undepressed efficient grate inlet can be determined by computing the flow in the section occupied by the grate width and then determining the length of grate required to intercept all the flow. The appropriate correction for clogging should be made.

The following example illustrates the design procedure for an undepressed grate:

Given: Grate Inlet on Continuous Grade
Standard C-15.04 Type 4 CB, Single
Standard C-15.06 LW-1.2 Grate

Length of Grate	= 3' - 5 1/4"
Width of Grate	= 2.0
S_o	= .01
S_{x1} (Pavement)	= .02
S_{x2} (Gutter)	= .062
n	= .015
d_b	= 3 1/2" = .29'

Find: Spread on pavement and discharge intercepted by the grate inlet for Q = 5.0 cfs.

Solution: 1. Calculate depth of flow and pavement spread from Chart 1 as explained in the gutter flow example.
d = .33 ft.
T = 12.5 ft.

2. The depth of flow d' at outer edge of the grate (X = 2.0) is: (See previous example.)
d' = .21 ft.

3. Calculate the flow beyond the edge of the grate from Chart 1: (See previous example.)
with d = .21
Q = 2.8 cfs
Then the flow into the grate equals
Q = 5.0 - 2.
= 2.2 cfs
provided that the grate is long enough.

4. Check required length of grate

a. The mean velocity in the 2.0 ft. section over grate is

$$V = \frac{Q}{A}$$

where the area is

$$A = \frac{.33 + .21}{2}(2) = .54 \text{ ft.}^2$$

$$\text{and } V = \frac{2.2}{.54} = 4.1 \text{ fps}$$

The clear opening of grate then is

$$L = \frac{V}{2}(d + d_b)^{1/2}$$

$$= \frac{4.1}{2}(.33 + .29)^{1/2}$$

$$= 1.6 \text{ ft.}$$

Because of possible clogging the actual length of the grate should be twice the required length as described in c/.
The actual grate length should therefore be about 3.2 ft. Since the Standard C-15.06 grate is 3'-5¼" the design is satisfactory. The 2.8 cfs outside of the grate will have to be intercepted by a grate further downstream.

Capacity of Curb-Opening Inlets on a Continuous Grade

The capacity of a curb-opening inlet depends upon the length of opening (L_i), the depression of the inlet lip (a), the depth of flow at the curb line in the gutter (d), and both the cross slope (S_x) and the longitudinal slope of the gutter (S_o). Capacity charts prepared by the FHWA will be used for determining inlet capacities.

Charts 6 through 14 show the ratio of discharge intercepted (Q_i) to the total discharge (Q) for a given inlet geometry (L_i, W, and a) and n = 0.016. The figure contains two families of curves, one for longitudinal slope and one for cross slope. To use the capacity charts, select the appropriate chart for the inlet geometry. Then from the spread on pavement, in the lower left horizontal scale, move vertically to the curve representing the longitudinal slope. From this point move horizontally to the curve representing the pavement cross slope. Then move vertically to the $\frac{Q_i}{Q}$ scale, in the upper right horizontal scale and read the interception ratio. The discharge intercepted by the curb-opening inlet is the product of this ratio and the gutter discharge.

The charts presented are for the following three standard depression configurations:

c/Office Memo: Catch Basin Design Effective Areas 2-1-72

Depression Depth, Inches	Depression Width, Feet
1	1
2	2
3	3

Approximate capacities for inlets with different depression geometries may be obtained by applying the following relationships:

Dimension from Graph	Actual Dimension of Depression	Effect on Q_i/Q from Graph
W = 2', a = 2"	W = 2', a = 1"	Reduce by one-fourth
W = 1', a = 1"	W = 1', a = 2"	Increase by one-fourth
W = 2', a = 2"	W = 1', a = 2"	Reduce by one-fourth
W = 1', a = 1"	W = 2', a = 1"	Increase by one-fourth

Interception by curb-opening inlets of lengths other than for which charts are given may be computed by interpolating between the given lengths.

The spread of water T shown on the charts is limited to 10 ft. and 15 ft. The charts should not be extended beyond 12 ft±.

The following example illustrates the use of these charts:

Given: 10 ft. long curb-opening inlet on a continuous grade.
Pavement and gutter geometry are given in the example on gutter flow; curb-opening depression = 2.0"
Discharge Q = 5.0 cfs

Find: Discharge intercepted by the inlet.

Solution: From the previous gutter flow example

d = .33 Gutter flow depth
T = 12.5 Pavement spread

1. From Chart 10 with T = 12.5 ft.
   a. extend S_o = .01 line to T = 12.5 ft.
   b. interpolate between S_x = .015 - .03
   c. read Q_i/Q ratio at .60

2. Discharge intercepted is
   Q_i = (.60)(5.0)
   = 3.0 cfs
   Because of clogging the actual inlet capacity will be 2.4 cfs, or the inlet may be increased to 12.5' to intercept 3.0 cfs.

Capacity of Combination Inlets on a Continuous Grade

The capacity of a combined curb-opening and grate inlet will be computed by ignoring the capacity of the curb-opening and computing the capacity of the grate opening alone without a clogging factor. The curb-opening will provide the required safety factor against clogging.

Inlet Spacing

Generally, the spacing of inlets will depend on the type of roadway to be drained.

In the spacing of inlets on a limited access type of roadway where only on-site runoff will be encountered the full permissible flooding width of the gutter and pavement should be utilized. The depth of flow along the curb will be greatest under these conditions and the inlets will operate under maximum efficiency.

The first inlet will be located by determining the length of roadway necessary to generate the discharge that will occupy the maximum permissible pavement spread. The second and successive inlets will be located at a distance which is just long enough to generate the discharge intercepted by the previous inlet.

Thus, at the design discharge the pavement will be flooded to its maximum permissible spread along the section under design.

The last inlet must intercept both the flow that bypassed the previous inlet and the discharge generated by the length of the last pavement reach. The last inlet must therefore be designed for 100% interception.

On urban type roadways where cross-roads and off-site drainage areas contribute flow, inlets may have to be located before the maximum pavement spread is utilized.

Inlets may have to be located at intersections to prevent gutter flow from crossing traffic lanes of the intersecting roads, at major points of off-site drainage inflow or located due to other than hydraulic considerations. Since the maximum permissible pavement spread and gutter flow depth cannot be utilized, inlet interception will not be very efficient and a greater number of inlets will be required.

It should be emphasized that gutter inlets are not efficient for intercepting off-site drainage. Every effort should be made to intercept off-site drainage with open channels or special inlet structures before the flow gets on the pavement.

Sewer Design

After the inlets have been located and proportioned the sewer runs will be located and the quantity of flow to be carried by each pipe and the gradient and size of each pipe must be determined.

The following factors must be considered:

Sewer Run Location: The location of sewer runs, including inlets, leterals, main lines, and outfalls must be determined.

Generally, medians offer the most desirable storm sewer location. In the absence of medians, a location beyond the edge of pavement or under sidewalks may be desirable. It is generally recommended that when a storm sewer is placed beyond the edge of pavement that one main line with connecting leterals be used instead of running two trunks, one down each side of the road.

Outfalls: The tailwater depth in the outfall channel may significantly affect the sizing and operation of the sewer runs. The outfall pipe may empty into an open channel or another pipe; in each case, the depth of flow in the outfall channel must be determined.

Sewer Run Discharge: The design discharge carried by any particular section of pipe is not necessarily the sum of the inlet design quantities of all inlets above that section of pipe. As a general rule the sewer discharge is somewhat less than the sum of the inlet discharges.

In determining the discharge for each sewer run the rainfall intensity used in the Rational Equation is based on a time of concentration that is made up of two parts:

Inlet time: Which is the time required for water to flow from the hydraulically most distant point of the drainage area to the inlet. Generally, inlet time can be determined from the appropriate figure in a/ if the drainage area is off the roadway. The time of concentration for pavement flow can be determined as described previously or a minimum time of concentration of 10 min. may be used.

---

a/ Hydrologic Design for Highway Drainage in Arizona.

Sewer time: Which is the time required for the water to flow through the sewer line from the upstream inlet to the point of the sewer line under design. The sewer time can be determined by calculating the sewer flow velocity with the Manning equation ($V = \frac{1.49}{n} R^{2/3} S^{1/2}$) and the time can be calculated by:

$$t = \frac{L}{V}$$

where t = time of flow
L = length of the sewer run
V = velocity of flow

Thus the design time of concentration for any point on a sewer line is the inlet time for the inlet at the upper end of the line plus the time of flow through the sewer from the upper end of the sewer to the reach under design, unless the time for another branch or inlet at that point is greater. The minimum time of concentration of 10 minutes should be used.

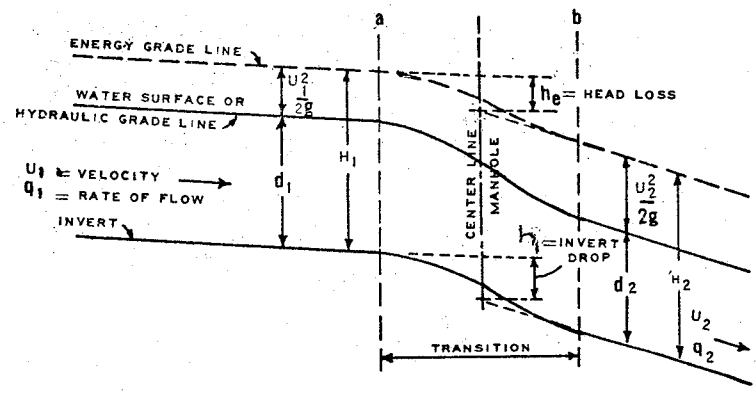
Sewer Profile: A tentative sewer crown profile will be required for the calculation of pipe sizes. The crown line profile is drawn to indicate the inside top of the pipes. The inverts of the pipes will be established after the required pipe diameters have been computed. At changes in size of pipe the top insides should be placed at the same level rather than placing the flow lines at the same level. Where flow lines are placed at the same level , the smaller pipe will discharge against a head and it will be necessary to plot a hydraulic grade line.

Pipes should be placed on such a slope that the velocity of flow will not be less than 3 fps.

When profiles are comparatively flat, it is desirable that the sewer sections and slopes be so designed that the velocity of flow will increase progressively. Thus, sediments washed into the sewer will be carried on through and will not be dropped at some point due to a sharp decrease in velocity.

Pipe Size Reduction: Generally, pipe sizes should not be reduced in the downstream direction. In special cases a downstream pipe may be reduced

in size if the grade and capacity are sufficient to carry the flow. To determine the amount of drop (h_i) of the ℄ of the manhole the following method should be used:



ENERGY AND HYDRAULIC GRADE LINES AT TRANSITIONS IN SIZE OR GRADE

$$H_1 = d_1 + V_1^2/_{2g}$$

$$H_2 = d_2 + V_2^2/_{2g}$$

$$h_e = K \frac{\Delta V^2}{2g}$$

V = full flow velocity

"h_e" is always positive

Where ΔV^2 is the algebraic difference between the velocities.

The value of "K" is 0.1 if $V_2 > V_1$ and 0.2 if $V_1 > V_2$

Therefore, the drop (h_i) is equal to

$$h_i = H_2 - H_1 + h_e$$

If "h_i" is positive the drop is required otherwise the inverts will be at the same elevations.

Pipe Junctions: The crowns of pipes at the center of manholes or catch basins should be at the same elevation. If a lateral must be placed so its flow is directed against the main flow through the manhole or catch basin, the lateral invert should be raised to match the crown of the inlet pipe.

The angle in the direction of flow between pipes at junctions should be less than 90° to prevent excessive turbulence and head loss.

Minimum Pipe Diameter: The minimum pipe diameter shall be 12 inches for laterals and 18 inches for trunk lines.

Manholes: Manholes are generally required for maintenance and cleanout purposes. They should be provided at changes in pipe gradient, changes in pipe size, changes in direction, and at 300 ft. intervals in long runs of constant grade, direction, and size unless access through catch basins or junction boxes is provided.

Sewer Calculations

For most conditions the sewer runs should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The capacity of pipes should be calculated with the Manning equation. Chart 15 can be used to determine the capacity of corrugated metal or concrete pipes at either part full or full flow.

The hydraulic gradient is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a sewer run. The difference in elevation for the water surfaces in the successive tubes represents the friction loss for that length of sewer, and the slope of the line between water surfaces is the friction slope. Therefore, if a sewer run were placed on a calculated friction slope corresponding to a certain quantity of water, cross-section, and roughness factor, the surface of flow (hydraulic gradient) would be parallel to the top of the conduit, and the sewer run would not be under pressure. This is desirable as previously stated. If there is reason to place the sewer run on a slope less than friction slope, then the hydraulic gradient would be steeper than the slope of the sewer run. Depending on the elevation of the hydraulic gradient at the downstream end of the run under design, it is possible to have the hydraulic gradient go above the top of the conduit which would mean that the sewer is under pressure until, at some point upstream, the hydraulic gradient is once again at or below the top of the gradient.

It will not be necessary to compute the hydraulic grade line of a sewer run, where the slope and the run sizes are chosen so that the slope is equal to or greater than friction slope and the top surfaces of successive runs are lined up at changes in size rather than the bottom surfaces, and the surface of the water at the point of discharge does not lie above the top of the outlet. In such cases the pipe will not operate under pressure and the slope of the water surface under capacity discharge will approximately parallel the slope of the invert of the pipe. There will be small head losses at inlets, manholes, etc., but if these are properly designed these losses may be neglected.

Whenever all of these conditions do not exist however, and particularly in those instances where the inverts in the pipes are placed on the same grade at changes in pipe size (which forces the smaller pipe to discharge against head) or when it is desired to check the sewer system against a larger flood than that used in the proportioning of the pipes, it will be necessary to compute the hydraulic grade line of the entire sewer system.

The hydraulic grade line is computed by starting with the tailwater elevation at the point where the sewer is finally discharged and working back up the length of the sewer, computing the friction loss for each run and plotting the elevation of the total head at each pipe junction, manhole and inlet.

The friction slope or the friction loss for each run can be calculated from Chart 15 by the following procedure:

Given: 36" diameter concrete pipe
discharge Q = 50 cfs

Find: Friction slope S_f and head loss h_L for a 500 ft. reach

Solution: Chart 15

a. Lay a straight edge on pipe diameter D = 36" and d/D ratio at full flow. Mark the intersection of the straight edge on the turning line.

b. Lay straight edge on intercept on the turning line and on the discharge line at Q = 50 cfs.

c. Read friction slope S_f on slope line S_f = .005.

d. Calculate head loss

$$h_f = S_f L$$
$$= (.005)(500)$$
$$= 2.5 \text{ ft.}$$

The head loss thus calculated is added to the tailwater elevation thus defining the hydraulic grade line.

The calculations are continued in the upstream direction by calculating the head loss for each reach and plotting the total head at each pipe junction, manhole and inlet.

If the hydraulic grade line as thus plotted does not rise above the top of any manhole or above the lip of any inlet the sewer system is considered satisfactory. Wherever it does rise above these points, however, blow-outs through inlet slots and manhole covers will occur and pipe sizes or gradients should be increased as necessary to eliminate such blow-outs.

Note that any hydraulic gradient must have an original base elevation no lower than the outlet tailwater elevation. Therefore, the backwater effects of a significant tailwater elevation should be checked very carefully.

Design Procedure

The design procedure for storm sewer system design is divided into the following three parts:

I The calculation of the design discharge.

II The sizing and spacing of the inlets.

III The hydraulic design of the sewer runs.

The recommended design procedure using the appropriate design calculation sheet is as follows:

I Runoff Calculation Sheet:

Location Data

1. The required input is self-explanatory.

Design Data:

1. Select the recommended sewer design frequency from a/.
2. Obtain 6-hour and 24-hour precipitation values for the project area from Precipitation Maps 1-6 in a/.
3. Calculate 1-hour precipitation value as described in a/.

Runoff Calculations:

1. Delineate contributing drainage areas, assign a reference number to each and indicate Station-Station limits of the drainage areas. It may be necessary to lay out a preliminary inlet location plan before the drainage areas can be delineated.
2. Measure the contributing drainage areas.
3. Determine the runoff coefficients for each drainage area. It may be possible to use an average runoff coefficient for the total area or it may be necessary to subdivide each drainage area according to the runoff coefficient. A table of C values is given in a/.
4. Compute and summate the CA values.
5. Calculate the time of concentration as outlined in a/ or as described in previous sections. Use a minimum time of concentration of 10 minutes.
6. Determine rainfall intensity as outlined in a/.
7. Calculate design discharge by the Rational Equation.

II Inlet Calculation Sheet:

Location Data

1. The required input is self-explanatory.

---

a/ Hydrologic Design for Highway Drainage in Arizona.

Design Data

1. Determine the allowable pavement spread T_{all} from b/.
2. Obtain the appropriate Manning's n value for the gutter and pavement from Chart 1.

Inlet Calculations

1. Assign a reference number to each inlet and list its location.
2. List the reference number of each drainage area that contributes flow to the inlet under design.
3. Indicate the type of inlet to be used.
4. Show the discharge Q for the inlet. This is the discharge obtained from the runoff calculation sheet.
5. Calculate the depth of flow D and pavement spread T for the given Q, the given gutter and pavement cross-slopes S_x, and the longitudinal slope S_o. If the depth of flow or the pavement spread exceeds the allowable limits, inlets at more frequent intervals will be required.
6. Indicate the inlet depression "a" to be used.
7. Calculate the inlet capacity. The preferred manner of calculating inlet capacity is to select a standard inlet, correct its length, perimeter, or area for clogging and compute its capacity. If the capacity thus calculated is not adequate, additional grates or curb-opening may be used, the inlet depression may be increased or a different type of inlet may be selected.

   Thus indicate the actual and the corrected inlet dimension (length L_i, perimeter P, or area A) in the appropriate column.
8. Calculate the interception rate Q_i/Q and the discharge intercepted Q_i for depressed curb-openings from Charts 6-14.
9. Calculate the intercepted discharge Q_i for grate inlets and combination inlets from the appropriate equations.
10. Calculate the carry-over flow Q_c
    where

    $$Q_c = Q - Q_i$$

    At the first inlet the total discharge Q_T is equal to the discharge Q from the contributing drainage area. At all other inlets the total discharge Q_T is equal to the discharge Q from the contributing drainage area plus the carry-over discharge Q_c from the previous inlet.
    Thus $Q_T = Q + Q_c$
11. Continue with the inlet calculations. There should be no carry-over discharge at the last inlet.

III Sewer Calculation Sheet:

Location Data

1. The required input is self-explanatory.

Design Data

1. Generally, the design frequency and the precipitation values will be identical in both the Runoff-Inlet Calculations and the Sewer Calculations. If for some reason the design frequency for the inlets and the storm sewer is not identical, the appropriate frequency and precipitation values should be determined as previously outlined.
2. Calculate the depth of flow in the sewer outfall channel.
3. Determine the coefficient of friction for the sewer pipe. A table of friction factors is found in HEC #5. If alternate types of pipes will be specified on the final plans, a separate storm sewer design will be required for each type of pipe.

Sewer Calculation:

Location

1. Select a line reference number such as:

   Lateral 1
   Trunk Line 1

   for each line and indicate its station to station location.

Drainage Area

1. Indicate the reference number of the drainage areas contributing flow to the upstream end of the sewer run under design.
2. Show the incremental CA value of the drainage areas contributing flow to the inlet at the upstream end of the sewer run under design. The incremental CA values should be obtained from the Runoff Calculation Sheet.
3. The ΣCA value is the total of all the CA values of the contributing drainage areas at the upstream end of the sewer run under design.

Time of Flow

1. Calculate the inlet time of concentration or use the time of concentration determined previously for the runoff calculations.
2. Calculate the sewer flow time. There is no sewer time for the first sewer run. For all subsequent runs the sewer time should be calculated as previously outlined.

3. Design time: For the first run the design time will be equal to the time of concentration of the first inlet. For all successive runs the design time will be the inlet time plus the time required for flow through the sewer to the reach under design. The minimum time of concentration of 10 min. should be used.

Calculate the rainfall intensity for the design time of concentration which was previously determined.

Calculate the design discharge by the Rational Equation with previously determined ΣCA and the rainfall intensity I.

Sewer Profile

1. Plot a tentative sewer crown profile and determine the inlet and outlet elevations for each run.
2. The length of each sewer run is the length from center to center of inlets or manholes. This length is used in determining the time of flow from one inlet or manhole to another.
3. Calculate the slope of each run.

Sewer Design

1. Select a trial pipe diameter and calculate its full flow capacity from Chart 15. The size and gradient of the pipe must be chosen in such manner that the pipe when flowing full, but not under head, will carry an amount of water approximately equal to or greater than the computed discharge, Q. In other words, Q_{full} must be approximately equal to or greater than Q. If the first trial pipe diameter is too small, continue checking successive pipe sizes until an adequate pipe size is found.
2. Calculate the velocity for the design discharge and the selected pipe with the Mannings equation (Chart 15) and Table 2 as follows:
   a. determine d/D ratio (depth of flow/Diameter) from Chart 15 for the actual flow conditions.
   b. determine C_a value from Table 2 for the above d/D ratio
   c. determine area of flow where $a = C_a D^2$
   d. calculate velocity by $V = Q/A$

Hydraulic Grade Line

Whenever one of the following conditions occurs,

a. the full flow capacity (Q_{full}) of the selected pipe

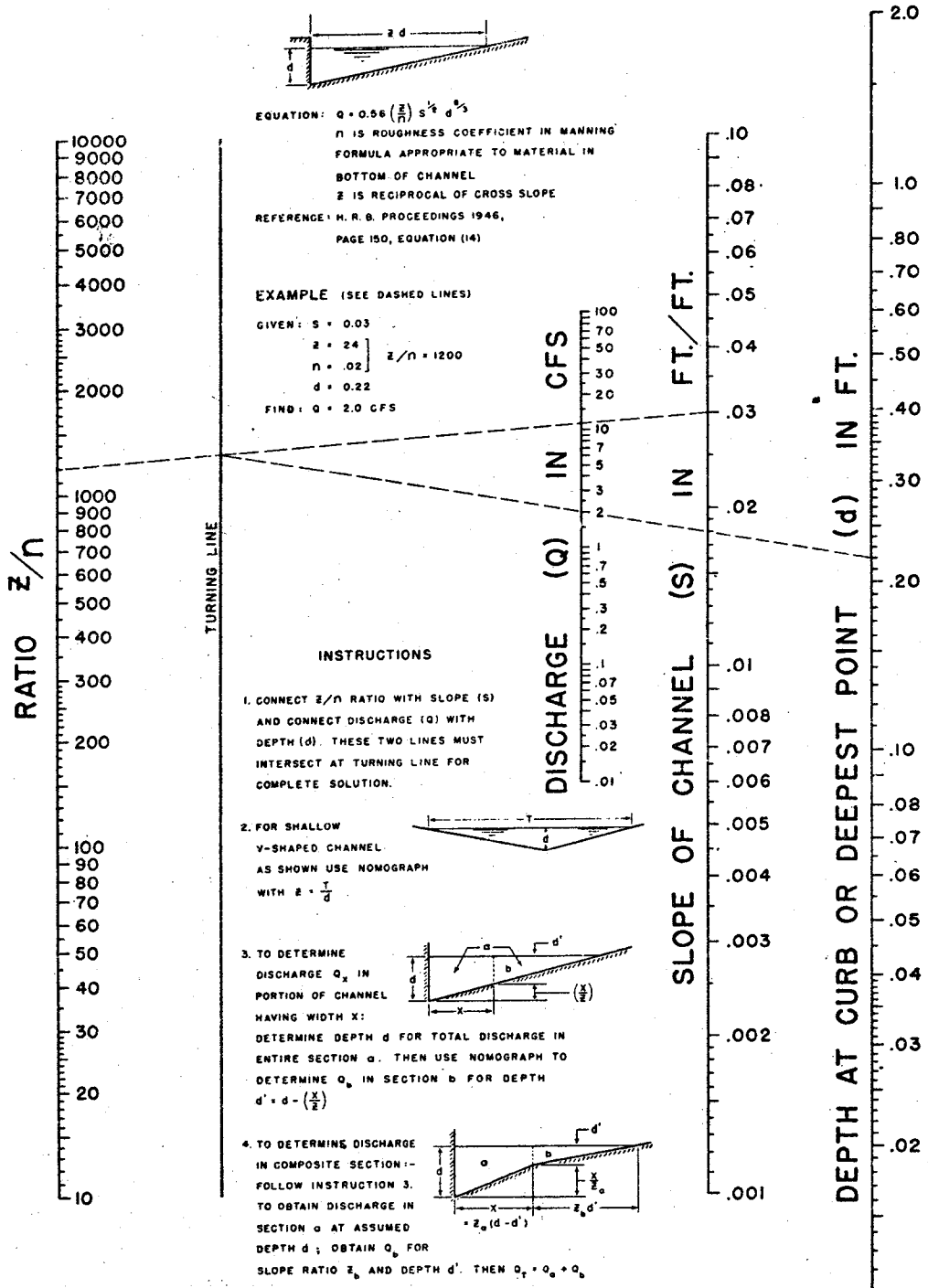
is less than the design discharge Q.

b. a pipe of a larger diameter discharges into a pipe of a smaller diameter
c. where it is impossible to line up the crowns of pipe runs at changes in pipe size
d. where the tailwater at the outlet of the sewer run submerges the sewer pipe

the hydraulic grade line should be calculated to be sure that the backwater head created by such a design is not large enough to cause blowouts at inlets or manholes above the run. The following procedure can be used to calculate the hydraulic grade line:

1. Calculate the friction slope S_f from Chart 15 and the head loss for each reach as previously outlined.
2. Plot the hydraulic grade line by adding the headloss to the tailwater elevation and continue upstream.

   If the hydraulic grade line is sufficiently below the inlets and manholes, the design is satisfactory. If the hydraulic grade line is at or above the inlets and manholes, blowouts will occur and the design should be revised.



Concrete gutter troweled finish	0.012
Asphalt pavement	
(1) Smooth texture	0.013
(2) Rough texture	0.016
Concrete gutter with asphalt pavement	
(1) Smooth	0.013
(2) Rough	0.015
Concrete pavement	
(1) Float finish	0.014
(2) Broom finish	0.016
	0.016

For gutters with small slope where sediment may accumulate, increase all above values of "n" by 0.002.

NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS

AHD
Structures Section
Hydraulics Branch
10-15-72

Chart 1

FHWA
HEC #12

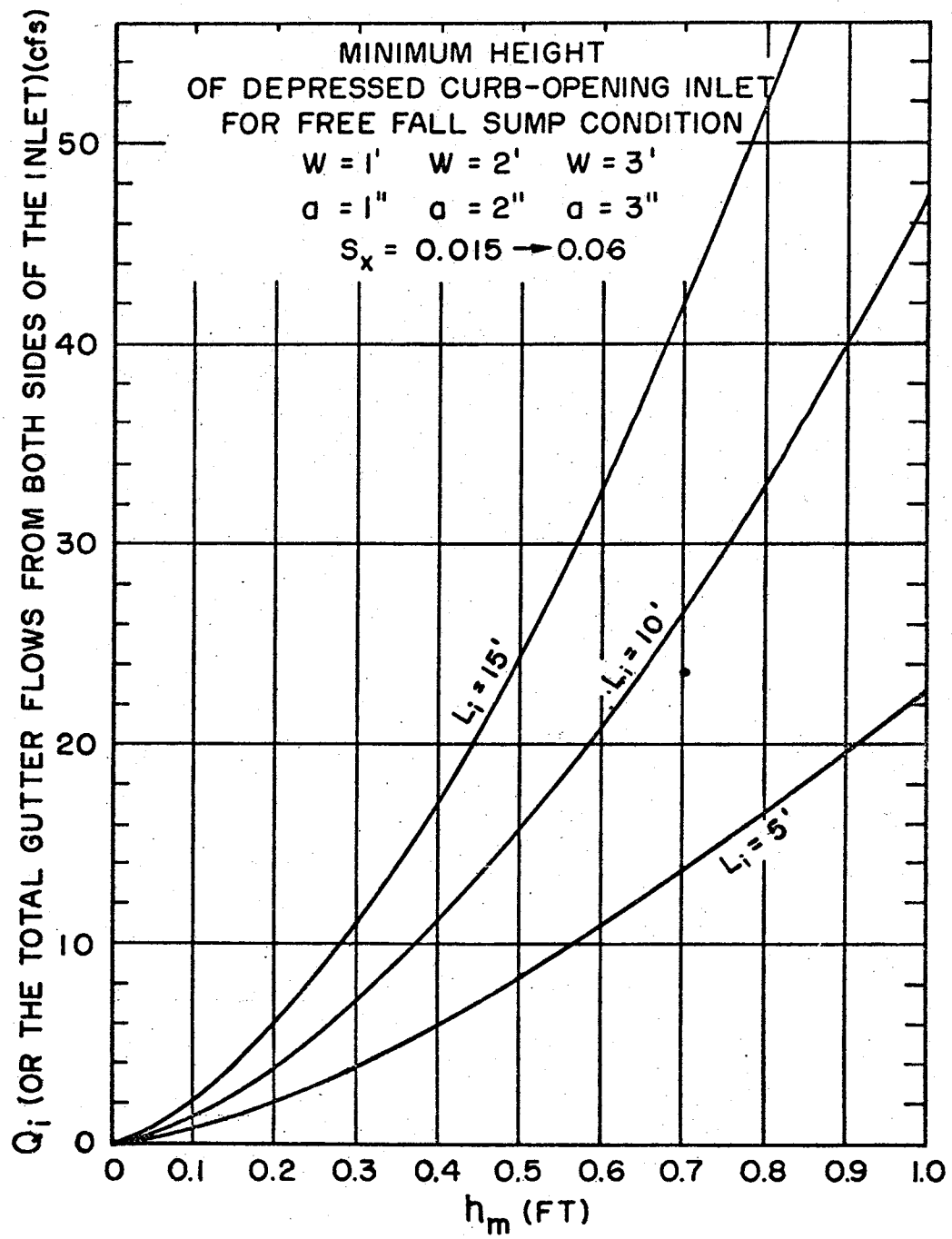


Chart 2

AHD
Structures Section
Hydraulics Branch
10-15-72

FHWA
HEC #12

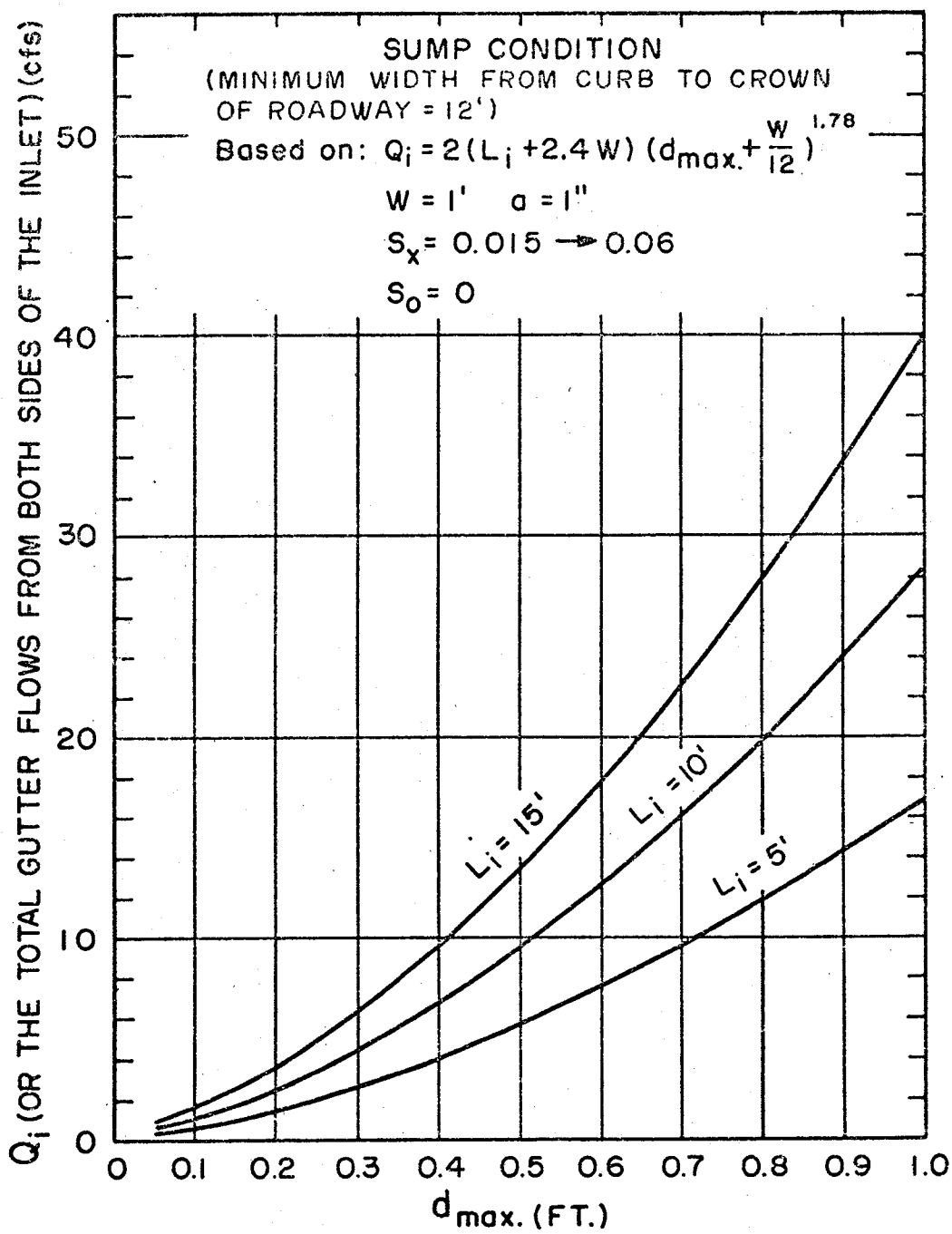

SUMP CONDITION
(MINIMUM WIDTH FROM CURB TO CROWN OF ROADWAY = 12')
Based on: $Q_i = 2(L_i + 2.4W)(d_{max.} + \frac{W}{12})^{1.78}$
W = 1' a = 1"
S_x = 0.015 → 0.06
S_o = 0
L_i = 15'
L_i = 10'
L_i = 5'
Q_i (OR THE TOTAL GUTTER FLOWS FROM BOTH SIDES OF THE INLET) (cfs)
0
10
20
30
40
50
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
$d_{max.}$ (FT.)

Chart 3

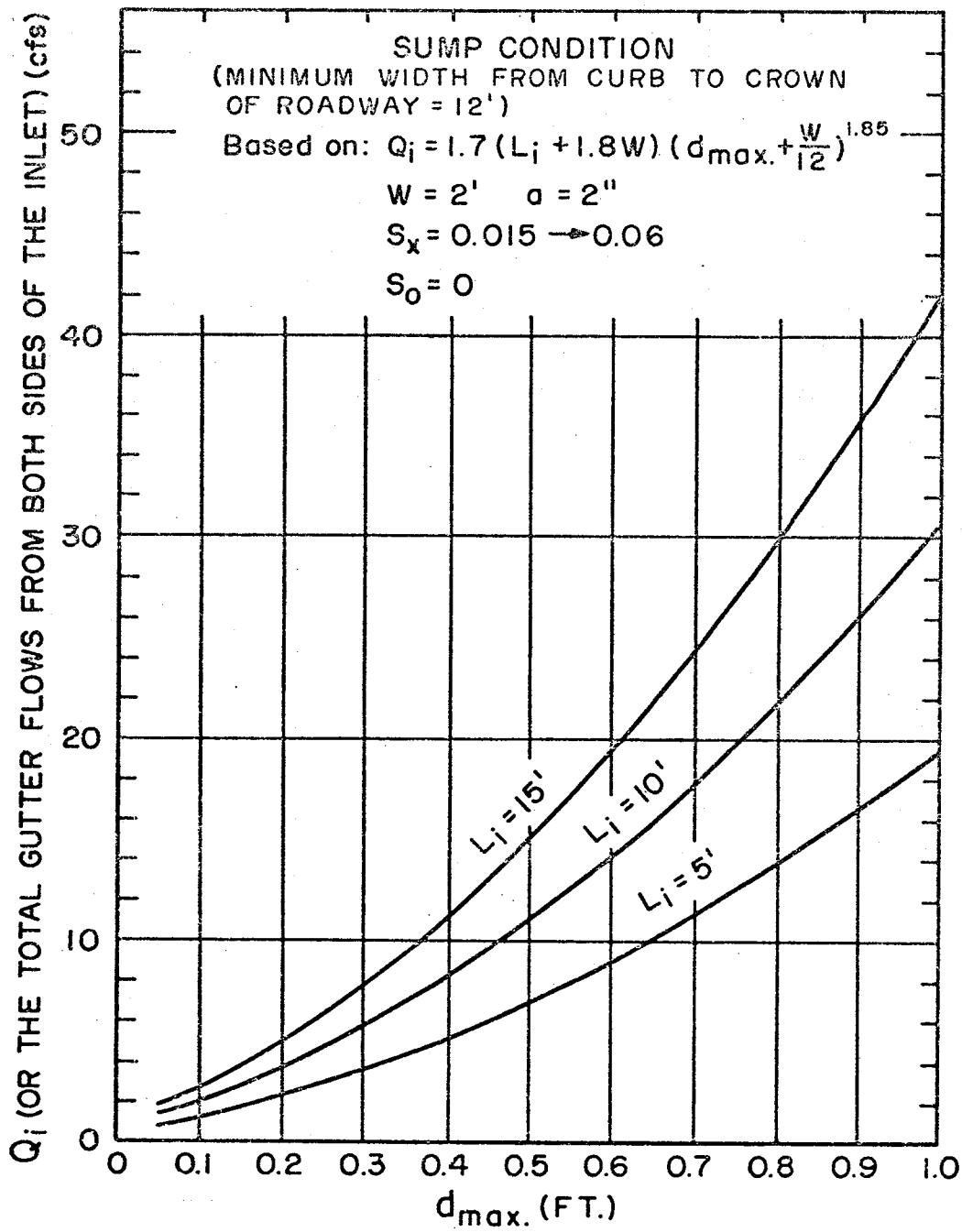

SUMP CONDITION
(MINIMUM WIDTH FROM CURB TO CROWN OF ROADWAY = 12')
Based on: $Q_i = 1.7(L_i + 1.8W)(d_{max.} + \frac{W}{12})^{1.85}$
W = 2' a = 2"
$S_x = 0.015 \rightarrow 0.06$
$S_o = 0$
$L_i = 15'$
$L_i = 10'$
$L_i = 5'$
Q_i (OR THE TOTAL GUTTER FLOWS FROM BOTH SIDES OF THE INLET) (cfs)
0
10
20
30
40
50
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
$d_{max.}$ (FT.)

Chart 4

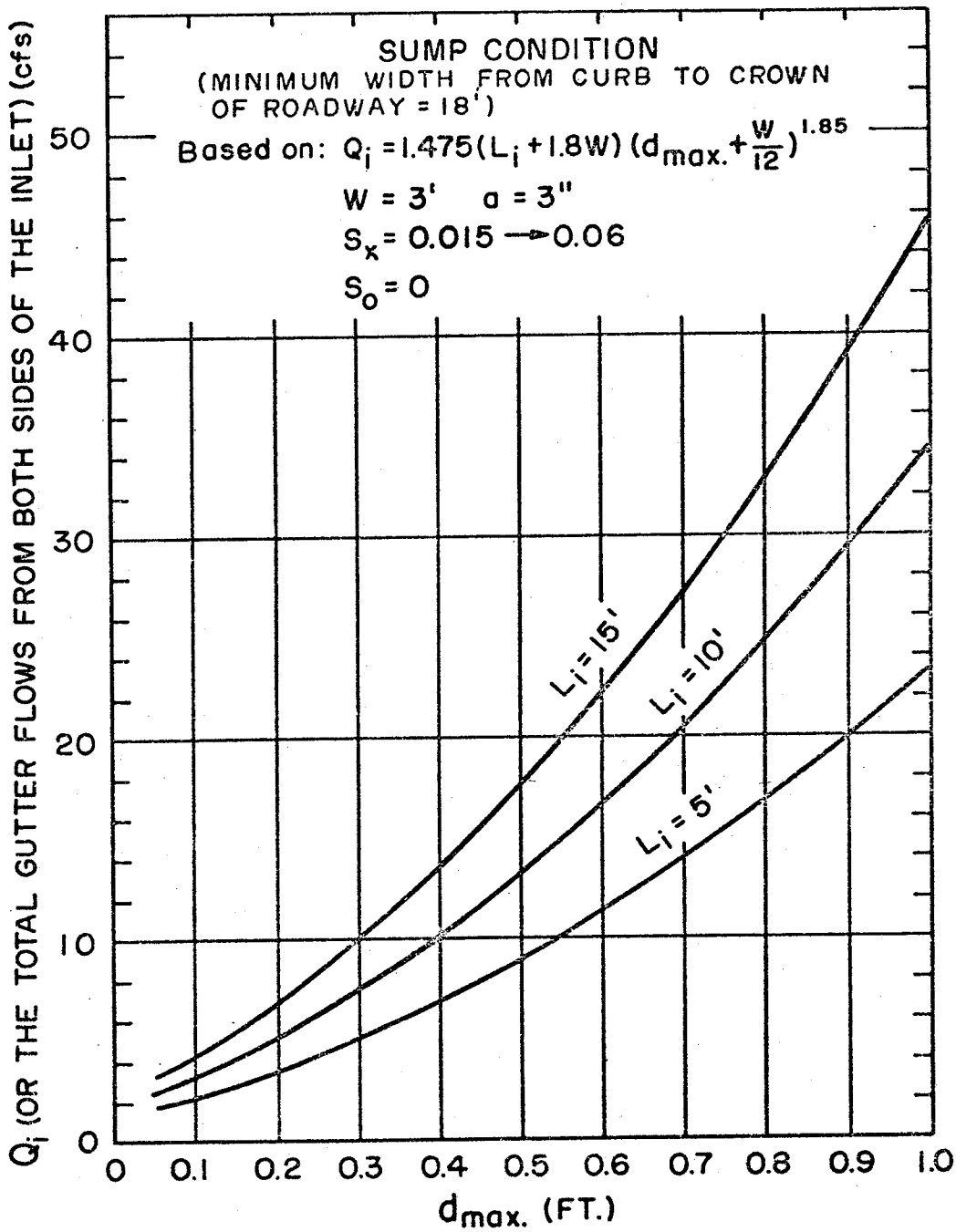
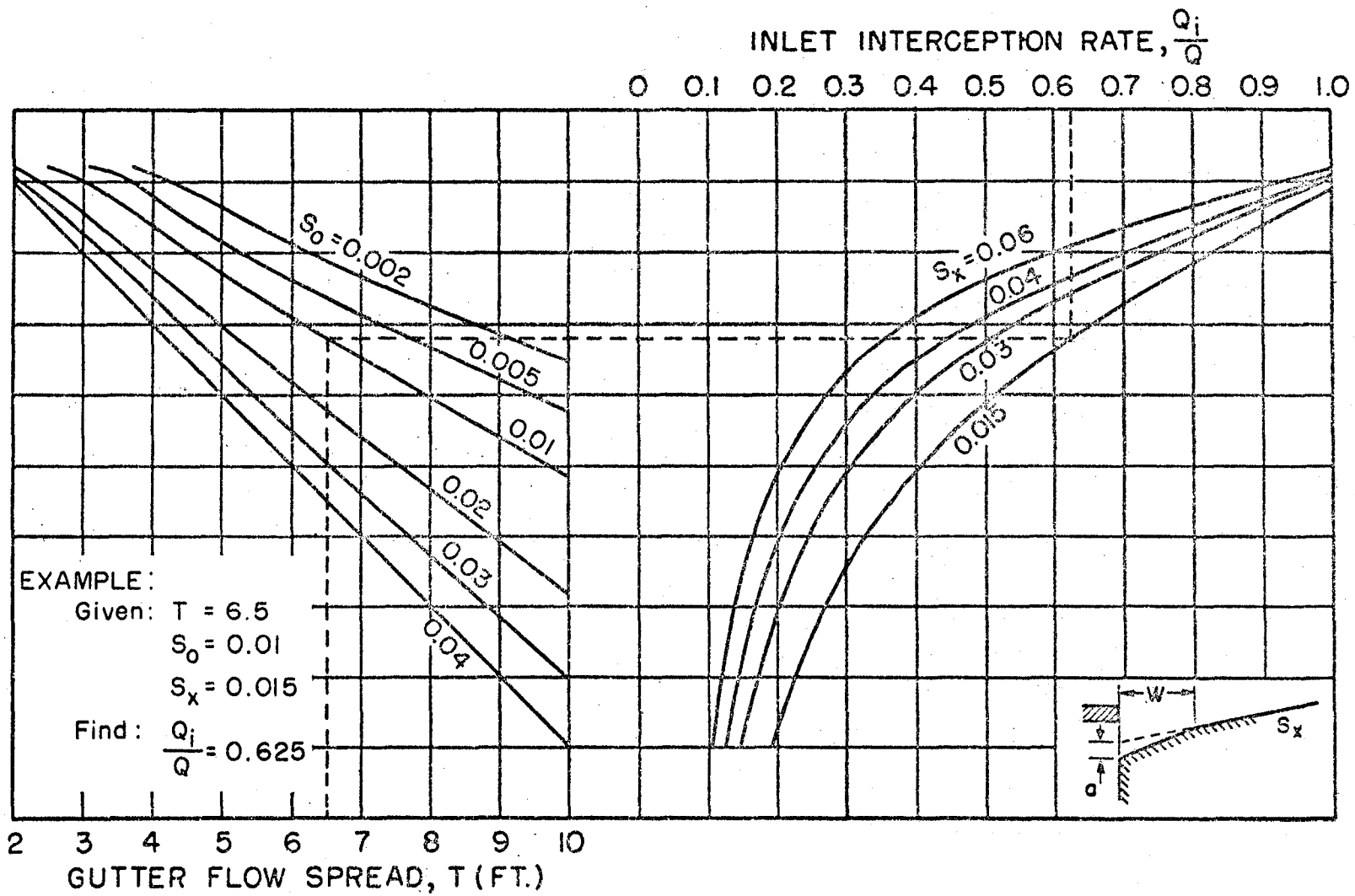
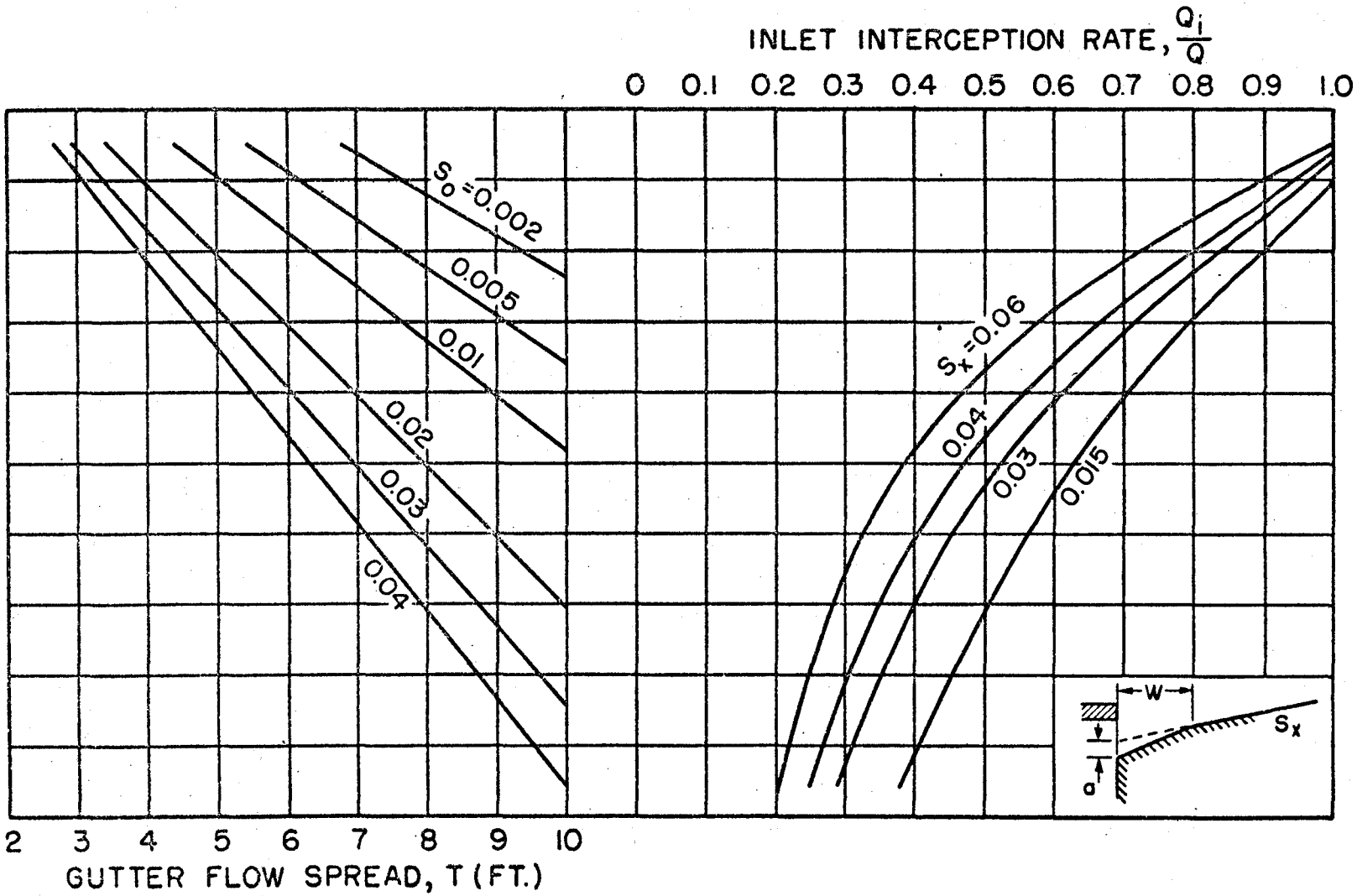

SUMP CONDITION
(MINIMUM WIDTH FROM CURB TO CROWN OF ROADWAY = 18')
Based on: $Q_i = 1.475(L_i + 1.8W)(d_{max.} + \frac{W}{12})^{1.85}$
W = 3' a = 3"
$S_x = 0.015 \rightarrow 0.06$
$S_o = 0$
$L_i = 15'$
$L_i = 10'$
$L_i = 5'$
Q_i (OR THE TOTAL GUTTER FLOWS FROM BOTH SIDES OF THE INLET) (cfs)
0
10
20
30
40
50
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
$d_{max.}$ (FT.)

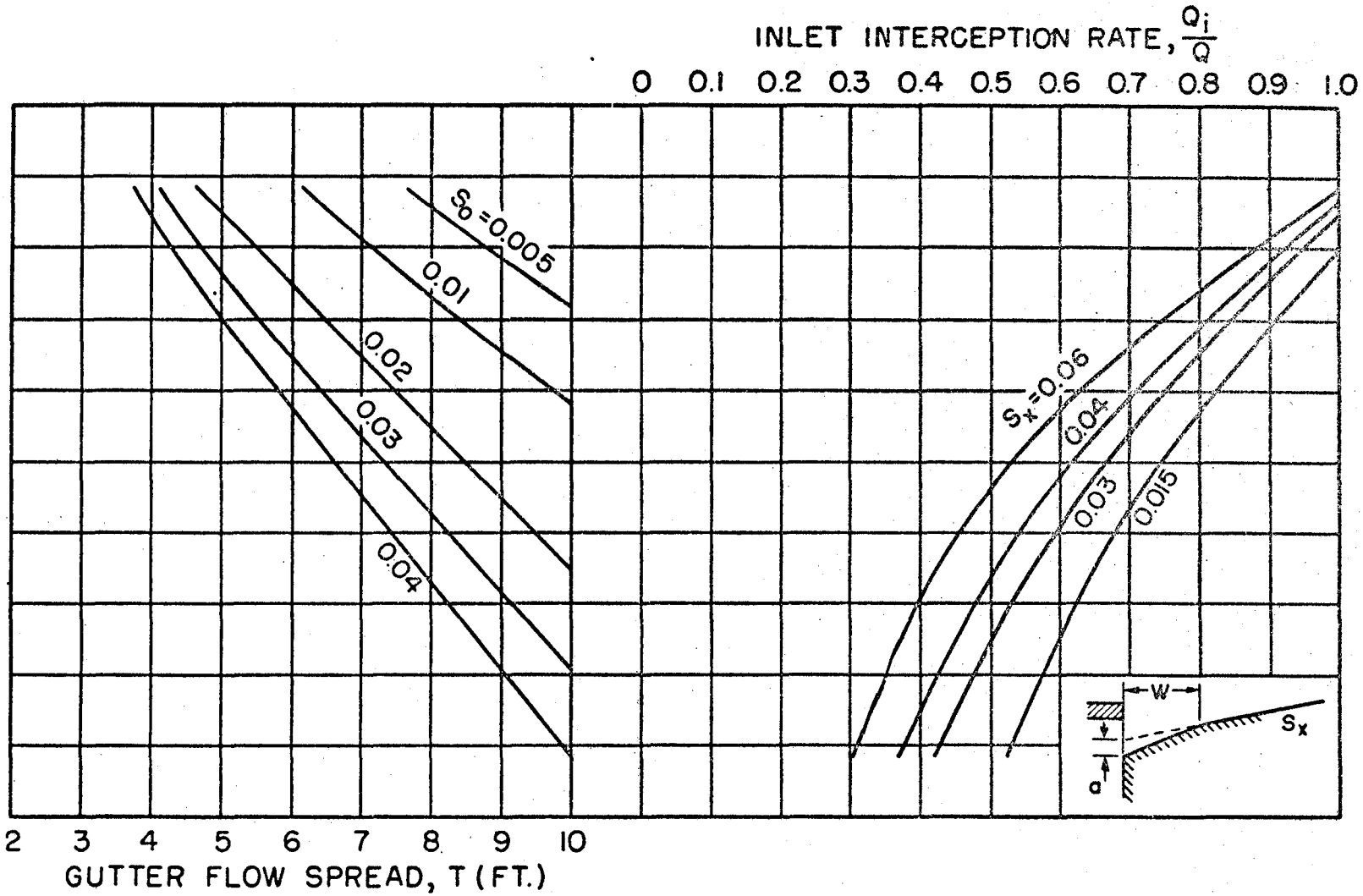
Chart 5



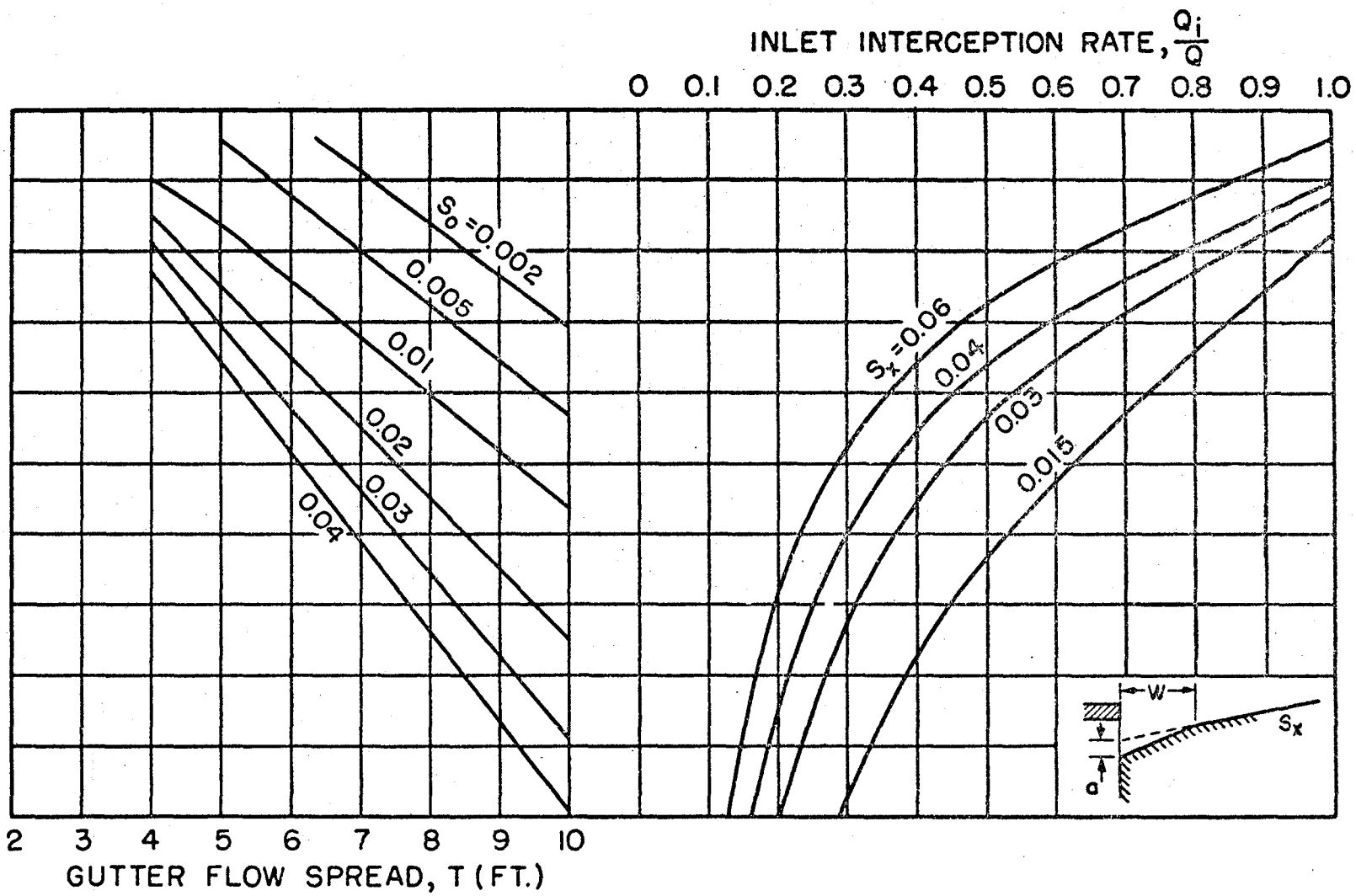
General condition
W = 1' a ≧ 1"
n = 0.016
Length of opening, L_i = 5'
Minimum height of curb opening, $h_m = T\ S_x$



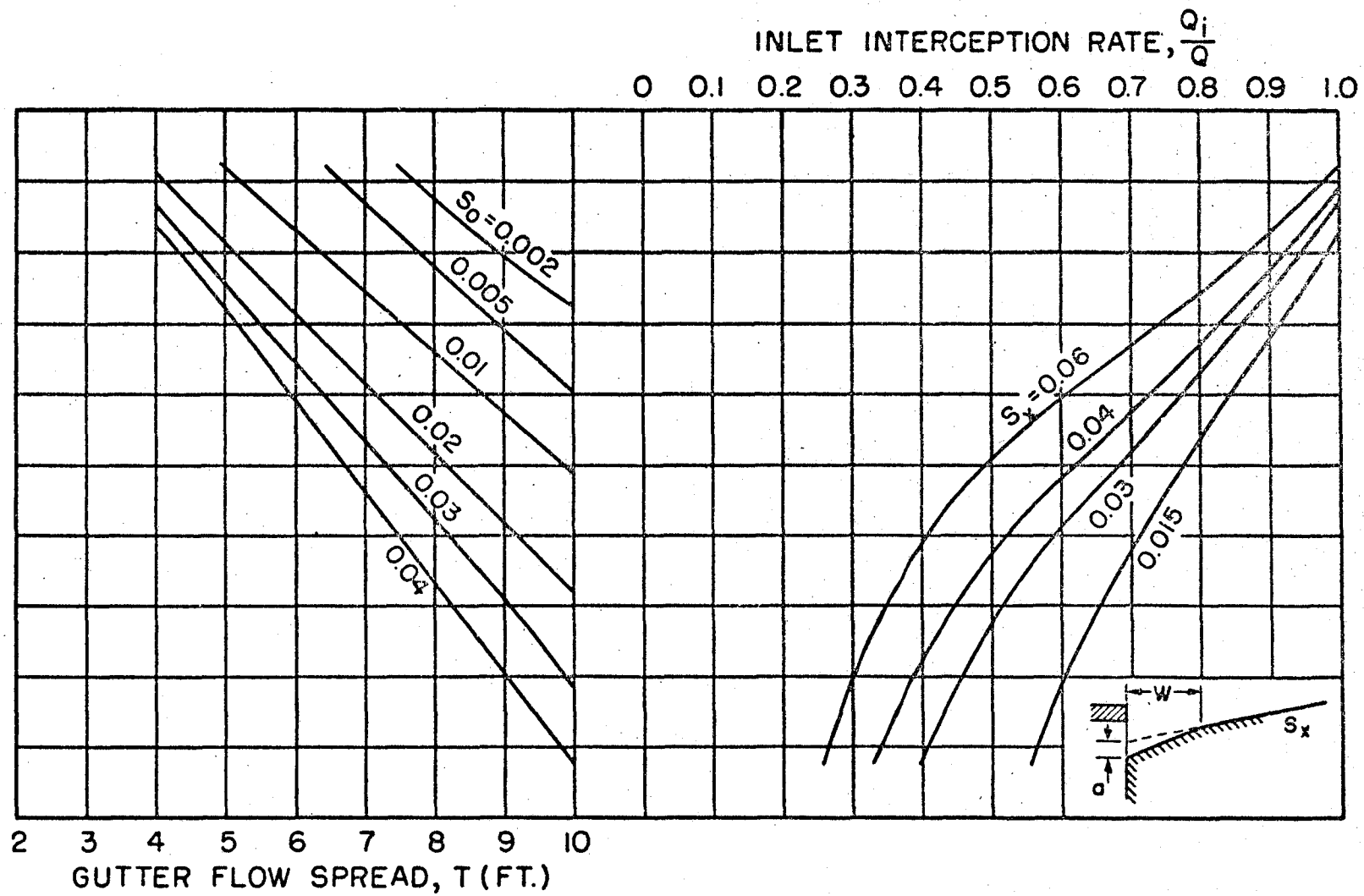
General condition
W = 1' a ≧ 1"
n = 0.016
Length of opening, L_i = 10'
Minimum height of curb opening, $h_m = T\ S_x$



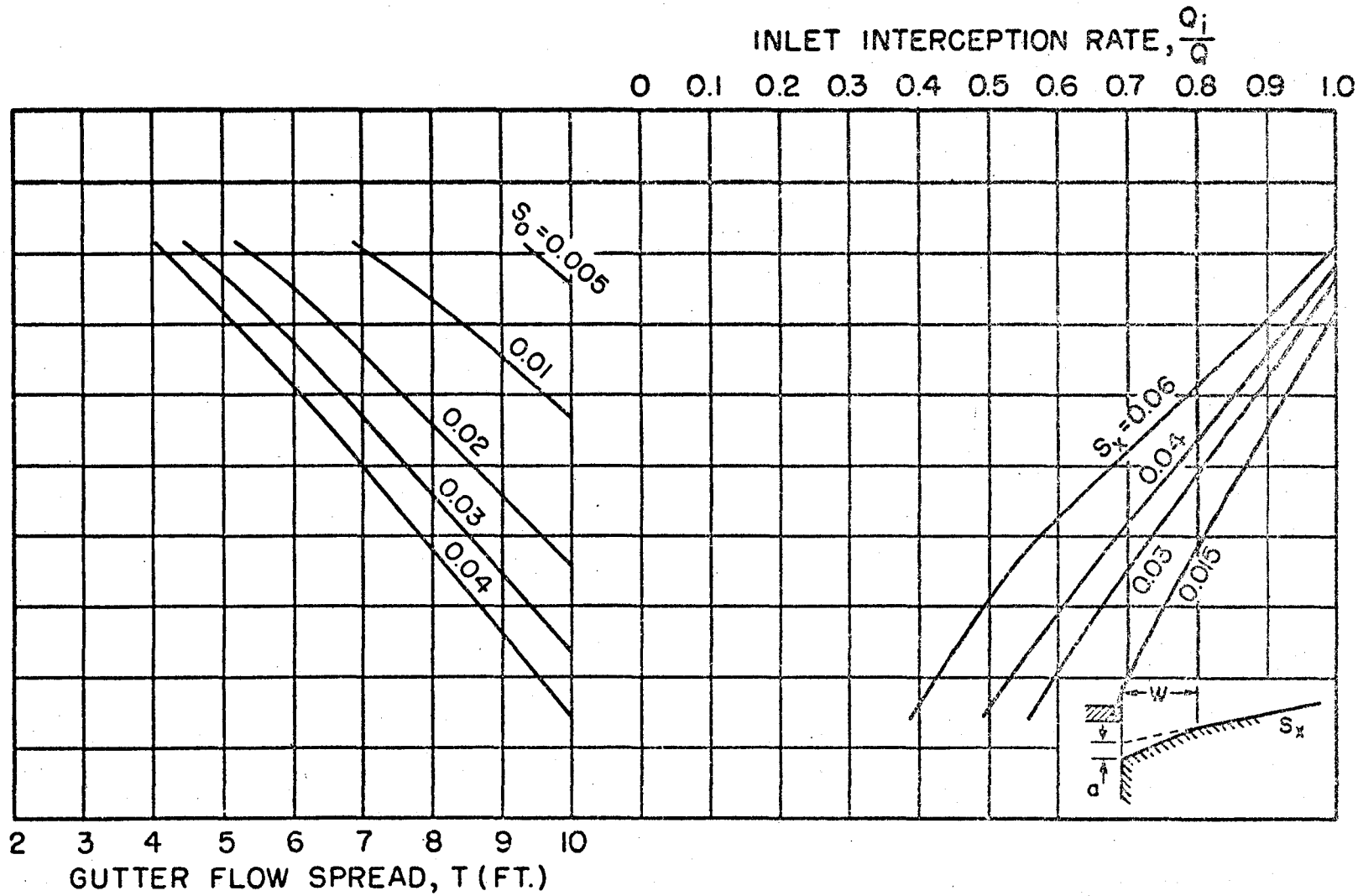
General condition
W = 1' a ≧ 1"
n = 0.016
Length of opening, L_i = 15'
Minimum height of curb opening, $h_m = T \cdot S_x$



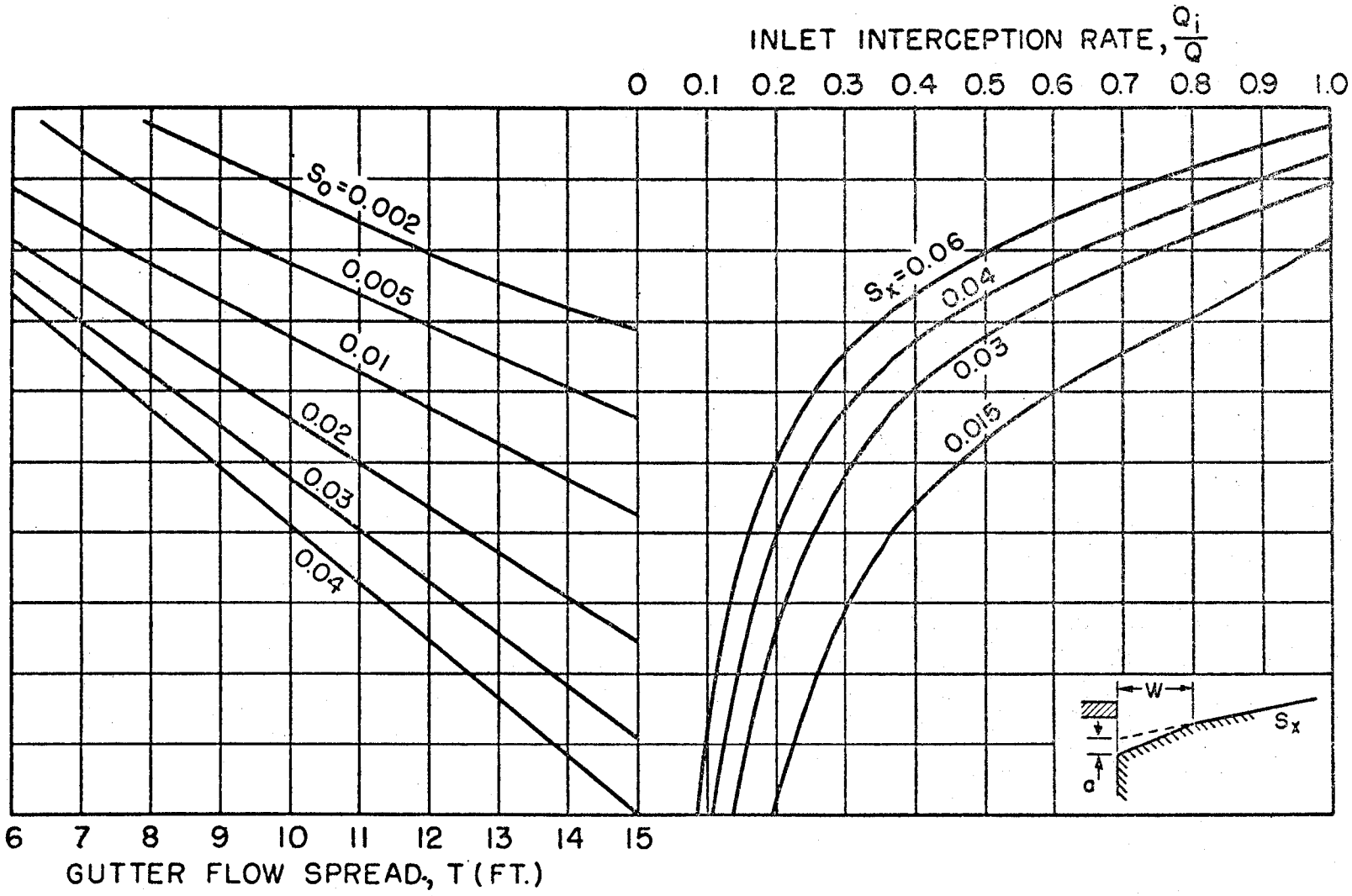
General condition
W = 2' a ≧ 2"
n = 0.016
Length of opening, L_i = 5'
Minimum height of curb opening, $h_m = T\ S_x$



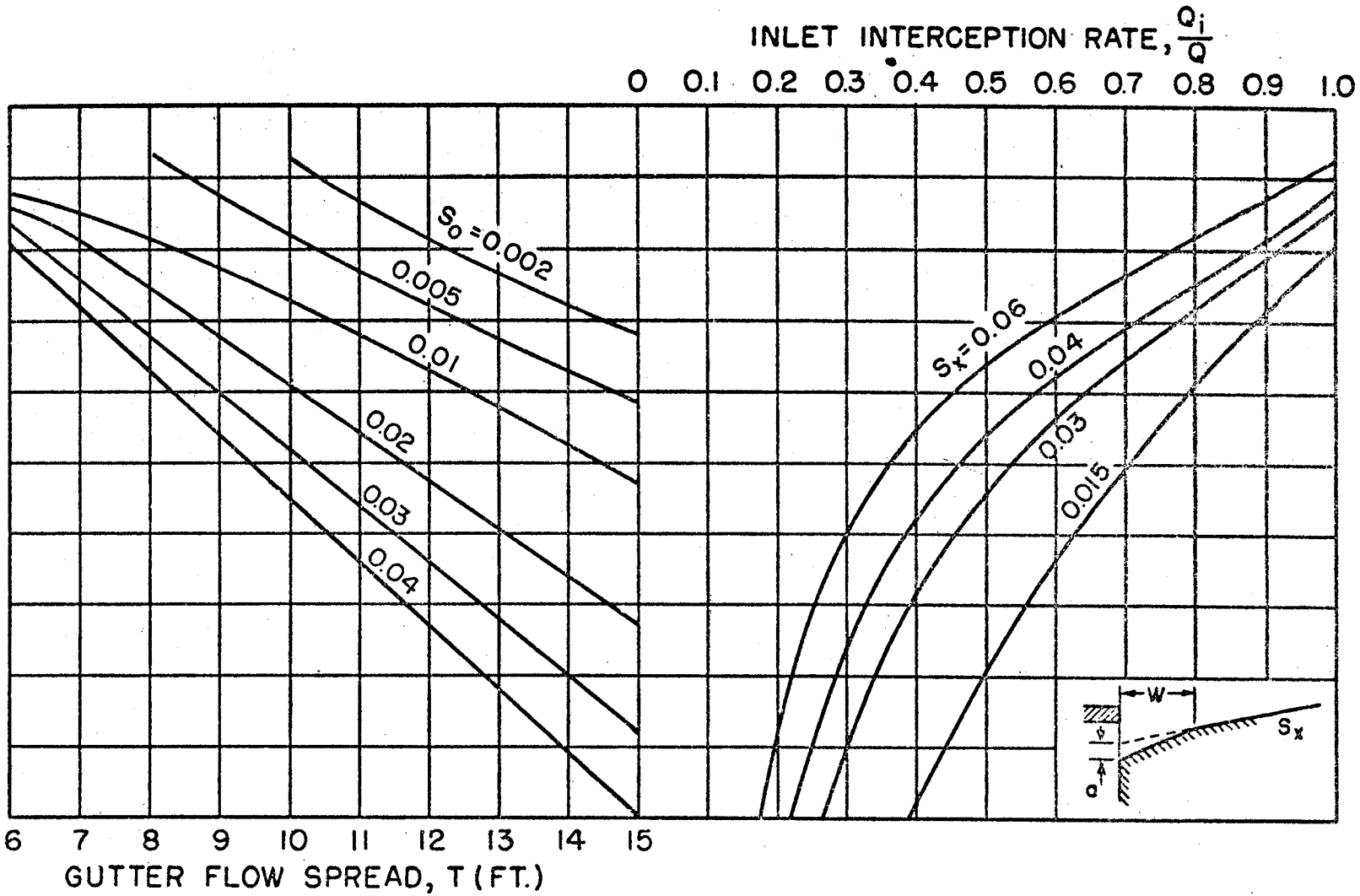
General condition
W = 2' a ≧ 2"
n = 0.016
Length of opening, L_i = 10'
Minimum height of curb opening, $h_m = T\ S_x$



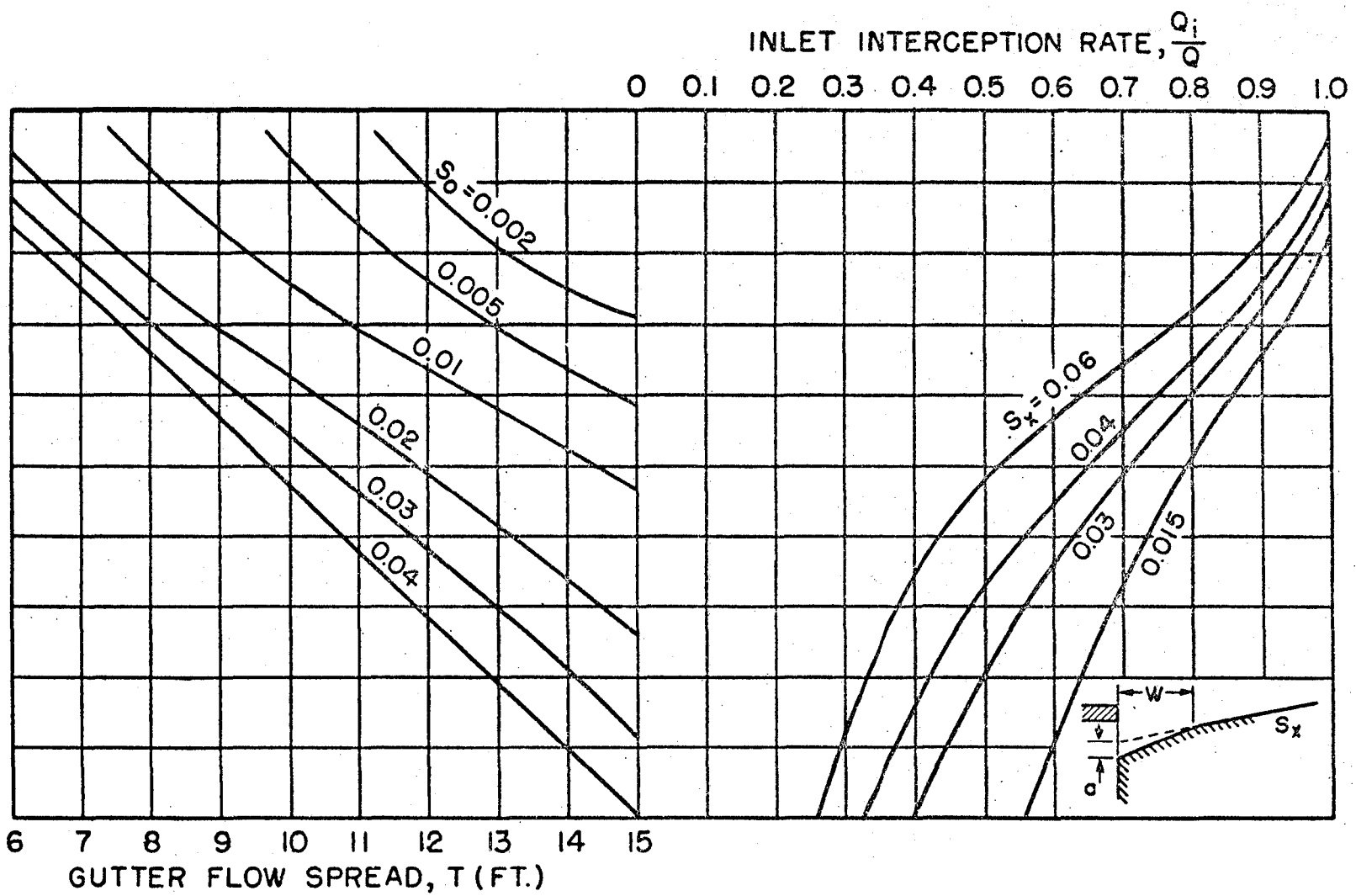
General condition
W = 2' a ≧ 2"
n = 0.016
Length of opening, L_i = 15'
Minimum height of curb opening, $h_m = T\ S_x$



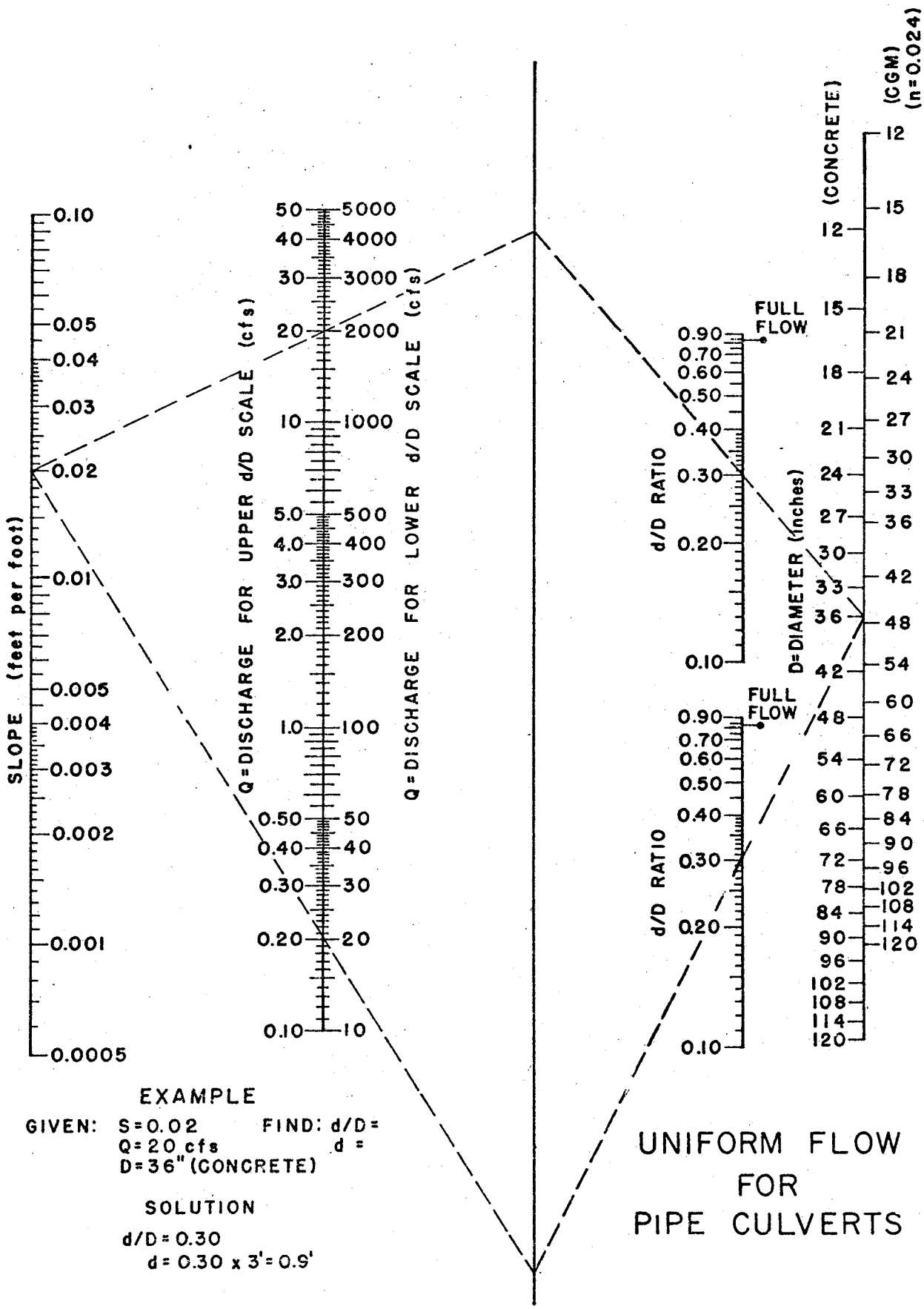
General condition
W = 3' a ≧ 3"
n = 0.016
Length of opening, L_i = 5'
Minimum height of curb opening, $h_m = T\ S_x$



General condition
W = 3' a ≧ 3"
n = 0.016
Length of opening, L_i = 10'
Minimum height of curb opening, $h_m = T\ S_x$



General condition
W = 3' a ≧ 3"
n = 0.016
Length of opening, L_i = 15'
Minimum height of curb opening, $h_m = T\ S_x$



EXAMPLE

GIVEN: S=0.02 FIND: d/D=
Q=20 cfs d =
D=36" (CONCRETE)

SOLUTION

d/D=0.30
d=0.30 x 3'=0.9'

UNIFORM FLOW FOR PIPE CULVERTS

AHD
Structures Section
Hydraulics Branch
10-15-72

Chart 15

Texas
Highway
Department

REVISED GRATE OPENING AREAS
Standards C-15.06 & C-15.07
Revised Oct. 15, 1972

CATCH BASIN GRATE C-15.06

Type	Clear Spacing	No. Bars	X	Frame Opening Sq. Ft.	Net Grate Opening Sq. Ft.
LW or LB-1.0	1"	16	5/16"	6.17	4.11
LW or LB-1.1	1 3/8"	12	1 1/4"	6.17	4.63
LW or LB-1.2	2"	9	1 9/16"	6.17	5.01
LW or LB-2.0	1"	12	5/16"	4.63	3.08
LW or LB-2.1	1 3/8"	9	1 1/16"	4.63	3.47
LW or LB-2.2	2"	7	1 1/16"	4.63	3.73

CATCH BASIN GRATE C-15.07

Type	Clear Spacing	No. Bars	X	Frame Opening Sq. Ft.	Net Grate Opening Sq. Ft.
TW or TB-1.0	1"	27	7/8"	5.59	3.70
TW or TB-1.1	1 3/8"	22	11/16"	5.59	4.08
TW or TB-1.2	2"	16	1 5/8"	5.59	4.53
TW or TB-2.0	1"	27	7/8"	4.05	2.68
TW or TB-2.1	1 3/8"	22	11/16"	4.05	2.96
TW or TB-2.2	2"	16	1 5/8"	4.05	3.28

ARIZONA HIGHWAY DEPARTMENT ROADWAY CONSTRUCTION STANDARDS

CATCH BASIN GRATES

Table 1

For Determining the Area "a" of the Cross Section
of a Circular Conduit Flowing Part Full

Let $\frac{\text{depth of water}}{\text{diameter of channel}} = \frac{D}{d}$ and C_a = the tabulated value. Then $a = C_a d^2$.

$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.0000	.0013	.0037	.0069	.0105	.0147	.0192	.0242	.0294	.0350
.1	.0409	.0470	.0534	.0600	.0668	.0739	.0811	.0885	.0961	.1039
.2	.1118	.1199	.1281	.1365	.1449	.1535	.1623	.1711	.1800	.1890
.3	.1982	.2074	.2167	.2260	.2355	.2450	.2546	.2642	.2739	.2836
.4	.2934	.3032	.3130	.3229	.3328	.3428	.3527	.3627	.3727	.3827
.5	.393	.403	.413	.423	.433	.443	.453	.462	.472	.482
.6	.492	.502	.512	.521	.531	.540	.550	.559	.569	.578
.7	.587	.596	.605	.614	.623	.632	.640	.649	.657	.666
.8	.647	.681	.689	.697	.704	.712	.719	.725	.732	.738
.9	.745	.750	.756	.761	.766	.771	.775	.779	.782	.784

Table 2

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: RUNOFF CALCULATION SHEET

LOCATION DATA

Highway _____

Location _____

Project No. _____

DESIGN DATA

Frequency _____ years

P_6= _____ in. P_{24}= _____ in. P_1= _____ in.

RUNOFF CALCULATIONS

Drainage Area			Pav't.			Comm.			Resid.										Tc	I	Q
No.	Station - Station	Acre	C	A	CA	C	A	CA	C	A	CA	C	A	CA	C	A	CA	ΣCA	min.	in. /hr.	cfs

Computed by: _____ Checked by: _____ Date: _____

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: INLET CALCULATION SHEET

LOCATION DATA

Highway _____
Location _____
Project No. _____

DESIGN DATA

Frequency _____ years
P_6= _____ in. P_{24}= _____ in. P_1= _____ in.
T_{all} _____
n _____

INLET CALCULATIONS

Inlet		DA	Inlet	Q	Q_c	Q_T	So	Gutter		Pavement		D	T	a	Inlet		Q_i/Q	Q_i	Q_c
No	Station	No	Type	cfs	cfs	cfs	%	S_x	z/n	S_x	z/n	ft.	ft.	in.	Act.	Corr.		cfs	cfs

Computed by: _____ Checked by: _____ Date: _____

F-44

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: SEWER CALCULATION SHEET

LOCATION DATA
Highway _____
Location _____
Project No. _____

DESIGN DATA
Frequency _____ years
P_6= _____ in. P_{24}= _____ in. P_1= _____ in.
Outlet TW Elev. = _____
Pipe n = _____

SEWER CALCULATIONS

Location		Drainage Area			Time of Flow, Min.			I	Q	Sewer Profile				Sewer Design			Hydraulic Grade Line			
Line No.	From - To	No.	Increm. CA	Σ CA	Inlet	Sewer	Design	in. /hr	cfs	Crown Elevation Inlet	Crown Elevation Outlet	Length ft.	Slope ft. / ft.	Diam. In.	Q_{full} cfs	V f. p. s.	S_f ft. / ft.	h_L ft.	Elevation Inlet	Elevation Outlet

Computed by: _____ Checked by: _____ Date: _____

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: SEWER CALCULATION SHEET

LOCATION DATA
Highway _____
Location _____
Project No. _____

DESIGN DATA
Frequency _____ years
P_6= _____ in. P_{24}= _____ in. P_1= _____ in.
Outlet TW Elev. = _____
Pipe n = _____

SEWER CALCULATIONS

Location		Drainage Area			Time of Flow, Min.			I	Q	Sewer Profile				Sewer Design			Hydraulic Grade Line			
Line No.	From - To	No.	Increm. CA	Σ CA	Inlet	Sewer	Design	in./hr	cfs	Crown Elevation Inlet	Crown Elevation Outlet	Length ft.	Slope ft./ft.	Diam. In.	Q_{full} cfs	V f.p.s.	S_f ft./ft.	h_L ft.	Elevation Inlet	Elevation Outlet

Computed by: _____ Checked by: _____ Date: _____

F-45

SCOUR AND BANK PROTECTION

Introduction

The purpose of this hand-out is to offer guidelines and procedures in scour and bank protection which is to be used along with sound engineering judgement. This material is not original but is rather a compilation of ideas and design procedures used by other agencies. Special acknowledgment is given to the California Division of Highways and the Federal Highway Administration for use of their publications.

Bank protection is often costly, but the lack of adequate protection may result in substantial property losses and loss of life. The size of rock used in riprap for streambank protection has commonly been based on the experience of flood control engineers. Success has increased with experience but is always subject to uncertainty under new and varied conditions. No criteria directly applicable to riprap has been developed.

Channel Bank Protection

Channel bank protection is necessary when natural bank material is unstable. Erosion frequently becomes a problem in a channel when grades become steep, drainage areas are fairly large, or where there is a constriction or a sharp curve in a channel.

The hazard of erosion in a channel may be estimated by first determining the probable maximum allowable velocity for the soil type known as well as the maximum velocity at the most critical cross section. The critical section of a channel will be in a reach of maximum slope and minimum channel bottom width. If the calculated maximum velocity in this location is not expected to cause scour there will be little need to investigate further. As a general rule, if the channel investigated has a velocity that is less than 1.5 times the average natural channel velocity bank protection will not be required.

Design Procedure for Graded Rock Riprap - California Method

This section summarizes the California Division of Highways'

practice of sizing graded riprap. The design procedure described in the publication entitled, "Bank and Shore Protection in California Highway Practice," will be used to determine the stone sizes for a sample problem. The results of this method have been highly successful and the only known failures have occurred due to overtopping or under cutting.

California Highways Method of Designing Riprap

Reference:

Bank and Shore Protection in California Highway Practice

Example Problem (See page G 8 for Final Recommendation.)

Symbols used:

V_B = velocity, bending or impinging flow
V_A = velocity, parallel flow
V_m = Mean stream velocity
W_c = Weight class
W = Weight of rock
S_{gr} = Specific gravity of rock
T = Riprap revetment thickness

Method A = Hand or machine place RSP
Method B = Dumping
RSP = Rock slope protection

Given:

V_A = 10 ft./sec.
V_B = 20 ft./sec.
V_m = 15 ft./sec.
S_{gr} = 2.65

1. Using Chart A solve for W

When V_A = 10 ft./sec. W = 57 lbs.
V_B = 20 ft./sec. W = 1.8 Tons

2. Using Table 2

Obtain values for W, W_c and T for placement Method B

When V_A = 10 ft./sec. (Parallel Flow)
W = 57 lbs., W_c = 1/4 Ton
T = 3.3 ft.

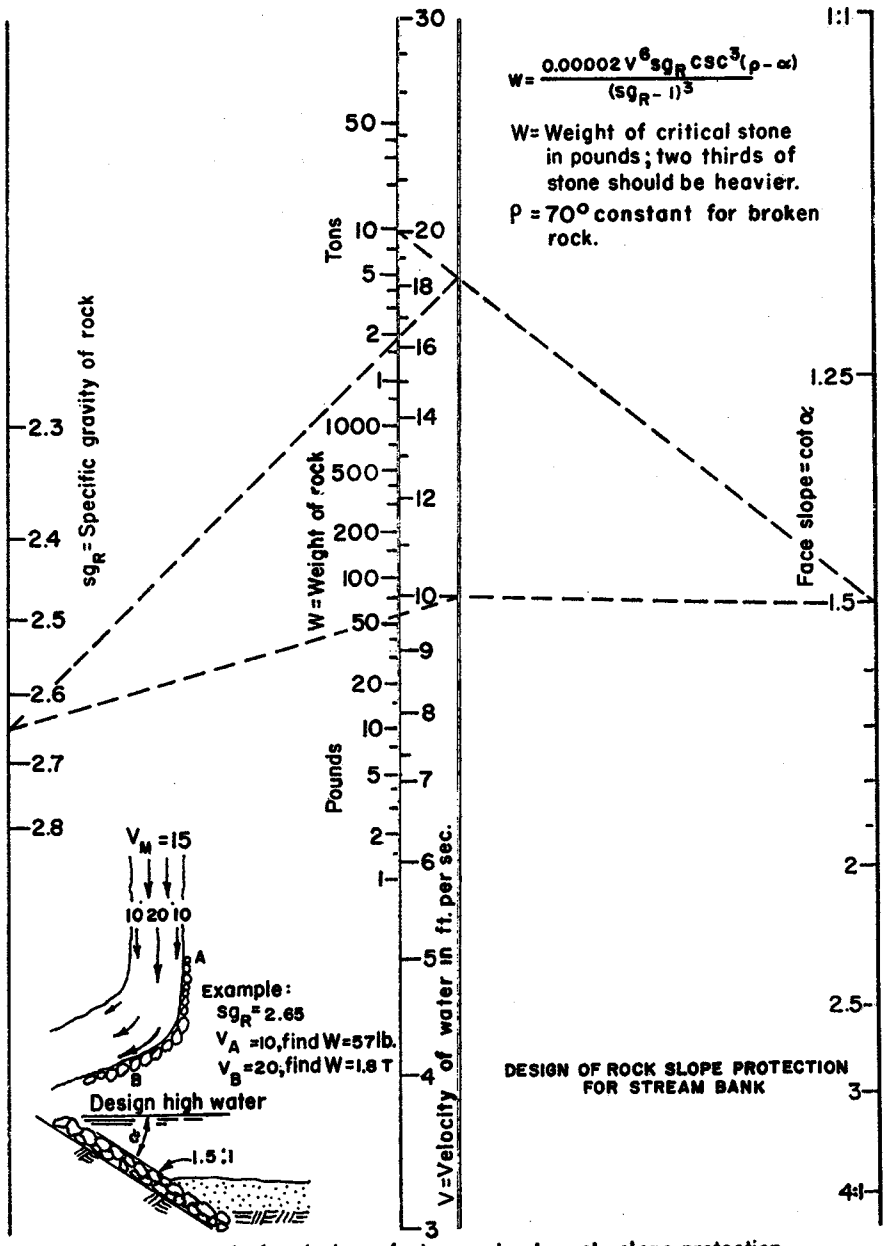
When V_B = 20 ft./sec. (Impinging Flow)
For Placement Method B
W = 1.8 Ton, W_c = 4, T = 6.7 ft.

3. Find Gradation for rock riprap - Chart page G 7.

W_c = 1/4 Ton, Placement Method B

Rock Sizes	% larger than
1/2 Ton	0-5
1/4 Ton	50-100
200 lbs.	-------
75 lbs.	95-100

4. Find gradation for Filter Blanket. This will be based on materials report based on the analysis of the channel bank. For this design select filter blanket No. 1. California Highway practice generally calls for a filter blanket thickness of one foot.



Nomograph for design of stream-bank rock slope protection

Chart A

TABLE 2. APPLICATION OF CHART D TO DESIGN OF STREAM-BANK REVETMENT

Mean stream velocity V_M	Parallel flow along tangent bank					Impingement flow against curved bank				
	Current velocity V_A	Minimum stone W	Protection class W_C	Placement method	Section thickness T	Current velocity V_B	Minimum stone W	Protection class W_C	Placement method	Section thickness
fps	fps	lb		A or B	ft	fps	lb or T		A or B	ft
4.5	3		None			6	3 lb	None		
6	4		None			8	15	Facing	B	1.8
7.5	5	1	None			10	57	¼ ton	B	3.3
9	6	3	None			12	170	¼ ton	B	3.3
10.5	7	7	Facing	B	1.8	14	430	½ ton	A	3.3
									B	4.2
12	8	15	Facing	B	1.8	16	950	1 ton	A	4.2
									B	5.3
13.5	9	30	Light	B	2.5	18	1.0 T	2 ton	A	5.3
15	10	57	¼ ton	B	3.3	20	1.8	4 ton	A	6.7
18	12	170	¼ ton	B	3.3	24	5.5	8 ton	A	8.3
21	14	430	½ ton	A	3.3	28	13.7	Special		
				B	4.2					
24	16	950	1 ton	A	4.2	32	30.4	Special		
				B	5.3					

Basic data and assumptions: velocity ratios $V_A:V_M:V_B = 2:3:4$; specific gravity of rock is $sg_r = 2.65$; face slope of revetment is 1.5:1; stones grade uniformly between specified minima for class with two thirds heavier than minimum required on face; $T = \frac{1}{3}\sqrt[3]{W_c}$, plus 25% for Method B.

$$W = \frac{2 \times 10^{-5}\, V^6\, sg_r}{(sg_r - 1)^3 \sin^3(\rho - \alpha)} = \frac{.00002\ V^6 \cdot 2.65}{1.65^3 \quad .592^3} = .000057\ V^6$$

Filter Material

Filter material is placed between the embankment and the rock slope protection to prevent fine embankment material from being washed out through the voids of the face stones. It should be uniformly graded from gravel to a size that will not work through the voids of the rock, or placed in two or more layers of progressively coarser sizes.

Where rock slope protection consists of quarry-run rock end-dumped into place, most of the finer material will naturally settle against the embankment face and the coarser stones will work to the outside, avoiding the need for backing material. But where the face stones are nearly uniform in size and embankment material is vulnerable, backing material will be necessary.

Embankment material should never be carried out over the rock slope protection so that the rock becomes a part of the fill. With this type of construction fill material will filter down through the voids of the large stones and the portion of fill above the rock will be lost.

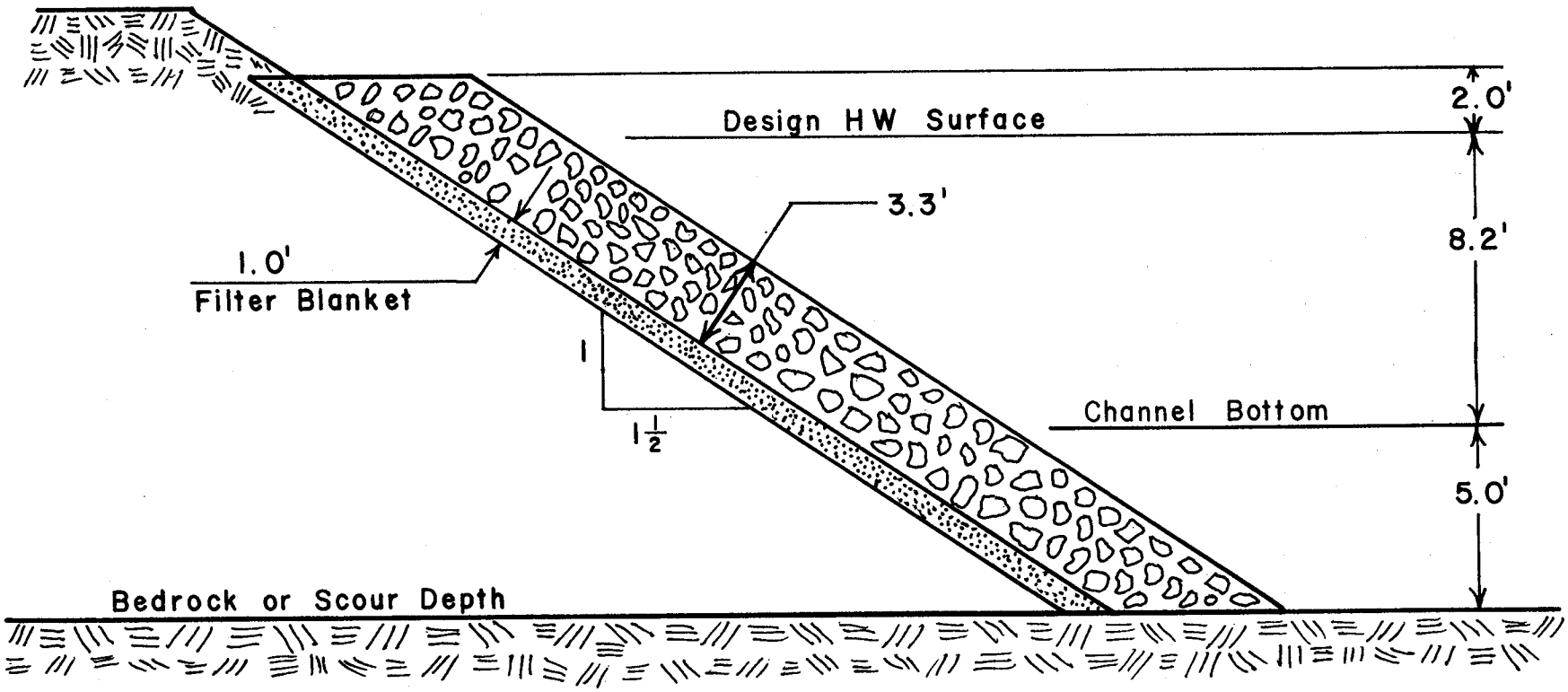
PERCENTAGE LARGER THAN *

Rock Sizes	Method A Placement					Method B Placement							
	Protection Classes Wc					Protection Classes Wc					Backing (Filter Blanket)		
	8 Ton	4 Ton	2 Ton	1 Ton	½ Ton	1 Ton	½ Ton	¼ Ton	Light	Facing	No. 1	No. 2	No. 3
16-Ton	0–5												
8-Ton	50–100	0–5											
4-Ton	95–100	50–100	0–5										
2-Ton		95–100	50–100	0–5		0–5							
1-Ton			95–100	50–100	0–5	50–100	0–5						
½-Ton				95–100	50–100	------	50–100	0–5					
¼-Ton					95–100	95–100	------	50–100	0–5				
200-Lb.							95–100	------	50–100	0–5	0–5		
75-Lb.								95–100	------	50–100	50–100	0–5	
25-Lb.									95–100	90–100	90–100	25–75	0–5
5-Lb.												90–100	25–75
1-Lb.													90–100

* The amount of material smaller than the smallest size listed in the table for any class of rock slope protection shall not exceed the percentage limit listed in the table determined on a weight basis.

Compliance with the percentage limit shown in the table for all other sizes of the individual pieces of any class of rock slope protection shall be determined by the ratio of the number of individual pieces larger than the specified size compared to the total number of individual pieces larger than the smallest size listed in the table for that class.

NOT TO SCALE



Adapted from:
Bank and Shore Protection in California Highway Practice — Calif. Div. of Highways

Specifications for Placement of Riprap:

Method A Placement - Hand or Machine Placed

A footing trench shall be excavated along the toe of slope as shown on the plans.

The larger rocks shall be placed in the footing trench.

Rocks shall be placed with their longitudinal axis normal to the embankment face and arranged so that each rock above the foundation course has a 3-point bearing on the underlying rocks. Foundation course is the course placed on the slope in contact with the ground surface. Bearing on smaller rocks which may be used for chinking voids will not be acceptable. Placing of rocks by dumping will not be permitted.

Local surface irregularities of the slope protection shall not vary from the planned slope by more than one foot measured at right angles to the slope.

Method B Placement - Dumping

A footing trench shall be excavated along the toe of the slope as shown on the plans.

Rocks shall be so placed as to provide a minimum of voids and the larger rocks shall be placed in the toe course and on the outside surface of the slope protection. The rock may be placed by dumping and may be spread in layers by bulldozers or other suitable equipment.

Local surface irregularities of the slope protection shall not vary from the planned slopes by more than one foot measured at right angles to the slope.

At the completion of slope protection work, the footing trench shall be filled with excavated material and compaction will not be required.

Creager's Equation

When the unit weight of the stone is other than 165 lbs./Ft.^{3}, the diameter of the stone should be corrected by Creager's equation.

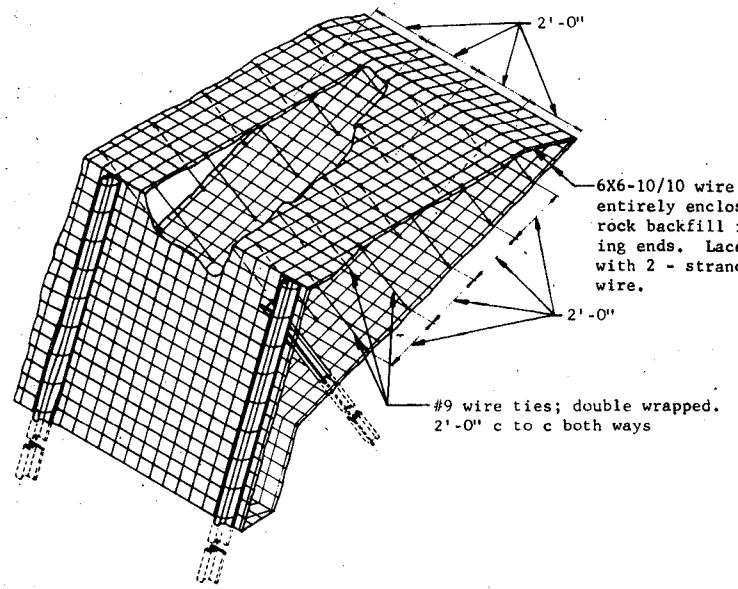
$$K_W = \frac{102.5\ K}{W - 62.5}$$

where K = dia. of stone (165 lbs./Ft.^{3}

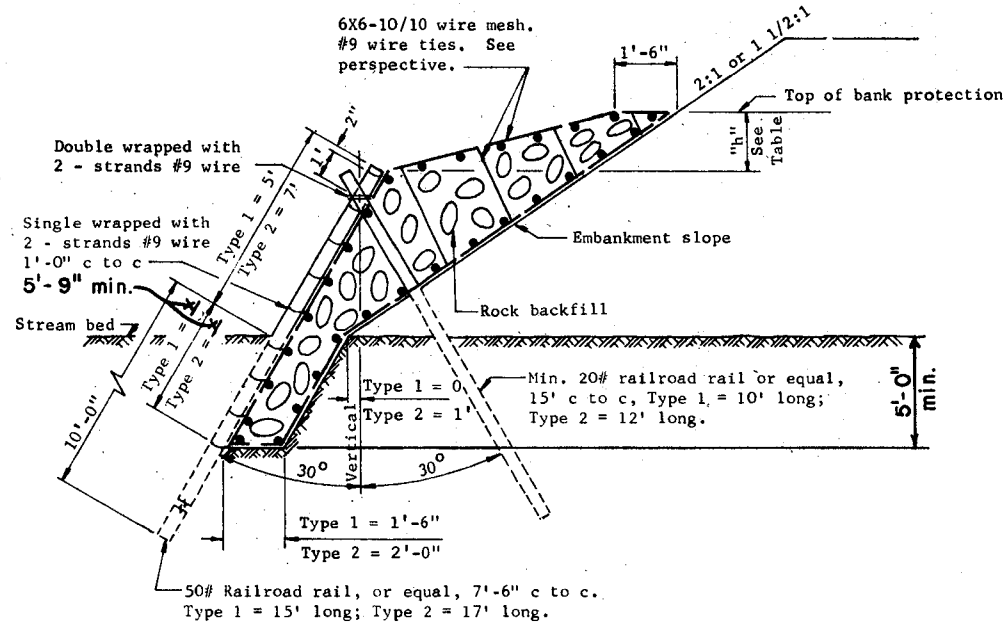
K_W = dia. of stone size for stone of W lbs./Ft.^{3}

AHD Standard Railbank Protection: See page G11.)

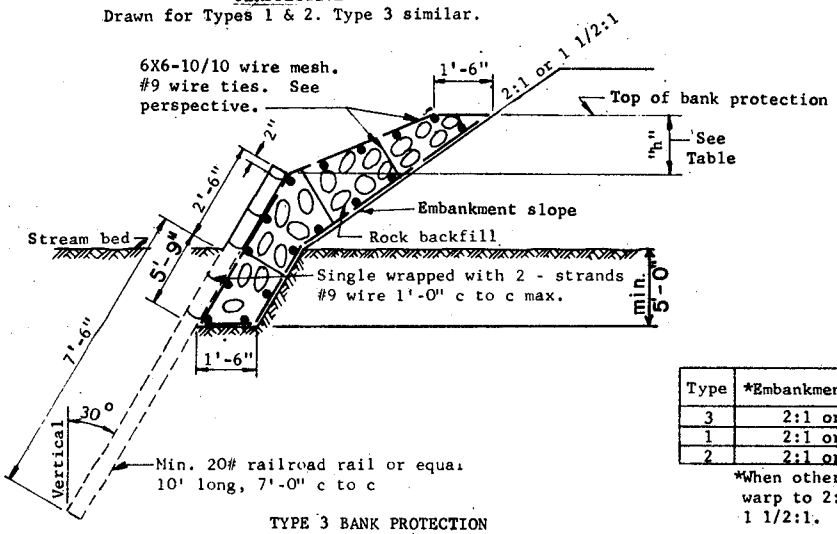
The Bridge Division of the Arizona Highway Department developed its Standard Railbank Protection in 1952. Over this 20-year period we have been well satisfied with the protection provided by this installation. Results must be considered good and failures have been caused only by overtopping or undercutting of the protection itself. We anticipate that failures or problems caused by undercutting will be significantly reduced by increasing the depth of protection below the stream bed. Our Maintenance Division has no record of basket movement or wire failure. The use of this railbank protection has some advantages over graded riprap in that it permits the utilization of smaller rock.



PERSPECTIVE
Drawn for Types 1 & 2. Type 3 similar.



TYPE : TYPE 1 & 2 BANK PROTECTION



TYPE 3 BANK PROTECTION

Type	*Embankment slope rate	"h"	Top of bank protection above stream bed
3	2:1 or 1 1/2:1	0' to 2'	2' to 4'
1	2:1 or 1 1/2:1	0' to 3'	4' to 7'
2	2:1 or 1 1/2:1	0' to 6'	6' to 12'

*When other embankment slope rates are encountered, warp to 2:1 or 1 1/2:1; that is, warp 1:1 slope to 1 1/2:1.

GENERAL NOTES
Rock for backfill shall be sound and durable and shall not pass a 6" square opening.

BANK PROTECTION
TYPE 1, 2, & 3

Design of Grouted Riprap

Grouted riprap is used where stone of suitable size for other types of riprap are not available. Wire can be embedded in the riprap to increase the tensile strength of the protective cover. The finished protection is rigid and has little strength. For this reason, the embankment protected must provide adequate support and the edges of the riprap cover must be protected from undermining at the toe and at the terminals. Design data are lacking for determining the specific thickness of the cover. The grouted riprap may be left with a rough surface by brushing the grout until from one-fourth to one-half the depth of the stone is exposed.

Weep holes should be provided in the blanket to provide rapid relief of any hydrostatic pressure behind the blanket. Filter blankets are generally necessary as in the case of other types of riprap.

Wire Basket Riprap

This type consists of a heavy, steel wire mesh basket, rectangular in shape and varying in size. The basket is filled with clean stone (larger than the mesh opening). This type can be used in place of other types of riprap due to its flexibility which allows it to yield to earth movement and still remain efficient and structurally sound. The selection of this type is usually made when a source of light or heavy loose riprap is not readily available. Tests have shown that a one foot deep wire basket is equal to approximately three feet of riprap. a/

a/ Refer to Wash. St. Hyd. Manual

Scour Protection Along Channel Banks

No design procedure directly applicable to riprap design has been developed for scour protection along channel banks. Through field experience, it was arbitrarily decided by this Department that the minimum depth for scour protection should be 5 feet.

Thickness of Riprap Revetment

The thickness of the riprap should be at least equal to the maximum size stone.

4.5 FILTER BLANKET FOR RIPRAP

A filter blanket may be needed beneath the riprap cover to prevent the water from removing bank material through voids in the riprap. Removal of bank material leaves cavities behind the riprap cover, and failure of the cover might result, particularly if the riprap cover is rigid and cannot slump to continue contact with the supporting soil. Whether a filter blanket is needed will depend upon the gradation of the bank material and the openings or voids in the riprap cover. For dumped riprap a filter ratio of 5 or less between layers will usually result in a stable condition. The filter ratio is defined as the ratio of the 15 percent particle size (D_{15}) of the coarser layer to the 85 percent particle size (D_{85}) of the finer layer. An additional requirement for stability is that the ratio of the 15 percent particle size of the coarser material to the 15 percent particle size of the finer material should exceed 5 but be less than 40. This requirement can be stated thus:

$$\frac{D_{15}\ \text{(riprap)}}{D_{85}\ \text{(bank)}} < 5 < \frac{D_{15}\ \text{(riprap)}}{D_{15}\ \text{(bank)}} < 40$$

If a single layer of filter material will not satisfy the filter requirements, one or more additional layers of filter material must be used. The filter requirement applies between the bank material and the filter blanket, between successive layers of filter blanket material if more than one layer is used, and between the filter blanket and the stone cover. In addition to the filter requirements, the grain size curves for the various layers should be approximately parallel to minimize the infiltration of the fine material into the coarser material. Not more than 5 percent of the filter material should pass the No. 200 sieve.

The thickness of the filter blanket ranges from 6 inches to 15 inches for a single layer or from 4 inches to 8 inches for individual layers of a multiple layer blanket. Where the gradation curves of adjacent layers are approximately parallel, thickness of the blanket layers should approach the minimum. Thickness of individual layers should be increased above the minimum proportionately as the gradation curve of the material comprising the layer departs from a parallel pattern.

An example of a filter design for dumped riprap follows. It is assumed that riprap is to be used to protect a streambank with the gradation in the filter shown in Figure 4-12. The gradation curves of the riprap and of the sand and gravel available for use are also shown on Figure 4-12.

Example 1

Material	Particle Size D_{15}		Particle Size D_{85}	
Riprap	90	mm	308	mm
Streambank	0.006	mm	0.10	mm
Sand	0.14	mm	2.4	mm
Gravel	4.0	mm	50	mm

Is filter required?

$$\frac{D_{15}\ \text{(riprap)}}{D_{85}\ \text{(streambank)}} = \frac{90}{0.10} = 900 > 5 \qquad \text{Yes}$$

Can a single layer of gravel be used?

$$\frac{D_{15}\ \text{(riprap)}}{D_{85}\ \text{(gravel)}} = \frac{90}{50} = 1.8 < 5 \qquad \text{Ok.}$$

$$\frac{D_{15}\ \text{(gravel)}}{D_{85}\ \text{(streambank)}} = \frac{4.0}{0.10} = 40 > 5 \qquad \text{No}$$

Can a layer of sand and a layer of gravel be used?

1st requirement

$$\frac{D_{15}\ \text{(riprap)}}{D_{85}\ \text{(gravel)}} = \frac{90}{50} = 1.8 < 5 \qquad \text{Ok.}$$

$$\frac{D_{15}\ \text{(gravel)}}{D_{85}\ \text{(sand)}} = \frac{4.0}{2.4} = 1.7 < 5 \qquad \text{Ok.}$$

$$\frac{D_{15}\ \text{(sand)}}{D_{85}\ \text{(streambank)}} = \frac{0.14}{0.10} = 1.4 < 5 \qquad \text{Ok.}$$

2nd requirement

$$\frac{D_{15}\ \text{(riprap)}}{D_{15}\ \text{(gravel)}} = \frac{90}{4.0} = 22 < 40 \qquad \text{Ok.}$$

$$\frac{D_{15}\ \text{(gravel)}}{D_{15}\ \text{(sand)}} = \frac{4.0}{0.14} = 29 < 40 \qquad \text{Ok.}$$

$$\frac{D_{15}\ \text{(sand)}}{D_{15}\ \text{(streambank)}} = \frac{0.14}{0.006} = 23 < 40 \qquad \text{Ok.}$$

The gradation of the sand and the gravel is satisfactory; if adequate placing methods are to be used, two minimum thickness layers (4 or 5 inches) can be used, one of sand and one of gravel.

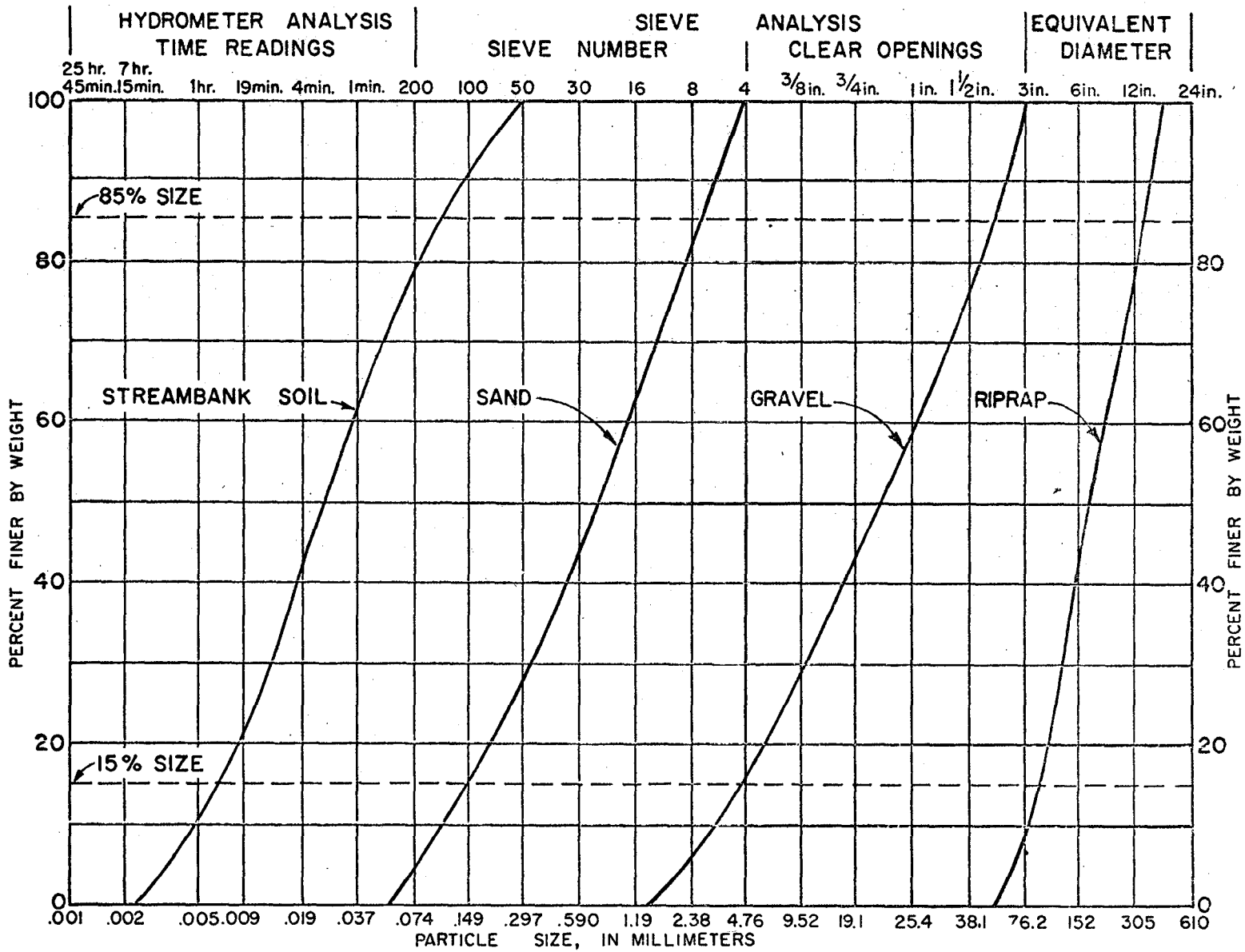


Figure 4-12

EXAMPLE OF GRADATION CURVES FOR FILTER DESIGN

Other acceptable procedures for design of rock slope protection are:

1. "Use of Riprap for Bank Protection,"
   Hydraulic Engineering Circular No. 11, U.S.B.P.R.

2. "Engineering Design Standards,"
   Far West States, U.S.S.C.S.

3. "Maccaferri Gabions Technical Handbook,"
   Rocky Mountain Gabions, LTD.

CULVERT OUTLET PROTECTION

Culvert outlet structures are primarily used for two purposes: (1) to protect the embankment and support the culvert, and (2) prevent damage by scour to the culvert, embankment, and stream bed.

I Outlet Protection Recommendations:

A. All drainage structures that have drainage areas less than 0.1 square miles in range or desert areas will not require outlet protection unless specifically requested at the field review.

B. All outlet velocities that are less than 1.5 times the average stream velocity will not require outlet protection.

C. If the outlet velocity ranges in value between 1.5 to 2.5 times the average stream velocity and (1) the velocity is less than 10 ft./sec., dumped riprap shall be used, however (2) for velocities greater than 10 ft./sec., wire-tied riprap will be required.

D. Outlet velocities greater than 2.5 times the average stream velocity will require the analyses of an energy dissipator or the possible increase in waterway opening. (See "Energy Dissipators.")

E. It is recommended that borrow pits be located a safe distance upstream and downstream from a culvert site so that degradation will not be hazardous to a culvert. If a borrow pit is proposed in a stream, it should be located upstream from the highway rather than downstream.

II Outlet Apron

Outlet aprons provide good protection to endwall structures. Aprons remove the potential erosion point away from the endwall foundation of structures by providing an area to spread and decrease the

flow concentration before outleting into the existing channel. By distributing the flow, bank-eroding turbulences are dampened and minimized.

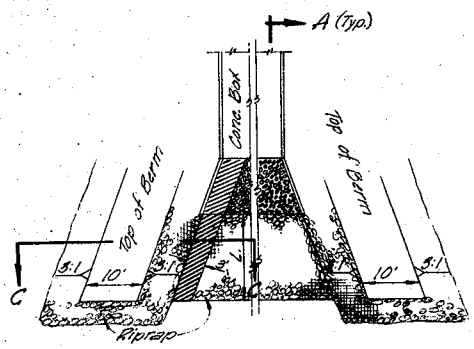
Outlet Apron (See drawing pg. G 20.)

(a) Wire Basket
(b) Loose Riprap
(c) Grouted Riprap
(d) Outlet Apron Dimension
(e) Outlet Apron Depth

FEDERAL AID PROJECT	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
	ARIZONA				

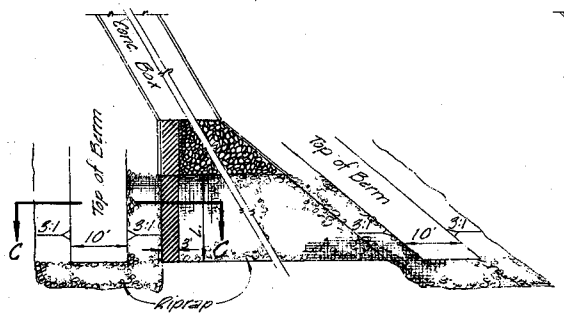
NOTE:
Provide 3' wide paved path where noted on riprap outlet sections designated as cattle pass.

NOTE
Riprap to be enclosed by #6x6-19/10 welded wire fabric. Baskets to be connected together w/#9 Galv. wire @ top & bottom of baskets. Top fabric mat to include top & sides and to be connected to bottom mat w/#9 Galv. wire. Min. baskets to contain 2c.y. of riprap w/a min. of 6" ø stones.



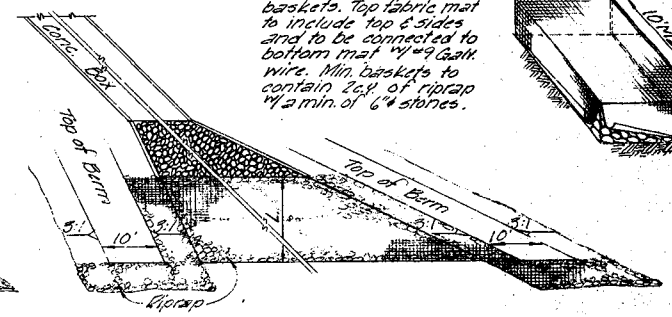
PLAN
RT.∡ BOX - CW-Wings

Riprap Schedule				
Station	L.	D.	C.Y.	



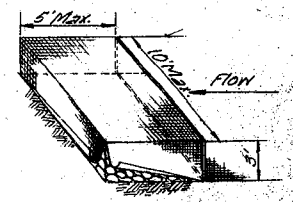
PLAN
50°Sk. BOX - CW-Wings

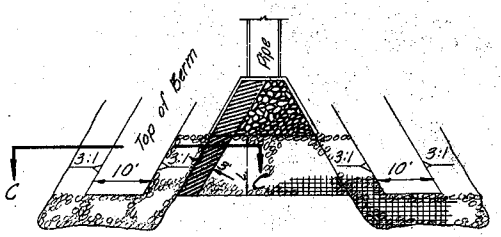
Riprap Schedule				
Station	L.	D.	C.Y.	



PLAN
45°Sk. BOX - CW-Wings

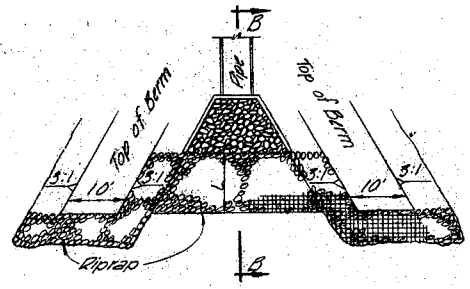
Riprap Schedule			
Station	L.	D.	C.Y.





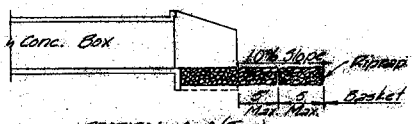
PLAN
RT.∡ SPECIAL PIPE

Riprap Schedule				
Station	L.	D.	C.Y.	

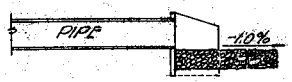


PLAN for
STD.: PIPE - STD. C14.02 WINGS

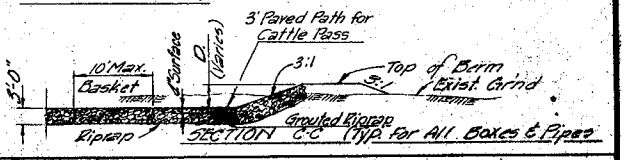
Riprap Schedule				
Station	L.	D.	C.Y.	Sk.



SECTION A-A (Typ.)



SECTION B-B (Typ.)



SECTION C-C (Typ. for All Boxes & Pipes)

RIPRAP DETAILS

		DATE
DESIGNED		
DRAWN		
CHECKED		
LAYOUT		
DESIGN		
ARCHITECTURAL		
DRAWN		
TRACED		
CHECKED		

SHEET NO. 1 OF 1

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

RIPRAP DETAILS AND SCHEDULES

BRIDGE NUMBER | DRAWING NUMBER | BRIDGE ENGINEER

ENERGY DISSIPATORS

1. St. Anthony Falls Stilling Basin (SAF)

2. USBR Type VI Basin (Impact – Energy Dissipator)

3. Riprap Plunge Basin

4. Straight Drop Spillway (Alaska Department of Highways)

Energy Dissipators

Outlet energy dissipators are usually required when the outlet energy is excessively high (supercritical flow) and riprap or wire basket will not be sufficient, to dissipate the excessive kinetic energy and to control damaging erosion.

Following are examples of energy dissipators that are most frequently used.

SAF BASIN

The St. Anthony Falls stilling basin is a hydraulic jump basin. All the dimensions of this basin are related in some way to the hydraulic jump. A reduction in the basin length from that of a natural hydraulic jump is achieved through the use of appurtenances consisting of chute blocks, floor blocks or baffle piers, and an end sill. General details of the SAF basin are shown in Figure 9. Dimensions of the chute blocks and floor blocks may be modified slightly to provide reasonable construction dimensions without materially affecting the efficiency of the structure.

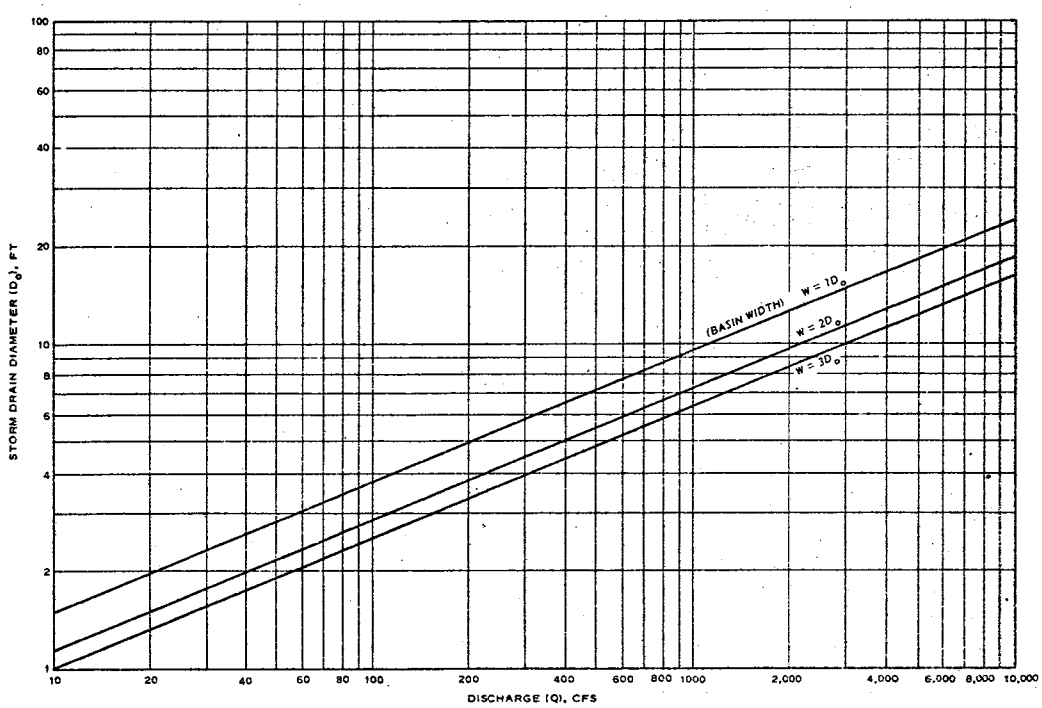


Figure 10. Storm drain diameter versus discharge for SAF stilling basin.

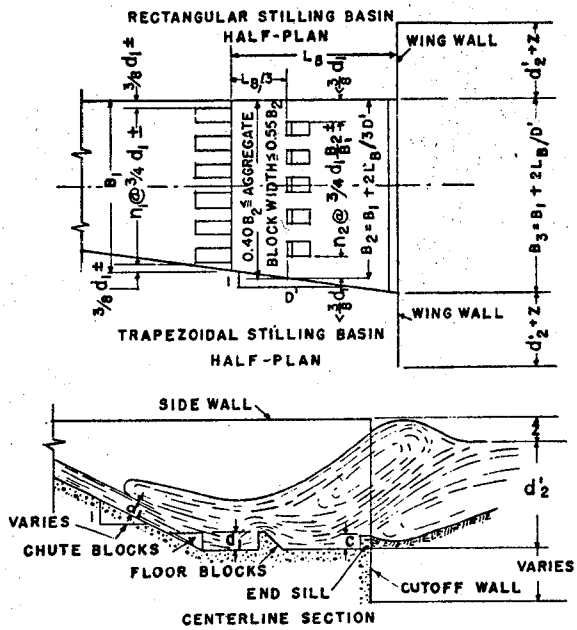


Figure 9. SAF stilling basin.

DESIGN EQUATIONS

(1) $F = \frac{V_1^2}{gd_1}$ (2) $d_2 = \frac{d_1}{2}(-1 + \sqrt{8F+1})$

(3a) $F = 3$ TO 30 $d_2^1 = (1.10 - F/120)\, d_2$

(3b) $F = 30$ TO 120 $d_2^1 = 0.85\, d_2$

(3c) $F = 120$ TO 300 $d_2^1 = (1.00 - F/800)\, d_2$

(4) $L_B = \frac{4.5 d_2}{F^{0.38}}$ (5) $Z = \frac{d_2}{3}$ (6) $c = 0.07 d_2$

Some form of protection consisting of expansions either paved or riprap-lined or both is required to prevent excessive scour downstream of energy dissipators. It is considered that either horizontal or vertical expansion or both to permit dissipation of excess kinetic energy in turbulence rather than direct attack of the channel boundaries is most practical. Guidance is needed in this area as well as for selection of the size and extent of riprap required downstream of energy dissipators. In general, the unpublished results of WES investigations of riprap protection downstream of hydraulic structures indicate that the minimum average size of stone required for protection of an exit channel downstream of an energy dissipator can be described by the following empirical relation:

where $$d_s = D\left(\frac{V}{\sqrt{gd}}\right)^3$$

d_s = minimum average size of stone, ft, usually termed d_{50} indicating that 50 percent by weight of a graded mixture is finer than the respective diameter,

D = depth of flow in channel downstream of structure, ft,

V = average velocity of flow in channel, fps, and

g = gravitational acceleration, ft/sec.

The protection should be extended downstream for a minimum distance equivalent to the width of the energy dissipator.

USBR TYPE VI BASIN

The Bureau of Reclamation impact-energy dissipator is an effective stilling device even with deficient tailwater. Dissipation is accomplished by the impact of the incoming jet on the vertical hanging baffle and by eddies that are formed by changing the direction of the jet after it strikes the baffle. Best hydraulic action is obtained when the tailwater elevation approaches, but does not exceed, a level halfway up the height of the baffle. Excessive tailwater, on the other hand, will cause some flow to pass over the top of the baffle. This should be avoided if possible. With velocities less than 2 fps, the incoming jet could possibly ride underneath the hanging baffle. Thus, this basin is not recommended with velocities less than 2 fps. It is believed that the possibility of cavitation or impact damage to the baffle can be prevented if an entrance velocity of 50 fps is not exceeded with this device. The general arrangement of the Type VI basin and the dimensional requirements based on the width of the structure are shown in Figure 6.

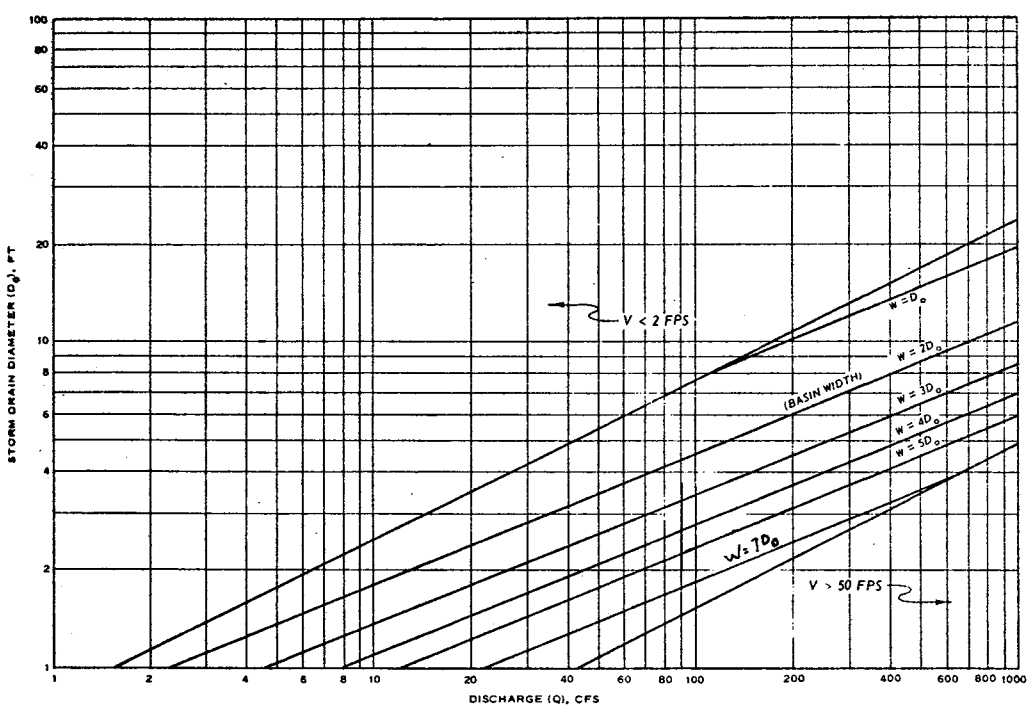


Figure 8. Storm drain diameter versus discharge for USBR Type VI basin.

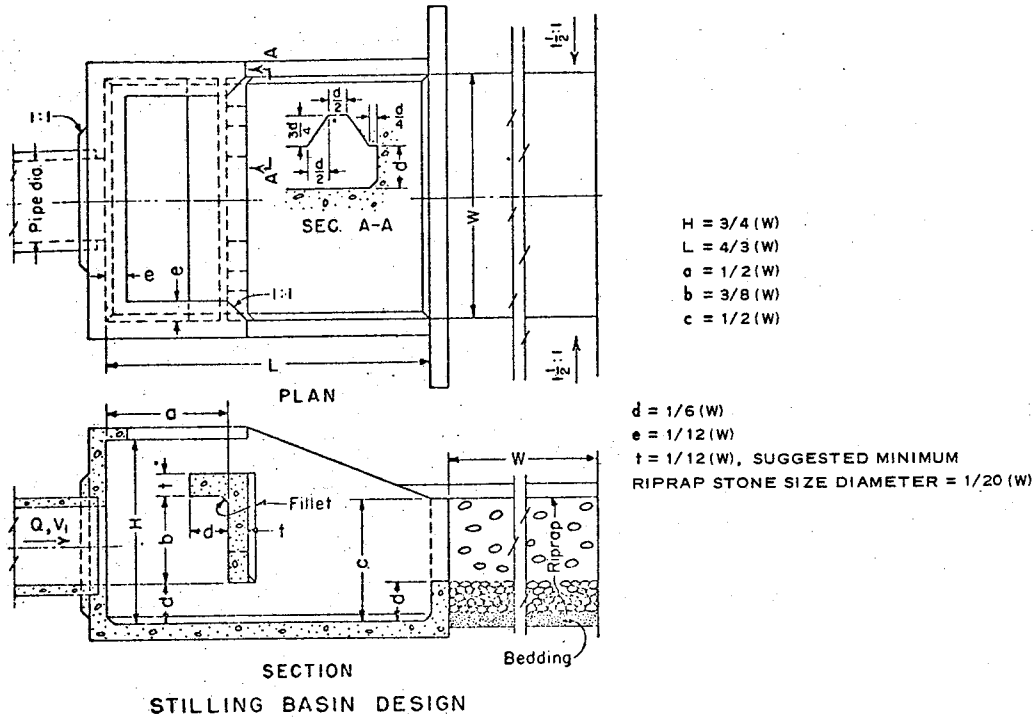


Figure 6. USBR Type VI basin.

Reference:
HIGHWAY RESEARCH BOARD
Number 373

National Academy of Sciences
2101 Constitution Avenue
Washington, D.C. 20418

RIPRAP PLUNGE BASIN

A plunge basin energy dissipator developed by the U.S. Bureau of Reclamation and refined by the Colorado State Highway Department is shown in Figure 2-8. The dimensions, based on incoming velocity, are listed in Table 2-2.

The obvious advantages to this design are ease of construction and relatively low cost. It would, of course, be neither as permanent nor as maintenance-free as a concrete structure.

Figure 2-8

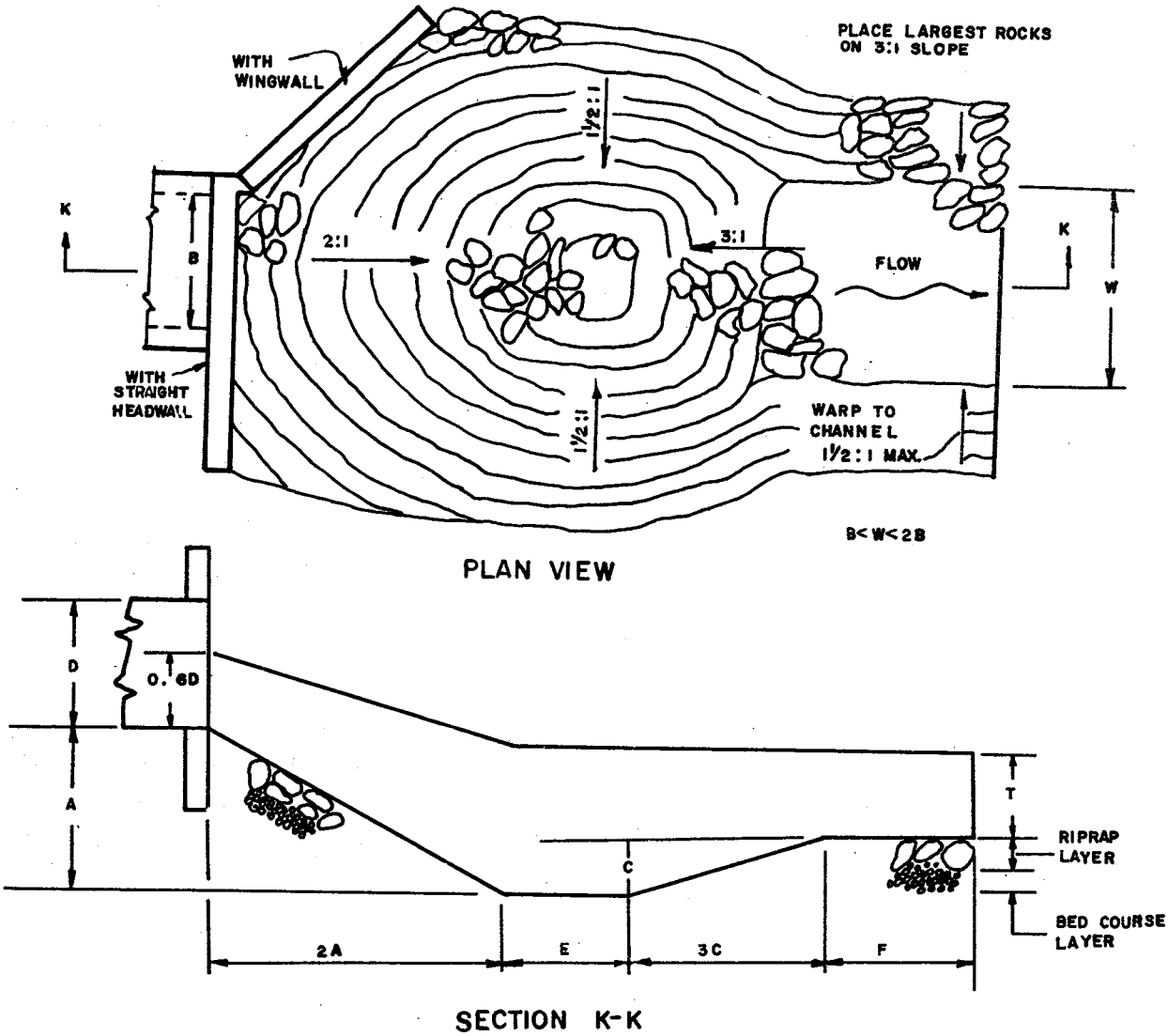


TABLE 2–2

PIPE DIA. (Inches)	BASIN DIMENSIONS (Feet)														
	V ≤18 f.p.s.					18 <V <24					24 <V <30				
	A	C	E	T	F	A	C	E	T	F	A	C	E	T	F
24	3	1.5	3	1.5	4	3	1.5	5	1.5	4	4	2	7	1.5	4
30	3	1.5	3	2	4	3	1.5	5	2	4	4	2	7	2	4
36	3	1.5	3	2.5	5	3	1.5	5	2.5	5	4	2	7	2.5	5
42	3	1.5	3	3	5	3	1.5	5	3	5	4	2	7	3	5
48	3	1.5	3	3	6	4	2	5	3	6	5	2.5	7	3	6
54	3	1.5	3	4	7	4	2	5	4	7	5	2.5	7	4	7
60	4	2	3	4	8	4	2	5	4	8	6	3	7	4	8
66	4	2	3	5	8	5	2.5	5	5	8	6	3	7	5	8
72	4	2	3	5	9	5	2.5	5	5	9	6	3	7	5	9
84	5	2.5	3	5.5	11	6	3	5	5.5	11	7	3.5	7	5.5	11
96	6	3	3	6	12	7	3.5	5	6	12	8	4	7	6	12

RIPRAP DIMENSIONS (Inches)									
RIPRAP SIZE	RIPRAP THICKNESS	BEDDING THICKNESS	RIPRAP SIZE	RIPRAP THICKNESS	BEDDING THICKNESS	RIPRAP SIZE	RIPRAP THICKNESS	BEDDING THICKNESS
16	18	6	24	24	8	24	30	10

It is a well-established fact that better performance of riprap results when it is placed over bed course material. Riprap placed directly on fine soil will often fail because the action of the water removes the fine material from beneath the riprap. Therefore, all riprap should be placed over a layer of bed course material unless the supporting base material is of similar gradation to the bed course specification. Table 2-2 shows the minimum layer thickness of the bed course material.

Plunge basins such as those illustrated in Figures 2-8 and 2-9 should be used only where the jet discharges into the air and then plunges downward into the basin. If the angle of impingement is too flat, the jet will ride across the surface at high velocity. This will cause waves and eddies in the basin sufficient to erode the side slopes, and there will be high exit velocities.

STRAIGHT DROP SPILLWAY

The straight drop spillway is essentially a straight overfall wier. The water flowing over the spillway falls on a horizontal apron where the energy is partly dissipated by blocks, sills, and tailwater. The water is discharged from the stilling basin into the downstream channel in such a manner as to prevent damaging scour. Figure 2-9 illustrates the design criteria for a straight drop spillway which has been shown to provide adequate protection from scour in the downstream channel.

Rules for the design of the straight drop spillway stilling basin are summarized below. Reference should also be made to Figure 2-9.

1. The minimum length of the stilling basin L_B is
   $L_B = x_a + x_b + x_c = x_a + 2.55\ d_c$

   a. The distance from the headwall to the point where the surface of the upper nappe strikes the stilling basin floor, x_a, is presented graphically in Figure 2-10.

   b. The distance from the point at which the surface of the upper nappe strikes the stilling basin floor to the upstream face of the floor blocks x_b is $x_b = 0.8\ d_c$, where d_c = critical depth.

   c. The distance between the upstream face of the floor blocks and the end of the stilling basin x_c is $x_c \geq 1.75\ d_c$.

2. The floor blocks are proportioned as follows:

   a. The height of the floor blocks is $0.8\ d_c$;

   b. The width and spacing of the floor blocks should be approximately $0.4\ d_c$ but a variation of $\pm\ 0.15\ d_c$ from this limit is permissible;

   c. The floor blocks should be square in plan; and

   d. The floor blocks should occupy between 50% and 60% of the stilling basin width.

3. The height of the end sill is $0.4\ d_c$.

4. Longitudinal sills may be used for structural purposes. They are neither beneficial nor harmful hydraulically. If used, they should pass through, not between, the floor blocks.

5. The sidewall height above the tailwater level should be 0.85 d_c.

6. The wingwalls should be located at an angle of 45 degrees with the outlet center line and should have a top slope of 1 on 1.

7. The minimum height of the tailwater surface above the floor of the stilling basin is $d_2 = 2.15\ d_c$.

8. The approach channel should have the following configuration:

   a. Be level with the crest of the spillway;

   b. Have the toe of the dike or the toe of the side slope intersect the approach channel floor at the ends of the spillway notch; the approach channel at the headwall should have a bottom width equal to the spillway notch length b; and

   c. Be protected by riprap or paving for a distance upstream from the headwall equal to two times the notch depth.

9. No special provision for aeration of the space beneath the nappe is required if the approach channel is shaped as recommended here.

The straight-drop spillway may be used effectively if the drop does not exceed 15 feet and if there is sufficient tailwater. An energy dissipator which has proven satisfactory beyond these limits is the baffle beam stilling basin.

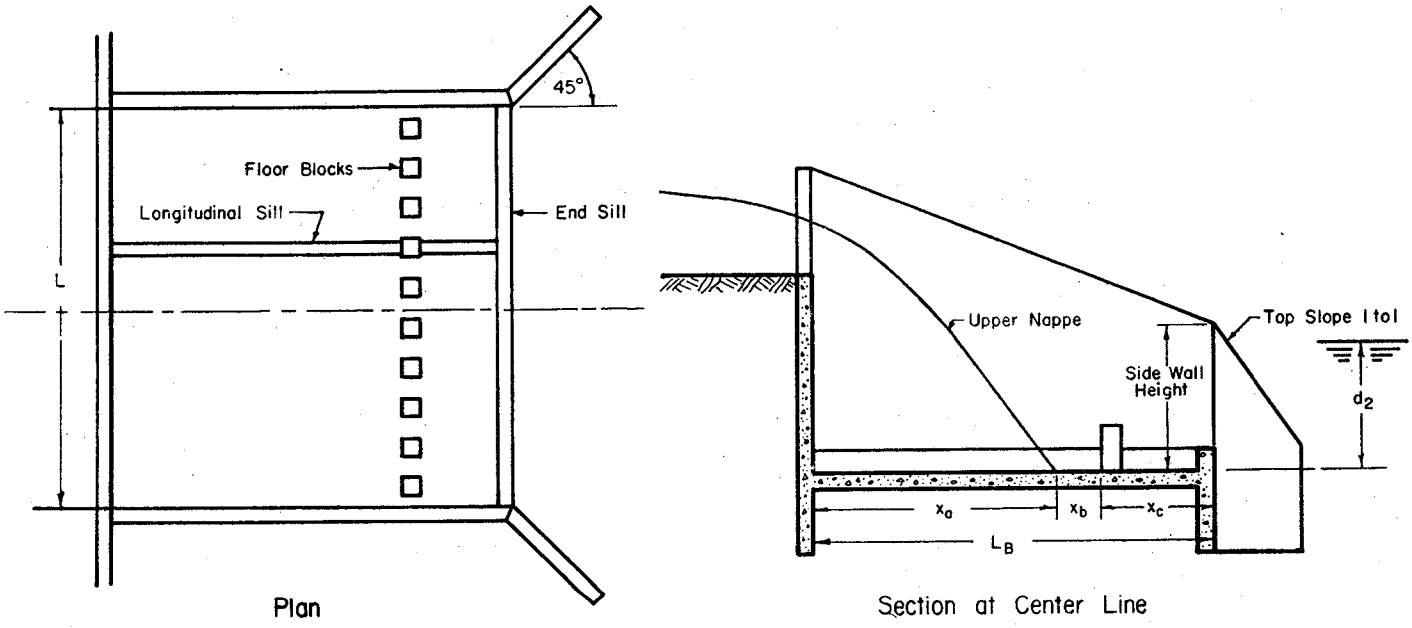


Figure 2-9

DESIGN CHART FOR DETERMINATION OF x_a

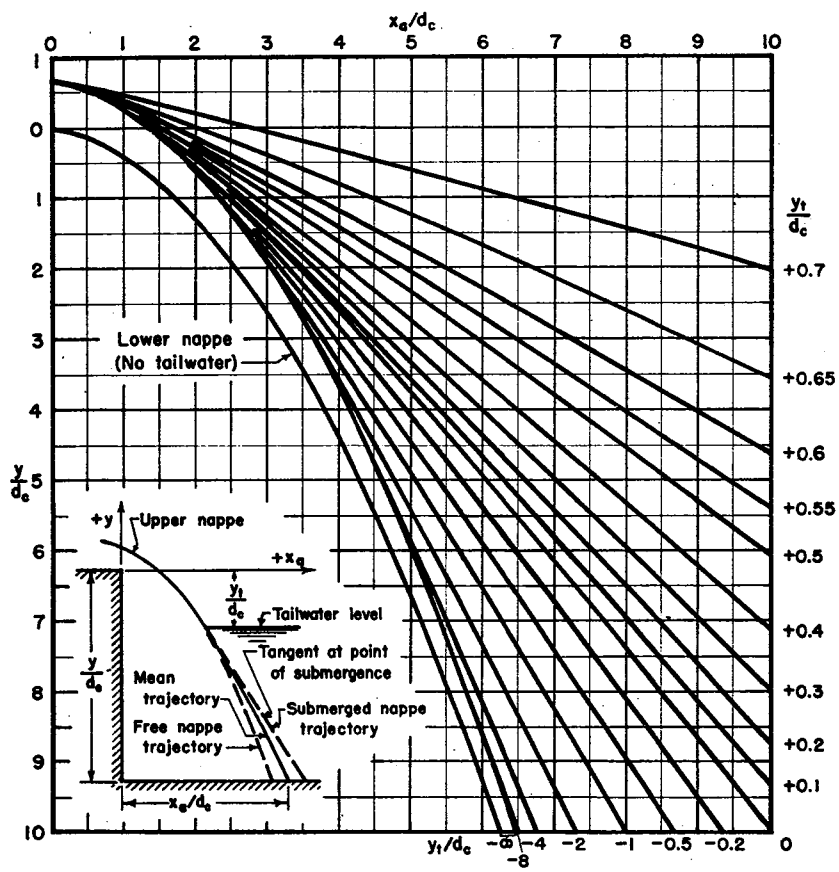


Figure 2-10

STRAIGHT DROP SPILLWAY STILLING BASIN

The following publications are also available for use in designing energy dissipators:

1. "Design of Small Dams," ---U.S.B.R.

2. "Engineering Handbook," Chute-Spillways
   Soil Conservation Service

3. "Open Channel Flow" by F.M. Henderson

4. "California Culvert Practice,"
   State of California, Department of Public Works,
   Division of Highways

5. "Handbook of Concrete Culvert Pipe Hydraulics,"
   Portland Cement Association

6. "Highway Research Record," Number 373
   Highway Research Board

COMPUTER APPLICATIONS

The following hydraulic computer programs are available to Arizona Highway Department personnel. If you have any questions about the use of the programs, do not hesitate to contact the Hydraulics Branch.

Fortran (IBM 360)

Program Number	Program Name
217-020	Hydraulic Analysis of (Box) Culverts
217-030	Natural Channel Backwater (California)
217-050	Hydraulics of Bridge Waterways (BPRHY-4-69)
217-060	Water Surface Profiles (USGS)
217-070	Log Pearson Type III Probability Distribution

ITF PROGRAMS

Name	Program Description
CH6 H3*	Manning's equation for regular sections (rect., trap., triangular)
DC1*	Critical depth of flow for trap. channels
DC3*	Critical depth of flow for circular sections
DN3*	Normal depth of flow for circular sections
HBC*	Hydraulic analysis of box culverts
HP1-HP3*	Hydraulic analysis of pipe culverts
HY1-HY2 H1-H2*	Manning's equation for irregular cross sections

ML1	Bridge pier losses by the momentum method
MN1*	Manning's equation for triangular gutter flow.
PRE* HPR	Equations for 30min., 1 hr., 2 hr., 3 hr., and 12 hr. precipitation values.
SF1	Friction slope for rectangular and circular sections
SIM	Flood routing by the storage indication method
SUM	Summation of triangular S.C.S. Hydrographs
TC1*	Time of concentration using Fig. 2-7 from the Hydrologic Manual
TC2	Time of concentration in a median using the Rational equation and Manning's equation
WS1*	Water surface profile in regular sections (down water)
WS2	Water surface profile in regular sections (backwater)

Improved inlet design calculations. These programs compute the required data as per the design sheets in HEC No. 13

II1	Improved Inlet Analysis-Outlet Control Design Calculations
ICD	Improved Inlet Analysis-Inlet Control Section Calculations (rectangular sections)
IC2	Improved Inlet Analysis-Inlet Control Section Design Calculations (circular sections)
ST1	Improved Inlet Analysis-Side-Tapered Design Calculations
ST2	Improved Inlet Analysis-Slope-Tapered Design Calculations

*/ Denotes programs in the Common Library.

NONUNIFORM FLOW

Uniform Flow occurs in a channel when the water surface is parallel to the channel bottom.

Nonuniform Flow occurs in a channel when the water surface is not parallel to the channel bottom.

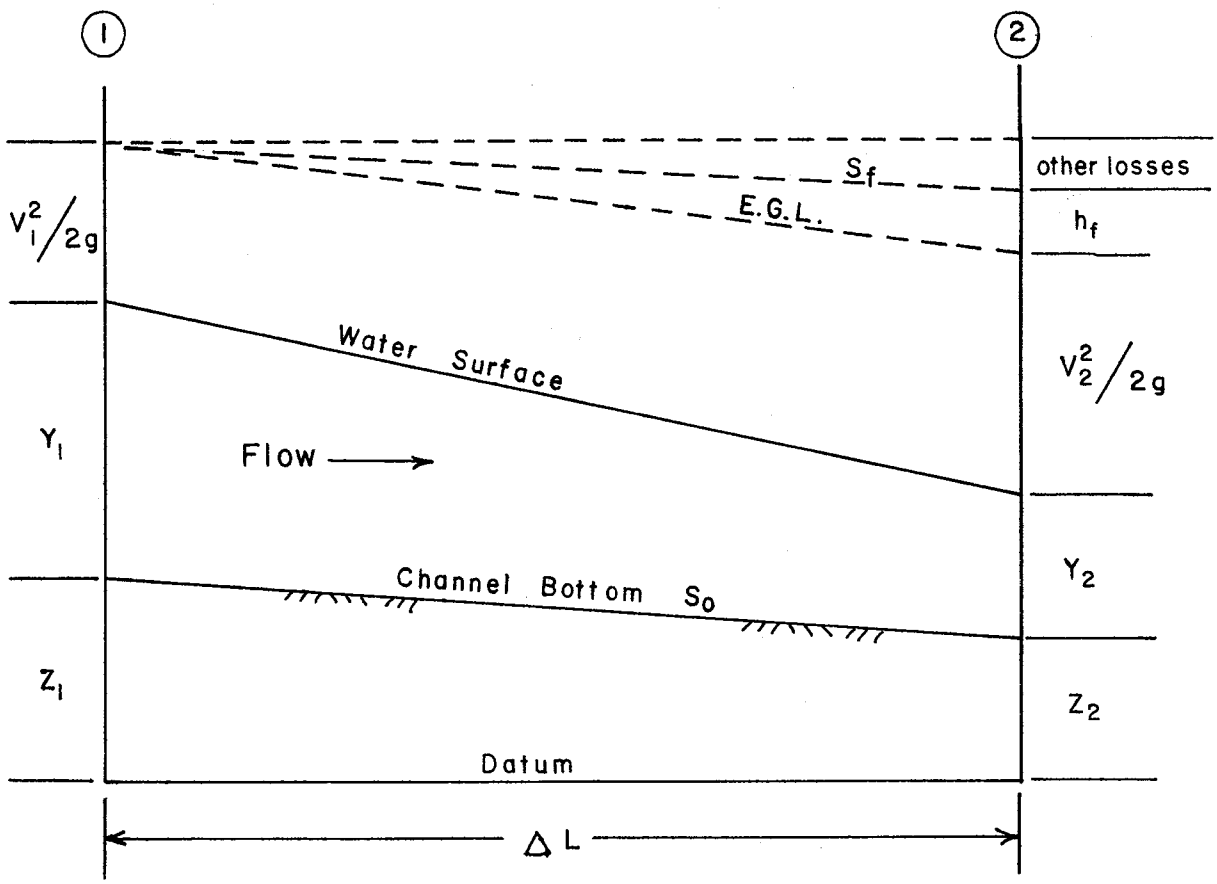


Fig. 1 — Nonuniform Flow[1/]

1/ See Table 1 for definition of symbols.

Table 1

Definition of symbols used in Nonuniform Flow Calculations:

Symbol	Definition	Units
A	Area of flow	ft.^{2}
b	Bottom width of channel	ft.
C^2	Constant = $(1.486/n)^2$	
d_c	Critical depth of flow	ft.
d_n	Normal depth of flow	ft.
E	Specific energy, $Y + V^2/2g$	ft.
h_e	Head loss due to expansion of contraction, $K(\triangle V^2/2g)$	ft.
h_f	Head loss due to friction	ft.
K	Coefficient used to compute h_e	
L	Length of reach	ft.
n	Mannings Coefficient	
Q	Discharge	cfs
R	Hydraulic radius, A/WP	ft.
S_f	Friction slope, $V^2/(C^2R^{4/3})$	ft./ft.
S_{fm}	Mean friction slope $\frac{S_{f1} + S_{f2}}{2}$	ft./ft.
S_o	Slope of channel	ft./ft.
V	Velocity of flow, Q/A	ft./sec.
$V^2/2g$	Velocity head	ft.
WP	Wetted perimeter	ft.
Y	Depth of flow	ft.

The Manning equation is sufficient to describe Uniform Flow. However, the Bernoulli equation (energy equation) must be used to describe Non-uniform Flow. Therefore, it is important to be able to recognize which type of flow condition prevails.

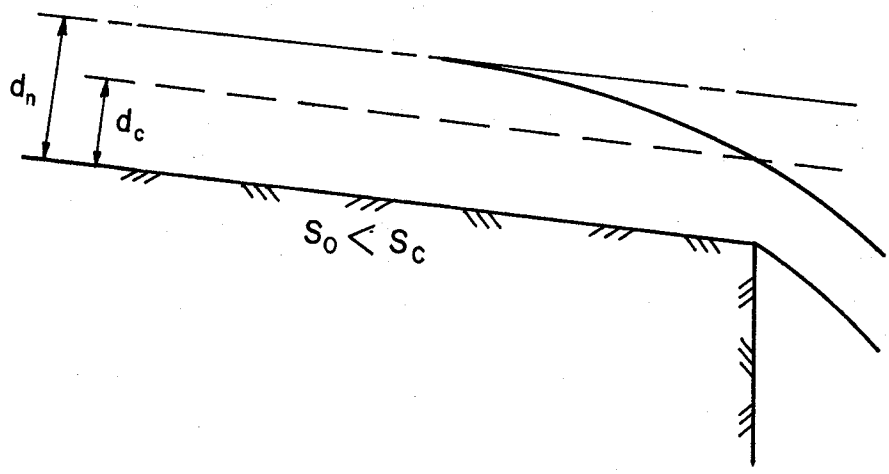
Uniform Flow occurs in long channel reaches of constant or nearly constant properties. Uniform Flow and the Manning equation were covered at the previous seminar and will not be repeated here.[2/]

When Non-uniform Flow occurs the designer must be able to recognize "controls," draw a generalized flow profile and be able to use the Bernoulli equation to describe the situation. "Controls" are channel cross sections for which the depth and velocity are known. The most frequently encountered controls are the occurrence of normal depth and critical depth. Fig. 2 provides examples of critical depth as a control sections. Supercritical flow is said to be subject to upstream control and subcritical flow is said to be subject to downstream control.

In order to determine the location of control sections and to sketch generalized flow profiles the critical depth and the normal depth of flow must be known for all sections of the channel reach under investigation.

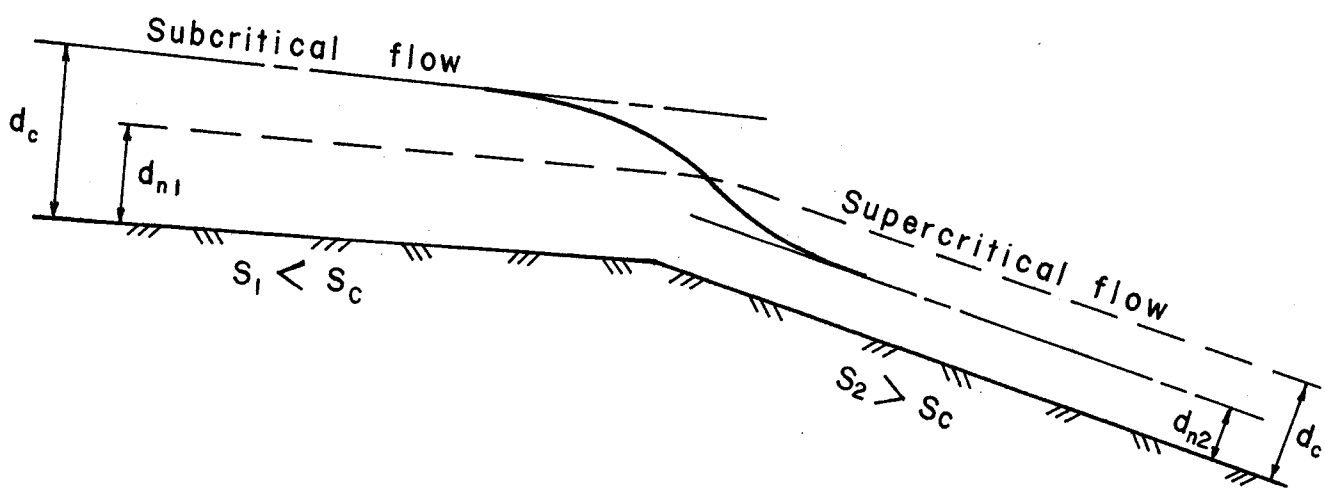
2/ See "Hydrologic and Hydraulic Training Session," May 1969

Free Outfall



For subcritical flow critical depth will occur at a free outfall.

Break in Slope from
Subcritical to Supercritical



Critical depth will occur at the break in Slope. Whenever the state of flow changes from subcritical to supercritical critical depth must occur.

Fig. 2 Critical Depth as a Control Section

CLASSIFICATION OF FLOW PROFILES

For a given discharge and channel condition the lines of normal depth and critical depth divide the flow area into three zones. These zones are shown in Figure 3.

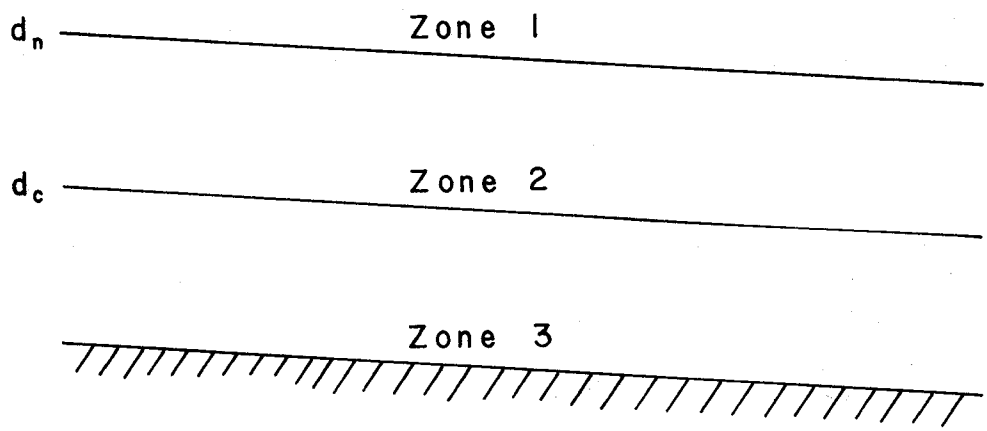


Figure 3: Zones of Channel (Longitudinal Section)

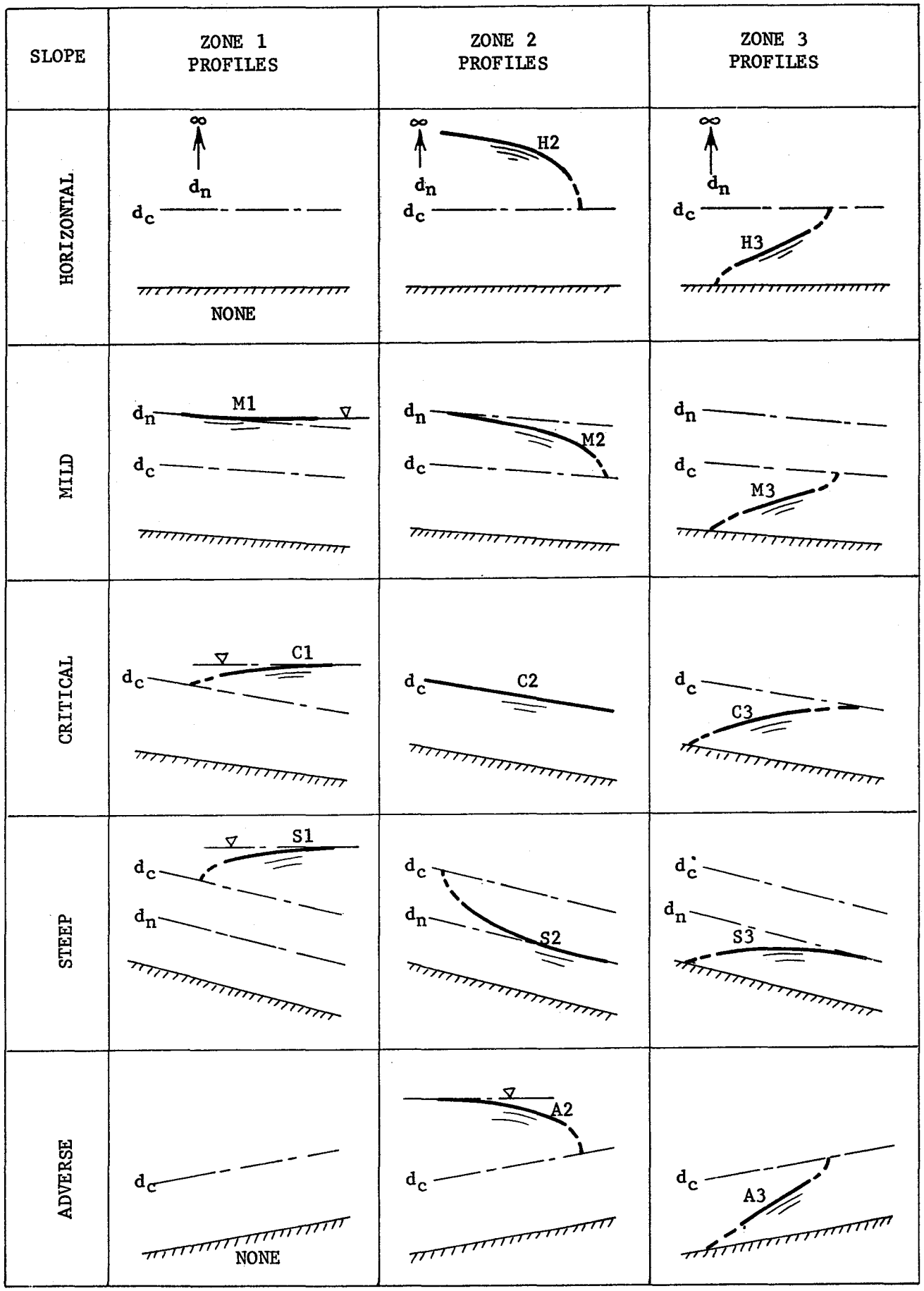
Flow profiles can be classified into thirteen different types according to the nature of the channel slope and the zone in which the flow surface is found. Table 2 summarizes these types. Notice that the designation of type utilizes a letter for the slope and a number for the zone. Except for the C2, the thirteen profiles are for non-uniform flow; C2 is for uniform flow. The shapes of these profiles are shown in Figure 4 and discussed further on pages I-8, I-9. Profiles near the critical depth and the channel bottom cannot be accurately defined, so they are shown with dashed lines.

Table 2 TYPES OF FLOW PROFILES IN PRISMATIC CHANNELS

CHANNEL SLOPE	ZONE	PROFILE TYPE	FLOW TYPE	GENERAL TYPE OF CURVE	RELATION OF d TO d_n AND d_c
Horizontal $s_o = 0$	1	None	None	None	$d > d_n > d_c$
	2	H2	SUBcritical	Drawdown	$d_n > d > d_c$
	3	H3	SUPERcritical	Backwater	$d_n > d_c > d$
Mild $s_c > s_o > 0$	1	M1	SUBcritical	Backwater	$d > d_n > d_c$
	2	M2	SUBcritical	Drawdown	$d_n > d > d_c$
	3	M3	SUPERcritical	Backwater	$d_n > d_c > d$
Critical $s_o = s_c > 0$	1	C1	SUBcritical	Backwater	$d > d_c = d_n$
	2	C2	Uniform-Critical	Parallel to Channel Bottom	$d_c = d = d_n$
	3	C3	SUPERcritical	Backwater	$d_c = d_n > d$
Steep $s_o > s_c > 0$	1	S1	SUBcritical	Backwater	$d > d_c > d_n$
	2	S2	SUPERcritical	Drawdown	$d_c > d > d_n$
	3	S3	SUPERcritical	Backwater	$d_c > d_n > d$
Adverse $0 > s_o$	1	None	None	None	$d > d_n^* > d_c$
	2	A2	SUBcritical	Drawdown	$d_n^* > d > d_c$
	3	A3	SUPERcritical	Backwater	$d_n^* > d_c > d$

* d_n assumed positive

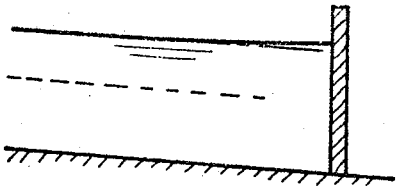
Figure 4 CLASSIFICATION OF FLOW PROFILES



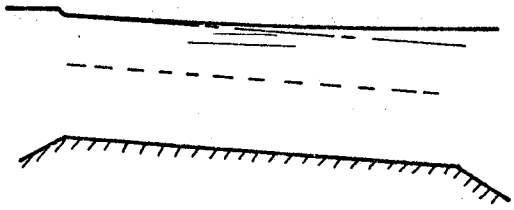
H PROFILES: These are the limiting cases of M profiles as the channel bottom becomes horizontal. Since d_n is infinite, no H1 profiles can be possible. The H2 and H3 profiles correspond to the M2 and M3 profiles.

M1: The M1 profile is the best known and most important of the profiles. Typical examples would be:

the profile behind a dam in a natural river, and

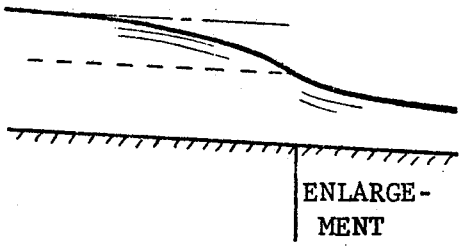


the profile in a canal joining two reservoirs.

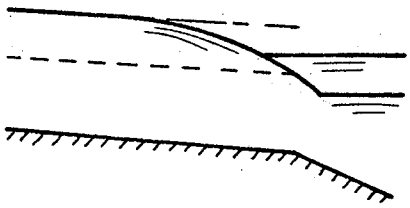


M2: This profile occurs when the lower end of the channel bottom is submerged in a reservoir to a depth less than the normal depth. Examples are:

the profile at the upstream end of a sudden enlargement of a canal cross section, and

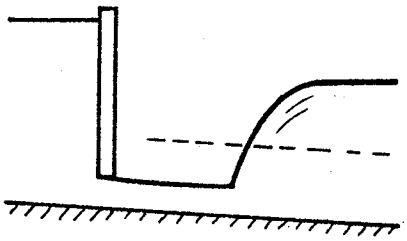


the profile of a canal leading to a reservoir.

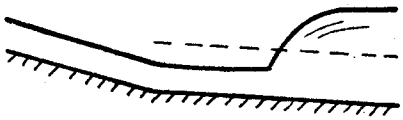


M3: The M3 profile theoretically starts from the upstream channel bottom and ends in a hydraulic jump at the downstream end. Examples are:

the profile of a stream below a sluice gate, and



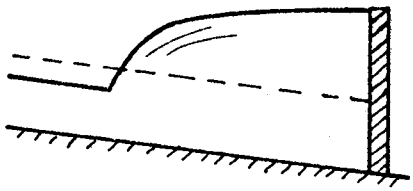
the profile after a change in bottom slope from steep to mild.



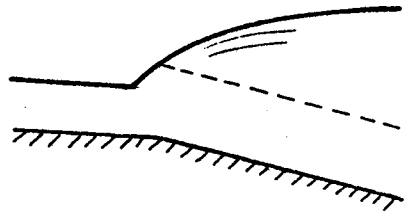
C PROFILES: These profiles are effectively the transition conditions between M and S profiles.

S1: This one begins with a jump and becomes tangent to the horinzontal pool level downstream. Examples are seen in:

the flow in a steep channel behind a dam, and

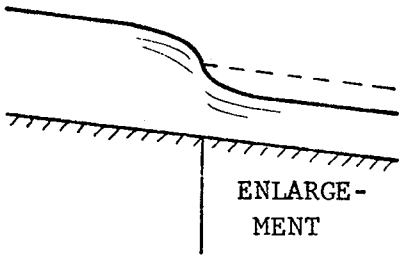


in a steep canal emptying into a pool of high elevation.

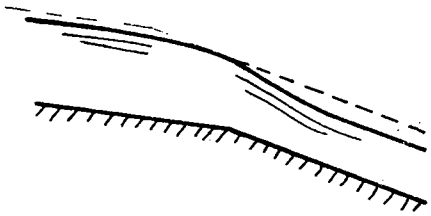


S2: This is a drawdown curve. Examples are:

the profile on the downstream end of a channel enlargement, and

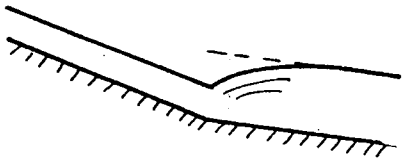


the profile on the steep side of a channel whose slope changes from steep to steeper.

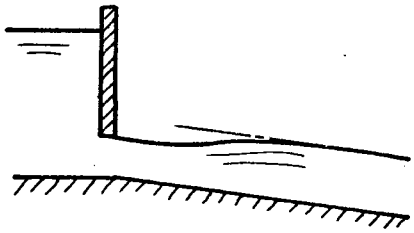


S3: This is a transitional type. Examples are:

the profile on the steep side of a channel whose slope changes from steep to milder steep, and



the profile below a sluice gate with the depth of the entering flow less than normal depth on a steep slope.



A PROFILES: The A1 profile is impossible since d_n is imaginary. The other two profiles rarely occur.

A more extensive treatment of this subject can be found in "Open Channel Hydraulics" by V. T. Chow. Once the type of flow profile is known, with the control sections located, the Bernoulli equation can be applied and the flow profile computed.

Water Surface Profiles by the Standard Step Method 3/

With the method as presented here it is assumed that the channel under consideration is of regular cross section (i.e., rectangular, trapezodial, or circular). The first point in the profile is determined by starting from a control point; additional points are determined by application of the Bernoulli equation.

Derivation of the equations

From Fig. 1 and the Bernoulli equation:

(1) $P_1/\gamma + Z_1 + Y_1 + V_1^2/2g = P_2/\gamma + Z_2 + Y_2 + V_2^2/2g + h_f$ + other losses

$Y_1 + V_1^2/2g = E_1$ Assume other losses $= h_e$

$Y_2 + V_2^2/2g = E_2$ $P_1/\gamma = P_2/\gamma$ = atmospheric pressure

$$Z_1 + E_1 = Z_2 + E_2 + h_f + h_e$$

$$(Z_1 + Z_2)/\Delta L = S_o$$

$$Z_1 - Z_2 = \Delta L S_o$$

$$\Delta L S_o - h_f = E_2 - E_1 - h_e$$

$$h_f/\Delta L = S_{fm}$$

$$h_f = \Delta L S_{fm}$$

where

$$h_f/\Delta L = \frac{29n^2}{R^{4/3}} \quad V^2/2g \quad \text{4/}$$

3/ "Open Channel Flow" by Henderson p. 126-130.

4/ BPR Hydraulic Engineering Circular No. 5, p. 5-6.

$$= \frac{V^2}{(1.486/n)^2 \; R^{4/3}}$$

$$= \frac{V^2}{C^2 \; R^{4/3}}$$

$$S_{fm} = \frac{S_{f1} + S_{f2}}{2}$$

$$\Delta L S_o - \Delta L S_{fm} = E_2 - E_1 - h_e$$

$$\Delta L (S_o - S_{fm}) = E_2 - (E_1 + h_e)$$

$$(2) \quad \Delta L = \frac{\Delta E}{S_o - S_{fm}}$$

All losses except those for friction (h_f) and expansion or contraction (h_e) are neglected. In a channel of constant cross section assume h_e is zero. For an expansion or contraction use the equation:

$$h_e = K(\Delta V^2/2g) \; \underline{5}/$$

and the following Table to compute h_e:

		Value of K
Expansion	Abrupt	.5
	Gradual	.2
Contraction	Abrupt	.3
	Gradual	.1

Table 3

5/ Note: h_e is always positive.

For flow which is everywhere deeper than critical depth (i. e. subcritical) one starts with the reach which is farthest downstream and computes the profile in the upstream direction. For flow which is everywhere less than critical depth (i. e. supercritical) one starts with the reach which is farthest upstream and computes the profile in the downstream direction.

The profile is determined by assuming successive changes in depth of flow in each of a number of small portions of the total length of channel under consideration. The length (ΔL) of these small portions or reaches is computed in turn using the equations previously derived. It should be noted that when the profile is run in the downstream direction ΔL is always positive. Similarly when the profile is run in the upstream direction, ΔL is always negative. The profile is ended when the summation of the calculated lengths ($\Sigma \Delta L$) is greater than the length of channel under consideration.

The reaches must be short enough so that the change in velocity head from one reach to the next is small. It is suggested that Fig. 5 be used when computing a water surface profile.

The example which follows provides a typical design problem for which a water surface profile would be desirable.

Note: Pages I-5 thru I-9 were adapted from " Manual Of Instruction, Part 4, Roadway Drainage ", Utah State Department of Highways, 1965.

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

HIGHWAY _____
LOCATION _____
PROJECT NO. _____

WATER SURFACE PROFILE COMPUTATION SHEET

CHANNEL TYPE _____

$n =$ _____, $C^2 = (1.486/n)^2 =$ _____, $Q =$ _____, $S_0 =$ _____, $b =$ _____, $d_c =$ _____, $d_n =$ _____.

Y	A	WP	R	$R^{4/3}$	V	$V^2/2g$	h_e	E	S_f $\frac{V^2}{(C^2R^{4/3})}$	S_{fm}	$S_0 - S_{fm}$	ΔE $E_2 - (E_1 - h_e)$	ΔL	$\Sigma\Delta L$

COMPUTED BY _____ CHECKED BY _____ DATE _____

Table 4 —*Four-thirds powers of numbers*

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0	0.0000	0.0022	0.0054	0.0093	0.0137	0.0184	0.0235	0.0288	0.0345	0.0403
0.1	.0464	.0527	.0592	.0659	.0727	.0797	.0869	.0942	.1016	.1092
0.2	.1170	.1248	.1328	.1409	.1491	.1575	.1659	.1745	.1832	.1920
0.3	.2008	.2098	.2189	.2280	.2373	.2467	.2561	.2656	.2752	.2849
0.4	.2947	.3046	.3145	.3246	.3347	.3448	.3551	.3654	.3758	.3863
0.5	.3969	.4075	.4182	.4289	.4397	.4506	.4616	.4726	.4837	.4948
0.6	.5061	.5173	.5287	.5401	.5515	.5631	.5746	.5863	.5980	.6097
0.7	.6215	.6334	.6453	.6573	.6693	.6814	.6936	.7058	.7180	.7303
0.8	.7427	.7551	.7675	.7800	.7926	.8052	.8178	.8305	.8433	.8561
0.9	.8689	.8818	.8948	.9078	.9208	.9339	.9470	.9602	.9734	.9867
1.0	1.000	1.013	1.027	1.040	1.054	1.067	1.081	1.094	1.108	1.122
1.1	1.136	1.149	1.163	1.177	1.191	1.205	1.219	1.233	1.247	1.261
1.2	1.275	1.289	1.304	1.318	1.332	1.347	1.361	1.375	1.390	1.404
1.3	1.419	1.433	1.448	1.463	1.477	1.492	1.507	1.522	1.536	1.551
1.4	1.566	1.581	1.596	1.611	1.626	1.641	1.656	1.671	1.687	1.702
1.5	1.717	1.732	1.748	1.763	1.778	1.794	1.809	1.825	1.840	1.856
1.6	1.871	1.887	1.903	1.918	1.934	1.950	1.966	1.981	1.997	2.013
1.7	2.029	2.045	2.061	2.077	2.093	2.109	2.125	2.141	2.157	2.173
1.8	2.190	2.206	2.222	2.238	2.255	2.271	2.287	2.304	2.320	2.337
1.9	2.353	2.370	2.386	2.403	2.420	2.436	2.453	2.470	2.486	2.503
2.0	2.520	2.537	2.553	2.570	2.587	2.604	2.621	2.638	2.655	2.672
2.1	2.689	2.706	2.723	2.741	2.758	2.775	2.792	2.809	2.827	2.844
2.2	2.861	2.879	2.896	2.913	2.931	2.948	2.966	2.983	3.001	3.018
2.3	3.036	3.054	3.071	3.089	3.107	3.124	3.142	3.160	3.178	3.195
2.4	3.213	3.231	3.249	3.267	3.285	3.303	3.321	3.339	3.357	3.375
2.5	3.393	3.411	3.429	3.447	3.466	3.484	3.502	3.520	3.539	3.557
2.6	3.575	3.594	3.612	3.630	3.649	3.667	3.686	3.704	3.723	3.741
2.7	3.760	3.778	3.797	3.815	3.834	3.853	3.871	3.890	3.909	3.928
2.8	3.946	3.965	3.984	4.003	4.022	4.041	4.060	4.079	4.098	4.117
2.9	4.136	4.155	4.174	4.193	4.212	4.231	4.250	4.269	4.288	4.308
3.0	4.327	4.346	4.365	4.385	4.404	4.423	4.443	4.462	4.481	4.501
3.1	4.520	4.540	4.559	4.579	4.598	4.618	4.637	4.657	4.676	4.696
3.2	4.716	4.735	4.755	4.775	4.794	4.814	4.834	4.854	4.873	4.893
3.3	4.913	4.933	4.953	4.973	4.993	5.013	5.033	5.053	5.073	5.093
3.4	5.113	5.133	5.153	5.173	5.193	5.213	5.233	5.253	5.274	5.294
3.5	5.314	5.334	5.355	5.375	5.395	5.415	5.436	5.456	5.477	5.497
3.6	5.517	5.538	5.558	5.579	5.599	5.620	5.640	5.661	5.682	5.702
3.7	5.723	5.743	5.764	5.785	5.805	5.826	5.847	5.868	5.888	5.909
3.8	5.930	5.951	5.972	5.992	6.013	6.034	6.055	6.076	6.097	6.118
3.9	6.139	6.160	6.181	6.202	6.223	6.244	6.265	6.286	6.307	6.328
4.0	6.350	6.371	6.392	6.413	6.434	6.456	6.477	6.498	6.519	6.541
4.1	6.562	6.583	6.605	6.626	6.648	6.669	6.690	6.712	6.733	6.755
4.2	6.776	6.798	6.819	6.841	6.863	6.884	6.906	6.927	6.949	6.971
4.3	6.992	7.014	7.036	7.057	7.079	7.101	7.123	7.145	7.166	7.188
4.4	7.210	7.232	7.254	7.276	7.298	7.319	7.341	7.363	7.385	7.407
4.5	7.429	7.451	7.473	7.495	7.518	7.540	7.562	7.584	7.606	7.628
4.6	7.650	7.672	7.695	7.717	7.739	7.761	7.784	7.806	7.828	7.850
4.7	7.873	7.895	7.918	7.940	7.962	7.985	8.007	8.030	8.052	8.074
4.8	8.097	8.119	8.142	8.164	8.187	8.210	8.232	8.255	8.277	8.300
4.9	8.323	8.345	8.368	8.391	8.413	8.436	8.459	8.482	8.504	8.527

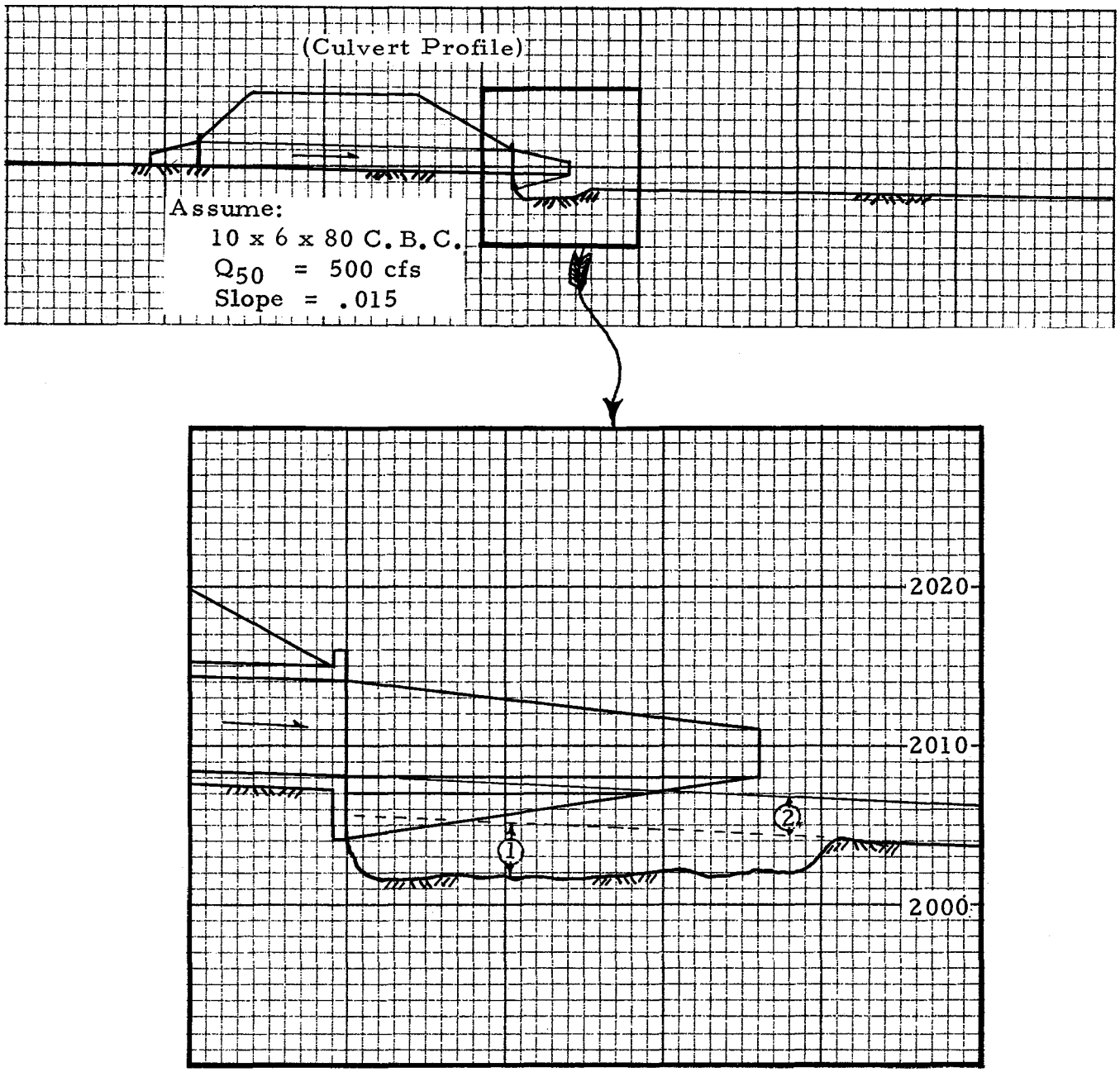
Table 4 .—*Four-thirds powers of numbers*—Continued

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
5.0	8.550	8.573	8.596	8.618	8.641	8.664	8.687	8.710	8.733	8.756
5.1	8.779	8.802	8.825	8.848	8.871	8.894	8.917	8.940	8.963	8.986
5.2	9.009	9.032	9.055	9.078	9.101	9.125	9.148	9.171	9.194	9.217
5.3	9.241	9.264	9.287	9.310	9.334	9.357	9.380	9.404	9.427	9.450
5.4	9.474	9.497	9.521	9.544	9.568	9.591	9.614	9.638	9.661	9.685
5.5	9.708	9.732	9.756	9.779	9.803	9.826	9.850	9.874	9.897	9.921
5.6	9.945	9.968	9.992	10.02	10.04	10.06	10.09	10.11	10.13	10.16
5.7	10.18	10.21	10.23	10.25	10.28	10.30	10.33	10.35	10.37	10.40
5.8	10.42	10.44	10.47	10.49	10.52	10.54	10.56	10.59	10.61	10.64
5.9	10.66	10.69	10.71	10.73	10.76	10.78	10.81	10.83	10.85	10.88
6.0	10.90	10.93	10.95	10.98	11.00	11.02	11.05	11.07	11.10	11.12
6.1	11.15	11.17	11.19	11.22	11.24	11.27	11.29	11.32	11.34	11.37
6.2	11.39	11.41	11.44	11.46	11.49	11.51	11.54	11.56	11.59	11.61
6.3	11.64	11.66	11.68	11.71	11.73	11.76	11.78	11.81	11.83	11.86
6.4	11.88	11.91	11.93	11.96	11.98	12.01	12.03	12.06	12.08	12.11
6.5	12.13	12.16	12.18	12.21	12.23	12.26	12.28	12.31	12.33	12.36
6.6	12.38	12.41	12.43	12.46	12.48	12.51	12.53	12.56	12.58	12.61
6.7	12.63	12.66	12.68	12.71	12.73	12.76	12.78	12.81	12.83	12.86
6.8	12.88	12.91	12.93	12.96	12.98	13.01	13.03	13.06	13.09	13.11
6.9	13.14	13.16	13.19	13.21	13.24	13.26	13.29	13.31	13.34	13.37
7.0	13.39	13.42	13.44	13.47	13.49	13.52	13.54	13.57	13.59	13.62
7.1	13.65	13.67	13.70	13.72	13.75	13.77	13.80	13.83	13.85	13.88
7.2	13.90	13.93	13.95	13.98	14.01	14.03	14.06	14.08	14.11	14.14
7.3	14.16	14.19	14.21	14.24	14.26	14.29	14.32	14.34	14.37	14.39
7.4	14.42	14.45	14.47	14.50	14.52	14.55	14.58	14.60	14.63	14.65
7.5	14.68	14.71	14.73	14.76	14.79	14.81	14.84	14.86	14.89	14.92
7.6	14.94	14.97	14.99	15.02	15.05	15.07	15.10	15.13	15.15	15.18
7.7	15.21	15.23	15.26	15.28	15.31	15.34	15.36	15.39	15.42	15.44
7.8	15.47	15.50	15.52	15.55	15.57	15.60	15.63	15.65	15.68	15.71
7.9	15.73	15.76	15.79	15.81	15.84	15.87	15.89	15.92	15.95	15.97
8.0	16.00	16.03	16.05	16.08	16.11	16.13	16.16	16.19	16.21	16.24
8.1	16.27	16.29	16.32	16.35	16.37	16.40	16.43	16.45	16.48	16.51
8.2	16.54	16.56	16.59	16.62	16.64	16.67	16.70	16.72	16.75	16.78
8.3	16.80	16.83	16.86	16.89	16.91	16.94	16.97	16.99	17.02	17.05
8.4	17.08	17.10	17.13	17.16	17.18	17.21	17.24	17.27	17.29	17.32
8.5	17.35	17.37	17.40	17.43	17.46	17.48	17.51	17.54	17.57	17.59
8.6	17.62	17.65	17.67	17.70	17.73	17.76	17.78	17.81	17.84	17.87
8.7	17.89	17.92	17.95	17.98	18.00	18.03	18.06	18.09	18.11	18.14
8.8	18.17	18.20	18.22	18.25	18.28	18.31	18.33	18.36	18.39	18.42
8.9	18.44	18.47	18.50	18.53	18.55	18.58	18.61	18.64	18.67	18.69
9.0	18.72	18.75	18.78	18.80	18.83	18.86	18.89	18.92	18.94	18.97
9.1	19.00	19.03	19.05	19.08	19.11	19.14	19.17	19.19	19.22	19.25
9.2	19.28	19.31	19.33	19.36	19.39	19.42	19.45	19.47	19.50	19.53
9.3	19.56	19.59	19.61	19.64	19.67	19.70	19.73	19.75	19.78	19.81
9.4	19.84	19.87	19.89	19.92	19.95	19.98	20.01	20.04	20.06	20.09
9.5	20.12	20.15	20.18	20.20	20.23	20.26	20.29	20.32	20.35	20.37
9.6	20.40	20.43	20.46	20.49	20.52	20.54	20.57	20.60	20.63	20.66
9.7	20.69	20.72	20.74	20.77	20.80	20.83	20.86	20.89	20.91	20.94
9.8	20.97	21.00	21.03	21.06	21.09	21.11	21.14	21.17	21.20	21.23
9.9	21.26	21.29	21.31	21.34	21.37	21.40	21.43	21.46	21.49	21.52

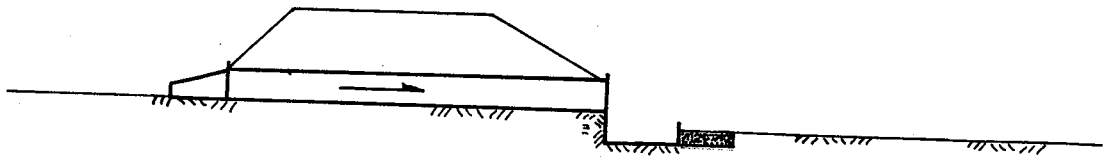
Example: Application of Water Surface Profile Computation

Assume it is your responsibility to provide the recommendations for the following hydraulic problem encountered on an existing primary highway:

Degradation② and Local scour① at the Outlet of an existing C. B. C.



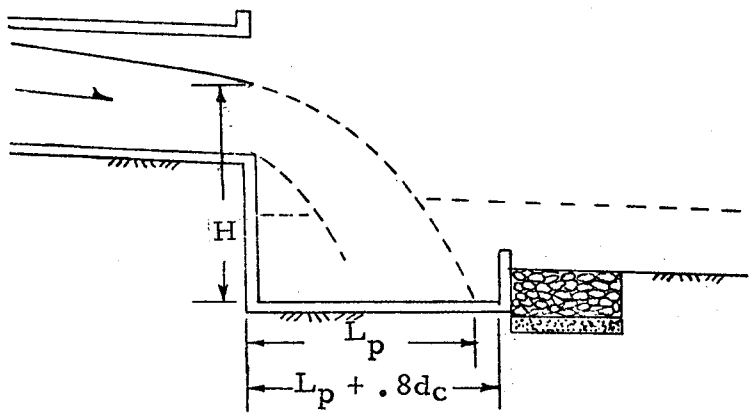
Possible solution to be analyzed in this example



Construct a concrete drop with an end-sill, provide rip-rap if needed.

Problem: Where should the endsill be located?

From p. 309 "Design of Small Dams" the endsill should be located a distance of $L_p + .8d_c$ from the beginning of the plunge; where L_p is defined as the length of the plunge.



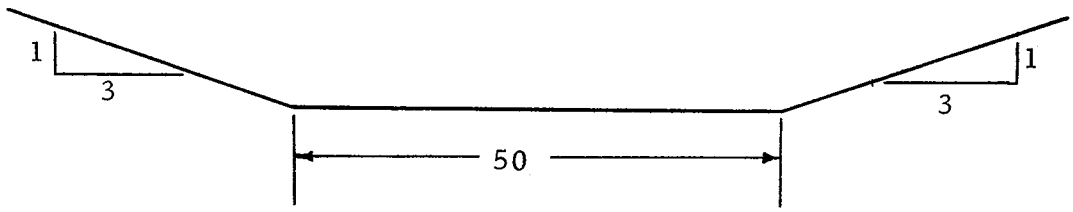
Min TW = 2.15 d_c

$$L_p = \sqrt{\frac{2V_o^2 H}{g}}$$ 6/

Therefore, the velocity and the depth of flow at the outlet of the C.B.C. is required. First determine normal depth and critical depth for all sections of the reach, locate control points, and draw a generalized flow profile.

Assume this cross section for the channel upstream:
Find D_n & D_c (Kings Handbook)

$S_o = .015$
$n = .035$
$Q = 500$ cfs.



6/ S.C.S. National Engineering Handbook, Sec. 5, p. 5.6-2, 1971

From page 7-38, & Table 7-11

$$Q = \frac{K'}{n} b^{8/3} S^{1/2}$$

$$K' = {}^{Qn}/_{b^{8/3} S^{1/2}} = {}^{(500)(.035)}/_{(50)^{8/3} (.015)^{1/2}}$$

$$K' = .00422$$

Interpolating in Table 7-11

$$D/b = .029$$
$$D = 1.45'$$

From page 8-55 & Table 8-5

$$Q = K'_c b^{5/2}$$

$$K'_c = {}^{Q}/_{b^{5/2}} = {}^{500}/_{(50)^{5/2}} = .0283$$

Interpolating in Table 8-5

$$D_c/b = .028$$

$$D_c = 1.4'$$

ARIZONA HIGHWAY DEPARTMENT

CULVERT COMPUTATION SHEET

PROJECT: As Built ****

STATION: *****

DESIGNER: R.C.J.

DATE: 9-28-72

HYDROLOGIC AND CHANNEL INFORMATION

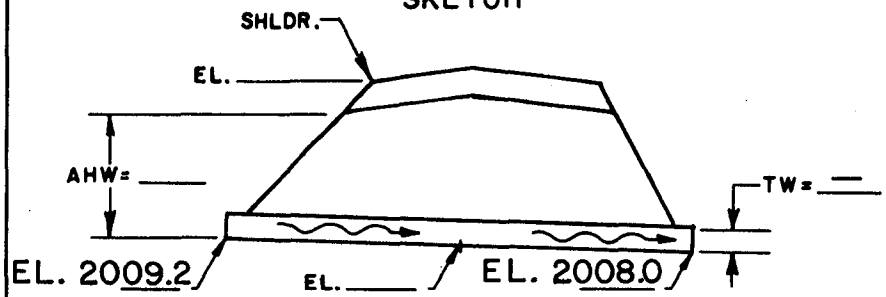
DRAINAGE AREA 1.2 mi.^{2}

Q_1 = ____ TW_1 = ____

Q_2 = 500 cfs TW_2 = ____

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

SKETCH



S_o = .015 L = 80 L/100 S_o = ____

MEAN STREAM VELOCITY = ____

CULVERT DESCRIPTION		Q	CAP. CHART HW	HEADWATER COMPUTATION										CONTROL HW	OUTLET VELOCITY	COST	DHW ELEV.	COMMENTS
				INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$												
SIZE	ENTR. TYPE			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c + D}{2}$	TW	h_o	LS_o	HW					
10x16	h/w with c w wings	500		1.22	7.3	.5	1.8	4.3	5.15	—	5.15	1.20	5.7	7.3				existing C.B.C.
									d_c = 4.3									
									d_n = 2.4									

SUMMARY & RECOMMENDATIONS

AHD 20-050 12-69

Normal Depth Calculation

Given: $Q = 500$ cfs
$n = .012$
$S_o = .015$
10 x 6 CBC

See Kings 5th ed. Table 7-19

$$K' = \frac{Qn}{b^{8/3}\, S^{1/2}} = \frac{500 \times .012}{10^{8/3}\,(.015)^{1/2}} = \frac{500 \times .012}{464 \times .1225}$$

$K' = .1056$

K'	$\frac{D}{d}$
.1061	.24
.1056	
.0997	.23

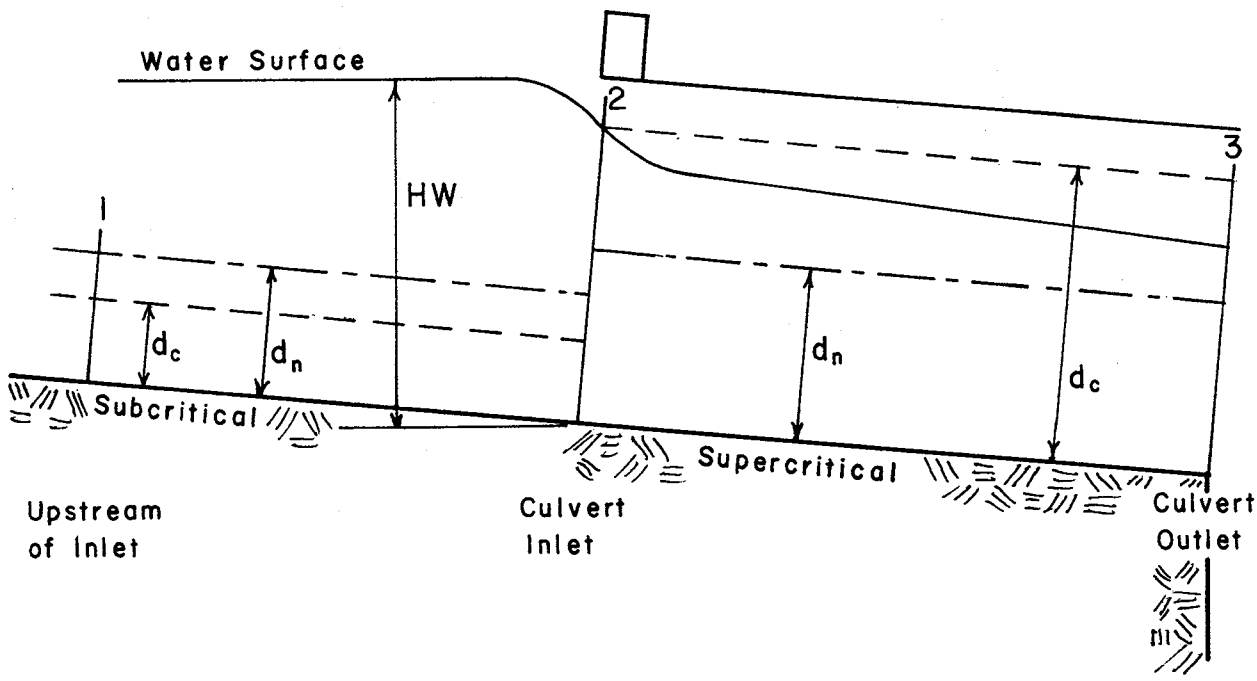
$$\frac{59}{64} = \frac{x}{.01} \qquad x = 0.009$$

$$\frac{D}{b} = 0.23 + 0.009 = 0.239$$

<u>D = 2.39 feet</u> Call 2.4 feet

Subcritical flow occurs upstream from the culvert inlet and supercritical flow occurs downstream from the inlet. Critical depth must occur when the state of flow changes from subcritical to supercritical (see Figure 2). Critical depth will be assumed at the culvert inlet. A water surface profile will be run downstream from the inlet to the culvert outlet by the Standard Step Method.

Generalized Flow Profile



ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

HIGHWAY As built ***
LOCATION *****
PROJECT NO. ****

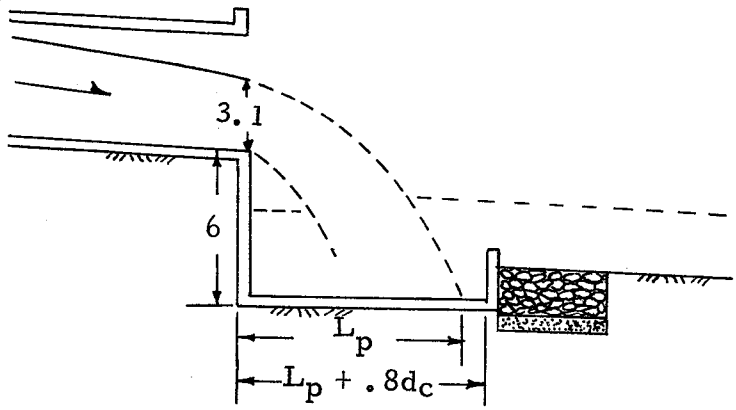
WATER SURFACE PROFILE COMPUTATION SHEET

CHANNEL TYPE rectangular, 10'x6' CBC

$n = .012$, $C^2 = (1.486/n)^2 = 15,335$, $Q = 500$, $S_0 = .015$, $b = 10$, $d_c = 4.27$, $d_n = 2.40$
$g = 32.2$

Y	A	WP	R	$R^{4/3}$	V	$V^2/2g$	h_e	E	S_f $\frac{V^2}{(C^2R^{4/3})}$	S_{fm}	$S_0 - S_{fm}$	ΔE $E_2 - (E_1 - h_e)$	ΔL	$\Sigma\Delta L$
4.27	42.7	18.54	2.30	3.036	11.7096	2.1291								
							0	6.3991	.0029	.0035	.0115	.0893	8	8
3.80	38.0	17.6	2.16	2.792	13.1579	2.6884								
							0	6.4884	.0040	.0048	.0102	.2697	26	34
3.40	34.0	16.8	2.02	2.553	14.7059	3.3581								
							0	6.7581	.0055	.0060	.0090	.2329	26	60
3.20	32.0	16.4	1.95	2.436	15.6250	3.7910								
							0	6.9910	.0065	.0072	.0078	.3223	41	101
3.00	30.0	16.0	1.88	2.320	16.6667	4.3133								
								7.3133	.0078					
3.10					16.14									
													@	80

COMPUTED BY RCJ CHECKED BY SB DATE 9-28-72



from water surface Profile $V_o = 16.1$ fps, $d_o = 3.1$ ft.

$\therefore H = 6 + 3.1 = 9.1$ ft.

$$L_p = \sqrt{\frac{2(16.1)^2 (9.1)}{32.2}} = 12.1 \text{ ft.}$$

$d_c = 4.3$ ft.

the distance to the endsill $= L_p + .8(d_c)$

$\boxed{15.5 \text{ ft.}}$

HYDROLOGIC AND HYDRAULIC DESIGN METHODS AND CRITERIA

AHD Hydrology Seminar

October 16 - 17, 1972

Phoenix, Arizona

I HYDROLOGIC DESIGN METHODS AND CRITERIA.
(To be incorporated in the future with AHD manual, "Hydrologic Design for Highway Drainage in Arizona.")

1. FREQUENCY:

   a. For Interstate, Primary, and Secondary storm drain design, the storm drain itself must convey a minimum Q_{10} event from the contributing watershed area.

   b. Median drainage structures shall be designed for the same design frequency as used for the cross culvert design.

2. PRECIPITATION:

   a. A minimum storm duration value to ten minutes shall be used with Figure 3-2 of "Hydrologic Design for Highway Drainage in Arizona."

   b. The following equations can be substituted for Figure 2-2 in the manual "Hydrologic Design for Highway Drainage in Arizona" for the determination of rainfall depths at other than 6 hour and 24 hour durations:

$$P_5 \text{ minutes} = .58\ P_6 - .29\ P_{24}$$

$$P_{10} \text{ minutes} = .90\ P_6 - .45\ P_{24}$$

$$P_{15} \text{ minutes} = 1.14\ P_6 - .57\ P_{24}$$

$$P_{30} \text{ minutes} = 1.58\ P_6 - .79\ P_{24}$$

$$P_1 \text{ Hour} = 2\ P_6 - P_{24}$$

$$P_2 \text{ Hour} = 1.77\ P_6 - .77\ P_{24}$$

$$P_3 \text{ Hour} = 1.55\ P_6 - .55\ P_{24}$$

$$P_{12} \text{ Hour} = .49\ P_{24} + .51\ P_6$$

3. DRAINAGE AREA:

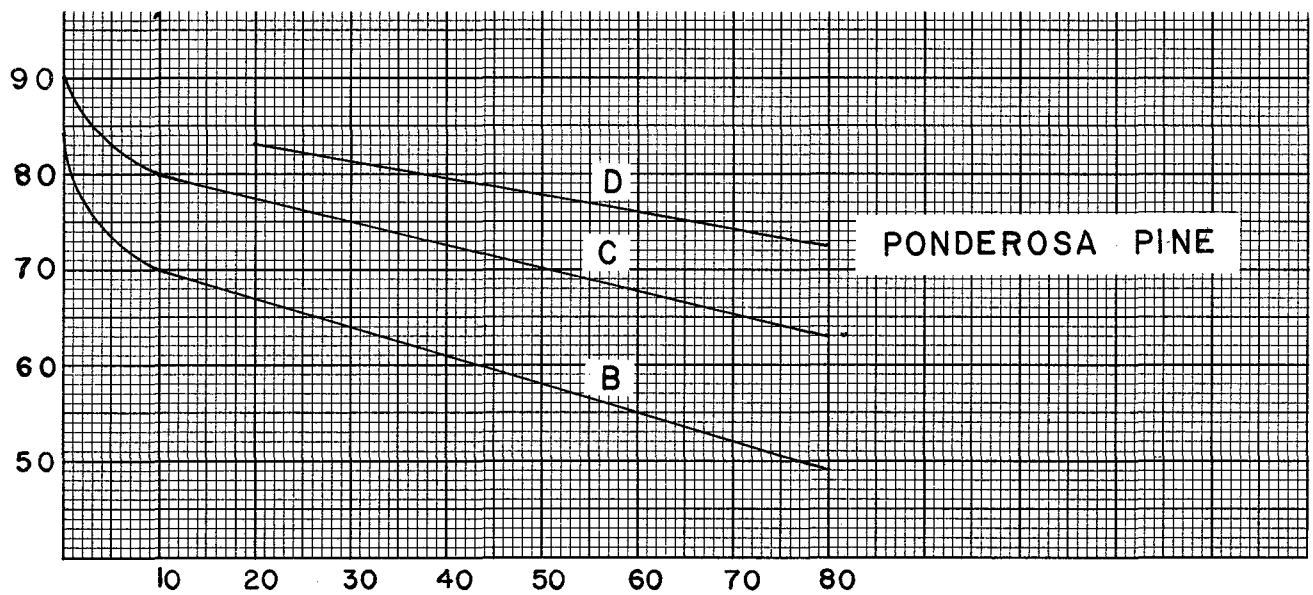
   a. Whenever the DHW value of a drainage structure exceeds the elevation of the drainage area boundary of the adjacent drainage structure, the two drainage areas will be analyzed for possible

combination into one hydrologic unit. A common DHW value will be determined for both structures.

4. VEGETATION:

a. A maximum cover density of 30% will be used for desert brush.

b. An additional hydrologic soil cover complex and associated curve number line for a D soil for Ponderosa Pine is shown below for Figure 2-3 of "Hydrologic Design for Highway Drainage in Arizona."



Cover Density-Percent

Fig. 2-3

HYDROLOGIC SOIL - COVER COMPLEXES AND ASSOCIATED CURVE NUMBERS

5. CURVE NUMBERS

a. A minimum curve number of 70 will be used for all SCS Part I and Part II Calculations for Runoff.

b. Runoff curve numbers for hydrologic soil - cover complexes

Land use	Cover: Treatment or practice	Cover: Hydrologic condition 2/	Hydrologic soil group B	C	D
Fallow	Straight row	----	86	91	94
Row crops	"	Poor	81	88	91
	"	Good	78	85	89
	Contoured	Poor	79	84	88
	"	Good	75	82	86
	"and terraced	Poor	74	80	82
	" " "	Good	71	78	81
Small grain	Straight row	Poor	76	84	88
		Good	75	83	87
	Contoured	Poor	74	82	85
		Good	73	81	84
	"and terraced	Poor	72	79	82
		Good	70	78	81
Close-seeded legumes 1/ or rotation meadow	Straight row	Poor	77	85	89
	" "	Good	72	81	85
	Contoured	Poor	75	83	85
	"	Good	70	78	83
	"and terraced	Poor	70	80	83
	"and terraced	Good	70	76	80
Pasture or range		Poor	70	86	89
		Fair	70	79	84
		Good	70	74	80
	Contoured	Poor	70	81	88
	"	Fair	70	75	83
	"	Good	70	70	79

1/ Close-Drilled Broadcast

2/

Vegetative Condition	Hydrologic Condition
Heavily grazed, has no mulch or has plant cover on less than 1/2 of area	Poor
Not heavily grazed, plant cover on 1/2 to 3/4 of area	Fair
Lightly grazed, plant cover on more than 3/4 of area	Good

6. RUNOFF:

a. No computations shall have a runoff value that is less than 10% of the precipitation value. Note: Use either (1) the amount of runoff obtained for this item, or (2) the runoff obtained from a minimum 70 curve number; whichever is greater.

b. The coefficient "c" used in the Rational Method shall conform approximately to the ratio of the one hour design runoff value to the one hour design precipitation value used in SCS Part I Hydrologic Design. (Example: CN = 90; $P_{(1\ hr.)}$ = 2.0 ins.; Q_1 = 1.1 ins.; C = 1.1/2.0 = .55)

c. The effect of storage may be considered whenever the volume calculated at the AHW elevation is 20% of the volume of the design runoff or greater. (Example: See flood routing examples.)

7. TIME OF CONCENTRATION:

a. A minimum time of concentration value of ten minutes shall be used for all peak discharge calculations.

b. No watershed velocity less than 1.0 ft./sec. shall be used for all time of concentration determinations.

c. For "long and narrow" drainage areas, see chart on Page J4a

II <u>HYDRAULIC DESIGN METHODS AND CRITERIA</u>.
(To be incorporated in the future with AHD manual, "Hydrologic Design for Highway Drainage in Arizona.")

1. OPEN CHANNELS:

a. Freeboard for improved channels:

.25 x depth for supercritical flow

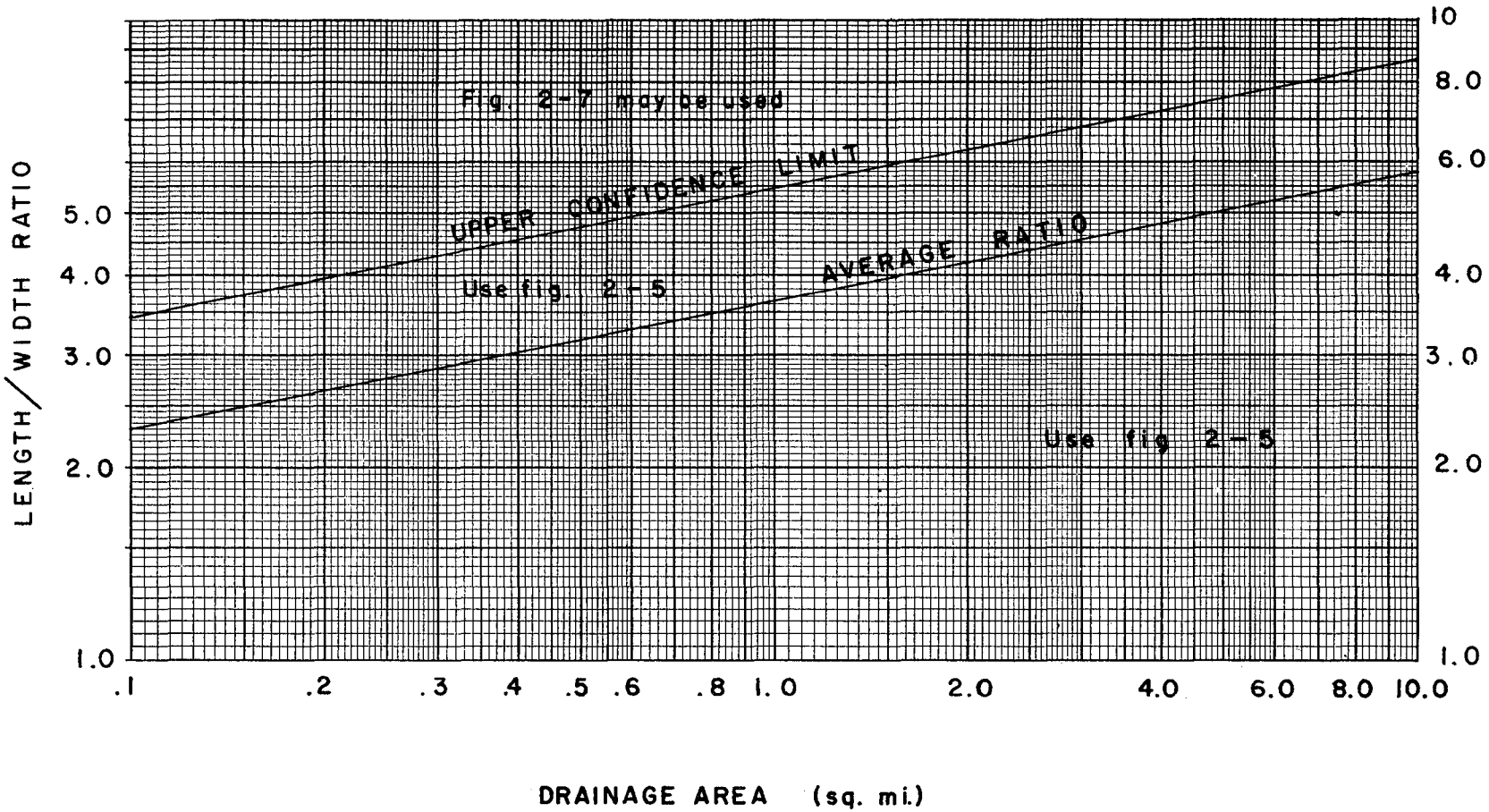
.20 x (depth + $V^2/2g$) for subcritical flow

But not less than 1.0 feet.

b. Channel changes:

All channel changes must be analyzed for an equivalent area and flow depth which should compare closely to the natural channel it is replacing.

DETERMINATION OF LONG AND NARROW DRAINAGE AREAS*



* Adapted From S. C. S. Engineering Field Manual for Conservation Practice, 1969, p. 2-5

2. CROSS CULVERTS:

a. DHW elevation shall not exceed the elevation of the top of subgrade.

b. Improved inlets as denoted in Hydraulic Circular No. 13 may be analyzed for unusually long and/or large structures which are operating under inlet control. Cost comparisons must be made to justify the selection of an improved inlet design.

c. Median drainage structures that connect to cross drainage structures shall be designed so that water will not back into the median.

d. Cattle guard structures that are utilized for conveying drainage will require hydrologic and hydraulic calculations.

e. All culverts having invert elevations more than six inches below the natural stream profile shall be assumed to have only the waterway opening above the stream bed profile for hydraulic capacity calculations.

f. All structures used for combination cattle pass and drainage will require hydrologic and hydraulic calculations.

g. All structures under existing roadways should be revised if prior knowledge indicates the existing drainage opening is inadequate (water over the roadway or upstream property damage from backwater).

h. For hydraulic design of structures under a new roadway that is adjacent to an existing highway, the new structure shall be equal or larger in hydraulic capacity than the existing structure.

i. When cost comparisons are made between different types of drainage structures, the same approximate DHW value shall be used for each installation.

3. STORM DRAINS:

a. At changes in size of pipes, crown elevations should be the same.

b. An adequate outlet must be analyzed and provided for all storm drain installations.

4. OUTLET PROTECTION:

a. All drainage structures that have a contributing drainage area of less than 0.1 square miles in range or desert areas will not require outlet protection unless specifically requested at the field review.

b. Outlet protection requirements should be determined for structures with drainage areas greater than .1 square mile. The following guidelines are suggested for determining the type of outlet protection required:

Ratio of Outlet Velocity to Natural Stream Velocity	Outlet Protection
1.0 to 1.5	No protection required
1.5 to 2.0 with outlet velocity less than 10 f.p.s.	Dumped rock riprap
1.5 to 2.5 with outlet velocity greater than 10 f.p.s.	Wire tied rock riprap
Greater than 2.5	Concrete energy dissipator

For ratios of outlet velocity to natural stream velocity greater than 2.5 consideration should be given to increasing the proposed structure size in lieu of providing a concrete energy dissipator.

c. 140 lbs./cu. ft. will be the rock density value assumed for use in the design of graded riprap.

5. DRAINAGE EASEMENTS:

a. The hydraulic report summary sheet must contain a column stating whether ponding occurs beyond the right of way. A ponding area map delineating the extent of ponding beyond the right of way will be required.

b. Drainage easements should be considered at all drainage outlets where flows are concentrated.

6. HYDRAULIC REPORT AND CONSTRUCTION PLANS REQUIREMENTS:

a. Ponding beyond right of way. (See attached memo of April 10, 1972.)

b. Signature of checker. (See attached memo of April 10, 1972.)

c. Significant figures for hydraulic values. (See attached memo of June, 1972.)

d. Stream cross sections and profiles. (See attached memo of November 1, 1971.)

e. Predesign Conference. (See attached memo of April 10, 1972.)

f. Flooded width of pavement. (See attached memo of August 24, 1970.)

g. It is the responsibility of the roadway designer to verify the information on the hydraulic survey report form. In all cases, this will require a field inspection by the designer.

h. The designer should determine the adequacy, condition, and past performance of all existing culverts by visual inspection and checking with the maintenance personnel or local residents.

i. For range areas upstream and downstream photographs of all drainage areas greater than 0.1 square milesshould be taken for inclusion in the hydraulic report. Additional photographs may be necessary in developed areas.

j. All drainage alternates including concepts and structural types must be analyzed and a recommendation made prior to the field review.

ARIZONA HIGHWAY DEPARTMENT

Phoenix, Arizona

April 10, 1972

ENGINEERING CONSULTANTS DIVISION
INFORMATIONAL BULLETIN NO. 44

RE: HYDRAULIC STUDY

The following procedures regarding hydraulic study are now in effect:

1. Hydraulic design work for new contracts and change orders will be authorized to begin only after a predesign conference is held. This conference will be scheduled through the Engineering Consultants Division at a time when the consultant's engineer, in charge of design and preparation of the hydraulic study, can be present.

2. Item #7 of the memo dated August 4, 1969 is amended to include the following:

n. Ponding beyond right of way.

3. Structure summary sheets submitted after May 1, 1972 should indicate "yes" or "no" for Item "n" on each pipe or box culvert listed. If ponding occurs beyond the right of way line, a map showing the ponding in relation to the drainage structure(s) should be submitted with the hydraulic report.

4. Hydraulic reports should be reviewed, checked and signed by a qualified designer other than the person who prepared the report.

Foster B. Allen

Foster B. Allen
Coordinator
Engineering Consultants

FBA:kg

June, 1972

ARIZONA HIGHWAY DEPARTMENT
Structures Section
Hydraulics Branch

GUIDELINES FOR DETERMINING SIGNIFICANT FIGURES FOR HYDRAULIC DATA ON CONSTRUCTION PLANS

DRAINAGE AREAS (Square Miles):

Use three significant figures for values 0.1 square miles and greater.
Use two significant figures for values from .011 square mile to .099 square mile.
Use one significant figure for values of .001 square mile to .009 square mile.

Examples: Calculated Values

3251	Use	3250
251.6	Use	252
52.75	Use	52.8
52.01	Use	52.0
2.754	Use	2.75
.7566	Use	.757
.07566	Use	.076
.0704	Use	.070
.00756	Use	.008

DISCHARGES (Cross Drainage Structures):

Use three significant figures for values greater than 99 cfs.
Use two significant figures for values from 10 to 99 cfs.
Use one significant figure for values from 1 to 9 cfs.

DISCHARGES (Pavement Drainage):

Use three significant figures for values from 10.0 to 99.9 cfs.
Use two significant figures for values from 1.0 to 9.9 cfs.
Use one significant figure for values from .1 to .9 cfs.

Examples: Calculated Values

(Cross Drainage Structures)

10,430	Use	10,400
3,251	Use	3,250
251.6	Use	252
52.75	Use	53
2.754	Use	3

(Pavement Drainage)

65.66	Use	65.7
7.62	Use	7.6
.77	Use	0.8

SLOPE (Cross Drainage Structures and Channel Reaches Less than 1,000 Feet in Length):

Use three significant figures for values greater than .1%.
Use two significant figures for values from .01 to .1%.
Use one significant figure for values from .001 to .01%.

SLOPE (Channel Reaches 1,000 to 10,000 Feet in Length):

Use four significant figures for values greater than .1%.
Use three significant figures for values from .01 to .1%.
Use two significant figures for values from .001 to .01%.
Use one significant figure for values from .0001 to .001%.

SLOPE (Channel Reaches Greater Than 10,000 Feet in Length):

Use five significant figures for values greater than .1%.
Use four significant figures for values from .01 to .1%.
Use three significant figures for values from .001 to .01%.
Use two significant figures for values from .0001 to .001%.
Use one significant figure for values from .00001 to .0001%.

Examples: Calculated Values

(Cross Drainage Structures and Channel Reaches Less Than 1,000 Feet in Length):

.1251	Use	12.5%
.02624	Use	2.62%
.003622	Use	.362%
.0004621	Use	.046%
.00001211	Use	.001%

(Channel Reaches 1,000 Feet to 10,000 Feet in Length):

.12511	Use	12.51%
.054321	Use	5.432%
.0072463	Use	.7246%
.00065483	Use	.0655%
.00001234	Use	.0012%

(Channel Reaches Greater Than 10,000 Feet in Length):

.236954	Use	23.695%
.0423697	Use	4.2370%
.00354218	Use	.35422%
.00096527	Use	.09653%
.0000123753	Use	.00124%

DESIGN HEADWATER (DHW):

Use closest 0.1 foot.

Example: Calculated Value

3251.26	Use	3251.3

CENTERLINE INVERT ELEVATIONS:

Use closest 0.01 foot.

Examples: Calculated Values

2526.321	Use	2526.32
4126.1	Use	4126.10

ARIZONA HIGHWAY DEPARTMENT

OFFICE MEMO

November 1, 1971

TO: FOREST JENNINGS
Assistant State Engineer - Location

FROM: R. C. BRECHLER
Bridge Engineer - Design

SUBJECT: Survey Requirements - Hydraulics
Stream Cross Sections and Profiles

We are attaching a copy of the August 4, 1969 memorandum from Engineering Consultants Division to Consulting Engineers which contains a resume of the information required for hydraulic reports.

The requirements concerning stream or channel cross sections and profiles listed under Item 8d of the memo should be amended to include the following additional sentence:

Stream cross sections and profiles will generally not be required for drainage areas of less than 0.1 square mile for range land and undeveloped areas within publicly owned lands.

R. C. Brechler

RCB/jw

Attachment

bcc: M. Sheldon

ARIZONA HIGHWAY DEPARTMENT

OFFICE MEMO

August 24, 1970

TO: MARTIN TONEY
Engineer of Bridges & Dams

FROM: JAMES P. OXLEY
Assistant Engineer of Plans

SUBJECT: Allowable Flooded Width used in the Design of Curbed Roadway Drainage

The Hydraulics Section has requested issuance of a memo providing criteria for allowable water spread on pavement as related to catch basin or inlet spacing determination for curbed roadways.

Below is a recommended tabulation of the requested information:

Roadway Type	Max. Water Surface Width	*Storm Frequency
Rural 4-lane divided	Lt. -Shoulder width Rt. -Shoulder width plus 1/2 adjacent traffic lane width.	10-years
Urban 4 or 6-lane divided by raised median	Lt. -Gutter width plus any shoulder width. Rt. -Shoulder, parking or distress lane width plus 1/2 adjacent traffic lane width.	10-years
4 or 6-lane undivided	Shoulder, parking or distress lane width plus 1/2 adjacent traffic lane width.	10-years
2-lane undivided	Gutter and/or shoulder, parking or distress lane width.	10-years

RECEIVED
AUG 25 1970
BRIDGE DIVISION

22' or 24' ramps (Inc'l. accel. & decel. lane)	Lt. -2' Rt. -8'	10-years

*50-years for underpasses or other depressed roadways where ponded water can be removed only through a storm drain system.

James P. Oxley

JAMES P. OXLEY
Assistant Engineer of Plans

JPO:DG:sd

ARIZONA HIGHWAY DEPARTMENT

OFFICE MEMO

February 1, 1972

TO: F. B. ALLEN
Coordinator
Engineering Consultants

FROM: R. C. BRECHLER
Bridge Engineer - Design

SUBJECT: Catch Basin Design
Effective Areas

Effective February 15, hydraulic reports for all projects that have not been submitted for Highway Department review must use the following factors to increase the computed clear areas or perimeters of catch basins to allow for possible clogging. (Actual catch basin capacities will be computed on the basis of required area or perimeter.)

Grates:

1. Sump Conditions:
   a. Orifice Flow: Actual Area = 2.0 x required area
   b. Weir Flow: Actual Perimeter = 2.0 x required perimeter

2. Continuous grade conditions:
   a. Actual length of opening = 2.0 x required length or greater

Curb Opening Inlets:

1. Sump Conditions:
   a. Actual length of opening = 1.25 x required length or greater

2. Continuous grade conditions:
   a. Actual length of opening = 1.25 x required length or greater

Combination Grate and Curb Opening:

1. Sump Conditions:
   a. Orifice Flow: Actual area = 1.0 x required area of grate only
   b. Weir flow: Actual perimeter = 1.0 x required perimeter of grate only

2. Continous grade conditions:
   a. Actual length of opening = 1.0 x required length of grate only

F. B. Allen February 1, 1972

AHD Standard C-15.05 Catch Basin:

1. Continuous Grade Conditions:
   a. Actual curb opening length upstream from catch basin = 1.25 x required length

   b. Actual length of grate = 1.0 x required length of grate only

R C Brechler

R. C. Brechler

RCB/sm

Technical Release No. 25, Planning and Design of Open Channels. December 15, 1964.

U.S. Geological Survey

Roughness Coefficients for Stream Channels in Arizona June, 1972.

Highway Research Board

Report 108 - Tentative Design Procedure for Riprap Lined Channels. 1970.

Report 373 - Design of Culverts, Energy Dissipators, and Filter Systems. 1971.

Arizona Highway Department

Rainfall-Frequency Maps for Arizona, 6 and 24 Hour Durations, March, 1967 (Revised 1970).

Highway Design and Legal Liability for Drainage Changes.. August 5, 1971.

Other

Lower Colorado Region Vegetal Cover Map.
Main Report - Lower Colorado Region Comprehensive Framework Study. November, 1970.

Bank and Shore Protection in California Highway Practice. California Division of Highways.

ADDENDUM TO
HYDROLOGIC DESIGN FOR HIGHWAY DRAINAGE
IN ARIZONA - DECEMBER 1968

REFERENCES

Bureau of Public Roads Publications

Hydraulic Engineering Circulars
HDS No. 1 Hydraulics of Bridge Waterways
Second edition, 1970.
HEC No. 12 Drainage of Highway Pavements, March, 1969.
HEC No. 13 Hydraulic Design of Improved Inlets for Culverts
November, 1971.

Soil Conservation Service

Engineering Design Standards, Far West States
October, 1970.

Hydrologic Soil Group Maps for Apache, Coconino, Maricopa, Mohave, Navajo, Pinal, Santa Cruz, and Yavapai Counties.
1971.

National Engineering Handbook
Section 5: Hydraulics
9: Retarding and Gradient Control Systems
11: Drop Spillways
14: Chute Spillways
16: Box Inlet Drop Spillways
TR 15: Computation of Water Surface Profiles and Related Parameters.
TR 16: Rainfall-runoff Tables for Selected Runoff Curve Numbers.

JUL 23 1974 JORDAN

RIPRAP PROTECTION DESIGN PROCEDURE FOR OPEN CHANNELS

When the erosive forces of moving water are too great for an unlined earth channel, a protective lining is required. This procedure relates only to riprap protection. The design discharges, slope, size, shape and roughness of the channel are the hydraulic parameters considered in riprap design. The following design procedure, based on tractive force, is condensed from the Highway Research Board No. 108 Bulletin entitled "Tentative Procedure for Riprap-Lined Channels".

Erosion occurs when the dynamic forces of water acting on the channel boundary exceed the resistive forces of the channel material. The unit shear created on the channel boundary by the flow is expressed as

$$\tau = 62.4\ CRS$$

where τ is the boundary unit shear (lbs./sq. ft.), R is the hydraulic radius, (ft.) and S is the slope of the channel (ft./ft.). The value of C, as determined by Figure 1, depends on the channel shape and velocity distribution.

The shear force the riprap can resist, τ_R, (lbs/sq. ft.) is the maximum shear at which there is no movement of the riprap particles. This is expressed as

$$\tau_R = K4D_{50}$$

where D_{50} (ft.) is the mean stone size. (50% of the stone is larger and 50% is smaller). The factor K reduces τ_R due to the riprap resting on a side slope. This factor is given in Figure 2. The designer should know the angle of repose of the riprap material. If this is not possible the use of the "very rounded" curve is recommended.

The selected riprap must be of sufficient stone size so that the resistive shear, τ_R, is greater than the boundary shear force, τ. Combining the equations for τ and τ_R, D_{50} can be solved for as follows:

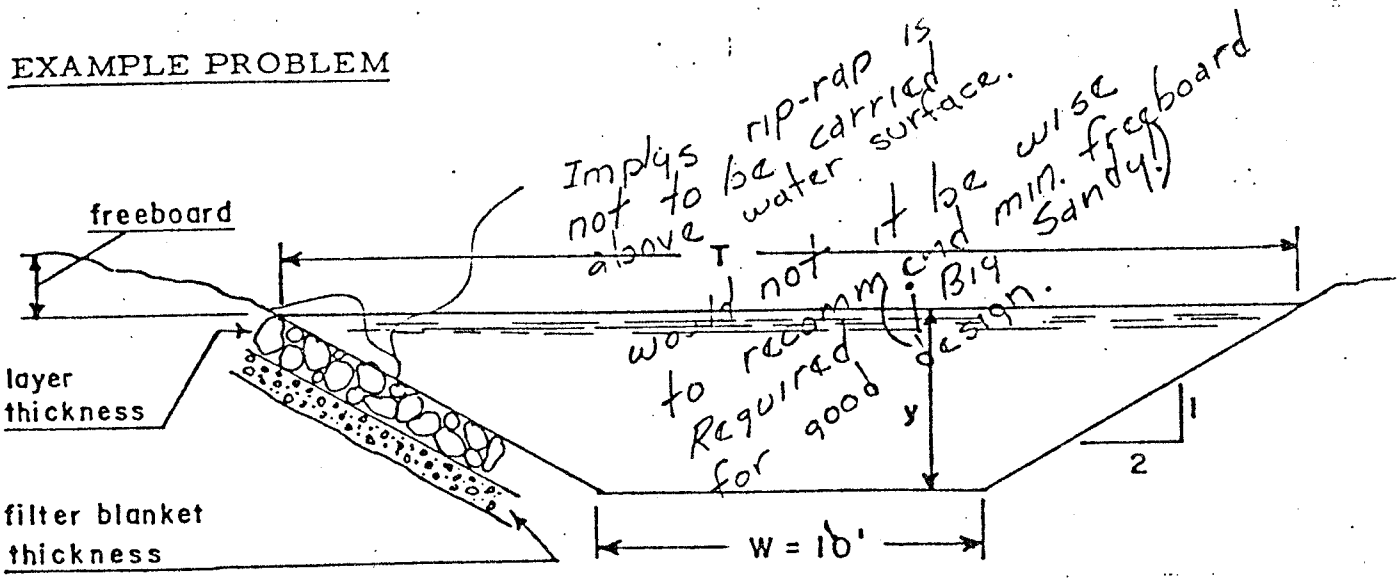
$$D_{50} \geq \frac{62.4\ CRS}{4K} \quad \text{or} \quad \frac{15.6\ CRS}{K}$$

The total thickness of the riprap layer is recommended to be at least $1\frac{1}{2}$ times the D_{50} size but not less than D_{max}.

Frequently a filter blanket is required to prevent leaching of the finer material underlying the riprap. Hydraulic Engineering Circular No. 11 may be used to design a filter blanket or to determine if one is required.

This method of riprap design applies to rock of specific weights ranging from 132 to 165+ pounds per cubic foot. Rock of specific weight lighter than 132 pounds per cubic foot are not recommended.

EXAMPLE PROBLEM



Given: Q=400 cfs S=0.010 ft./ft. Mannings "n"=0.035

Mean Bend Radius = 50 ft.

Z = 2 side slope

Riprap source is alluvial rock with a specific weight of 140 lb./cu. ft. (This value is within the 132-165 lbs./cu. ft. range; therefore, the equation $D_{50} \geq \frac{15.6\ CRS}{K}$ can be used).

Problem: Find the size of riprap and layer thickness required to stabilize the channel both in straight and curved reaches.

SOLUTION

By Manning's equation, the normal depth, y, is calculated to be 3.3 ft.

The other dimensions are:

V = 7.3 fps

R = 2.2 ft.
T = 23 ft.
W/y = 10/3.3 = 3.0
T-Radius = 23/50 = 0.46

From Figure 1 $C_s = 1.15$ for sides
$C_b = 1.42$ for bottom
$C_c = 1.7$ for curve

From Figure 2 $K = .64$ for $Z = 2$ and very rounded rock
$K = 1.0$ for level bottom

$$D_{50} \text{ for sides on a curve} = \frac{15.6\, C_c RS}{K} = \frac{15.6(1.7)2.2(.01)}{(.64)} = .91'$$

Say $D_{50} = 11''$

Required layer thickness $= 1\frac{1}{2} \times 11'' = 16.5''$ Say $18''$

$$D_{50} \text{ for sides in a straight reach} = \frac{15.6\, C_s RS}{K} = \frac{15.6(1.15)2.2(.01)}{(.64)} = .62'$$

Say $D_{50} = 8''$

Required layer thickness $= 1\frac{1}{2} \times 8'' = 12''$

$$D_{50} \text{ for channel bottom} = \frac{15.6\, C_b RS}{K} = \frac{15.6(1.42)2.2(.01)}{(1.0)} = .49'$$

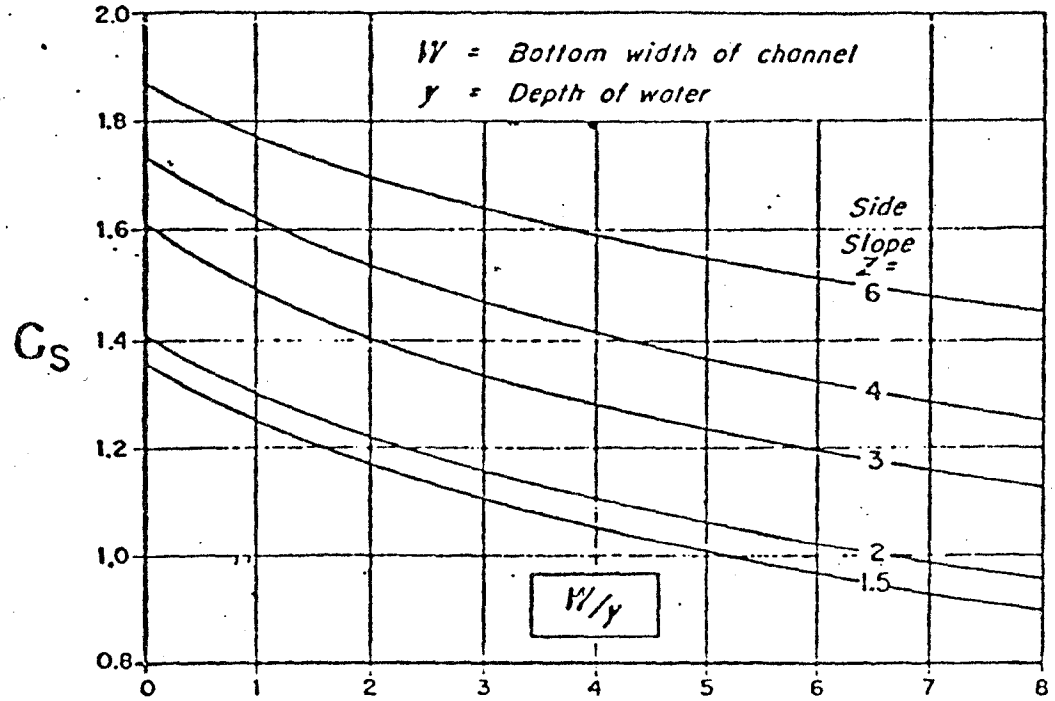
Say $D_{50} = 6''$

Required layer thickness $= 1\frac{1}{2} \times 6'' = 9''$

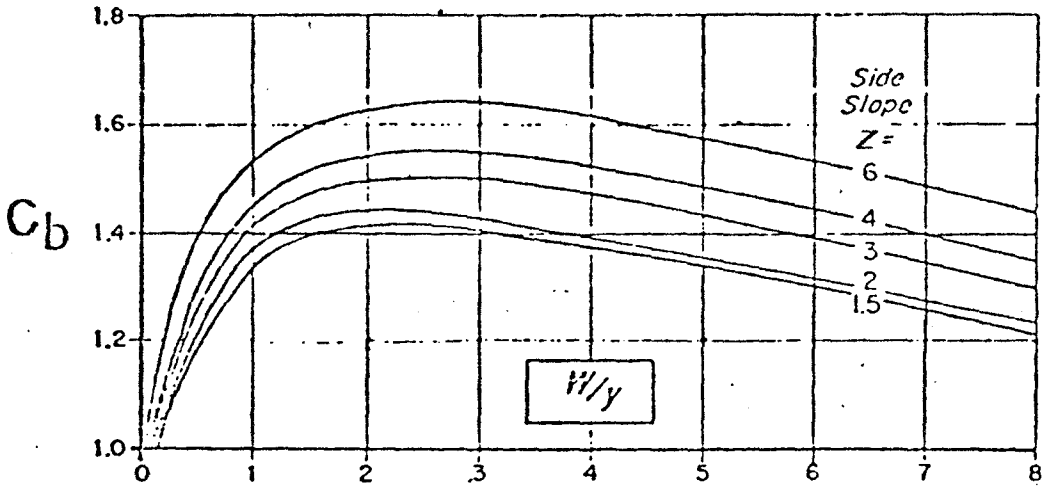
Hydraulic Engineering Circular No. 11, Figure 8, Curves A_1 or B, is recommended for determining a theoretical gradation for the riprap.

FIGURE 1

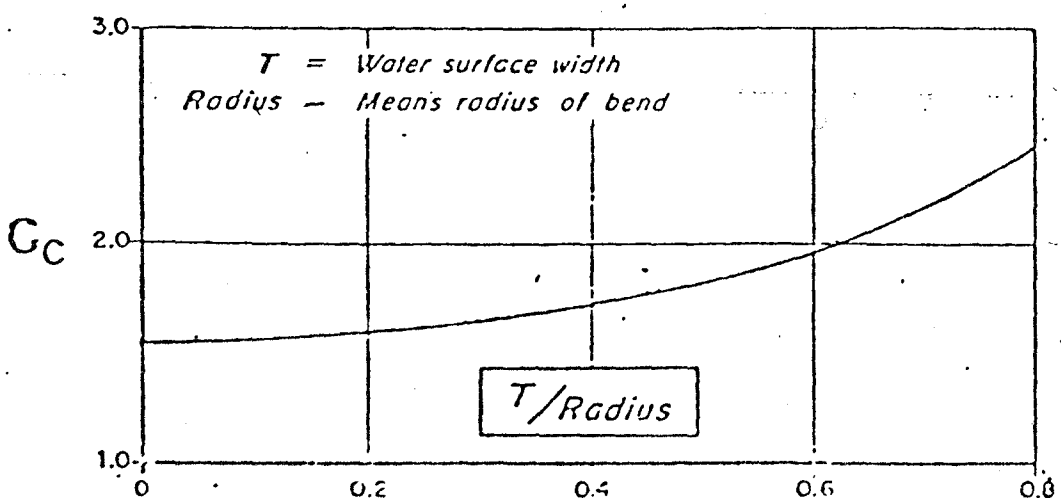
FLOW SHEAR



Maximum Boundary Shear Stress on SIDES of Trapezoidal Channels



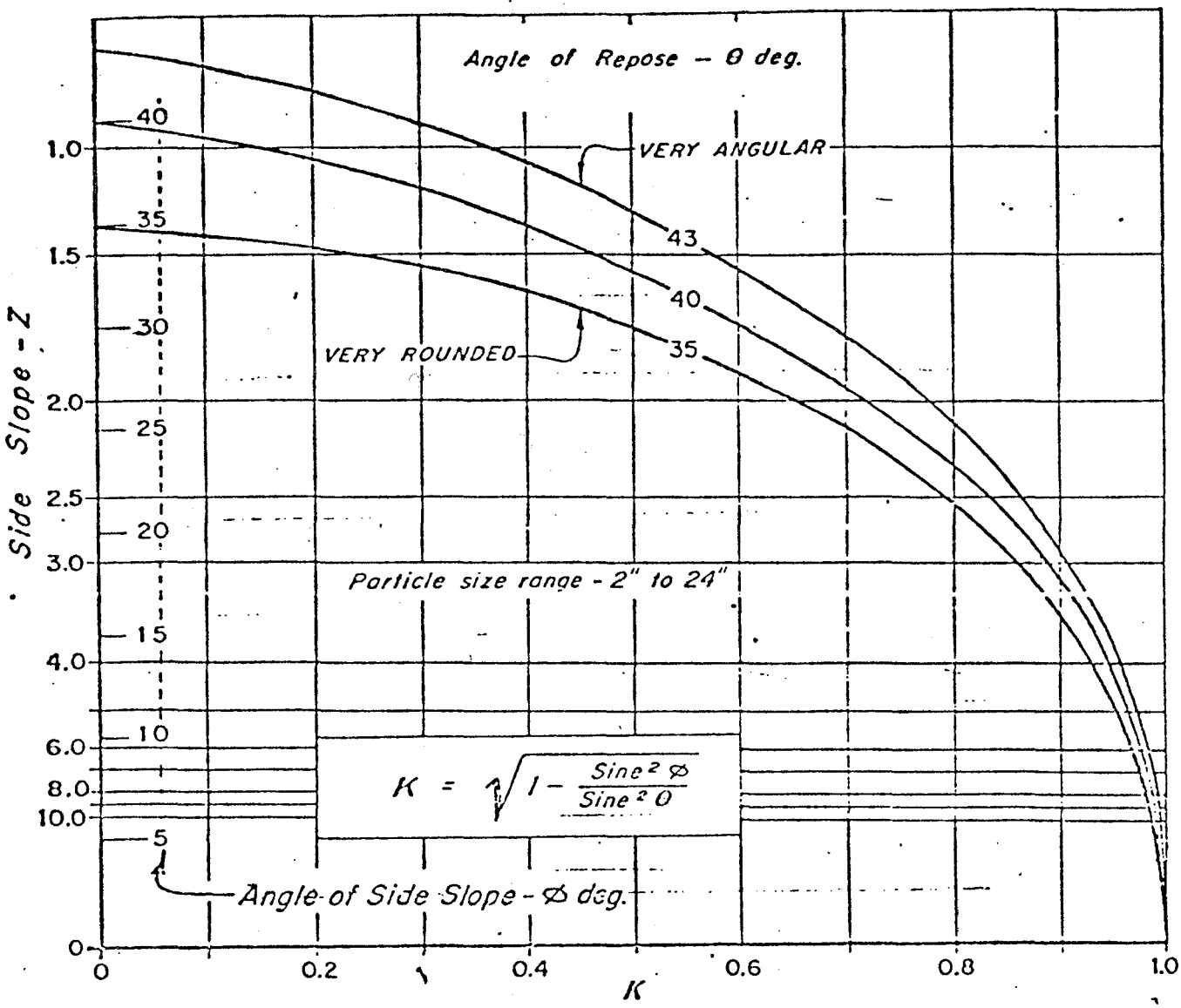
Maximum Boundary Shear Stress on BOTTOM of Trapezoidal Channels



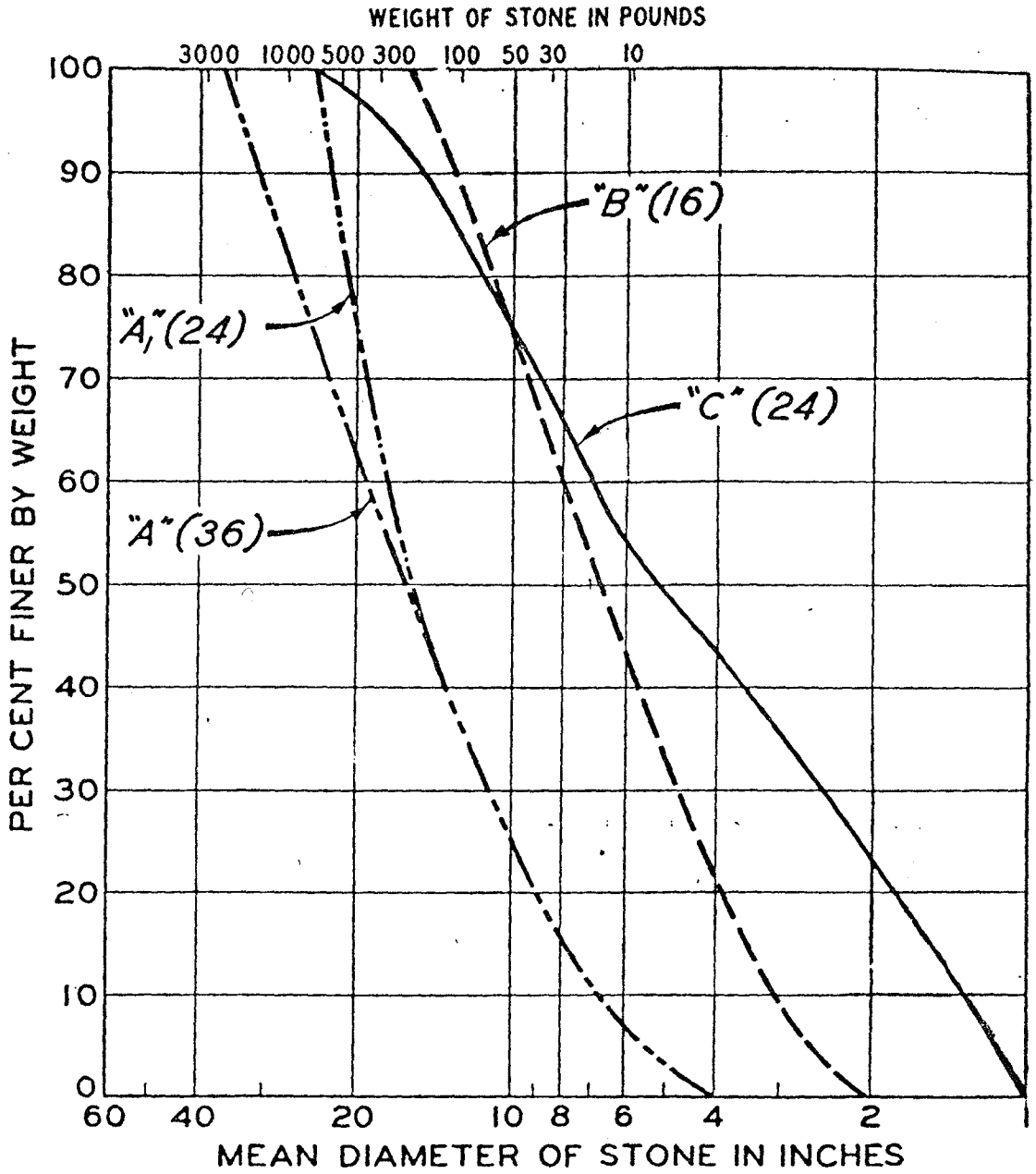
Maximum Boundary Shear Stress for Flow in Bends

FIGURE 2

LINING SHEAR



Reduction Factor of Resistive Unit Boundary Shear



PROTECTIVE STONE
GRADATION CURVES

FIG. 8 - GRADATION CURVES FOR STONE