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STORM SEWER SYSTEM DESIGN

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October, 1972

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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/.

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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.

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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: when d = .4'

$$T = zd$$

$(.56(z/n)(d)^{8/3} = 152.0079$

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

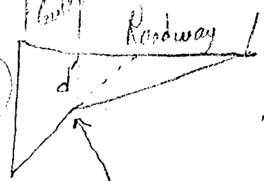
X = length of gutter or Roadway

2. From Chart 1 $\quad n = .015$

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

$Z_0 = \frac{1}{X}$ slope of gutter of the Gutter or the Road way

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.

From $T = Z_a d$
$T = X$

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 $\quad Q = .9$ 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:

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

d = 0 to .4'

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.

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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
$A = \frac{5.01}{2} = 2.5 \text{ ft.}^2$

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

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

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:

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

VERIFIED

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.,   T = zd-4
ACTUALLY = d'

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:

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

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'-5¾" 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:

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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$\pm$.

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.

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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

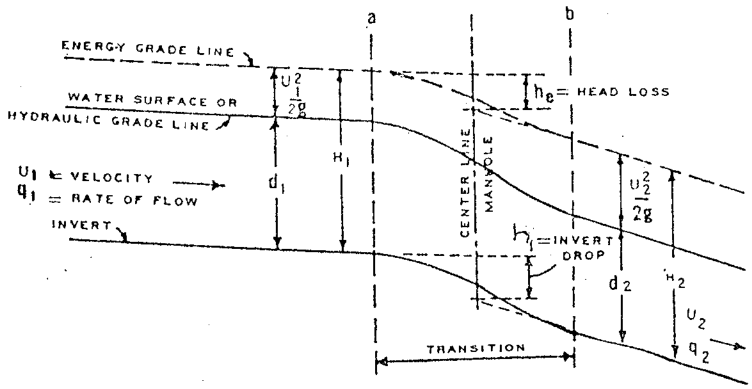
Thus the design time of concentration for any point on a sewer line is the inlet time for the inlet at the upper end of the line plus the time of flow through the sewer from the upper end of the sewer to the reach under design, unless the time for another branch or inlet at that point is greater. The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

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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

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

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,

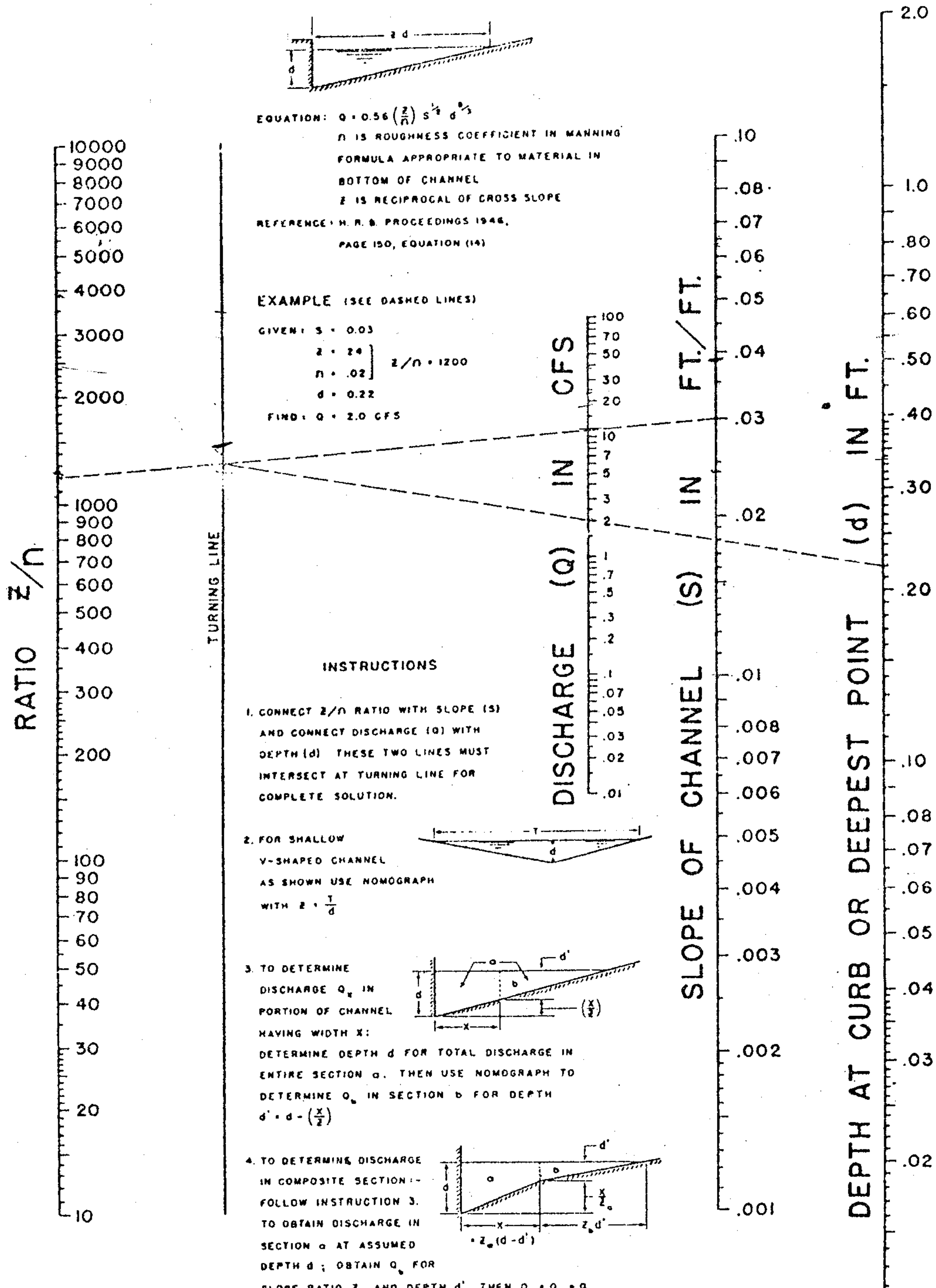
a. the full flow capacity (Q_{full}) of the selected pipe is less than the design discharge Q.

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

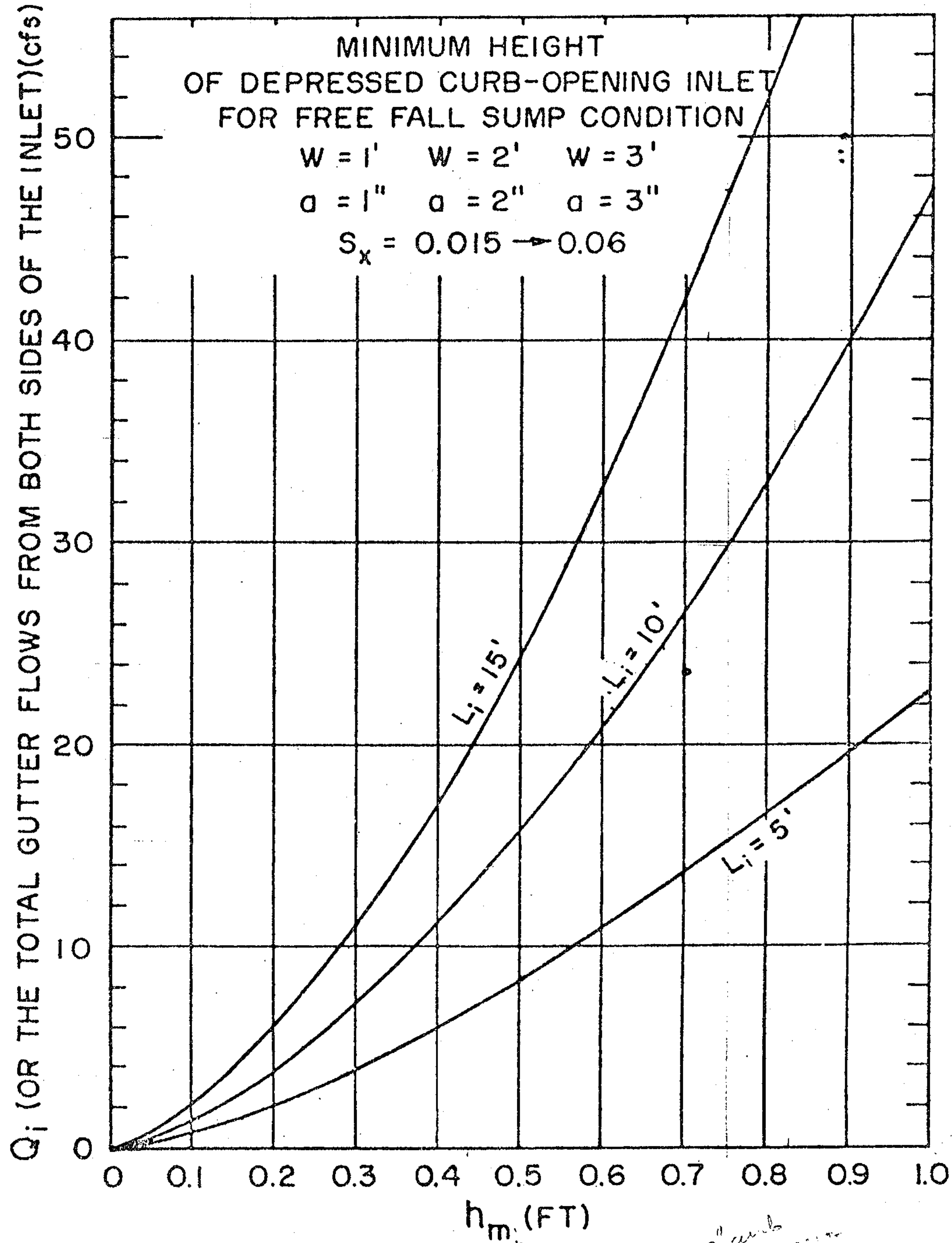
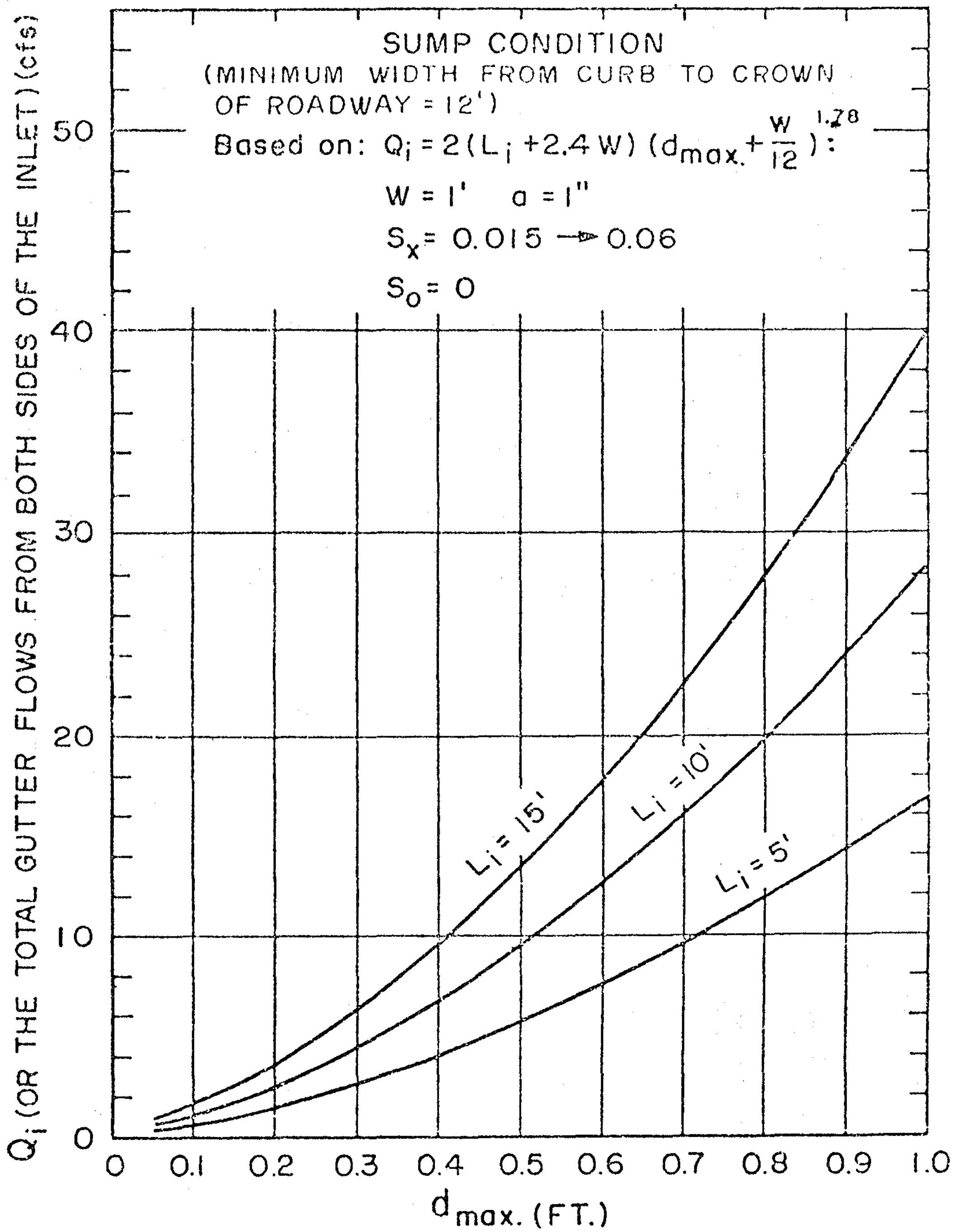

MINIMUM HEIGHT
OF DEPRESSED CURB-OPENING INLET
FOR FREE FALL SUMP CONDITION
W = 1' W = 2' W = 3'
a = 1" a = 2" a = 3"
S_x = 0.015 → 0.06
L_i = 15'
L_i = 10'
L_i = 5'
Q_i (OR THE TOTAL GUTTER FLOWS FROM BOTH SIDES OF THE INLET)(cfs)
0
10
20
30
40
50
0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0
h_m (FT)
7" curb
+2" depression

Chart 2



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

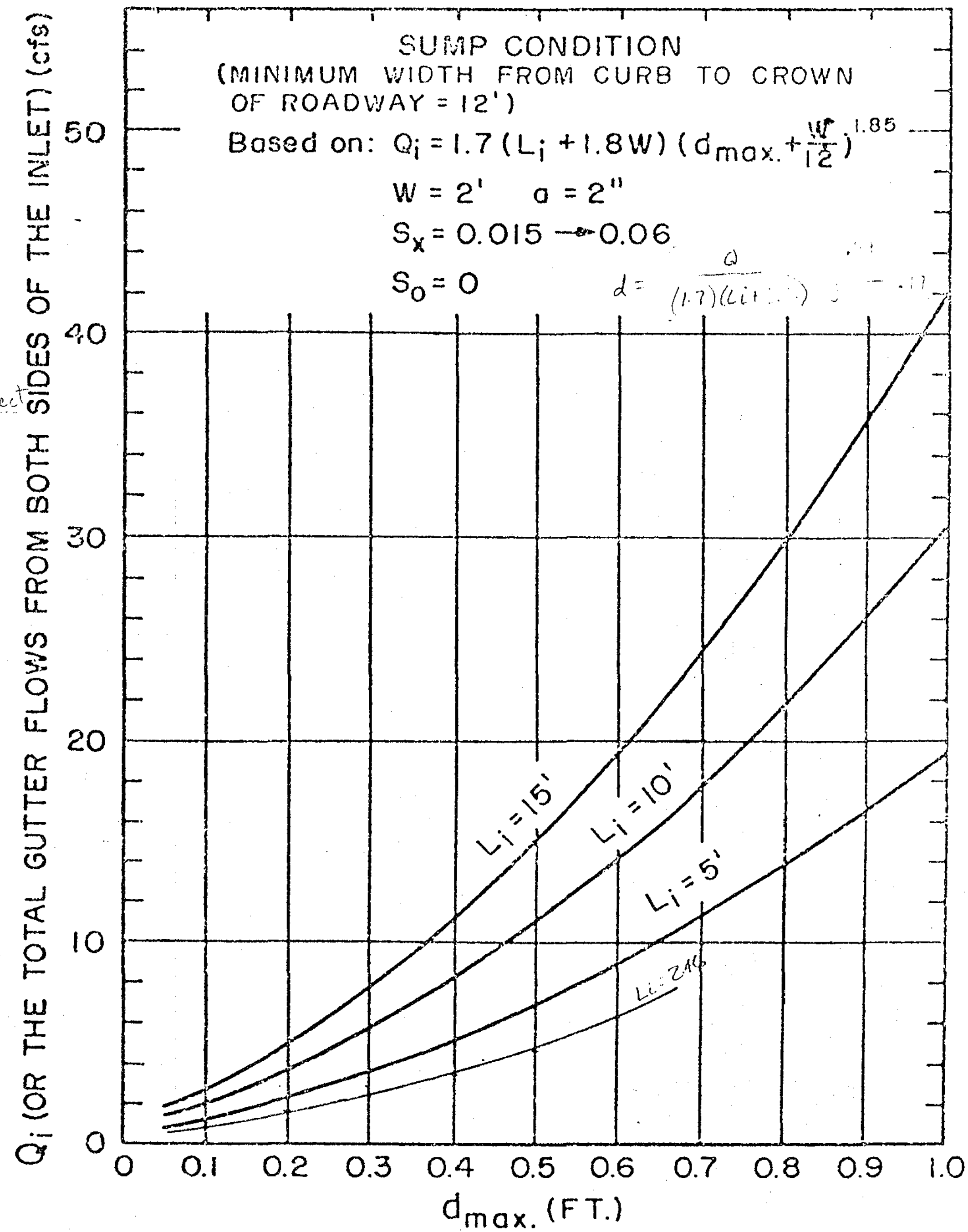

SUMP CONDITION
(MINIMUM WIDTH FROM CURB TO CROWN OF ROADWAY = 12')
Based on: $Q_i = 1.7\,(L_i + 1.8W)\,(d_{max.} + \frac{W}{12})^{1.85}$
W = 2' a = 2"
$S_x = 0.015 \rightarrow 0.06$
$S_o = 0$
$L_i = 15'$
$L_i = 10'$
$L_i = 5'$
$L_i = 2.46$
Q_i (OR THE TOTAL GUTTER FLOWS FROM BOTH SIDES OF THE INLET) (cfs)
$d_{max.}$ (FT.)
0
10
20
30
40
50
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

Chart 4

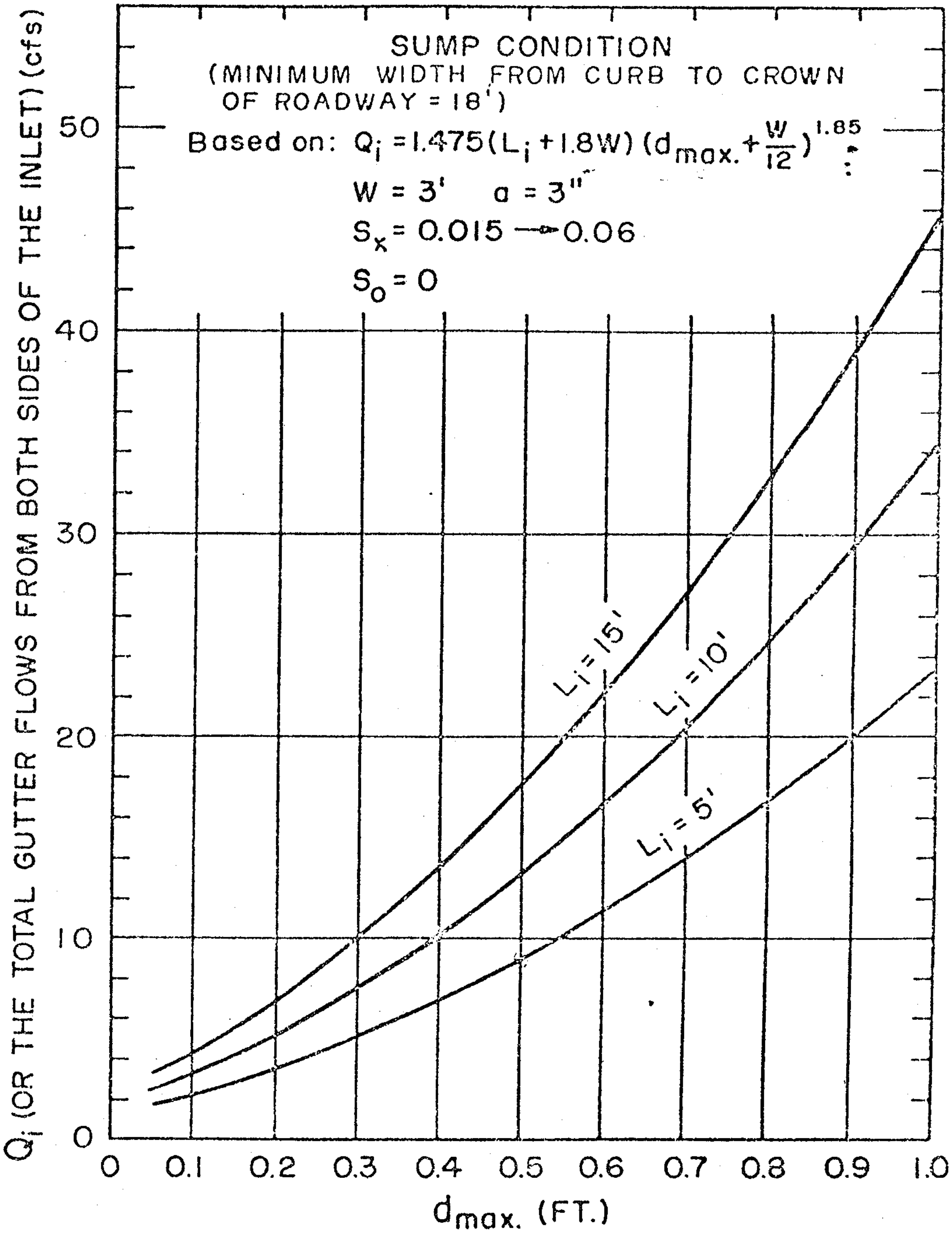
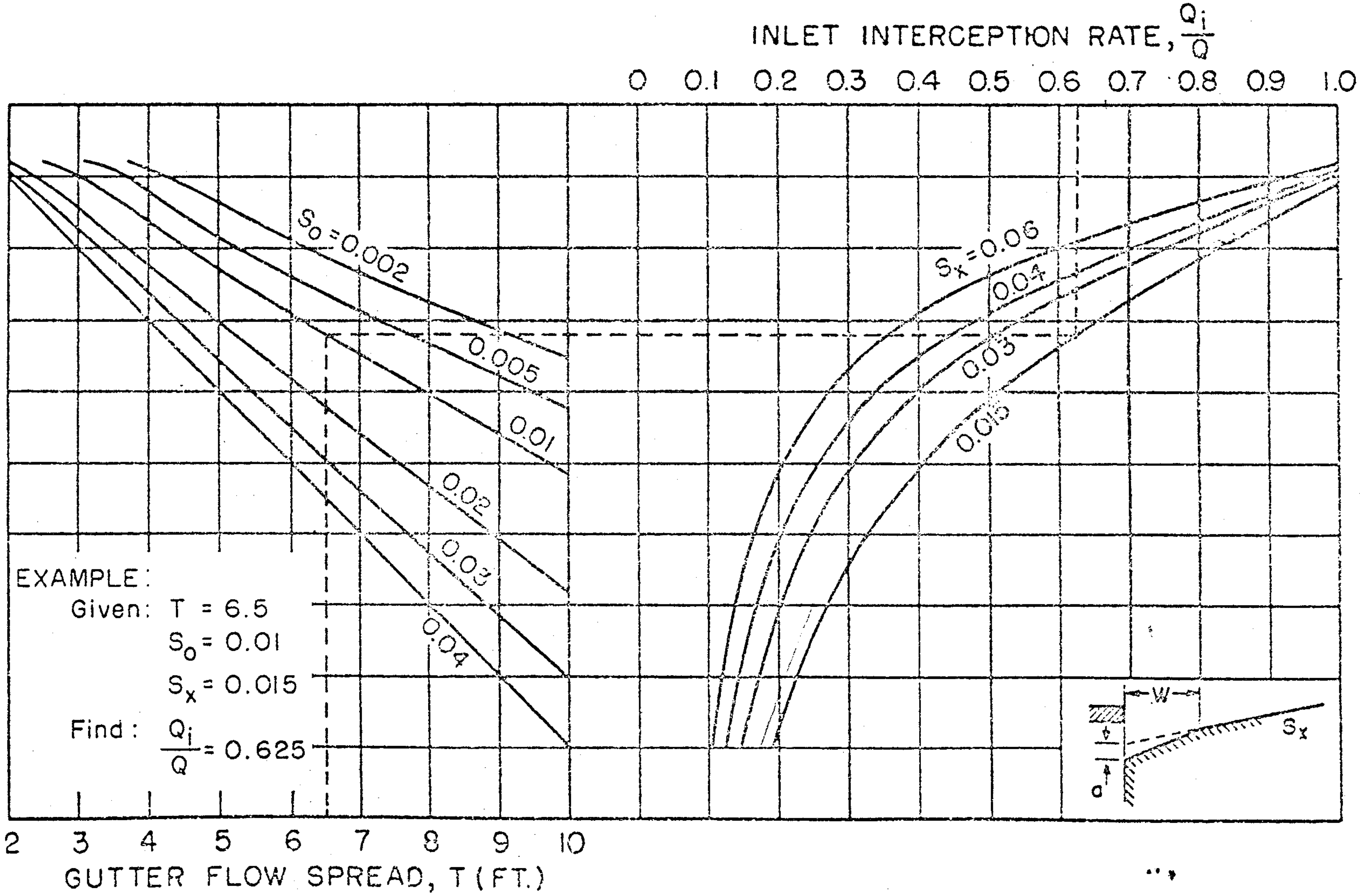


Chart 5

AHD
Structures Section
Hydraulics Branch
10-15-72

FHWA
HEC #12



General condition

W = 1' a ≧ 1"

n = 0.016

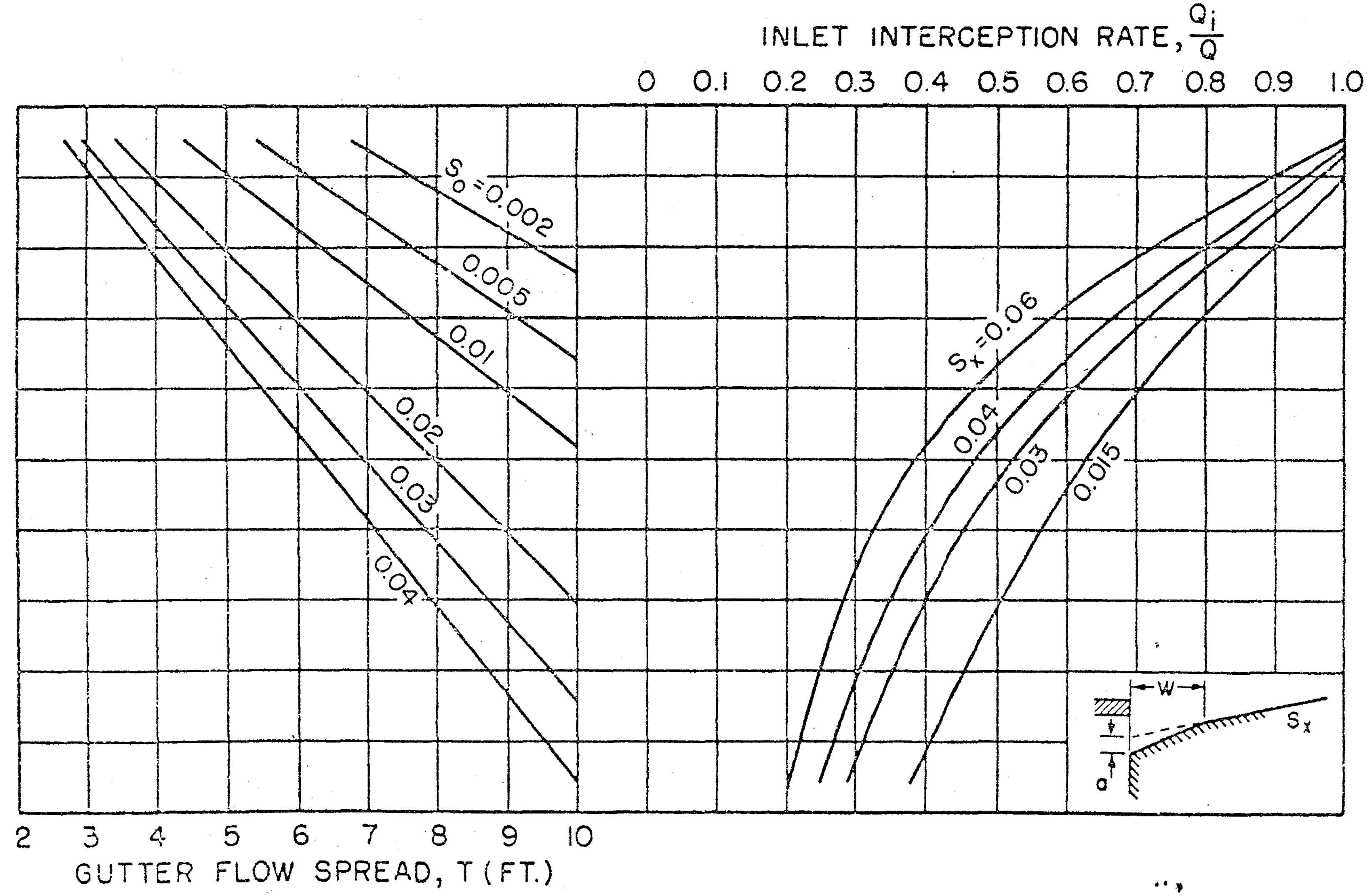
Length of opening, L_i = 5'

Minimum height of curb opening, $h_m = T\ S_x$

AHD
Structures Section
Hydraulics Branch
10-15-72

Chart 7
F-32

FHWA
HEC #12



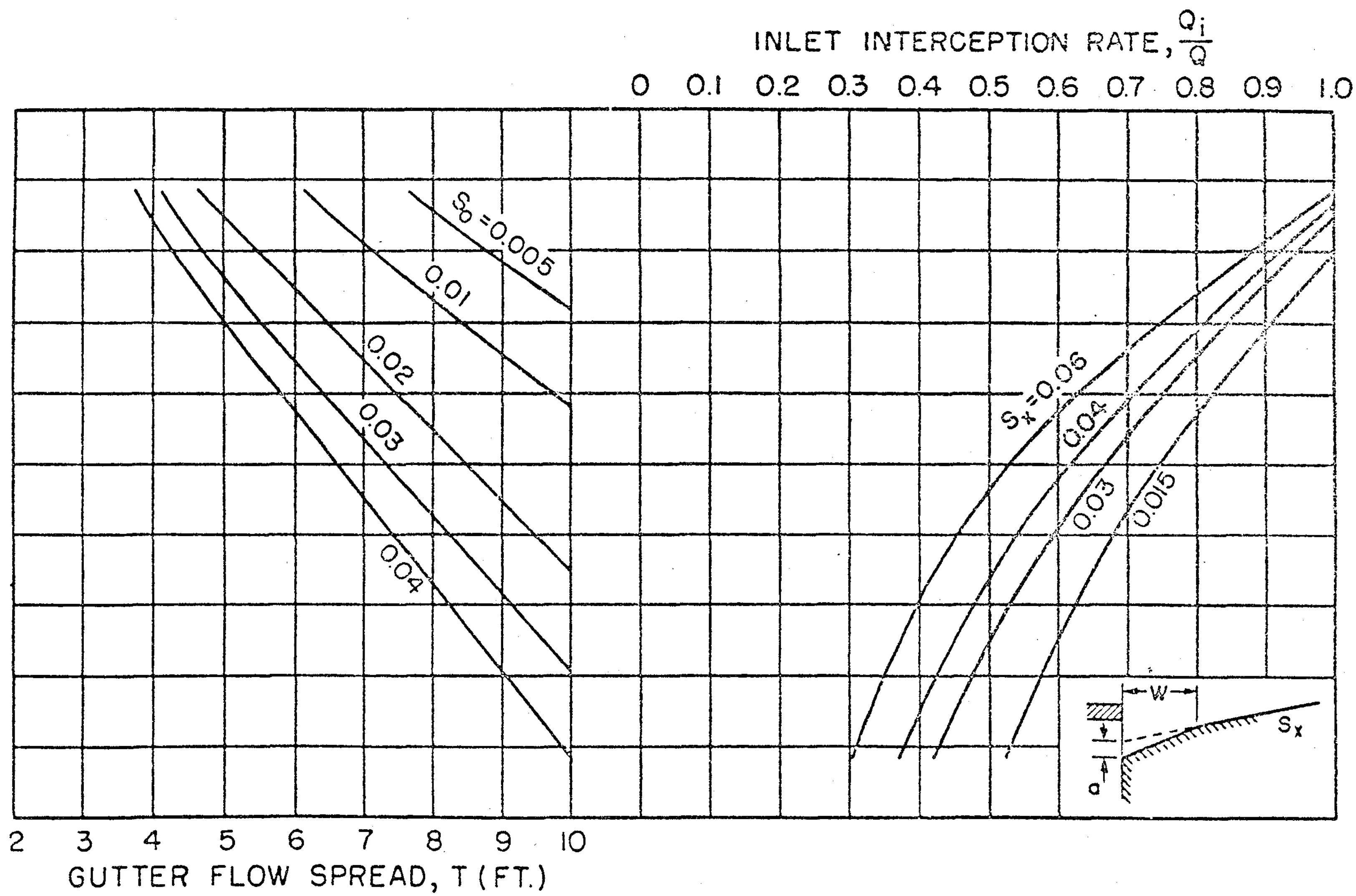
General condition

W = 1' a ≧ 1"

n = 0.016

Length of opening, L_i = 10'

Minimum height of curb opening, $h_m = T\,S_x$

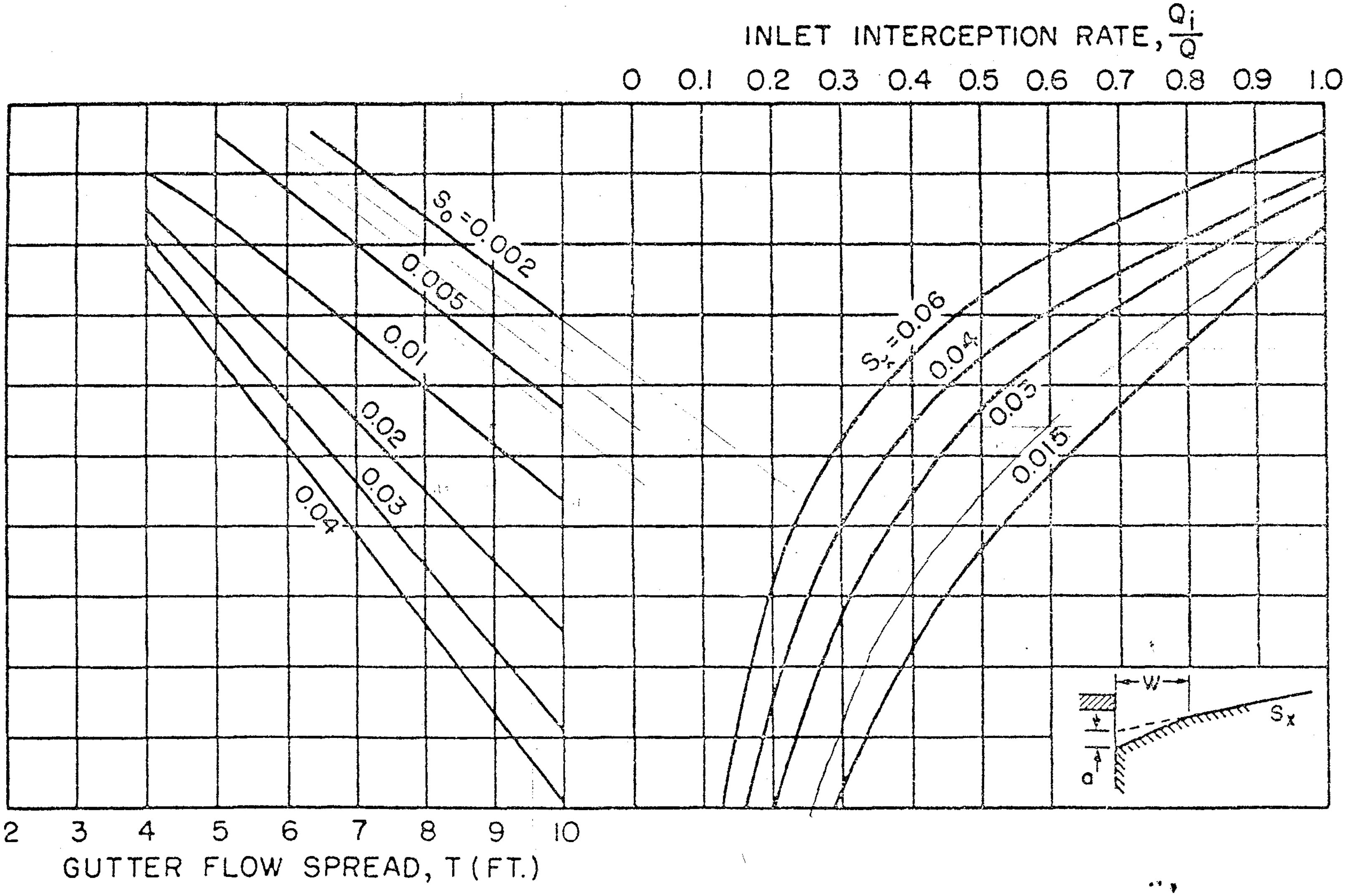


General condition
W = 1'   a ≧ 1"
n = 0.016
Length of opening, L_i = 15'
Minimum height of curb opening, $h_m = T\, S_x$

AHD
Structures Section
Hydraulics Branch
10-15-72

Chart 9
F-34

FHWA
HEC #12

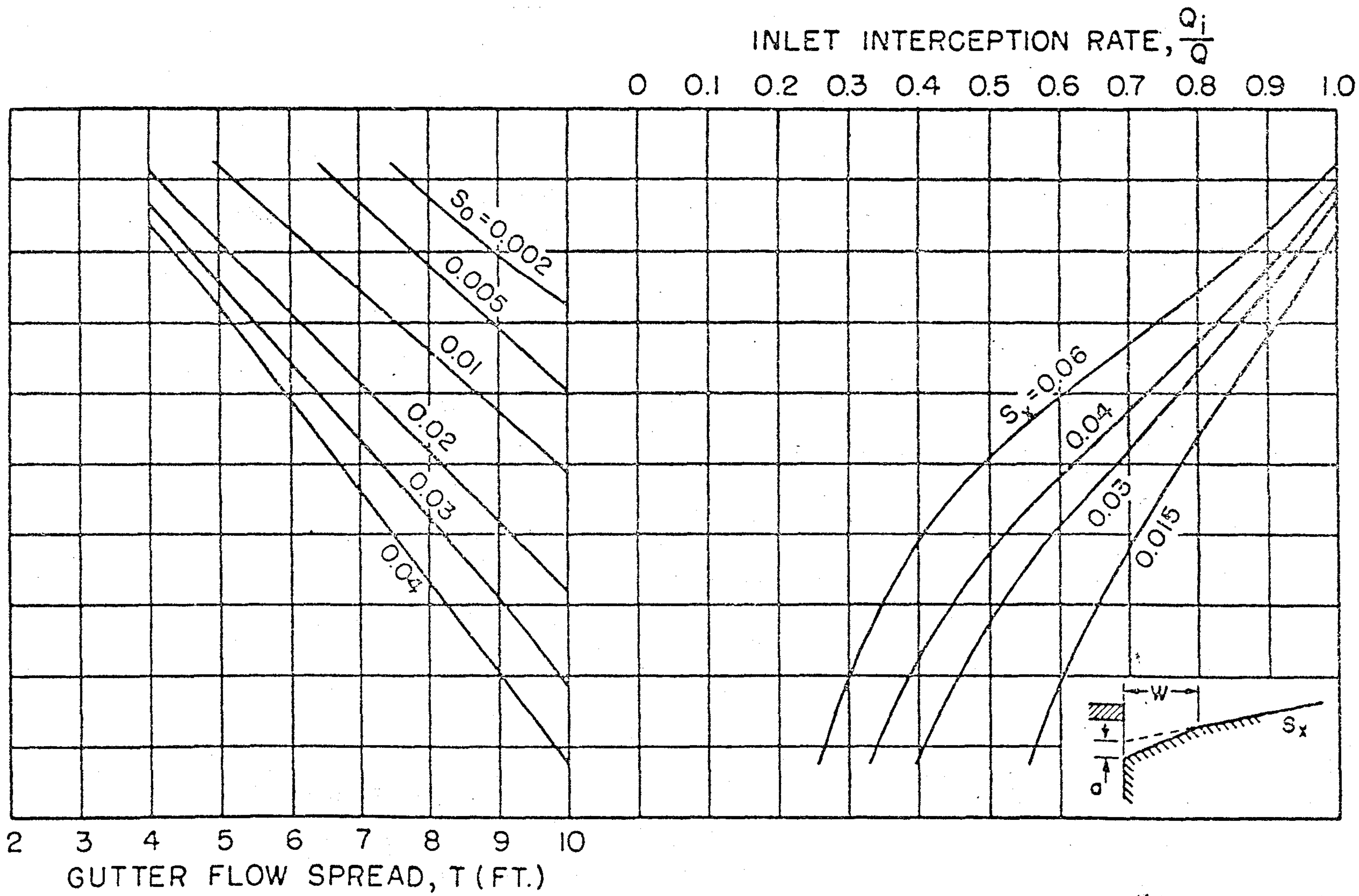


General condition
$W = 2'$ $a \geqq 2''$
$n = 0.016$
Length of opening, $L_i = 5'$
Minimum height of curb opening, $h_m = T\,S_x$

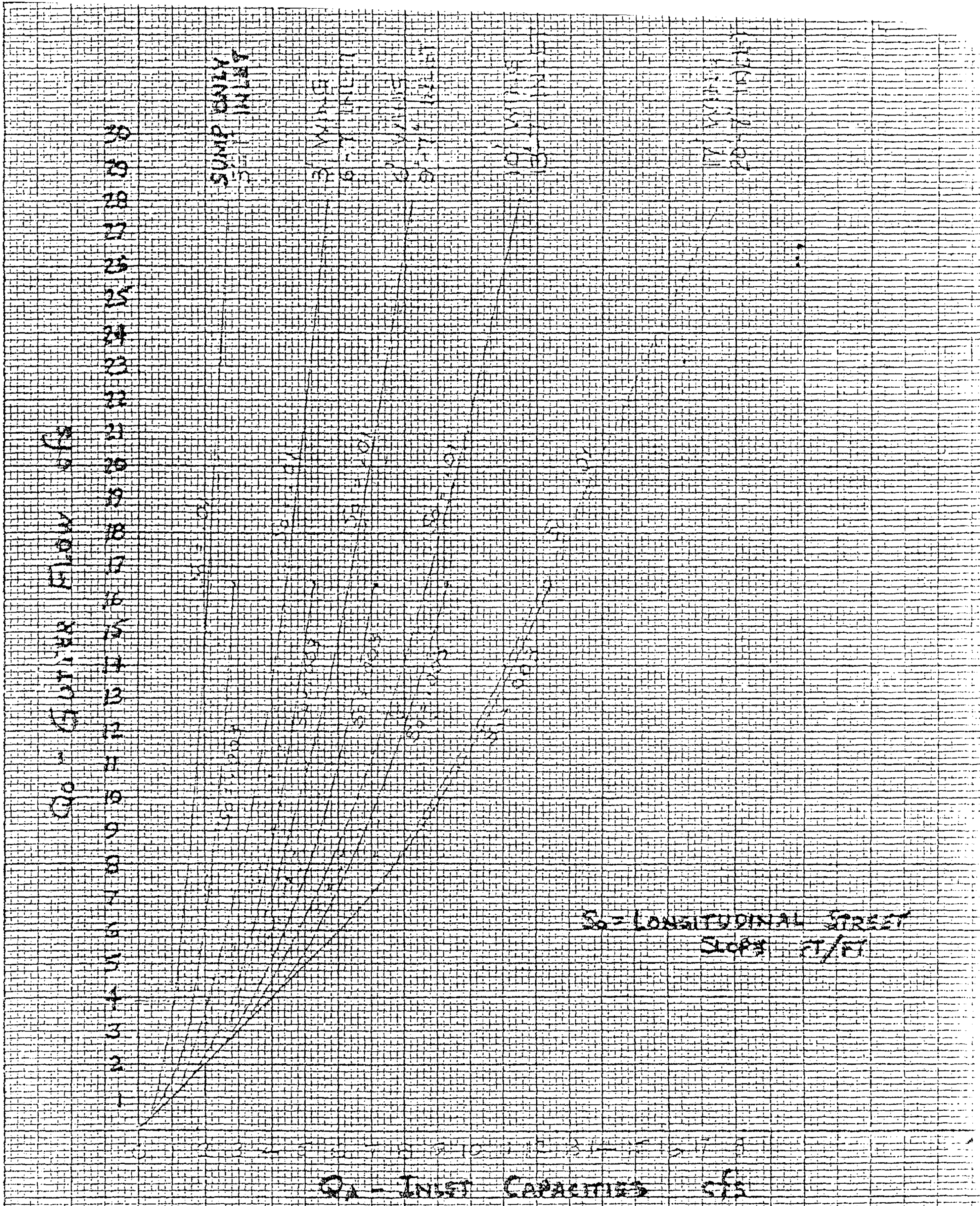
AHD
Structures Section
Hydraulics Branch
10-15-72

Chart 10
F-35

FHWA
HEC #12



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

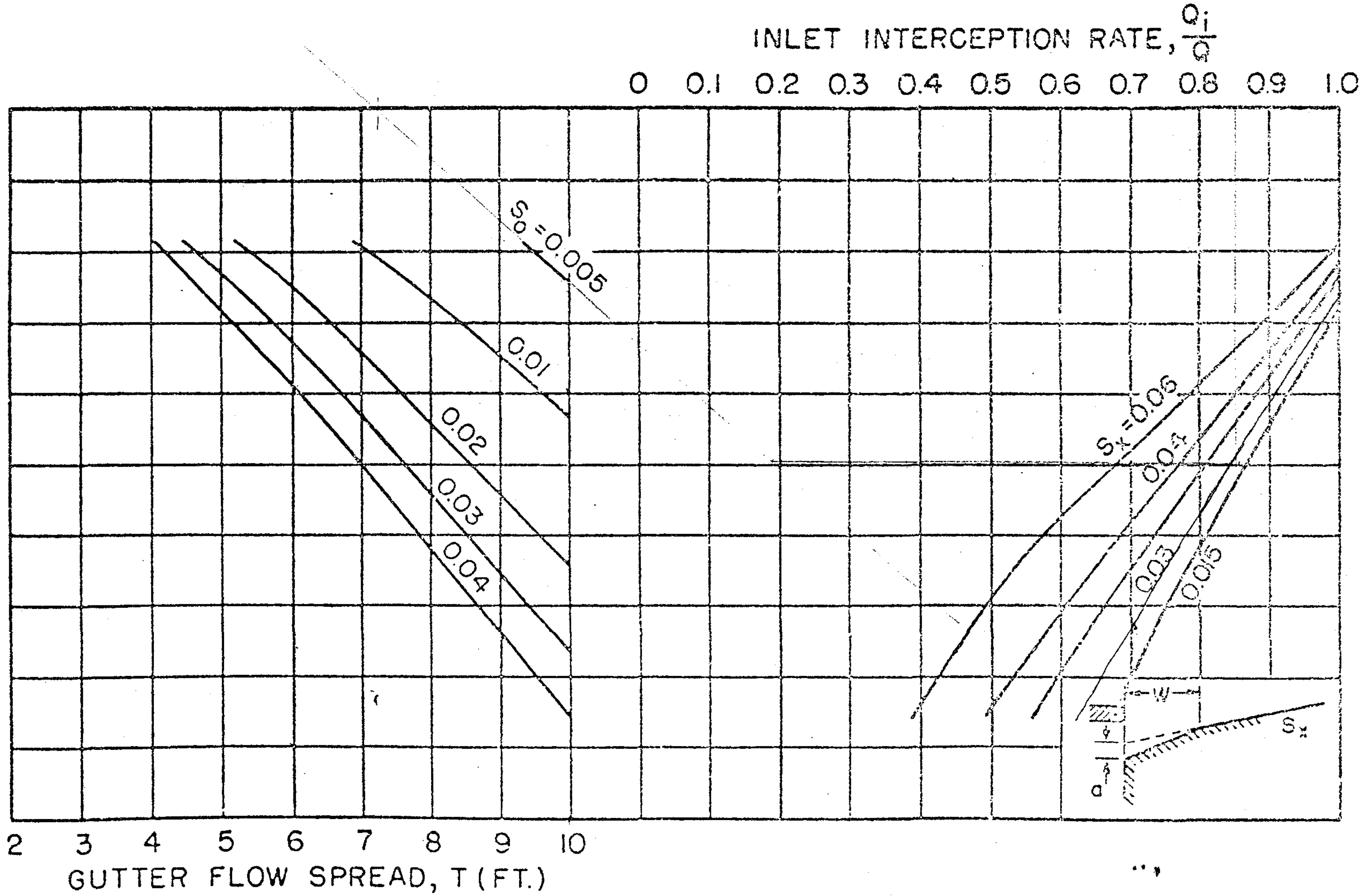


AHD TYPE 3 CATCH BASIN
PAVEMENT CROSS SLOPE = 2% n = .015
GUTTER DEPRESSION = 3"

AHD
Structures Section
Hydraulics Branch
10-15-72

Chart 11
F-36

FHWA
HEC #12



General condition

W = 2' a ≧ 2"

n = 0.016

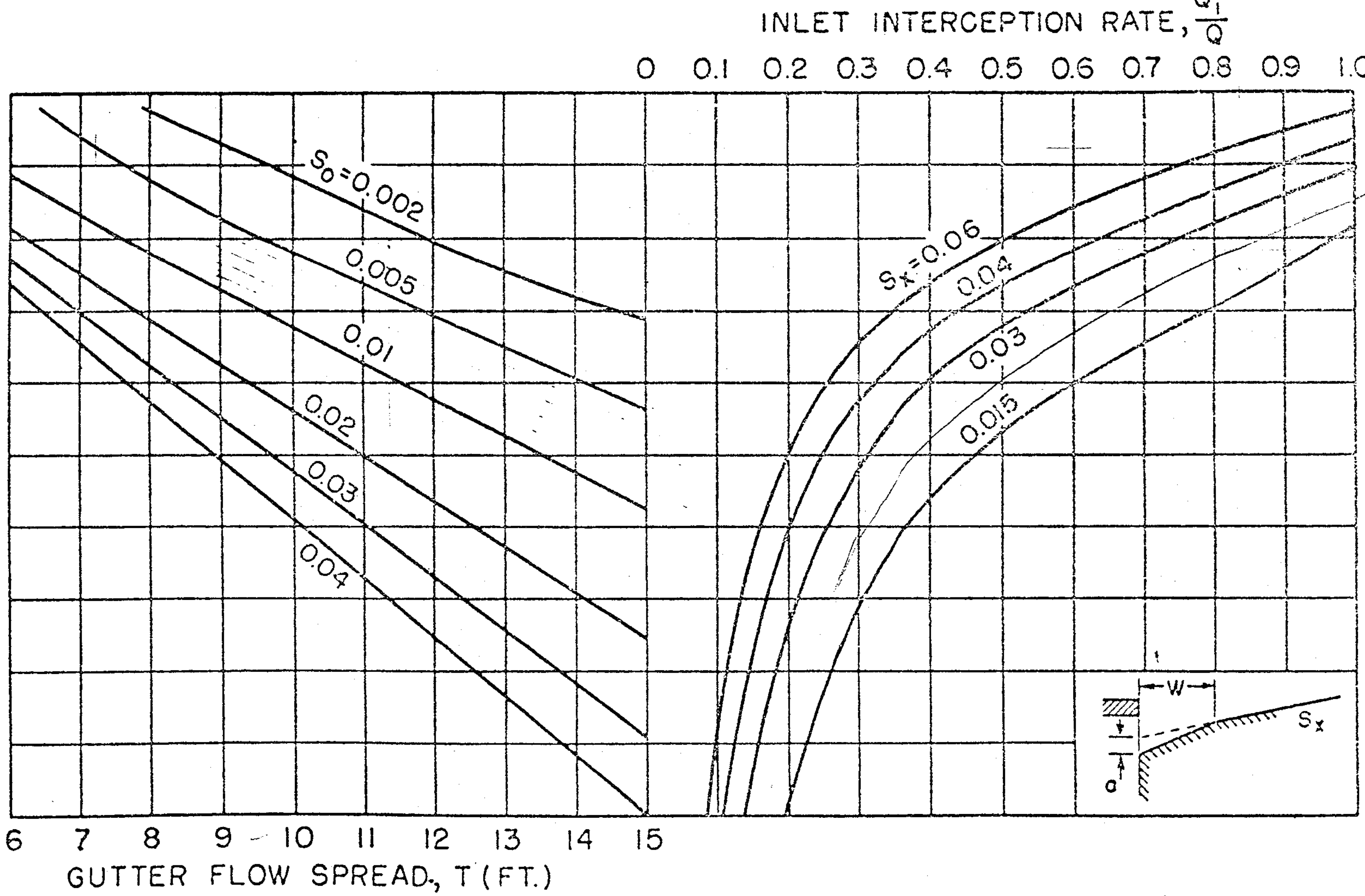
Length of opening, L_i = 15'

Minimum height of curb opening, $h_m = T S_x$

AHD
Structures Section
Hydraulics Branch
10-15-72

Chart 12
F-37

FHWA
HEC #12



General condition

W = 3' a ≧ 3"

n = 0.016

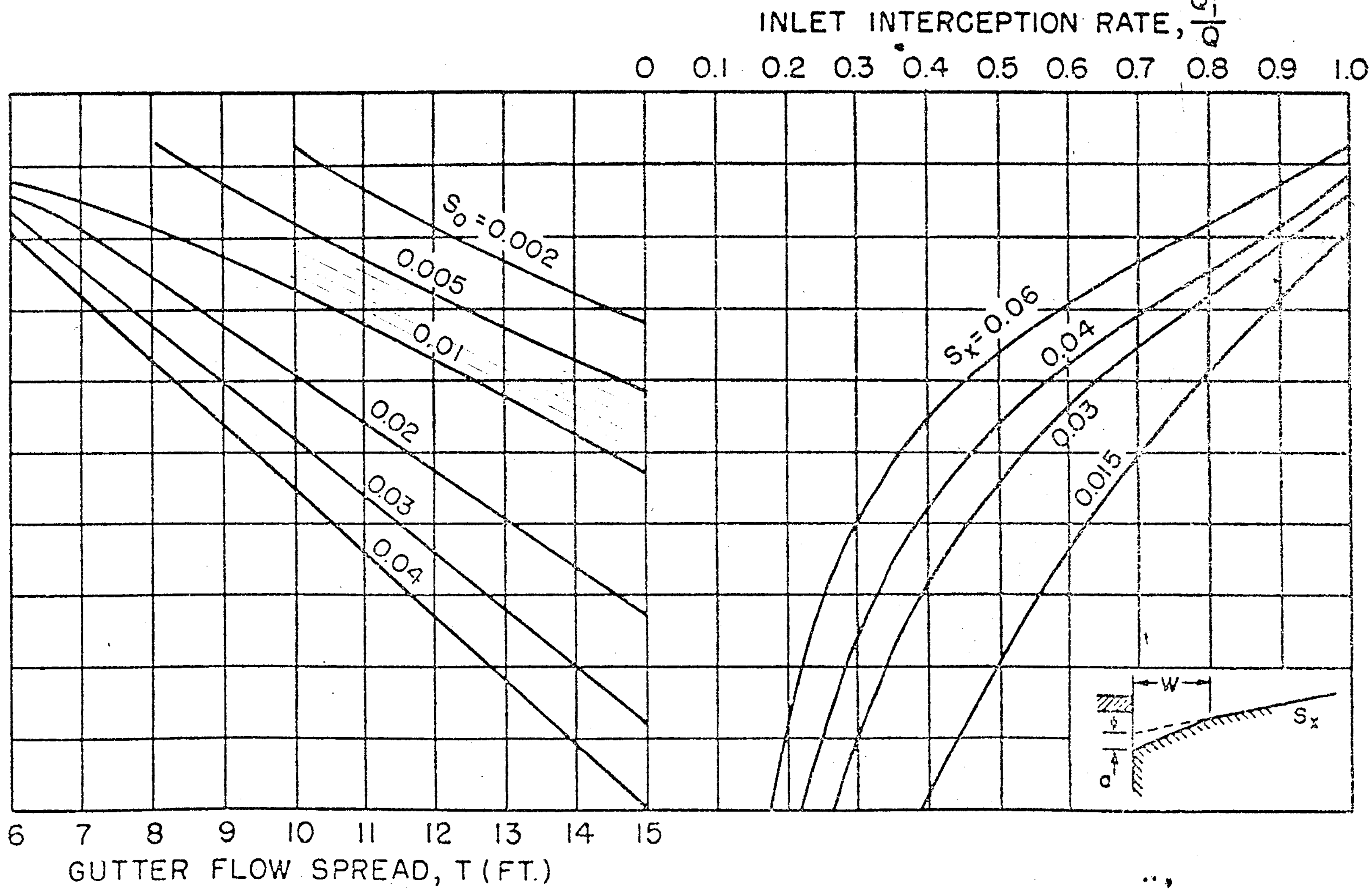
Length of opening, L_i = 5'

Minimum height of curb opening, $h_m = T\ S_x$

AHD
Structures Section
Hydraulics Branch
10-15-72

Chart 13
F-38

FHWA
HEC #12



General condition

W = 3' a ≧ 3"

n = 0.016

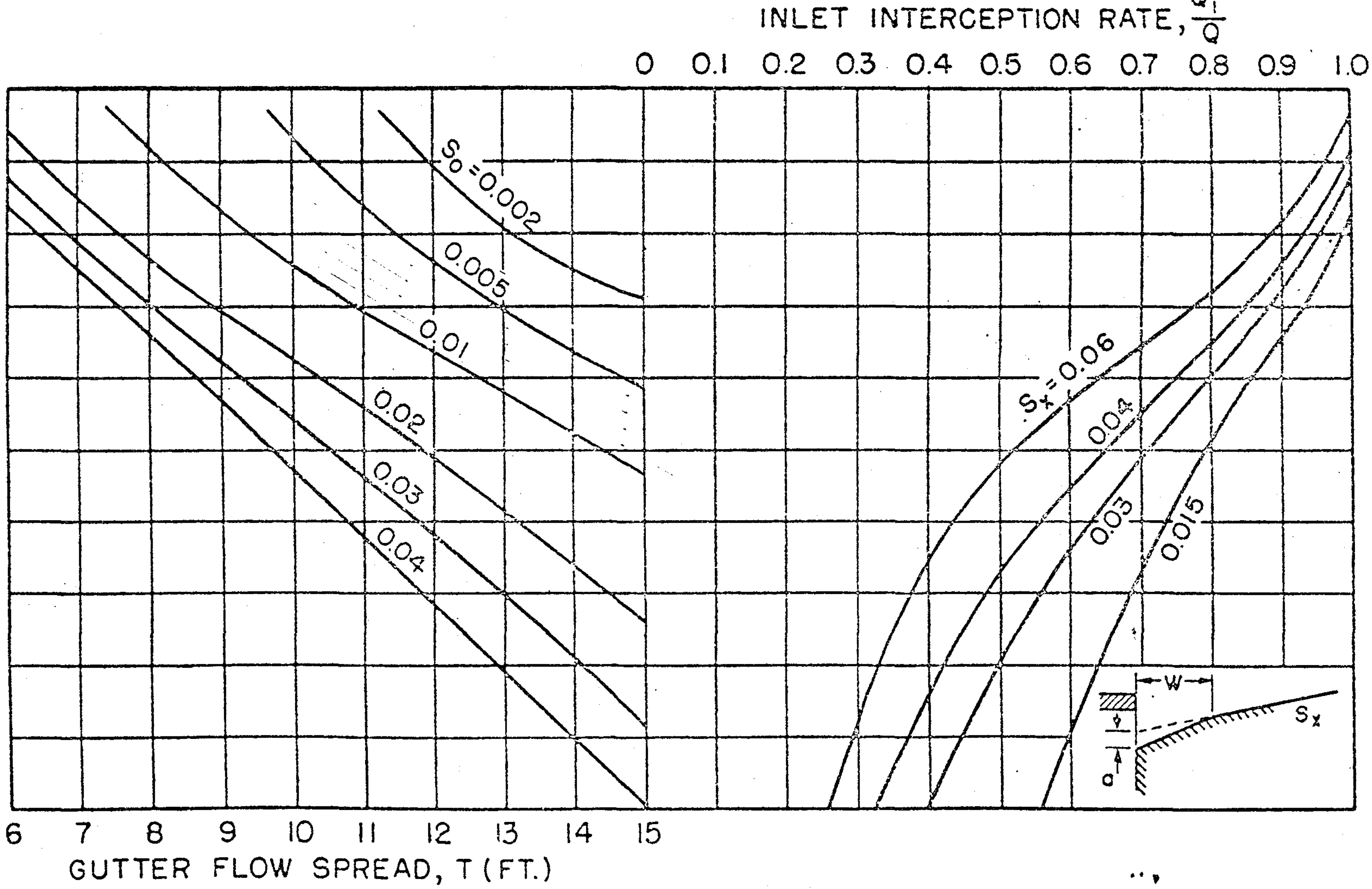
Length of opening, L_i = 10'

Minimum height of curb opening, $h_m = T\ S_x$

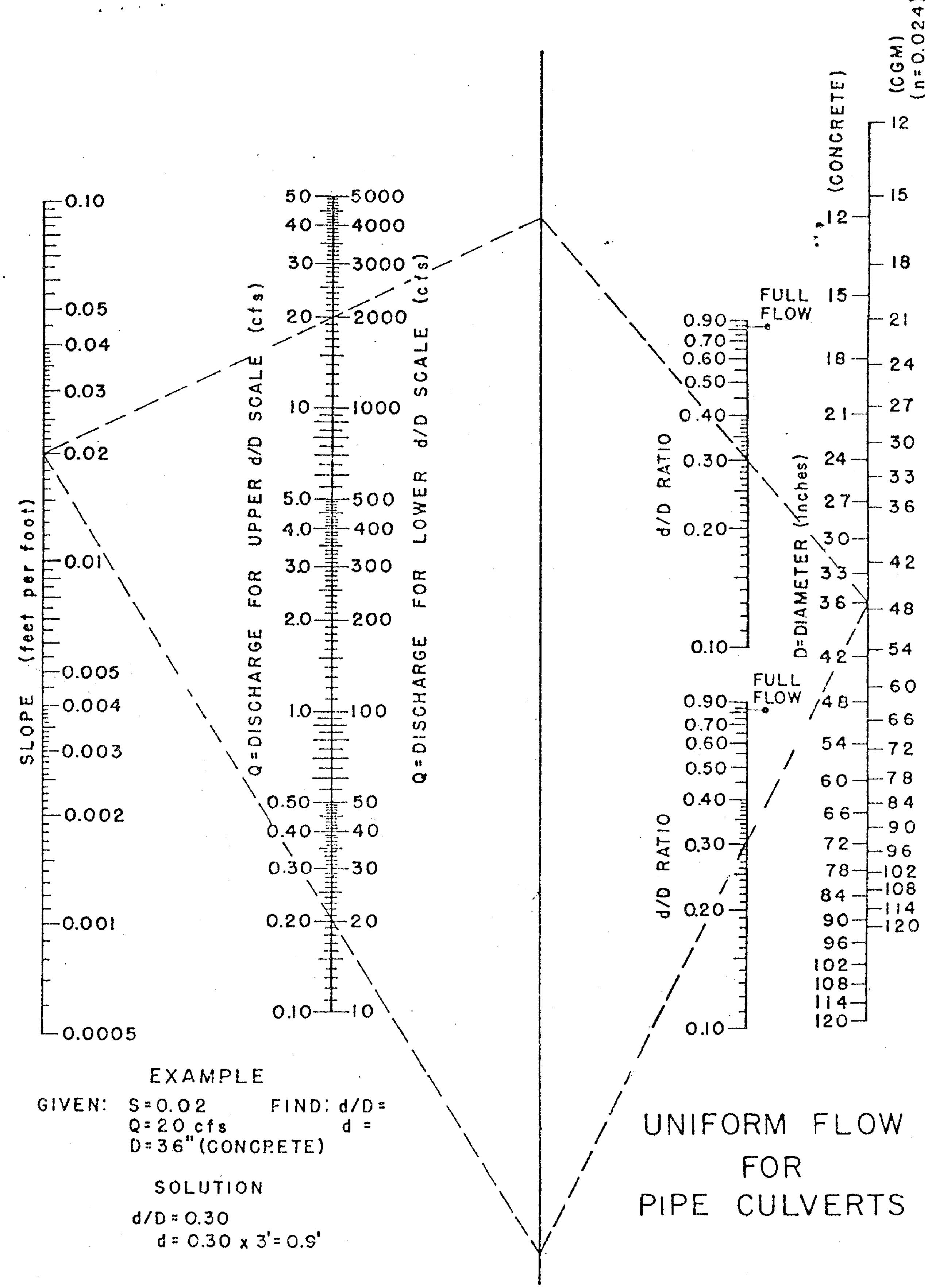
AHD
Structures Section
Hydraulics Branch
10-15-72

Chart 14
F-39

FHWA
HEC #12



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

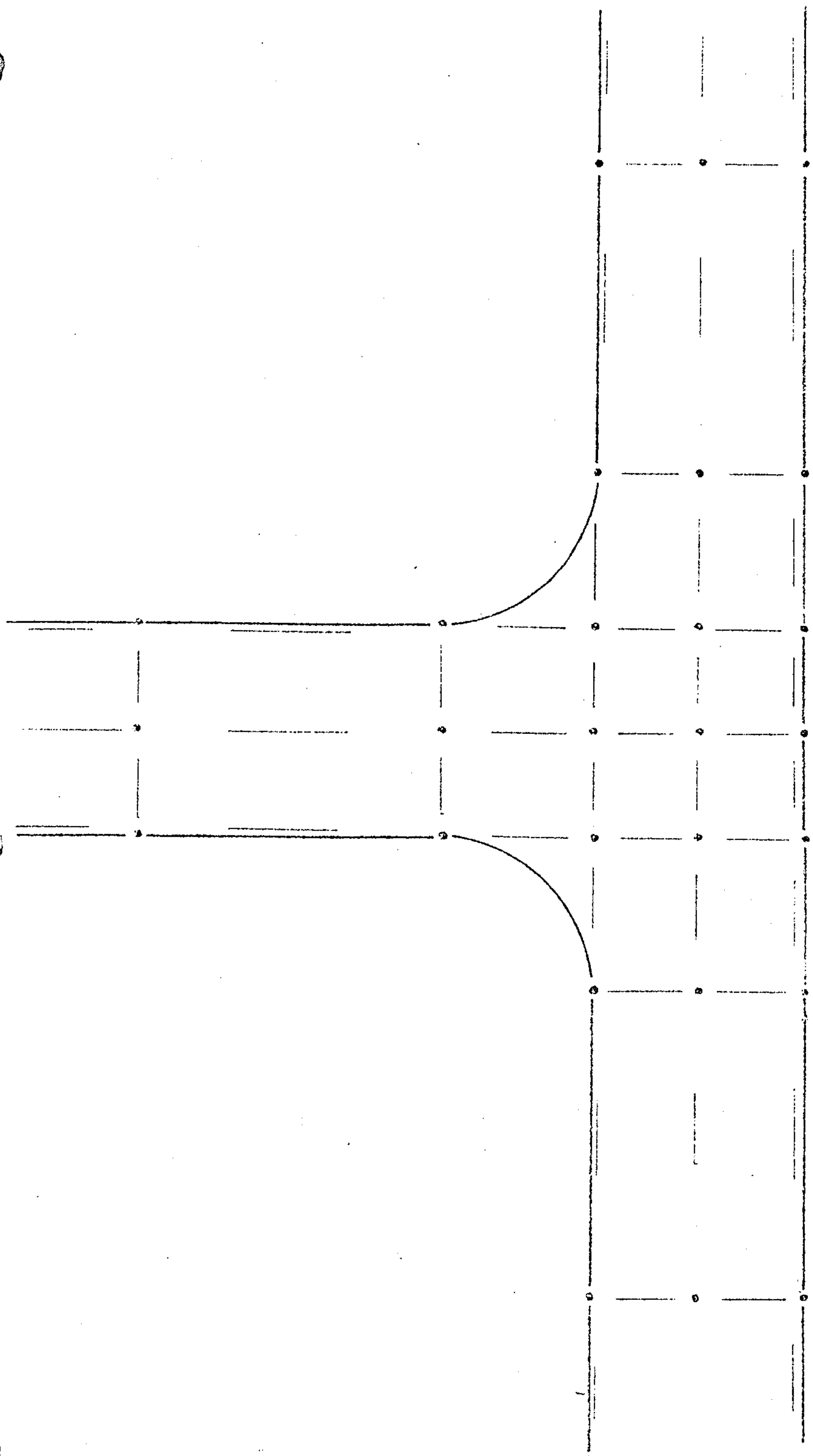
Table 1

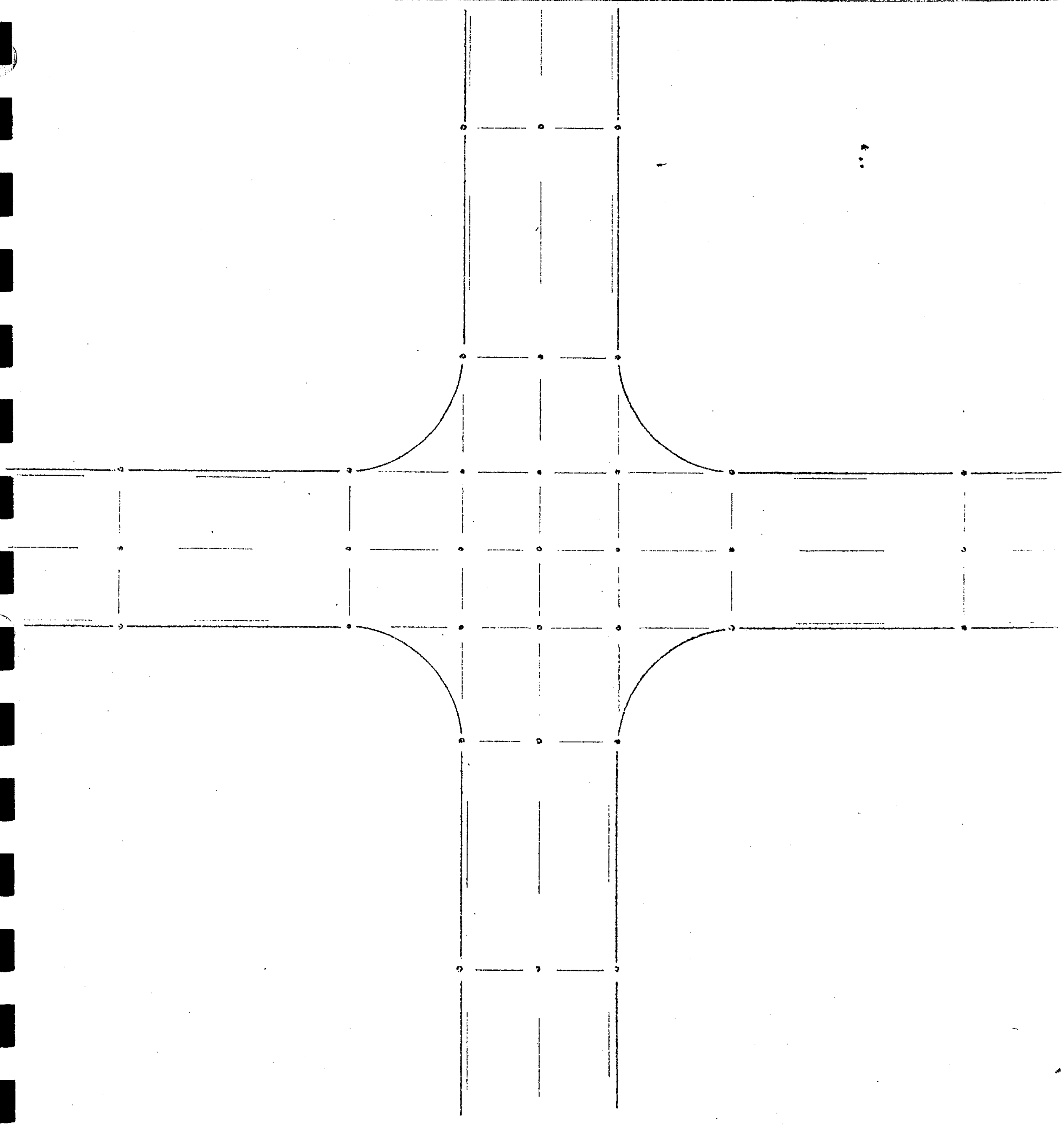
For Determining the Area "a" of the Cross Section of a Circular Conduit Flowing Part Full

Let $\dfrac{\text{depth of water}}{\text{diameter of channel}} = \dfrac{D}{d}$ 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
PHOENIX, ARIZONA

August 4, 1969

FLORIAN & COLLINS
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To: Consulting Engineers

From: Pre-Engineering (Consultants) Division

Re: Hydraulic Reports

The following requirements supplement and are in addition to the existing Hydrologic Manual.

Hydraulic Reports must be submitted in duplicate (one each for Plans Division and Bridge Division) bound in an 8½" x 11" folder, and must contain, but need not be limited to, the following:

1. Cover and title Sheet showing Project Name and Number, Title of Report, Authorship (Firm Name), and Date of Submittal.

2. Letter of Transmittal.

3. Index or Table of Contents.

4. A narrative describing the topographic features and hydrologic history of the project area, noting general drainage patterns, the history and effect of known flooding in the area, and recurring problem areas. The history of man-made developments and their effect upon natural drainage patterns, including an estimate of the effect the proposed highway construction might be expected to have upon major drainage flows. And finally, a projected estimate of future development in the project area which might affect the characteristics of future drainage flows and ultimately the performance of the hydraulic structures on the project.

5. An outline of the Scope of the Study, including the criteria, assumptions, and methods used in the design procedure, listing all references.

6. A general area map of the entire project at a small (reduced) scale showing topographic features and drainage areas.

7. A structure summary sheet listing all pipe and box culverts by station, and including the following:

   a. Station.
   b. Drainage Area.
   c. Discharge (Design Q).
   d. Type and Size of Structure, including length.
   e. Inlet and Outlet treatment (headwall, end section, etc.)
   f. Invert Elevation at centerline.
   g. Invert slope in %.
   h. Subgrade elevation.
   i. Allowable Headwater Elevation.
   j. Design Headwater Elevation.
   k. Natural Channel Velocity.
   l. Culvert Outlet Velocity.
   m. Outlet protection (Type).
   n. Ponding Outside R/W (Yes or No)

8. For each culvert, channel, or diversion, the following:

   a. A copy of Hydraulic Survey Report Form (A.H.D. 66-451).

   b. A copy of the drainage area map, aerial photo, or other source used, delineating and identifying contributing drainage area(s). One copy may be submitted for entire report, if feasible, with individual drainage areas identified. (See No. 6).

   c. All hydrologic computations on applicable Arizona Highway Department Hydrologic Design Data Sheets. The Rational Method shall be used for <u>ALL</u> drainage areas under 0.1 sq. mile (64 acres). SCS PART I for .1 to 1,000
   SCS PART II for 1 to 1000

   d. Stream or channel cross sections and profile necessary to develop natural stage-discharge relationships and velocity computations. The proposed culvert or bridge structure should be shown on the profile in its proper location along with the outline of the roadway prism. This data should be plotted to a reduced scale commensurate with the size of the report folder.

   e. Natural stage-discharge and velocity computations when applicable.

   f. Hydraulic computations for all culverts, channels, diversions, etc., including culvert extensions of sufficient length to affect the hydraulic characteristics of the structure.

Culvert computations should be submitted on A.H.D. Culvert Computation Sheets in accordance with the suggested Hydraulic Design Procedures (Green Cover) available from the Plans Division.

g. When a culvert is located in an area of high damage potential (i.e., urban, suburban or industrial area), it will be necessary to include a site sketch locating the various man-made improvements which may be affected by ponding or discharge. Where ponding exceeds the proposed right of way, the inundated area must be delineated by the limiting contour on the site sketch, and a recommendation for acquisition of additional right of way included in the report.

h. A concise summary of results and recommendations for each structure. Where a choice between a pipe culvert and a concrete box culvert is possible within the limits of the hydraulic analysis, a cost comparison should be submitted, including an annual cost based upon the estimated structure life, considering local acidity and abrasive characteristics and their effect upon the structure.

Soil characteristics should be considered when designing channels, ditches, and dykes. Unlined or unprotected slopes should be flat enough to withstand hydrostatic forces without excessive sloughing or erosion. Actual stability analysis may be required for dykes, levees, ditches, and channels. Analysis may include a slip circle analysis of slopes for rapid draw down condition, and allowable velocities procedure (Soil Conservation Service) for stability against erosion. The data compiled, if any, should be included in the report.

Roadway cross sections of each culvert should be plotted to a 10:10 scale and submitted in roll or sheet form with the Plans Division Report only. These sections should include the roadway prism, the culvert profile, and natural ground line extended to the right of way line downstream and the right of way line or limit of ponding upstream, whichever is greater. The design high water line should be indicated as well as the culvert length, slope, and inlet, outlet and center line elevations.

Don Phelps

DON PHELPS, Ass't. Coordinator
Engineering Consultants

DP:ba

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 <u>other</u> 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

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

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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

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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

<u>AHD Standard C-15. 05 Catch Basin:</u>

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

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

Compiled 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_i	Q_c
No	Station	No	Type	cfs	cfs	cfs	%	S_x	z/n	S_x	z/n	ft.	ft.	in.	Act.	Corr.	Q	cfs	cfs

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 ____________________

P_6= __________ in. P_{24}= __________

Outlet TW Elev. = ____________________

Pipe n = ____________________

SEWER CALCULATIONS

Location		Drainage Area			Time of Flow, Min.			I	Q
Line No.	From - To	No.	Increm. CA	Σ CA	Inlet	Sewer	Design	in./hr	cfs

Computed by: ____________________ Checked by: ____________________ Date: __________

P_1= ____________ in.

·file			Sewer Design			Hydraulic Grade Line			
·vation	Length ft.	Slope ft./ft.	Diam. In.	Q_{full} cfs	V f.p.s.	S_f ft./ft.	h_L ft.	Elevation	
Outlet								Inlet	Outlet

ARIZONA HIGHWAY DEPARTMENT
LOCATION DIVISION

HYDRAULIC SURVEY REPORT

LOCATION DATA:

Highway ______________________ County ______________
Location ______________________________________
Project No. ____________________ Station ______________
Name of Stream ________________________________

DRAINAGE AREA DATA:

Size ____
Source ______________________________________
Topography:
Flat __________ Hilly __________ Mountainous __________
Vegetation:
Type ____
____
Ground Cover Density - Percent ____________________
Land Use:
Urban ________ Agricultural ________ Range ________ Desert ________
Shape:
Width ______________ Length ______________
Slope ____

SITE SURVEY DATA:

Check List:

- ________ Contour map of proposed structure site.
- ________ Channel profile.
- ________ Channel cross sections.
- ________ High water elevations.
- ________ Limiting backwater conditions.
- ________ Dimensioned sketch.
- ________ Photographs.

EXISTING STRUCTURES:

Size ______________________________

Location ______________________________

Effective Waterway ______________________________

Adequacy ______________________________

Flood Damage ______________________________

STREAM AND CHANNEL DATA:

Maximum HW Mark Elevation ______________________________

Amount and Character of Drift ______________________________

Channel Alignment ______________________________

Channel Bed Material ______________________________

Channel and Bank Erosion ______________________________

Bank Protection Requirements ______________________________

Local Flood Information ______________________________

______________________________ (Name)

______________ 19____ ______________________________ (Title)

ARIZONA DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

HYDRAULIC DESIGN SUMMARY
BRIDGES

LOCATION DATA

Highway ______________________ County ______________

Location ____

Project No. ______________________ Station ______________

Name of Stream ____

DESIGN SUMMARY

Drainage Area	________	Sq. Miles
Q Maximum of Record	________	cfs
Frequency	________	years
Stage	________	feet
Q Design	________	cfs
Frequency	________	years
Stage (Unconstricted)	________	feet
Bridge Length	________	feet
Structure Skew	________	%
Backwater	________	
Design Velocity	________	
Design HW Elevation	________	
Depth of Scour	________	

Bank Protection ____

____

____

____

Designed by ______________________ Date ______________

ARIZONA DEPARTMENT OF TRANSPORTATION

CULVERT COMPUTATION RECORD

PROJECT: ______________ DESIGNER: ______________

STATION: ______________ DATE: ______________

HYDROLOGIC AND CHANNEL INFORMATION

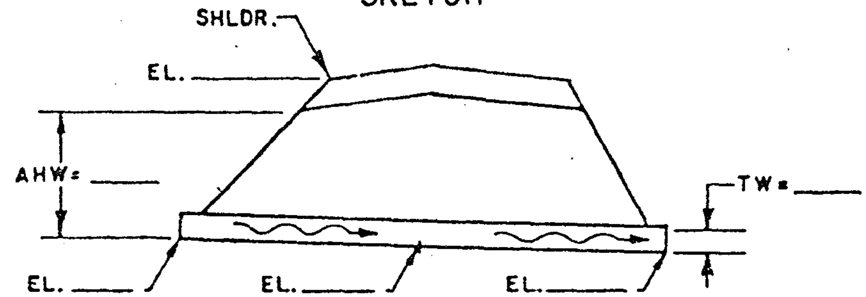
DRAINAGE AREA ______________

Q_1 = ________ TW_1 = ________

Q_2 = ________ TW_2 = ________

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

SKETCH



S_0 = ______________ L = ______________ 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			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c + D}{2}$	TW	h_o	LS_o	HW					

SUMMARY & RECOMMENDATIONS

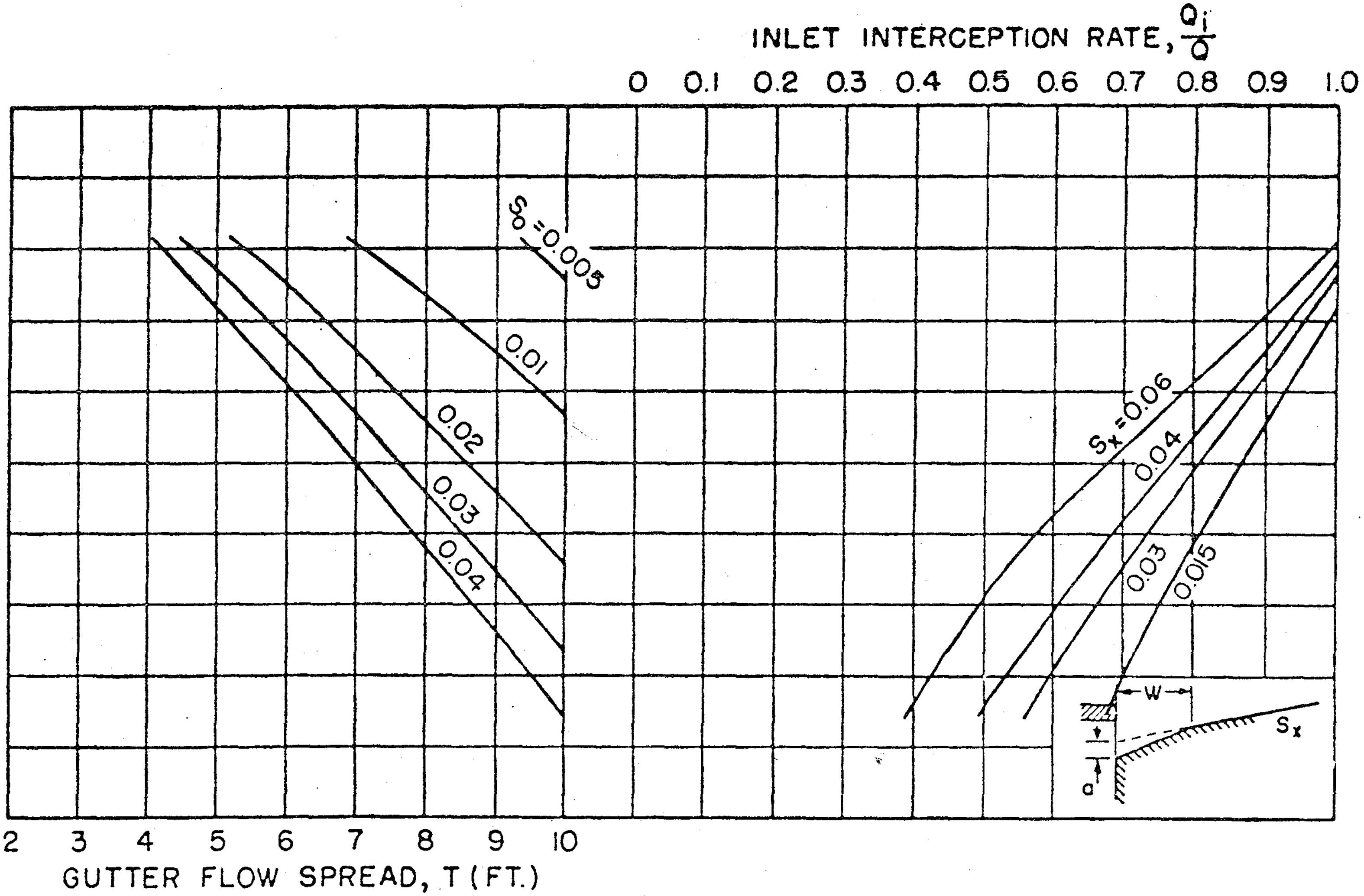


Figure 5.13 Curb Inlet Capacity Chart for General Condition (FHWA)

$W = 2'$ $a \geq 2'$ $n = 0.016$

Length of opening, $L_i = 15'$

Minimum height of curb opening, $h_m = T\ S_x$

<u>NOTE</u>: S_L (in text) = S_0 (this chart)

S_T (in text) = S_X (this chart)

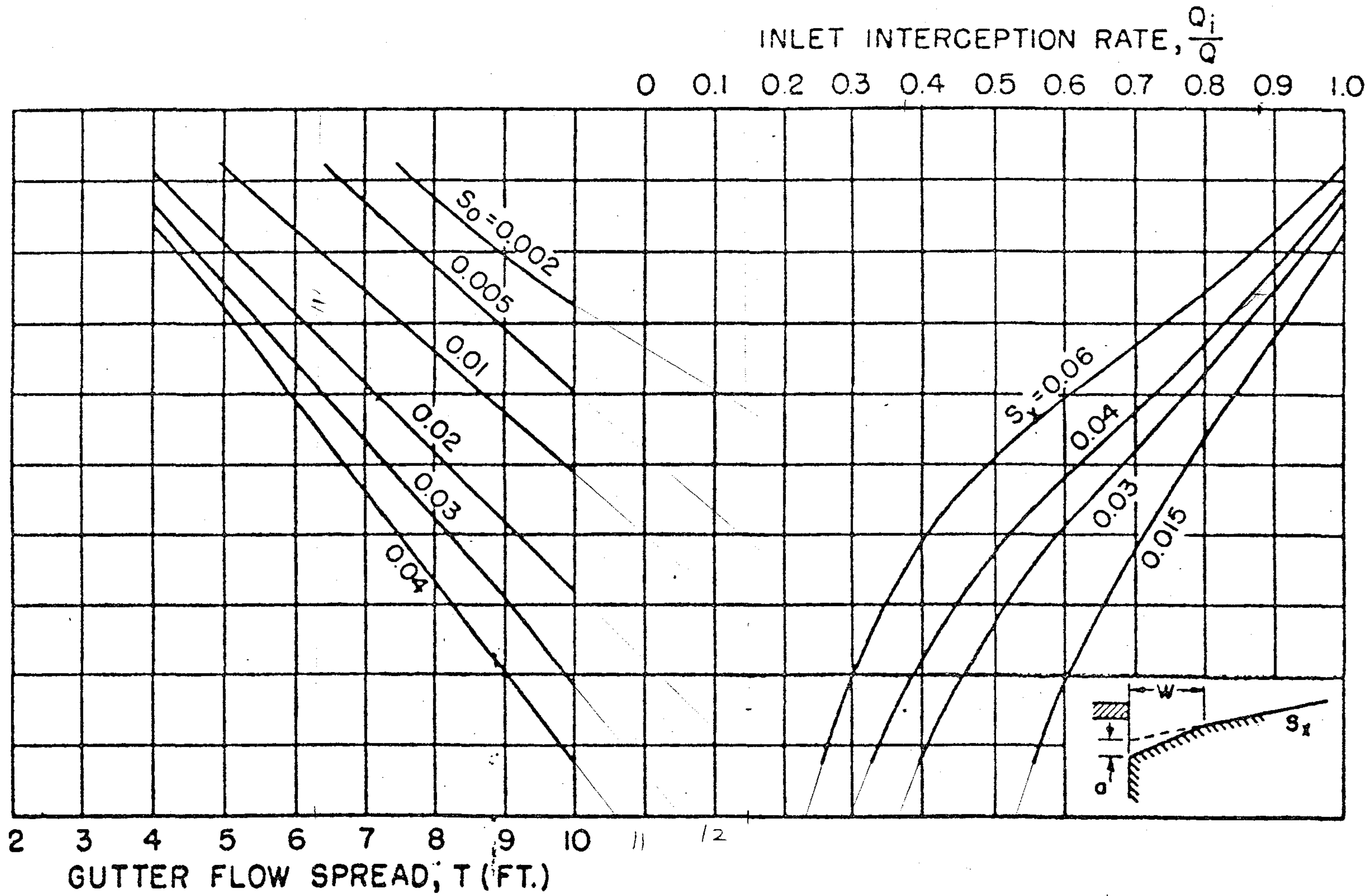


Figure 5.12 Curb Inlet Capacity Chart for General Condition (FHWA)

W = 2' a ≥ 2" n = 0.016

NOTE: S_L (in text) = S_0 (this chart)
S_T (in text) = S_x (this chart)

Length of opening, L_i = 10'

Minimum height of curb opening, $h_m = T\ S_x$

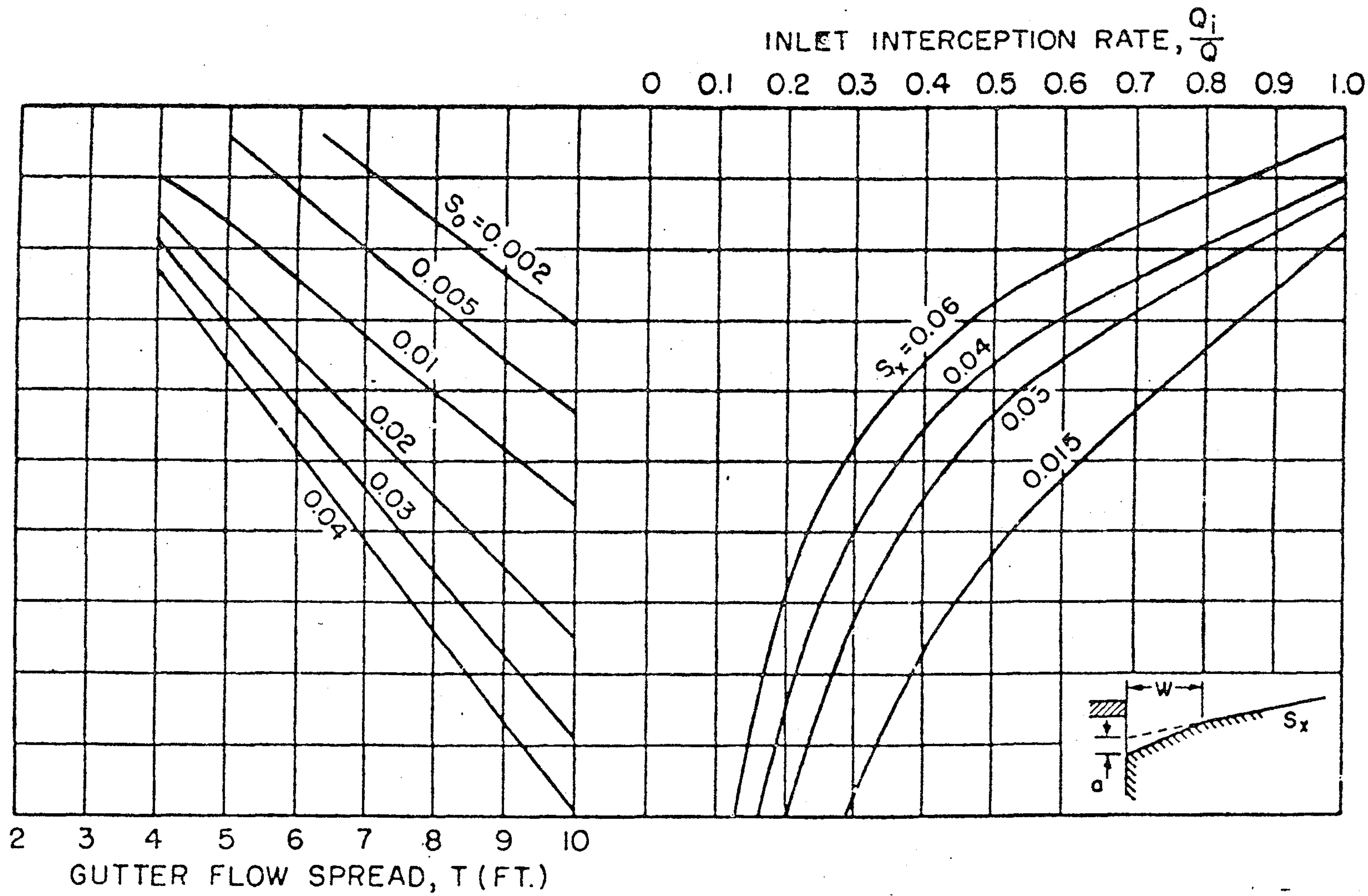


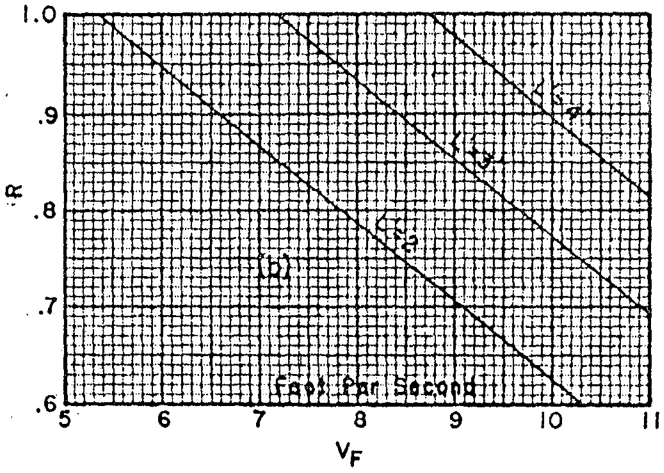
Figure 5.11 Curb Inlet Capacity Chart for General Condition (FHWA)

W = 2' a $\geq$ 2" n = 0.016

NOTE: S_L (in text) = S_o (this chart) Length of opening, L_i = 5'

S_T (in text) = S_x (this chart) Minimum height of curb opening, h_m = T S_x

GRATE INLET DESIGN CURVES



One foot is 0.3048m

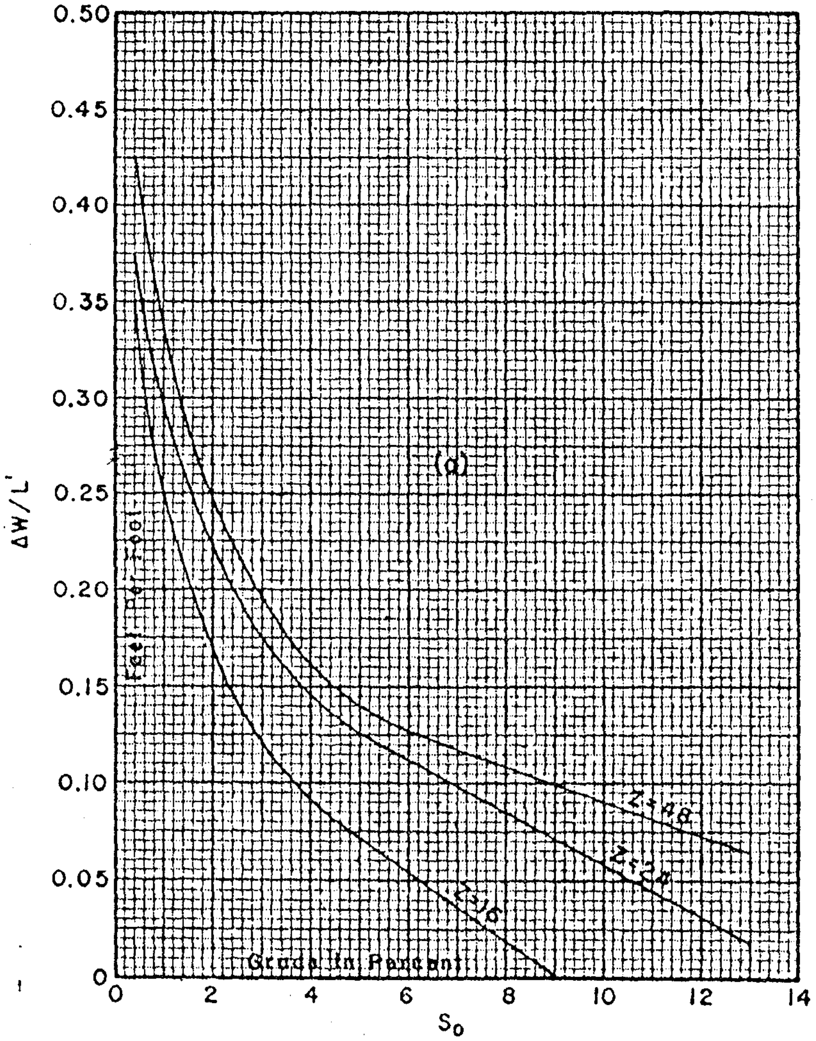


FIG. 5-27(a) & (b) P-1 7/8"-4" Grate

Dooley-Jones & Associates, Inc.

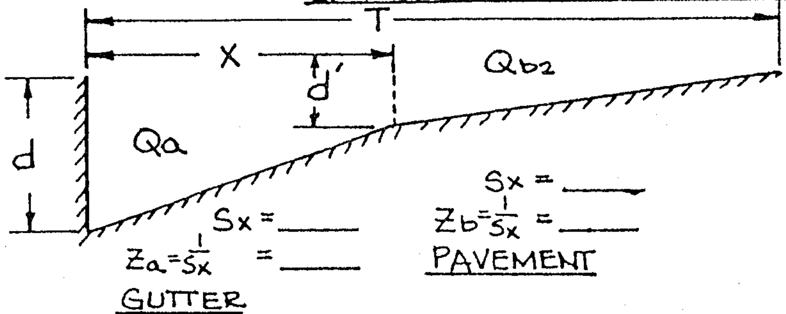
consulting engineers

Job No. ________ Sht. ______ of ______

Project ____________________ Calc. ________ Chk. ______ B'chk. ______

Date ________ Date ______ Date ______

GRATED INLET DESIGN



INLET LOCATION STA. __________ SHT. OF

LONGITUDINAL SLOPE ________. DESIGN DISCHARGE ________

GUTTER FLOW

	TRIAL 1	TRIAL 2	TRIAL 3
ASSUME d =			
$d' = d - \frac{X}{Z_a}$ =			
(USING Z_a) Q_{T1} =			
(USING Z_a) Q_{b1} =			
$Q_{T1} - Q_{b1} = Q_a$ =			
(USING Z_b) Q_{b2} =			
$Q_a + Q_{b2} = Q_{T2}$ =			

d = ________

$T = Z_b d' + X$ = ________

FRONTAL AREA = ________

FRONTAL Q = ________

FRONTAL V = ________

LENGTH OF GRATE = $L_b = \frac{V_F}{2}(d + \text{GRATE THICKNESS (FT.)})^{1/2}$ = ________

DEBRI CLOG FACTOR ______ X L_b = ________

OVERFLOW = $Q_T - Q_F$ = ________

COMMENTS: