Section 3. HYDRAULIC DESIGN

A. Weirs and Orifices

NOTE: Some of the graphs contained in this section are copied from the Los Angeles Hydraulics Manual and we wish to give them credit for their efforts. Also, applicable graphs are for 8” curb heights which may not meet Rio Rancho standards.

A.1. WEIRS

A weir is a barrier in an open channel, over which water flows. A weir with a sharp upstream corner or edge such that the water springs clear of the crest is a "sharp crested weir". All other weirs are classified as "weirs not sharp crested". Weirs are to be evaluated using the following equation:

$$Q = CLH^{3/2}$$

where:

Q = Discharge in cfs

C = Discharge coefficient from Handbook of Hydraulics, King and Brater, 5th Edition (or comparable)

L = Effective length of crest in feet

H = Depth of flow above elevation of crest in feet (approach velocity shall be disregarded in most applications)

Applications

Weirs are generally used as measuring and hydraulic control devices. Emergency spillways in which critical depth occurs and overflow-type roadway crossings of channels are the most common applications of weirs. Channel drop structures and certain storm drain inlets may also be analyzed as weirs. Special care must be exercised when selecting weir coefficients in the following cases:

a. Submerged weirs

b. Broad crested weirs

c. Weirs with obstructions (i.e., guardrails, piers, etc.)

A.2. ORIFICES

An orifice is a submerged opening with a closed perimeter through which water flows. Orifices are analyzed using the following equation:

$Q = CA \sqrt{2gh}$

where:

Q = Discharge in cfs

C = Coefficient of discharge from Handbook of Hydraulics, King and Brater, 5th Edition (or comparable)

A = Area of opening in square feet

g = 32.2 ft/sec

h = Depth of water measured from the center of the opening

Approach velocity shall be disregarded in most applications.

Applications

Orifices are generally used as measuring and hydraulic control devices. Orifice hydraulics control the function of many "submerged inlet - free outlet" culverts, primary spillways in detention facilities, manholes in conduit flow, and in storm drain inlets.

B. Criteria for Hydraulic Design: Closed Conduits

B.1. GENERAL HYDRAULIC CRITERIA

Closed conduit sections (pipe, box or arch sections) will be designed as flowing full and, whenever possible, under pressure except when the following conditions exist:

a. In some areas of high sediment potential, there is a possibility of stoppage occurring in drains. In situations where sediment may be expected, the City Engineer/SSCAFCA will use 18% for undeveloped conditions and 6% for developed conditions.

b. In certain situations, open channel sections upstream of the proposed closed conduit may be adversely affected by backwater.

If the proposed conduit is to be designed for pressure conditions, the hydraulic grade line shall not be higher than the ground or street surface, or encroach on the same in a reach where interception of surface flow is necessary. However, in those reaches where no surface flow will be intercepted, a hydraulic grade line which encroaches on or is slightly higher than the ground or street surface may be acceptable provided that pressure manholes exist or will be constructed.

B.2. WATER SURFACE PROFILE CALCULATIONS

a. Determination of Control Water Surface Elevation

A conduit to be designed for pressure conditions may discharge into one of the following:

(1) A body of water such as a detention reservoir

(2) A natural watercourse or arroyo

(3) An open channel, either improved or unimproved

(4) Another closed conduit

The controlling water surface elevation at the point of discharge is commonly referred to as the control and, for pressure flow, is generally located at the downstream end of the conduit.

Two general types of controls are possible for a conduit on a mild slope, which is a physical requirement for pressure flow in discharging conduits.

b. Control elevation above the soffit elevation. In such situations, the control must conform to the following criteria:

(1) In the case of a conduit discharging into a detention facility, the control is the 100 -year water surface reservoir elevation.

(2) In the case of a conduit discharging into an open channel, the control is the 100-year design water surface elevation of the channel.

(3) In the case of a conduit discharging into another conduit, the control is the design hydraulic grade line elevation of the outlet conduit immediately upstream of the confluence.

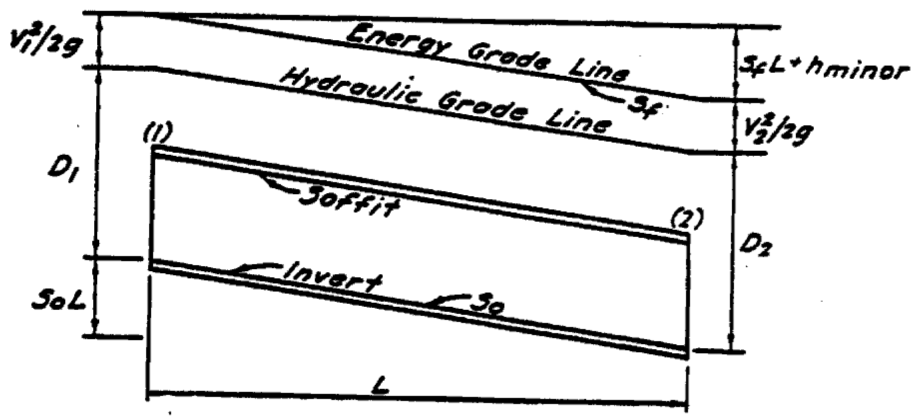
Whenever case (1) or (2) above is used, the possibility of having flow out of manholes or inlets due to discharge elevations at the 100-year level must be investigated and appropriate steps taken to prevent its occurrence.

c. Control elevation at or below the soffit elevation. The control is the soffit elevation at the point of discharge. This condition may occur in any one of the four situations described above in 2b.

d. Instructions for Hydraulic Calculations

Most procedures for calculating hydraulic grade line profiles are based on the Bernoulli equation. This equation can be expressed as follows:

$$\frac{V_1^2}{2g} + D_1 + S_o L = \frac{V_2^2}{2g} + D_2 + S_f L + h_{minor}$$



in which D = Vertical distance from invert to H.G.L
So = Invert slope
L = Horizontal projected length of conduit
Sg = Average friction slope between Sections 1 and 2
V = Average velocity (g/A)
h_{minor} = Minor head losses

Minor head losses have been included in the Bernoulli equation because of their importance in calculating hydraulic grade line profiles and are assumed to be uniformly distributed in the above figure.

When specific energy (E) is substituted for the quantity $\left(\frac{V^2}{2g} + D\right)$ in the above equation and minor head losses are ignored and the result rearranged,

$$L = \frac{E2 - E1}{S_o - S_f}$$

The above is a simplification of a more complex equation and is convenient for locating the approximate point where pressure flow may become unsealed.

e. Head Losses

(1) Friction Loss

Friction losses for closed conduits carrying storm water, including pump station discharge lines, will be calculated from the Manning equation or a derivation thereof. The Manning equation is commonly expressed as follows:

$$Q = \frac{1.486}{n} AR^{2/3} S_f^{1/2}$$

in which Q = Discharge, in c.f.s.

n = Roughness coefficient

A = Area of water normal to flow in ft.

R = Hydraulic radius

Sf = Friction slope

When rearranged into a more useful form,

in which

$$S_f = \left[\frac{Qn}{1.486AR^{2/3}} \right]^2 = \left[\frac{Q}{K} \right]^2$$

in which:

$$K = \frac{1.486\ AR^{2/3}}{n}$$

The loss of head due to friction throughout the length of reach (L) is calculated by:

$$h_f = S_f L = \left[\frac{Q}{K} \right]^2 L$$

The value of K is dependent upon only two factors: the geometrical shape of the flow cross section as expressed by the quantity ($AR^{2/3}$), and the roughness coefficient (n). The values of n are shown in Plate 22.3 B-1.

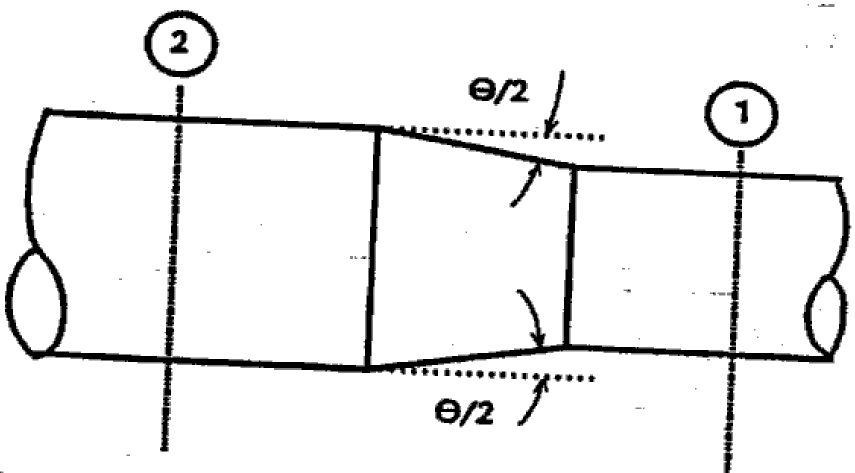
PLATE 22.3 B-1 VALUES OF MANNING'S n

	n
Tined Concrete	0.018
Shotcrete	0.025
Reinforced Concrete Pipe	0.013
Troweled Concrete	0.013
No-joint cast in place concrete pipe	0.014
Reinforced Concrete Box	0.015
Reinforced Concrete Arch	0.015
Streets	0.017
Flush Grouted Riprap	0.020
Corrugated Metal Pipe	0.025
Grass Lined Channels (sodded & irrigated)	0.025
Earth Lined Channels (smooth)	0.030
Wire Tied Riprap	0.040
Medium Weight Dumped Riprap	0.045
Grouted Riprap (exposed rock)	0.045
Jetty Type Riprap (D_{50} > 24")	0.050

See SSCAFCA's Sediment and Erosion Design guide for recommended Manning's n values for naturalistic channels. For materials not listed contact City Engineer/SSCAFCA prior to use.

(2) Transition Loss

Transition losses will be calculated from the equations shown below. These equations are applicable when no change in Q occurs and where the horizontal angle of divergence or convergence (θ /2) between the two sections does not exceed 5 degrees 45 minutes.



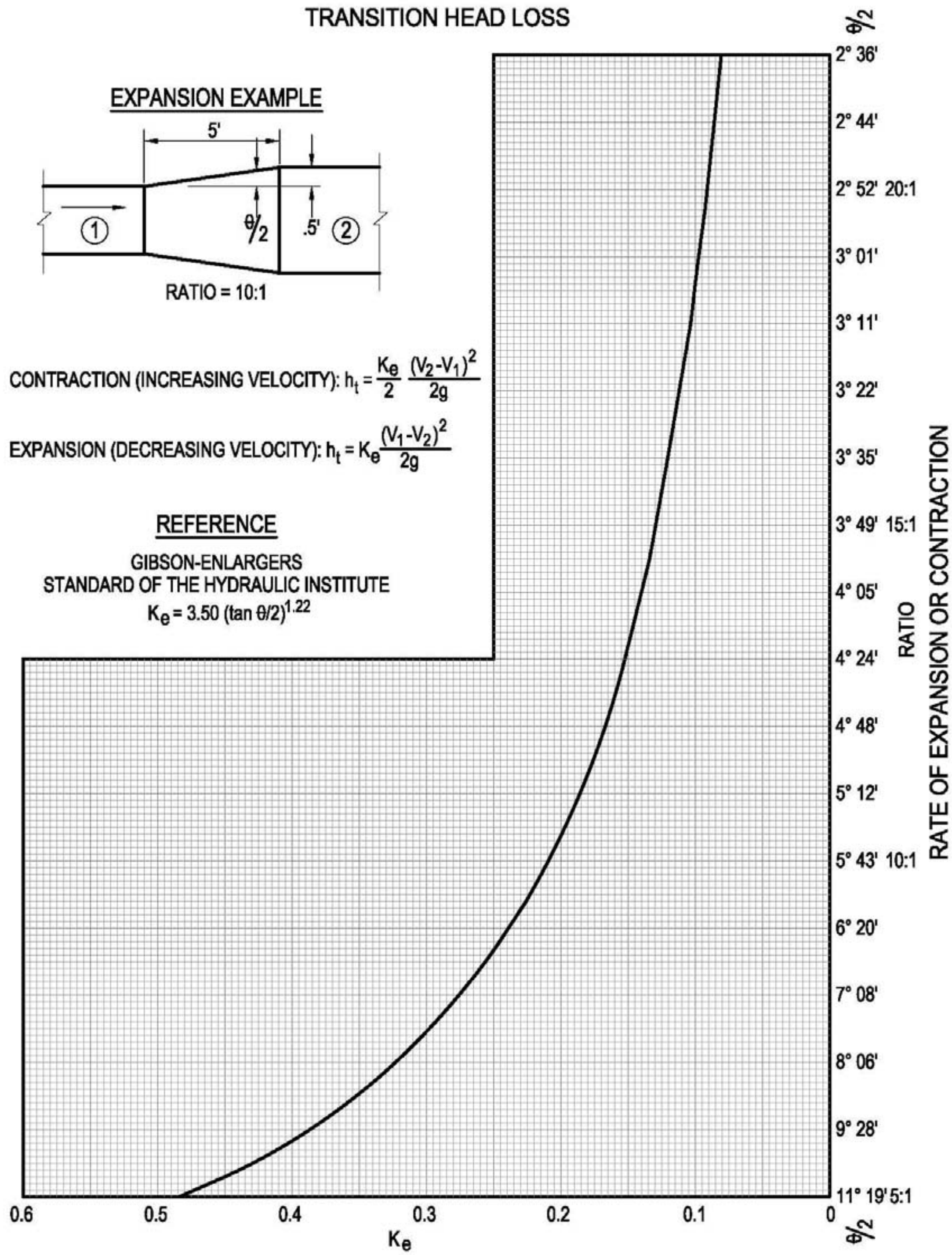
For increasing velocities in the direction of flow from 2 to 1

$$h_t = 0.1 \left[\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right]$$

For decreasing velocities in the direction of flow from 1 to 2

$$h_t = 0.2 \left[\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right]$$

Deviations from the above criteria must be approved by the City Engineer/SSCAFCA. When such situations occur, the angle of divergence or convergence (θ/2) may be greater than 5 degrees 45 minutes. However, when it is increased beyond 5 degrees 45 minutes, the above equation will give results for h_t that are too small, and the use of more accurate methods, such as the Gibson method shown Plate 22.3 B-2, will be acceptable.



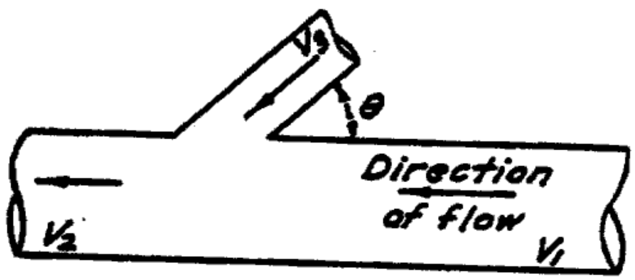
TRANSITION HEAD LOSS
EXPANSION EXAMPLE
5'
θ/2
.5'
①
②
RATIO = 10:1
CONTRACTION (INCREASING VELOCITY): $h_t = \frac{K_e}{2} \frac{(V_2-V_1)^2}{2g}$
EXPANSION (DECREASING VELOCITY): $h_t = K_e \frac{(V_1-V_2)^2}{2g}$
REFERENCE
GIBSON-ENLARGERS
STANDARD OF THE HYDRAULIC INSTITUTE
$K_e = 3.50 (\tan \theta/2)^{1.22}$
θ/2
2° 36'
2° 44'
2° 52' 20:1
3° 01'
3° 11'
3° 22'
3° 35'
3° 49' 15:1
4° 05'
4° 24'
4° 48'
5° 12'
5° 43' 10:1
6° 20'
7° 08'
8° 06'
9° 28'
11° 19' 5:1
RATIO
RATE OF EXPANSION OR CONTRACTION
0.6
0.5
0.4
0.3
0.2
0.1
0
K_e

PLATE 22.3 B-2

(3) Junction Losses

In general, junction losses are calculated by equating pressure plus momentum through the confluences under consideration. This can be done by using either the P + M method or the Thompson equation, both of which are shown in Section 22, Section 8. Both methods are applicable in all cases for pressure flow and will give the same results.

For the special case of pressure flow with $A_1 = A_2$ and friction neglected,



$$h_j = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} - \frac{2A_3}{A_2} \cdot \frac{V_3^2}{2g} \cdot \cos\theta$$

(4) Manhole Loss

Manhole losses will be calculated from the equation shown below. Where a change in pipe size and/or change in Q occurs, the head loss will be calculated in accordance with Sections (2) and (3), preceding.

$$h_{m.h.} = .05 \left[\frac{V^2}{2_g} \right]$$

(5) Bend Loss

Bend losses will be calculated from the following equations:

$$h_b = K_b \left[\frac{V^2}{2_g} \right]$$

BEND LOSSES

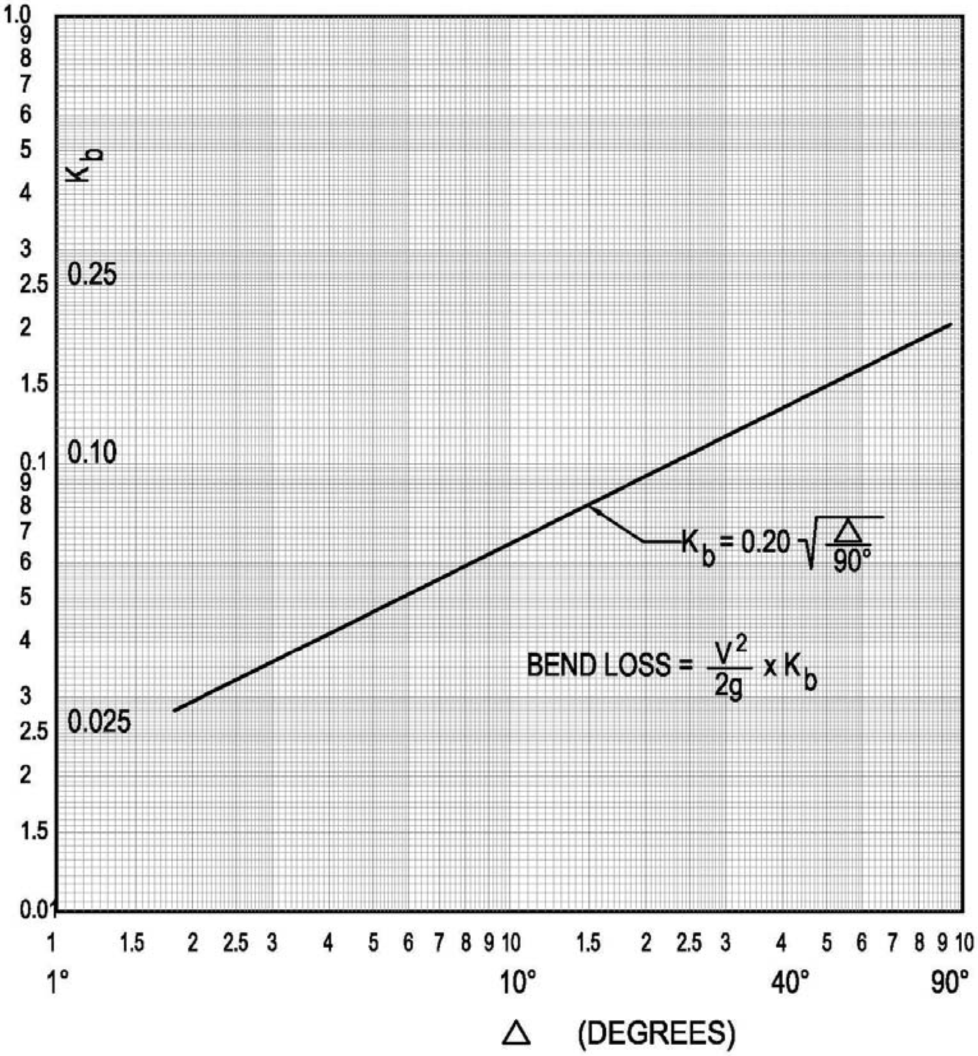


PLATE 22.3 B-3

in which:

$$K_b = 0.20 \sqrt{\frac{\Delta}{90^\circ}}$$

where Δ = Central angle of bend in degrees

K_b may be evaluated graphically from 22.3 B-3 for values of not exceeding 90 degrees.

Bend losses should be included for all closed conduits, those flowing partially full as well as those flowing full.

(6) Angle Point Loss

Angle point losses shall be calculated from the following equation:

$$h_{a.pt.} = .0033\, \Theta \left[\frac{V^2}{2_g}\right]$$

in which Θ = Deflection angle in degrees, not to exceed 6^0 without prior approval.

B.3. SPECIAL CASES

a. Transition From Large to Small Conduit

As a general rule, storm drains will be designed with sizes increasing in the downstream direction. However, when studies indicate it may be advisable to decrease the size of a downstream section, the conduit may be decreased in size in accordance with the following limitations:

(1) For slopes of .0025 (.25 percent) or less, conduit sizes may be decreased to a minimum diameter of 72 inches. Each reduction is limited to a maximum of 6 inches.

(2) For slopes of more than .0025, conduit sizes may be decreased to a minimum diameter of 30 inches. Each reduction is limited to a maximum of 3 inches for pipe 48 inches in diameter or smaller, and to a maximum of 6 inches for pipe larger than 48 inches in diameter. Reductions exceeding the above criteria must have prior City Engineer/SSCAFCA approval.

In any case the reduction in size must result in a more economical system.

Where conduits are to be decreased in size due to a change in grade, the criteria for locating the transition will be as shown on Plate 22.3 B-4.

B.4. DESIGN REQUIREMENTS FOR MAINTENANCE AND ACCESS

a. Manholes

(1) Spacing

Where the proposed conduit is 60" and larger, manholes should be spaced at intervals of approximately 800 feet to 1000 feet. Where the proposed conduit is less than 60 inches in diameter and the horizontal alignment has numerous bends or angle points, the manhole spacing should be reduced to approximately 500 feet.

The spacing requirements shown above apply regardless of design velocities. Deviations from the above criteria are subject to City Engineer/SSCAFCA approval.

(2) Location

Manholes should be located outside of street intersections wherever possible, especially when one or more streets are heavily traveled.

In situations where the proposed conduit is to be aligned both in easement and in street right-of-way, manholes should be located in street right-of-way, wherever possible.

Manholes should be located as close to changes in grade as feasible when the following conditions exist:

(a) When the upstream conduit has a steeper slope than the downstream conduit and the change in grade is greater than 10 percent, sediment tends to deposit at the point where the change in grade occurs.

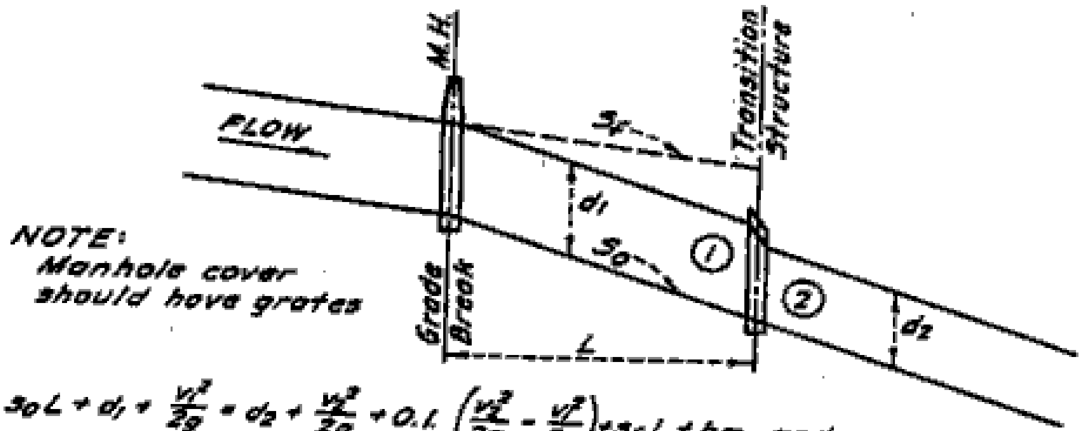
(b) When transitioning to a smaller downstream conduit due to an abruptly steeper slope downstream, sediment tends to accumulate at the point of transition.

(3) Design

When the design flow in a pipe flowing full has a velocity of 20 f.p.s. or greater, or is supercritical in a partially full pipe, the total horizontal angle of divergence or convergence between the walls of the manhole and its center line should not exceed 5º45'.

LOCATION OF TRANSITION

Large to Small Conduit



$$s_oL + d_1 + \frac{v_1^2}{2g} = d_2 + \frac{v_2^2}{2g} + 0.1\left(\frac{v_2^2}{2g} - \frac{v_1^2}{2g}\right) + s_fL + hm, \text{ and}$$

$$s_oL - s_fL = d_2 - d_1 + 1.1\left(\frac{v_2^2}{2g} - \frac{v_1^2}{2g}\right) + hm \text{..... therefore:}$$

$$L = \frac{d_2 - d_1 + 1.1\left(\frac{v_2^2}{2g} - \frac{v_1^2}{2g}\right) + hm}{s_o - s_f}$$

where
- s_o = slope of conduit
- s_f = friction slope of larger conduit
- d_1 = diameter or depth of larger conduit
- v_1 = velocity in larger conduit flowing full
- d_2 = diameter or depth of smaller conduit
- v_2 = velocity in smaller conduit flowing full
- h_m = other losses occuring between the transition and the grade break such as bend and confluence losses

EXAMPLE PROBLEM

$Q = 400$ cfs

$d_1 = 84'' = 7'$	$d_2 = 78'' = 6.5'$
$A_1 = 38.49$ sq. ft.	$A_2 = 33.18$ sq. ft.
$v_1 = 10.4$ fps	$v_2 = 12.0$ fps
$\frac{v_1^2}{2g} = 1.68'$	$\frac{v_2^2}{2g} = 2.24'$

$S_o = .00474$
$S_f = .00395$

$$L = \frac{6.5 - 7.0 + 1.1(2.24 - 1.68)}{.00474 - .00395} = 147$$

PLATE 22.3 B-4

b. Pressure Manholes

Pressure manholes should be avoided whenever possible. When unavoidable a pressure manhole shaft and a pressure frame and cover will be installed in a pipe or box storm drain whenever the design water surface is at the ground surface.

c. Special Manholes

Special 36-inch diameter manholes or vehicular access structures will be provided when required. The need for access structures will be determined by the City Engineer/SSCAFCA during the review of preliminary plans.

d. Deep Manholes

A manhole shaft safety ledge or other structural designs should be considered when the manhole shaft is 20 feet or greater in depth. Installation will be in accordance with City Engineer/SSCAFCA requirements.

e. Inlets into Main Line Drains

Lateral pipe entering a main line pipe storm drain generally will be connected radially. Lateral pipe entering a main line structure will conform to the following:

(1) The invert of lateral pipe 24 inches or less in diameter will be no more than five feet above the invert.

(2) The invert of lateral pipe 27 inches or larger in diameter will be no more than 18 inches above the invert, with the exception that storm inlet connector pipe less than 50 feet in length may be no more than five feet above the invert.

Exceptions to the above requirements may be permitted where it can be shown that the cost of bringing laterals into a main line conduit in conformance with the above requirements would be excessive.

f. Minimum Pipe Size

In cases where the conduit may carry significant amounts of sediment, the minimum diameter of main line conduit will be 24 inches.

g. Minimum Slope

The minimum slope for main line conduit will be 0.003 (0.30 percent), unless otherwise approved by the City Engineer/SSCAFCA. Minimum flow velocity for ¼ full pipe will be 2 f.p.s.

h. Inlet Structures

An inlet structure will be provided for storm drains located in natural channels. The structure should generally consist of a headwall, wingwalls to protect the adjacent banks from erosion, and a paved inlet apron. The apron slope should be limited to a maximum of 2:1. Wall heights should conform to the height of the water upstream of the inlet, and be adequate to protect both the fill over the drain and the embankments. Headwall and wingwall fencing and a protection barrier to prevent public entry will be provided.

If trash and debris are prevalent, barriers consisting of vertical 3-inch or 4-inch diameter steel pipe at 24 inches to 36 inches on centers should be embedded in concrete immediately upstream of the inlet apron. Trash rack designs must have City Engineer/SSCAFCA approval.

i. Outlet Structures

(1) Where a storm drain discharges into a detention reservoir, the designer should check with the City Engineer/SSCAFCA for up-to-date criteria as to location and type of structure to be used.

(2) When a storm drain outlets into a natural channel, an outlet structure will be provided which prevents erosion and property damage. Velocity of flow at the outlet should match as closely as possible with the existing channel velocity. Fencing and a protection barrier will be provided where deemed necessary by the City Engineer/SSCAFCA.

(a) When the discharge velocity is low, or subcritical, the outlet structure will consist of a headwall, wingwalls, and an apron. The apron may consist of a concrete slab, grouted rock, or well designed dumped riprap depending on conditions.

(b) When the discharge velocity is high, or supercritical, the designer will, in addition, design bank protection in the vicinity of the outlet and an energy dissipater structure. The City Engineer/SSCAFCA will furnish, upon request, guidance on types of energy dissipators appropriate for each application.

j. Protection Barriers

A protection barrier is a means of preventing people from entering storm drains. Protection barriers will be provided wherever necessary to prevent unauthorized access to storm drains. In some cases the barrier may be one of the breakaway type. In other cases the barrier may be a special design. It will be the designer's responsibility to provide a protection barrier appropriate to each situation and to provide details of such on the construction drawings.

k. Debris Barriers

A debris barrier or deflector is a means of preventing large debris or trash, such as tree limbs, logs, boulders, weeds, and refuse, from entering a storm drain and possibly plugging the conduit. The debris barrier should have openings wide enough to allow as much small debris as possible to pass through and yet narrow enough to protect the smallest conduit in the system downstream of the barrier. One type that has been used effectively in the past is the debris rack. This type of debris barrier is usually formed by a line of posts, such as steel pipe filled with concrete or steel rails, across the line of flow to the inlet. Other examples of barriers are presented in Hydraulic Engineering Circular No. 9, "Debris-Control Structures," published by the United States Department of Commerce, Bureau of Public Roads, which is available upon request from its Office of Engineering and Operations. It will be the designer's responsibility to provide a debris barrier or deflector appropriate to the situation.

l. Debris Basins

Debris basins, check dams and similar structures are a means of preventing mud, boulders and debris held in suspension and carried along by storm runoff from depositing in storm drains. Debris basins constructed upstream of storm drain conduits, usually in canyons, trap such material before it reaches the conduit. Debris basins must be cleaned out on a regular basis, however, if they are to continue to function effectively. Refer to the City Engineer/SSCAFCA and State Engineer regarding the criteria to be used in designing these structures.

m. Safety

Entry into any of these structures should be in accordance with OSHA requirements.

B.5. OTHER CLOSED CONDUIT CRITERIA

a. Angle of Confluence

In general, the angle of confluence between main line and lateral must not exceed 45 degrees and, as an additional requirement, must not exceed 30 degrees under any of the following conditions:

(1) Where the peak flow (Q) in the proposed lateral exceeds 10 percent of the main line peak flow.

(2) Where the velocity of the peak flow in the proposed lateral is 20 f.p.s. or greater.

(3) Where the size of the proposed lateral is 60 inches or greater.

(4) Where hydraulic calculations indicate excessive head losses may occur in the main line due to the confluence.

Connector pipe may be joined to main line pipe at angles greater than 45 degrees up to a maximum of 90 degrees provided none of the above conditions exist. If, in any specific situation, one or more of the above conditions does apply, the angle of confluence for connector pipes may not exceed 30 degrees. Connections must not be made to main line pipe which may create conditions of adverse flow in the connector pipes without prior approval from the City Engineer/SSCAFCA.

The above requirements may be waived only if calculations are submitted to the City Engineer/SSCAFCA showing that the use of a confluence angle larger than 30 degrees will not unduly increase head losses in the main line.

b. Flapgates (FLAPGATES ARE DISCOURAGED AND WILL ONLY BE USED ON A CASE BY CASE BASIS AND WITH APPROVAL FROM THE CITY ENGINEER/SSCAFCA)

A flapgate must be installed in all laterals outletting into a main line storm drain whenever the potential water surface level of the main line is higher than the surrounding area drained by the lateral.

The flapgate must be set back from the main line drain so that it will open freely and not interfere with the main line flow. A junction structure will be constructed for this purpose in accordance with City Engineer/SSCAFCA standards.

c. Rubber-Gasketed Pipe

Rubber-gasketed pipe will be used in all storm drain construction unless otherwise approved by the City Engineer/SSCAFCA.

d. Non-Reinforced Concrete Pipe

Non-reinforced concrete pipe may not be used for storm drain applications.

e. Junctions

Junctions will only be permitted on mains storm drain lines that are $\geq$42 inches. Junction locations cannot be more that 24' from the downstream manhole. An exception to this requirement may be laterals with slopes of 5% or greater. The City Engineer/SSCAFCA approval will be required for this exception and all other variances.

PLATE 22.3 B-5 FACTORS FOR CLOSED CONDUITS FLOWING FULL

Manning's Formula: $Q = \dfrac{1.486}{n} AR^{2/3}s^{1/2}$

$K = \dfrac{Q}{S^{1/2}} = \dfrac{1.486\ AR^{2/3}}{0.013}$, for pipe $K = 35.6259\ d^{8/3}$, for box $K = 114.3077\ \dfrac{A^{5/3}}{p2/3}$

$Q = K\ s^{1/2}$

$s = \left[\dfrac{Q}{K}\right]^2$

Where:

Q	=	discharge in cfs
s	=	friction slope
A	=	area of conduit
R	=	hydraulic radius of conduit
n	=	0.013
d	=	diameter of pipe
"	=	height of equivalent box
w	=	width of equivalent box
p	=	wetted perimeter

PLATE 22.3 B-5

PIPE & BOX		PIPE		EQUIVALENT BOX			
d		A	K	w		A	K
ft.	in.	sq.ft.		ft.-in.	ft.	sq. ft.	
1.25	15	1.227	64.6				
.50	18	1.767	105.0				
.75	21	2.405	158.4				
2.00	24	3.142	226.2				
.25	27	3.976	309.7				
.50	30	4.909	410.1				
.75	33	5.939	528.7				
3.00	36	7.068	666.9				
.25	39	8.295	825.8				
.50	42	9.621	1,006				
.75	45	11.044	1,209				
4.00	48	12.566	1,436				
.25	51	14.186	1,688				
.50	54	15.904	1,967				
.75	57	17.721	2,272				
5.00	60	19.635	2,604				
.25	63	21.648	2,966				
.50	66	23.758	3,358				
.75	69	25.967	3,780				
6.00	72	28.274	4,236				
.25	75	30.680	4,720				
.50	78	33.183	5,244				
.75	81	35.785	5,796				
7.00	84	38.485	6,388	5'-10"	5.83	40.3	6,357
.25	87	41.283	7,015				
.50	90	44.179	7,677	6'-4"	6.33	47.0	7,780
.75	93	47.173	8,379				
8.00	96	50.266	9,120	6'-9"	6.75	53.5	9,256
.50	102	56.745	10,720	7'-1"	7.08	59.7	10,685
9.00	108	63.617	12,487	7'-6"	7.50	67.0	12,452
.50	114	70.882	14,421	8'-0"	8.00	75.4	14,598
10.00	120	78.540	16,538	8'-5"	8.42	83.6	16,726
.50	126	86.590	18,835	8'-10"	8.83	92.1	19,026
11.00	132	95.033	21,322	9'-2"	9.17	100.3	21,303
.50	138	103.879	24,005	9'-7"	9.58	109.5	23,954
12.00	144	113.098	26,890	10'-0"	10.00	119.4	26,849

PARTIALLY FILLED CIRCULAR CONDUIT SECTIONS

D/d	area/d^2	wet. per/d	hyd.rad/d	D/d	area/d^2	wet. per/d	hyd. rad/d
0.01	0.0013	0.2003	0.0066	0.51	0.4027	1.5908	0.2531
0.02	0.0037	0.2838	0.0132	0.52	0.4127	1.6108	0.2561
0.03	0.0069	0.3482	0.0197	0.53	0.4227	1.6308	0.2591
0.04	0.0105	0.4027	0.0262	0.54	0.4327	1.6509	0.2620
0.05	0.0147	0.4510	0.0326	0.55	0.4426	1.6710	0.2649
0.06	0.0192	0.4949	0.0389	0.56	0.4526	1.6911	0.2676
0.07	0.0242	0.5355	0.0451	0.57	0.4625	1.7113	0.2703
0.08	0.0294	0.5735	0.0513	0.58	0.4723	1.7315	0.2728
0.09	0.0350	0.6094	0.0574	0.59	0.4822	1.7518	0.2753
0.10	0.0409	0.6435	0.0635	0.60	0.4920	1.7722	0.2776
0.11	0.0470	0.6761	0.0695	0.61	0.5018	1.7926	0.2797
0.12	0.0534	0.7075	0.0754	0.62	0.5115	1.8132	0.2818
0.13	0.0600	0.7377	0.0813	0.63	0.5212	1.8338	0.2839
0.14	0.0668	0.7670	0.0871	0.64	0.5308	1.8546	0.2860
0.15	0.0739	0.7954	0.0929	0.65	0.5404	1.8755	0.2881
0.16	0.0811	0.8230	0.0986	0.66	0.5499	1.8965	0.2899
0.17	0.0885	0.8500	0.1042	0.67	0.5594	1.9177	0.2917
0.18	0.0961	0.8763	0.1097	0.68	0.5687	1.9391	0.2935
0.19	0.1039	0.9020	0.1152	0.69	0.5780	1.9606	0.2950
0.20	0.1118	0.9273	0.1206	0.70	0.5872	1.9823	0.2962
0.21	0.1199	0.9521	0.1259	0.71	0.5964	2.0042	0.2973
0.22	0.1281	0.9764	0.1312	0.72	0.6054	2.0264	0.2984
0.23	0.1365	1.0003	0.1364	0.73	0.6143	2.0488	0.2995
0.24	0.1449	1.0239	0.1416	0.74	0.6231	2.0714	0.3006
0.25	0.1535	1.0472	0.1466	0.75	0.6318	2.0944	0.3017
0.26	0.1623	1.0701	0.1516	0.76	0.6404	2.1176	0.3025
0.27	0.1711	1.0928	0.1566	0.77	0.6489	2.1412	0.3032
0.28	0.1800	1.1152	0.1614	0.78	0.6573	2.1652	0.3037
0.29	0.1890	1.1373	0.1662	0.79	0.6655	2.1895	0.3040
0.30	0.1982	1.1593	0.1709	0.80	0.6736	2.2143	0.3042
0.31	0.2074	1.1810	0.1755	0.81	0.6815	2.2395	0.3044
0.32	0.2167	1.2025	0.1801	0.82	0.6893	2.2653	0.3043
0.33	0.2260	1.2239	0.1848	0.83	0.6969	2.2916	0.3041
0.34	0.2355	1.2451	0.1891	0.84	0.7043	2.3186	.03038
0.35	0.2450	1.2661	0.1935	0.85	0.7115	2.3462	0.3033
0.36	0.2546	1.2870	0.1978	0.86	0.7186	2.3746	0.3026
0.37	0.2642	1.3078	0.2020	0.87	0.7254	2.4038	0.3017
0.38	0.2739	1.3284	0.2061	0.88	0.7320	2.4341	0.3008
0.39	0.2836	1.3490	0.2102	0.89	0.7384	2.4655	0.2996
0.40	0.2934	1.3694	0.2142	0.90	0.7445	2.4981	0.2980
0.41	0.3032	1.3898	0.2181	0.91	0.7504	2.5322	0.2963
0.42	0.3130	1.4101	0.2220	0.92	0.7560	2.5681	0.2944
0.43	0.3229	1.4303	0.2257	0.93	0.7642	2.6061	0.2922
0.44	0.3328	1.4505	0.2294	0.94	0.7662	2.6467	0.2896
0.45	0.3428	1.4706	0.2331	0.95	0.7707	2.6906	0.2864
0.46	0.3527	1.4907	0.2366	0.96	0.7749	2.7389	0.2830
0.47	0.3627	1.5108	0.2400	0.97	0.7785	2.7934	0.2787
0.48	0.3727	1.5308	0.2434	0.98	0.7816	2.8578	0.2735
0.49	0.3827	1.5508	0.2467	0.99	0.7841	2.9412	0.2665
0.50	0.3927	1.5708	0.2500	1.00	0.7854	3.1416	0.2500

FACTORS FOR CIRCULAR CONDUITS FLOWING PARTLY FULL

D = depth of water
d = diameter of conduit

Tabulated Values

$K = \frac{\text{momentum}}{(Q/d)^2}$ $C = \frac{\text{pressure}}{d^3}$ $F = \frac{\text{Velocity Head}}{(Q/d^2)^2}$

D/d	K	C	F	D/d	K	C	F	D/d	K	C	F	D/d	K	C	F
.00	00	.0000	00	.25	.2026	.0157	0.659	.50	.0792	.0833	.1007	.75	.0492	.2121	.0389
.01	23.919	.0000	9188.	.26	.1916	.0173	0.589	.51	.0773	.0873	.0958	.76	.0485	.2185	.0379
.02	8.403	.0000	1134.	.27	.1817	.0190	0.530	.52	.0753	.0914	.0912	.77	.0479	.2249	.0369
.03	4.507	.0001	326.	.28	.1727	.0207	0.479	.53	.0736	.0956	.0869	.78	.0473	.2314	.0359
.04	2.961	.0002	140.9	.29	.1645	.0226	0.435	.54	.0719	.0998	.0829	.79	.0467	.2380	.0351
.05	2.115	.0003	71.9	.30	.1569	.0255	0.395	.55	.0703	.4042	.0793	.80	.0462	.2447	.0342
.06	1.620	.0005	42.1	.31	.1493	.0266	0.361	.56	.0687	.1087	.0758	.81	.0456	.2515	.0334
.07	1.285	.0007	26.5	.32	.1435	.0287	0.331	.57	.0672	.1133	.0726	.82	.0451	.2584	.0327
.08	1.058	.0010	17.97	.33	.1376	.0309	0.304	.58	.0658	.1179	.0696	.83	.0446	.2653	.0320
.09	0.888	.0013	12.68	.34	.1320	.0332	0.280	.59	.0645	.1227	.0668	.84	.0441	.2723	.0313
.10	0.760	.0017	9.28	.35	.1269	.0356	0.259	.60	.0632	.1276	.0641	.85	.0437	.2794	.0.07
.11	0.662	.0021	7.03	.36	.1221	.0381	0.240	.61	.0620	.1326	.0617	.86	.0433	.2865	.0301
.12	0.582	.0026	5.45	.37	.1177	.0407	0.222	.62	.0608	.1376	.0594	.87	.0429	.238	.0295
.13	0.518	.0032	4.31	.38	.1135	.0434	0.207	.63	.0597	.1428	.0572	.88	.0425	.3011	.0290
.14	0.466	.0038	3.48	.39	.1096	.0462	.1931	.64	.0586	.1481	.0551	.89	.0421	.3084	.0285
.15	0.421	.0045	2.84	.40	.1060	. 0491	.1804	.65	.0575	.1534	.0532	.90	.0418	.3158	.0280
.16	0.383	.0053	2.36	.41	.1026	.0520	.1689	.66	.0565	.1589	.0514	.91	.0414	.3222	.0276
.17	0.351	.0064	1.982	.42	.0993	.0551	.1585	.67	.0559	.1644	.0496	.92	.0411	.3308	.0272
.18	0.325	.0070	1.681	.43	.0963	.0583	.1489	.68	.0547	.1700	.0480	.93	.0408	.3384	.0266
.19	0.299	.0080	1.438	.44	.0934	.0616	.1402	.69	.0538	.1758	.0465	.94	.0406	.3460	.0265
.20	0.278	.0091	1.242	.45	.0907	.0650	.1321	.70	.0530	.1816	.0450	.95	.0403	.3537	.0261
.21	0.259	.0103	1.080	.46	.0882	.0684	.1248	.71	.0521	.1875	.0437	.96	.0401	.3615	.0259
.22	0.243	.0115	0.946	.47	.0857	.0720	.1180	.72	.0514	.1935	.0424	.97	.0399	.3692	.0256
.23	0.228	.0128	0.833	.48	.0834	.0757	.1118	.73	.0506	.1996	.0411	.98	.0398	.3770	.0254
.24	0.215	.0143	0.740	.49	.0813	.0795	.1060	.74	.0499	.2058	.0400	.99	.0397	.3848	.0253
												1.00	.0396	.3927	.0252

C. Criteria for Hydraulic Design: Open Channels

C.1. GENERAL HYDRAULIC CRITERIA

In general, all open channels should be designed with the tops of the walls or levees at or below the adjacent ground to allow for interception of surface flows. If it is unavoidable to construct the channel without creating a pocket, a means of draining the pocket must be provided on the drawings. All local drainage should be completely controlled. External flows must enter the channel at designated locations and through designated inlets unless specifically authorized by the City Engineer/SSCAFCA.

In making preliminary layouts for the routing of proposed channels, it is desirable to avoid sharp curvatures, reversed curvatures, and closely-spaced series of curves. If this is unavoidable, the design considerations in Section C-3 below must be followed to reduce super elevations and to eliminate initial and compounded wave disturbances.

It is generally desirable to design a channel for a Froude number of just under 2.0. In areas within the City of Rio Rancho and SSCAFCA jurisdiction, this is not always possible because of steep terrain. If the Froude number exceeds 2.0, any small disturbance to the water surface is amplified in the course of time and the flow tends to proceed as a series of "roll waves". Reference is made to Section C-3 for criteria when designing a channel with a Froude number that exceeds 2.0.

In the design of a channel, if the depth is found to produce a Froude number between 0.7 and 1.3 for any significant length of reach, the shape or slope of the channel should be altered to secure a stable flow condition. All analyses should be performed for the 10-year and 100-year design discharges.

C.2. WATER SURFACE PROFILE CALCULATIONS

a. General

Water surface profile calculations must be calculated using the Bernoulli energy equation (see Section B-2) combined with the momentum equation for analyzing confluences and functions. For use in expediting such calculations, computer programs are available from many sources, such as the U.S. Army Corps of Engineers and from industry accepted commercial software.

b. Determination of Controlling Water Surface Elevation

The following are general control points for the calculation of the water surface profile:

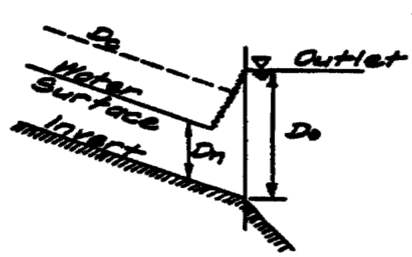
(1) Where the channel slope changes from mild to steep or critical, the depth at the grade break is critical depth.

(2) Where the channel slope changes from critical to steep, the depth at the grade break is critical depth.

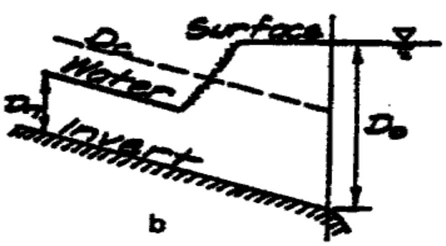
(3) Where a discharging or outletting channel or conduit is on a mild slope, the water surface is generally controlled by the outlet (see Section B-2.1).

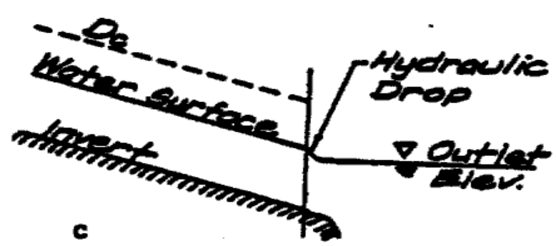
(4) When a channel on a steep slope discharges into a facility that has a water surface depth greater than the normal depth of the channel, calculate pressure plus momentum for normal depth and compare it to the pressure plus momentum for the water surface depth at the outlet according to the equation, $P_n + M_n \sim P_o + M_o$.

(a) If $P_n+M_n > P_o+M_o$, this indicates upstream control with a hydraulic jump at the outlet.



(b) If $P_n + M_n < P_o + M_o$, this indicates outlet control with a hydraulic jump probably occurring upstream.





(c) Where the water surface of the outlet is below the water surface in the channel or conduit, control is upstream and the outflow will have the form of a hydraulic drop.

When there is a series of control points, the one located farthest upstream is used as a starting point for water surface calculation.

c. Direction of Calculation

Calculations proceed upstream when the depth of flow is greater than critical depth and proceed downstream when the depth of flow is less than critical depth.

d. Head Losses

(1) Friction Loss

Friction losses or open channels shall be calculated by an accepted form of the Manning equation. The Manning equation is commonly expressed as follows:

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

in which

Q	=	Flow rate, in c.f.s.
n	=	Roughness coefficient
A	=	Area of water normal to flow, in ft.2
R	=	Hydraulic radius
S_f	=	Friction slope

When arranged into a more useful form,

$$S_f = \frac{2gn^2}{2.21} \left[\frac{V^2}{2_g} \Big/ R^{4/3} \right]$$

The loss of head due to friction throughout the length of reach involved (L) is calculated by:

$$h_f = S_f \cdot L$$

Refer to the appendix for values of "n" for different materials and corresponding values of

$$\frac{2gn^2}{2.21}$$

(2) Junction Loss

Junction losses will be evaluated by the pressure plus momentum equation and must conform to closed conduit angle of confluence criteria, Section B-5. Refer to Section G for cases and alternate solutions.

e. Channel Inlets

(1) Side Channels

Flow rates of 25% or more of the main channel flow must be introduced to the main channel by a side channel hydraulically similar to the main channel. Piping systems can be used to introduce side flows, if justification is provided satisfactory to SSCAFCA. The centerline radius of the side channel may not be less than the quantity (QV/l00) in feet.

Velocity and depth of the flows in the side channel when introduced into the main channel must be matched to within 1 foot of velocity head and to within 20% of the flow depth for both the 10-year and l00-year design discharges and the four combinations of side inlet and main channel flows which result. Energy and momentum balance type calculations must be provided to support all designs involving side channels.

(2) Surface Inlets

When the main channel is relatively narrow and when the peak discharge of side inflow is in the range between 3 and 6 percent of the main channel discharge, high waves are usually produced by the side inflow and are reflected downstream for a long distance, thus requiring additional wall height to preclude overtopping of the channel walls. This condition is amplified when the side inflow is at a greater velocity than the main channel. To eliminate these wave disturbances, the Los Angeles District of the Corps of Engineers has developed a side channel spillway inlet. The City or SSCAFCA may require this type of structure when outletting into one of their facilities, and its use should be considered for city channels if high waves above the normal water surface cannot be tolerated. See Subsection "f" below titled "Transitions" for the Corp's procedure and criteria.

Surface-type inlets shall be constructed of concrete having a minimum thickness of 7 inches and shall be reinforced with the same steel as 8" concrete lining. The upstream end of the surface inlet shall be provided with a concrete cutoff wall having a minimum depth of three feet and the downstream end of the inlet shall be connected to the channel lining by an isolation joint. Side slopes of a surface inlet shall be constructed at slopes no greater than l vertical to 10 horizontal to allow vehicular passage across the inlet where a service road is required.

Drainage ditches or swales immediately upstream of a surface inlet shall be provided with erosion protection consisting of concrete lining, rock riprap or other non-erosive material.

Surface inlets shall enter the channel at a maximum of 90° to the channel centerline, i.e., they may not point upstream.

(3) Direct Pipe to Channel

Junctions involving direct pipe connection to a channel must conform to the criteria listed in Section 5 of the closed conduit criteria. Additionally, pipe and box culvert inlets to channels shall be isolated by expansion joints. Continuously reinforced channels shall be designed to accommodate any extra stress resulting from these discontinuities. Paragraph 18(h), Corps of Engineers EM 1110-2-1061 has additional design criteria.

f. Transitions

(l) Subcritical Flow

For subcritical velocities less than 12 f.p.s., the angle of convergence or divergence between the center line of the channel and the wall must not exceed 12° 30'. The length of the transition (L) is determined from the following equation:

$$L \geq 2.5 \; \Delta \; B$$

For subcritical velocities equal to or greater than 12 f.p.s., the angle of convergence or divergence between the center line of the channel and the wall must not exceed 5° 45'. The length (L) is determined from the following equation:

$$L \geq 5.0 \; \Delta \; B$$

Head losses for transitions with converging walls in subcritical flow conditions can be determined by using either the P + M method or the Thompson equation, both of which are shown in Section G-5. For transitions, both methods are applicable in all cases and will give the same results.

(2) Supercritical Flow

(a) Divergent Walls

The angle of divergence between the center line of the channel and the wall must not exceed 5^{o} 45' or $\tan^{-1} F/3$ whichever is smaller. The length of the transition (L) is the longest length determined from the following equations:

$$L \geq 5.0 \, \Delta \, B$$

$$L \geq 1.5 \, \Delta \, B \cdot F$$

where F = **Upstream Fourde number based on depth of flow**

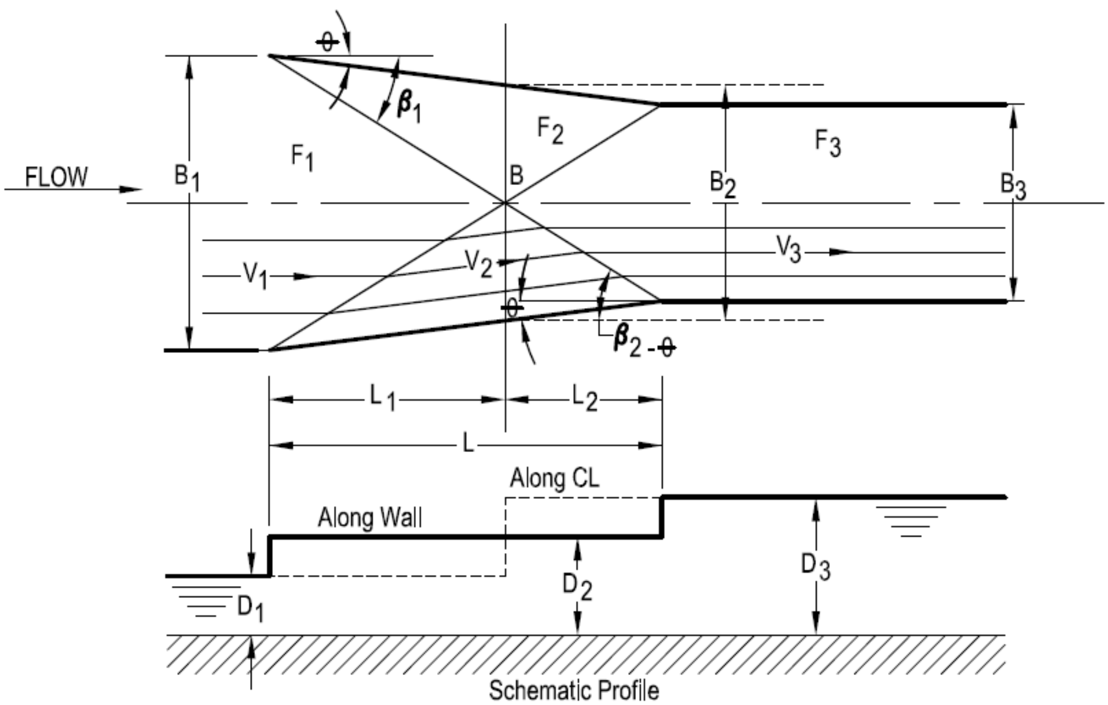
ΔB = **The difference in channel width at the water surface**

(b) Convergent Walls

Converging walls should be avoided when designing channels in supercritical flow; however, if this is impractical, the converging transition will be designed to minimize wave action and to avoid unstable flow regimes. The walls of the transition should be straight lines.

i. For convergent walls less than or equal to 5 degrees, design to avoid unstable flow regime in accordance with Section 3.C.1 and/or account for increased freeboard in accordance with Section 3.C.4. See Example problem No. 7 at the end of this section.

ii. Convergent walls > 5 degrees shall only be used at the discretion of SSCAFCA and based on an approved oblique wave analysis.



CONVERGENT WALL SCHEMATIC

$$L = \frac{B_1 - B_3}{2 \tan \theta}$$

With the initial Froude number and the contraction ratio fixed, and with the continuity equation:

$$\frac{B_1}{B_3} = \left(\frac{D_3}{D_1}\right)^{3/2} * \left(\frac{F_3}{F_1}\right)$$

trial curves can produce the geometry of the construction suggested above. The curves represent the equation

$$\tan\theta = \frac{\tan\beta_1\,(\sqrt{1+8F_1^2\sin^2\beta_1} - 3)}{2\tan^2\beta_1 + \sqrt{1+8F_1^2\sin^2\beta_1} - 1}$$

Refer to Plate 22.3 C-1 and to examples in the COA DPM and LA Hydraulic Design Manual.

(3) Transitions Between Channel Treatment Types

(a) Earth Channel to Concrete Lining Transition

The mouth of the transition should match the earth channel section as closely as practicable. Wing dikes and/or other structures must be provided to positively direct all flows to the transition entrance.

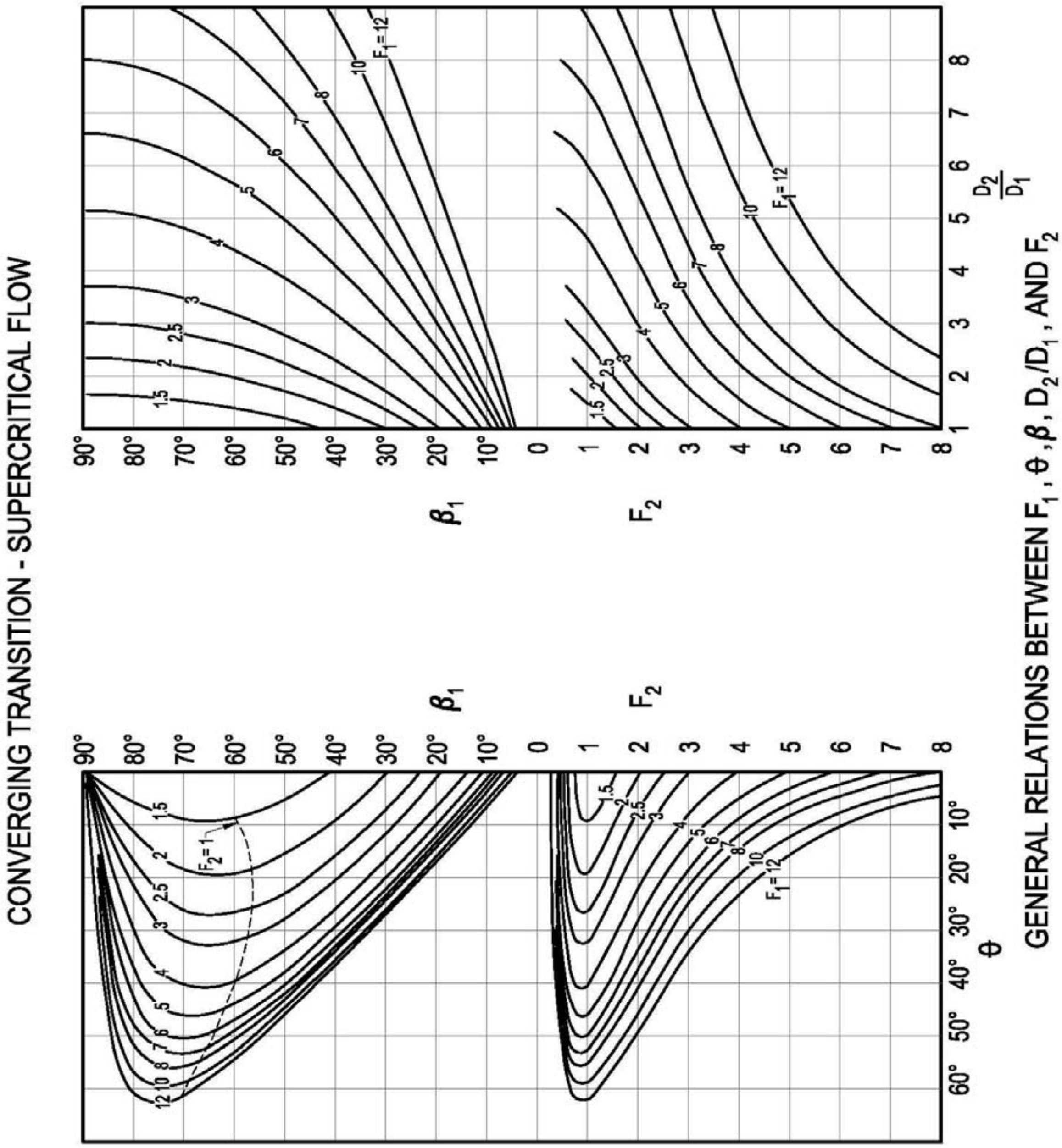

CONVERGING TRANSITION - SUPERCRITICAL FLOW
GENERAL RELATIONS BETWEEN F_1, θ, β, D_2/D_1, AND F_2
θ
β_1
F_2
$\frac{D_2}{D_1}$
$F_2 = 1$
$F_1 = 12$

PLATE 22.3 C-1

The upstream end of the concrete lined transition will be provided with a cutoff wall having a depth of 1.5 times the design flow depth but at least 3.0 feet and extending the full width of the concrete section. Erosion protection directly upstream of the concrete transition consisting of grouted or dumped rock riprap at least 12 feet in length and extending full width of the channel section must be provided. Grouted riprap must be at least 12 inches thick. Dumped riprap must be properly sized, graded and projected with gravel filter blankets.

The maximum allowable rate of bottom width transition is 1 to 7.5 maximum. Grout, dumped, or wire-tied material may also be used if approved on a case-by-case basis by the City Engineer/SSCAFCA. Grouted and wire-tied materials require gravel filters as well.

(b) Concrete Lining to Earth Channel Transition

The transition from concrete lined channels to earth channels will include an energy dissipator as necessary to release the designed flows to the earth channel at a relatively non-erosive condition.

Since energy dissipator structures are dependent on individual site and hydraulic conditions, detailed criteria for their design has been purposely excluded and only minimum requirements are included herein for the concrete to earth channel transition.

On this basis, the following minimum standards govern the design of concrete to earth channel transitions:

- ► Maximum rate of bottom width transitions:

Water Velocity	
0-15 f.p.s.	1:10
16-30 f.p.s.	1:15
31-40 f.p.s.	1:20

- ► The downstream end of the concrete transition structure will be provided with a cutoff wall having a minimum depth of 6 feet and extending the full width of the concrete section or as recommended by the engineer and accepted by the City Engineer/SSCAFCA.

- ► Directly downstream of the concrete transition structure erosion protection consisting of rough, exposed surface, grouted rock riprap and extending full width of the channel section shall be provided. The grouted rock riprap should be a minimum of 12 inches thick. Grout, dumped, or wire-tied material may also be used if approved on a case-by-case basis by the City Engineer/SSCAFCA. Grouted and wire-tied material require gravel filters as well. The length of riprap shall be determined by engineering analysis.

g. Piers

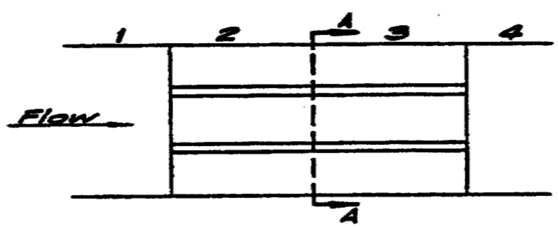
(1) General

The effect of piers on open channel design must be considered at bridge crossings and where an open channel or box conduit not flowing full discharges into a length of multi-barreled box. This effect is especially important when flow is supercritical and when transported debris impinges on the piers.

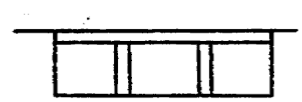
The total pier width includes an added width for design purposes to account for debris. Inasmuch as the debris width to be used in design will vary with each particular situation, the City Engineer/SSCAFCA will be contacted during the preliminary design stages of a project for a determination of the appropriate width. Streamline piers should be used when heavy debris flow is anticipated. Refer to Section 22.8 for design data regarding streamline piers.

The water surface elevations at the upstream end of the piers is determined by equating pressure plus momentum. The water surface profile within the pier reach is determined by the Bernoulli equation. The water surface elevations at the downstream end of the piers may be determined by applying either the pressure plus momentum equation or the Bernoulli equation.

(2) Pressure plus Momentum (P + M) Equation as Applied to Bridge Piers



$$P_1 + M_1 \frac{A_2}{A_1} - P_p = P_2 + M_2$$



Section A-A

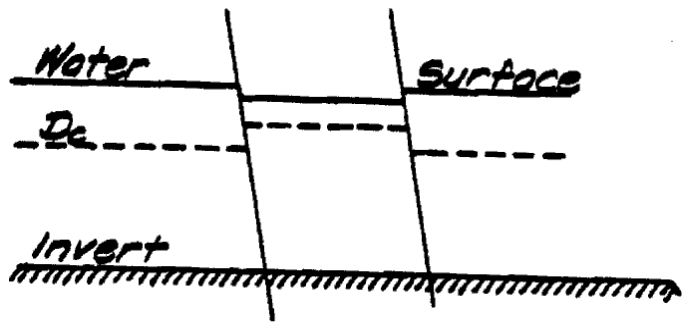
where P_1 = Hydrostatic pressure in unobstructed channel
M_1 = Kinetic momentum in unobstructed channel
where A_1 = Area of unobstructed channel
A_2 = A1 - Kp Ap = Area of water within bridge
P_2 = Hydrostatic pressure within bridge based on net flow area
M_2 = Kinetic momentum within bridge based on net flow area
P_p = Kp Ap Yp = Hydrostatic pressure of bridge pier
A_p = Area of piers
Y_p = Centroidal moment arm of A_p about the hydraulic grade at the section
K_p = Pier factor
K_p = 1.0 for square-nosed piers
K_p = 2/3 for round-nosed piers
(Subscripts indicate the applicable section)

Plate 22.3 C-2 is a graphical representation of the method presented above. Plate 22.3 C-3 and 22.3 C-4 are a graphical solution of the above P + M equation.

(3) Hydraulic Analysis

For subcritical or critical flow, the following cases, numbers 1 or 2, generally apply.

(a) If the depth which balances the P + M equation at the downstream end is equal to or above D_C within the piers, continue the water surface calculations to the upstream face of the bridge piers. Calculate the depth upstream of the piers by equating pressure plus momentum.



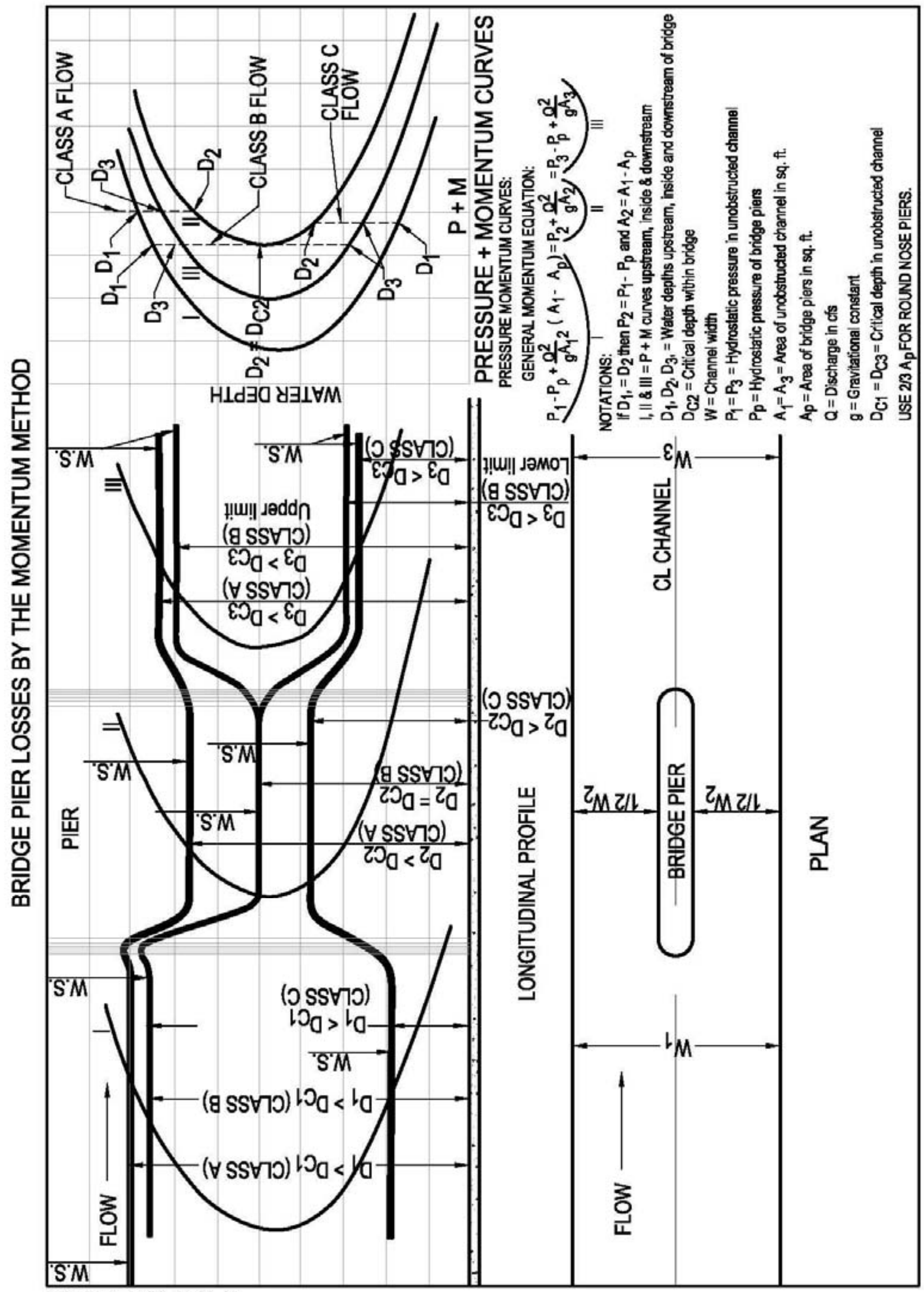

BRIDGE PIER LOSSES BY THE MOMENTUM METHOD
PIER
FLOW
W.S.
D1 > DC1 (CLASS A)
D1 > DC1 (CLASS B)
D1 < DC1
(CLASS C)
D2 > DC2
(CLASS A)
D2 = DC2
(CLASS B)
D2 < DC2
(CLASS C)
D3 > DC3
(CLASS A)
D3 > DC3
(CLASS B)
Upper limit
D3 > DC3
(CLASS B)
Lower limit
D3 < DC3
(CLASS C)
WATER DEPTH
LONGITUDINAL PROFILE
W1
1/2 W2
BRIDGE PIER
CL CHANNEL
W3
PLAN
CLASS A FLOW
CLASS B FLOW
CLASS C FLOW
D1
D2
D3
D2 = DC2
P + M
PRESSURE + MOMENTUM CURVES
PRESSURE MOMENTUM CURVES:
GENERAL MOMENTUM EQUATION:
NOTATIONS:
If D1 = D2 then P2 = P1 - Pp and A2 = A1 - Ap
I, II & III = P + M curves upstream, inside & downstream
D1, D2, D3 = Water depths upstream, inside and downstream of bridge
DC2 = Critical depth within bridge
W = Channel width
P1 = P3 = Hydrostatic pressure in unobstructed channel
Pp = Hydrostatic pressure of bridge piers
A1 = A3 = Area of unobstructed channel in sq. ft.
Ap = Area of bridge piers in sq. ft.
Q = Discharge in cfs
g = Gravitational constant
DC1 = DC3 = Critical depth in unobstructed channel
USE 2/3 Ap FOR ROUND NOSE PIERS.

PLATE 22.3 C-2

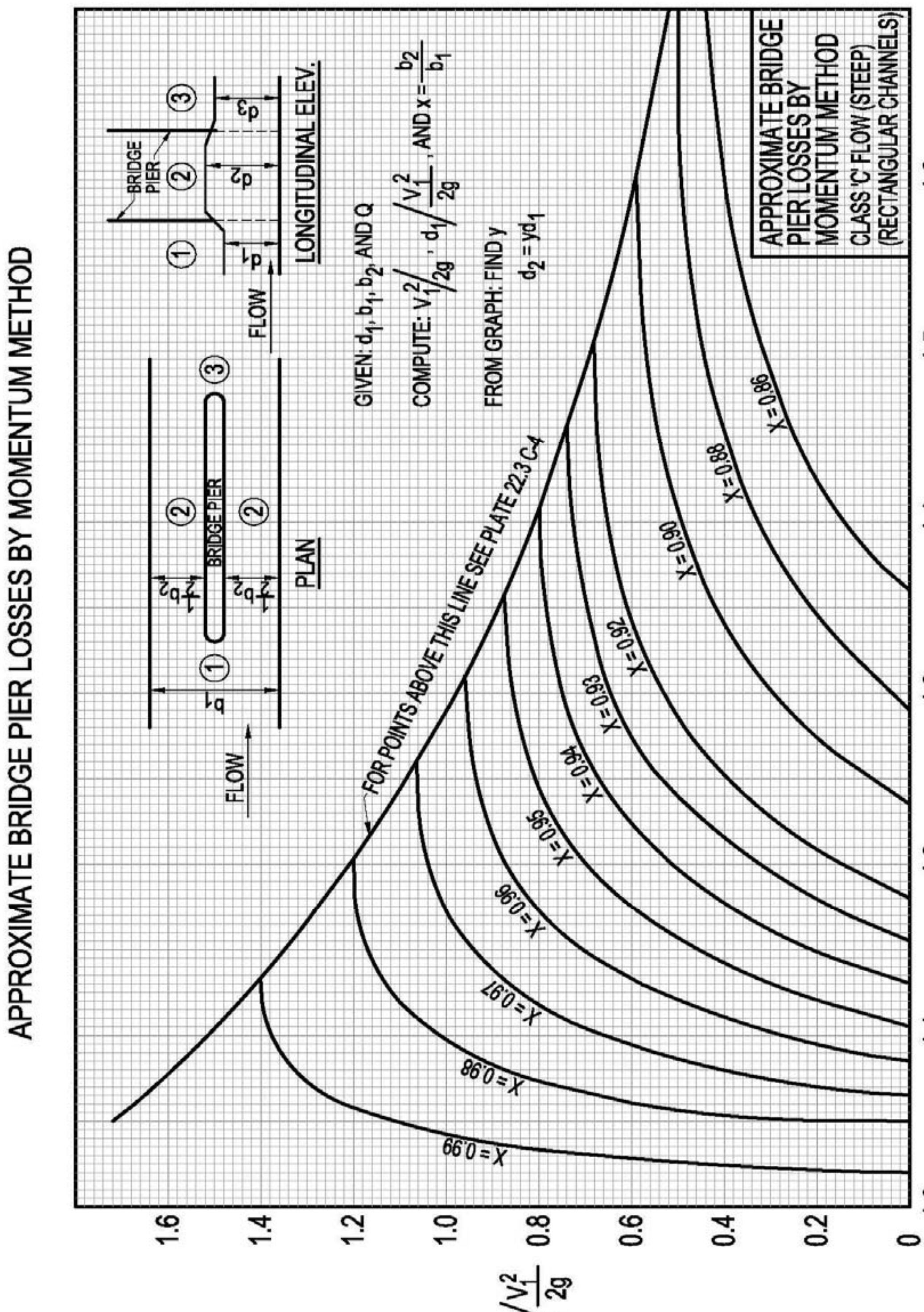

APPROXIMATE BRIDGE PIER LOSSES BY MOMENTUM METHOD
FLOW
BRIDGE PIER
PLAN
BRIDGE
PIER
LONGITUDINAL ELEV.
GIVEN: d1, b1, b2, AND Q
COMPUTE: V1²/2g, d1/(V1²/2g), AND x = b2/b1
FROM GRAPH: FIND y
d2 = yd1
FOR POINTS ABOVE THIS LINE SEE PLATE 22.3 C-4
X = 0.99
X = 0.98
X = 0.97
X = 0.96
X = 0.95
X = 0.94
X = 0.93
X = 0.92
X = 0.90
X = 0.88
X = 0.86
APPROXIMATE BRIDGE
PIER LOSSES BY
MOMENTUM METHOD
CLASS 'C' FLOW (STEEP)
(RECTANGULAR CHANNELS)
y = d2/d1
d1/(V1²/2g)

PLATE 22.3 C-3

APPROXIMATE BRIDGE PIER LOSSES BY MOMENTUM METHOD

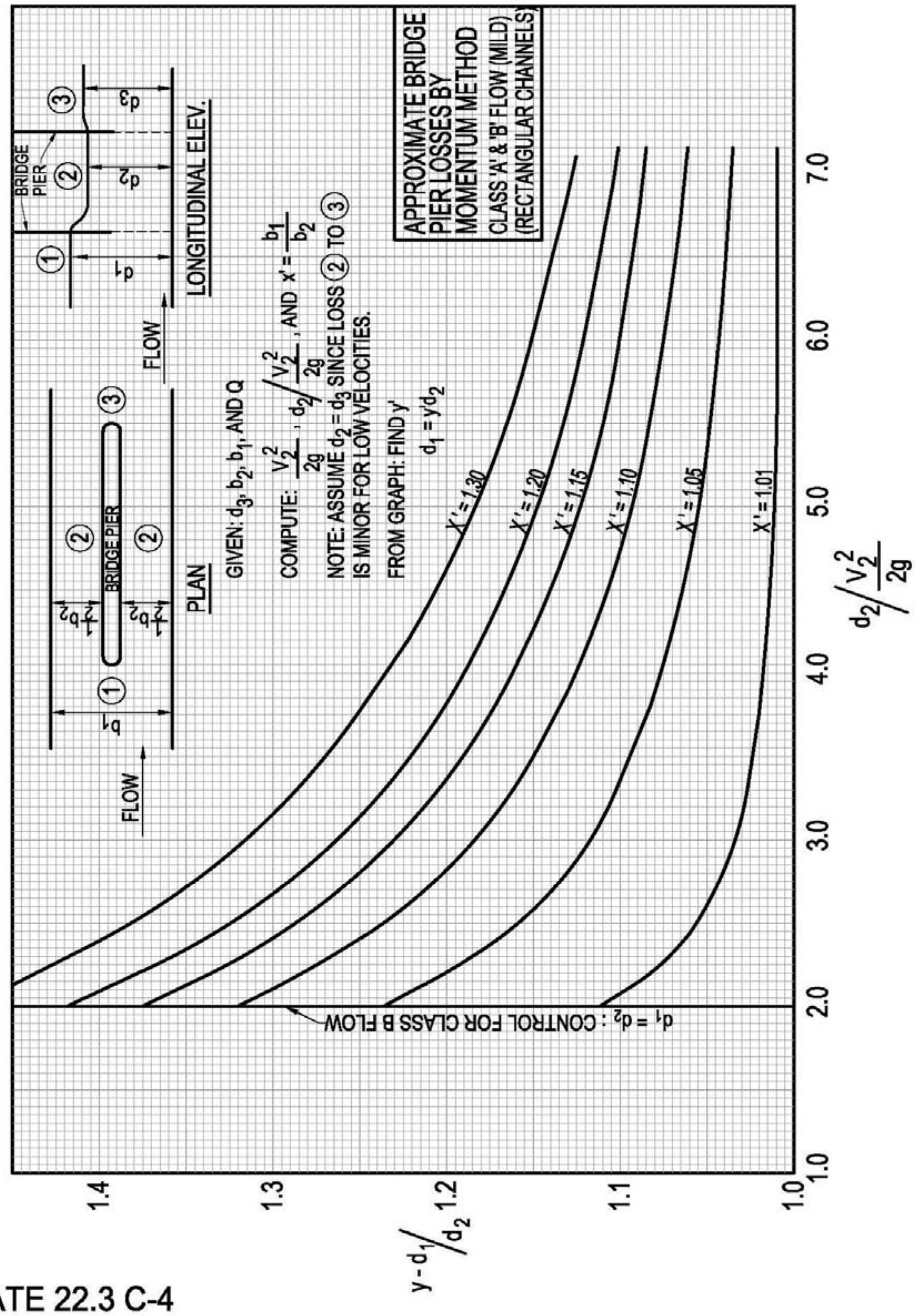
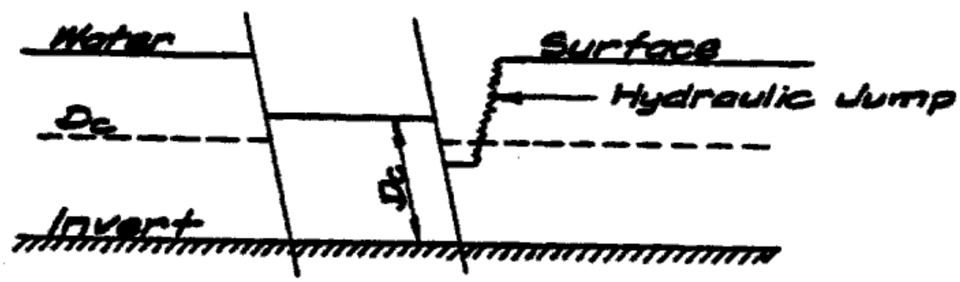


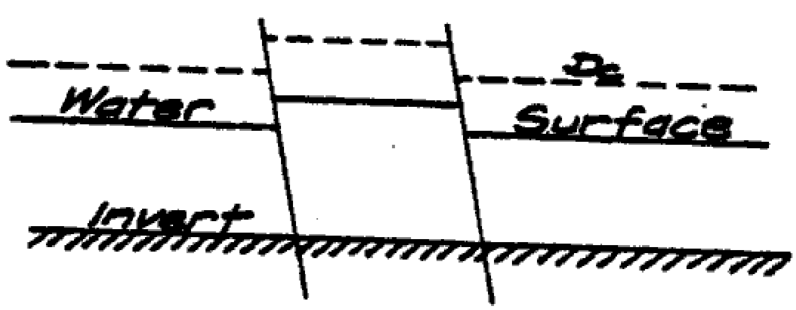
PLATE 22.3 C-4

(b) If at the downstream end of the piers no depth can be found to balance the P + M equation, assume critical depth within the pier and calculate the water surface just downstream from the end of the pier. Calculate P + M for this depth and its sequence depth. If the upper sequence depth provides a greater sum (P + M), a hydraulic jump occurs at the downstream end of the pier. If the lower sequent depth results in a greater sum (P + M) the hydraulic jump occurs some distance downstream from the pier. Within the pier, calculate the water surface to the upstream face and then calculate the depth just upstream of the face of the pier using the P + M equation.

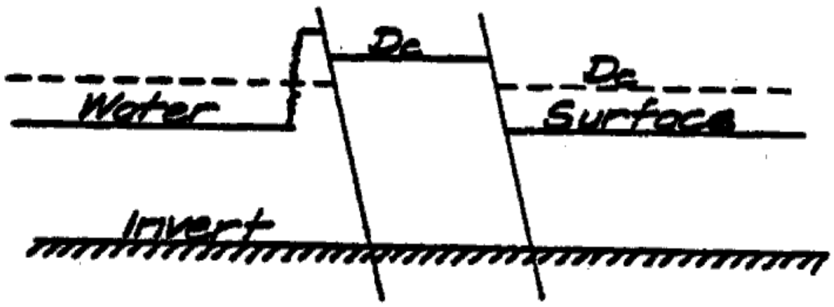


For supercritical flow the following cases, numbers 3 or 4, generally apply.

(c) If the depth calculated by the P + M equation just inside the upstream face of the pier is equal to or below critical depth continue the water surface to the downstream end of the pier and then calculate the depth just outside the pier by either the P + M equation or the Bernoulli equation.



(d) If, at the upstream end of the pier, no depth can be found to balance the P + M equation, calculate P + M for the depth of flow just outside the upstream end of the pier and its sequent depth. If the lower stage results in the greater sum (P + M), this indicates a hydraulic jump at the upstream face of the pier. If the upper stage results in the greater sum (P + M), this indicates a hydraulic jump some distance upstream from the pier. Assume critical depth just inside the upstream pier face and continue the water surface to the downstream end of the pier, and then calculate the depth just outside the pier by either the P + M equation or the Bernoulli equation.



C.3. CURVING ALIGNMENTS

a. Superelevation

Superelevation is the maximum rise in water surface at the outer wall above the mean depth of flow in an equivalent straight reach, caused by centrifugal force in a curving alignment.

(1) Rectangular Channels

For subcritical velocity, or for supercritical velocity where a stable transverse slope has been attained by an upstream easement curve, the superelevation (s) can be calculated from the following equation:

$$S = \frac{V^2 b}{2g r}$$

For supercritical velocity in the absence of an upstream easement curve, the superelevation (S) is given by the following equation:

$$S = \frac{V^2 b}{g r}$$

where V = velocity of the flow cross section, in f.p.s.
b = Width of the channel, in ft.
g = Acceleration due to gravity
r = Radius of channel center line curve, in ft.
X = Distance from the start of the circular curve to the point of the first S in ft.
D = Depth of flow for an equivalent straight reach
B = Wave front angle

$$X = \frac{\pi b V}{\sqrt{12 g D}} = \frac{.16 b V}{\sqrt{D}} = \frac{0.908 b}{\sin \beta}$$

$$\sin \beta = \frac{\sqrt{g D}}{V} = \frac{1}{F}$$

"S" will not be uniform around the bend but will have maximum and minimum zones which persist for a considerable distance into the downstream tangent.

(2) Trapezoidal Channels

For subcritical velocity, the superelevation (S) can be calculated from the following equation:

$$S = 1.15 \quad V^2 (b + 2 z D) / 2 g r$$

where z = cotangent of bank slope
b = channel bottom width, in ft.

For supercritical velocity, curving alignments shall have easement curves with a superelevation (S) given by the following equation:

$$S = 1.3 \quad V^2 (b + 2 z D) / 2 g r$$

(3) Unlined Channels

Unlined channels will be considered trapezoidal insofar as superelevation calculations are concerned. However, this does not apply to calculations of stream or channel cross-sectional areas.

4. Freeboard:

Freeboard is the additional wall height applied to a calculated water surface. This criteria can be superseded by other government regulations/requirements.

a. Rectangular Channels will not be used except with City Engineer/SSCAFCA's approval)

b. Trapezoidal Channels and Associated Types

Adequate channel freeboard above the designed water surface must be provided and will not be less than the amount determined by the following:

(l) For flow rates of less than 100 c.f.s. and average flow velocity of less than 35 f.p.s.:

Freeboard (Feet) = $1.0 + 0.025\ Vd^{1/3}$

(2) For flow rates of 100 c.f.s. or greater and average flow velocity of 35 f.p.s. or greater:

Freeboard (Feet) = $0.7\ (2.0 + 0.025\ Vd^{1/3})$

Freeboard will be in addition to any superelevation of the water surface, standing waves and/or other water surface disturbances. When the total expected height of disturbances is less than 0.5 feet, disregard their contribution.

Unlined portions of the drainage way may not be considered as freeboard unless specifically approved by the City Engineer/SSCAFCA.

For supercritical flow where the specific energy is equal to or less than 1.2 of the specific energy at D_c, the wall height will be equal to the sequent depth, but not less than-the heights required above. This condition should be avoided.

c. Roll Waves

Roll waves are intermittent surges on steep slopes that will occur when the Froude Number (F) is greater than 2.0 and the channel invert slope (S_0) is greater than the quotient, twelve divided by the Reynolds Number. When they do occur, it is important to know the maximum wave height at all points along the channel so that appropriate wall heights may be determined based on the experimental results of roll waves as identified by Richard R. Brock, so that the maximum wave height can be estimated.

For details, see "Development of Roll Waves in Open Channels", Report No. KH-R-16, California Institute of Technology, July 1967. Refer also to Plates 22.3 C-5, 22.3 C-6 and 22.3 C-7.

5. Other Criteria

a. Unlined Channels

After full consideration has been given to the soil type, velocity of flow, desired life of the channel, economics, availability of materials, maintenance and any other pertinent factors, an unlined earth channel may be approved for use.

Generally, its use is acceptable where erosion is not a factor and where mean velocity does not exceed 3 f.p.s. Old and well-seasoned channels will stand higher velocities than new ones; and with other conditions the same, deeper channels will convey water at a higher nonerodible velocity than shallower ones.

Maximum side slopes are determined pursuant to an analysis of soil reports. However, in general, slopes should be 6:l or flatter with erosion protection measures.

b. Composite Linings

In case part of the channel cross section is unlined or the linings are composed of different materials, a weighted coefficient must be determined using the roughness factors for the materials as given in Table 22.3 B-l. If the lining materials are represented by the subscripts "a", "b" and "c", and the wetted perimeters by "P", the weighted value of "n" for the composite section is given by the following equation:

$$n = \frac{\left[P_a n_a^{3/2} + P_b n_b^{3/2} + P_c n_c^{3/2} \right]^{2/3}}{P}$$

ROLL WAVES

Maximum Wave Height

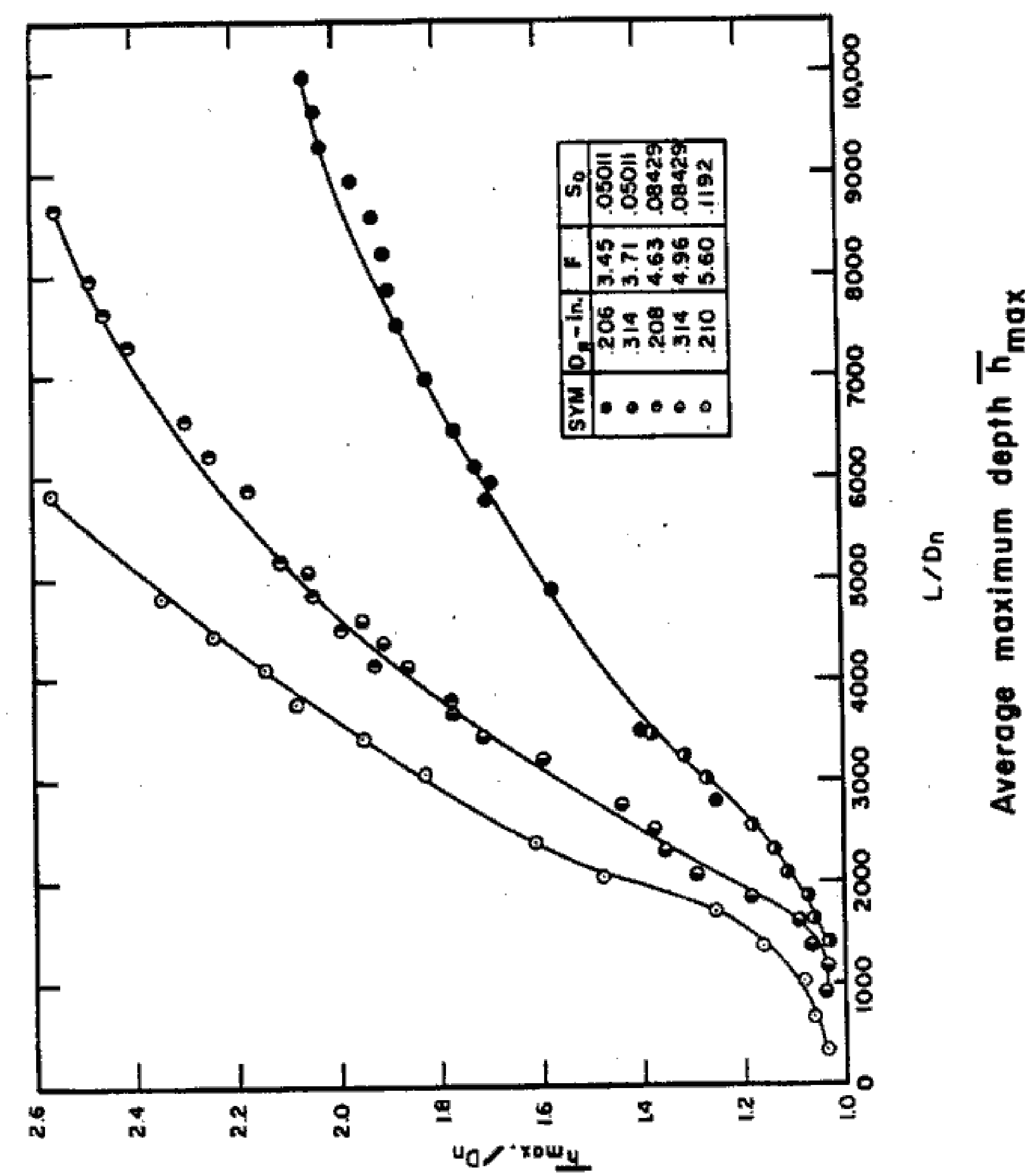
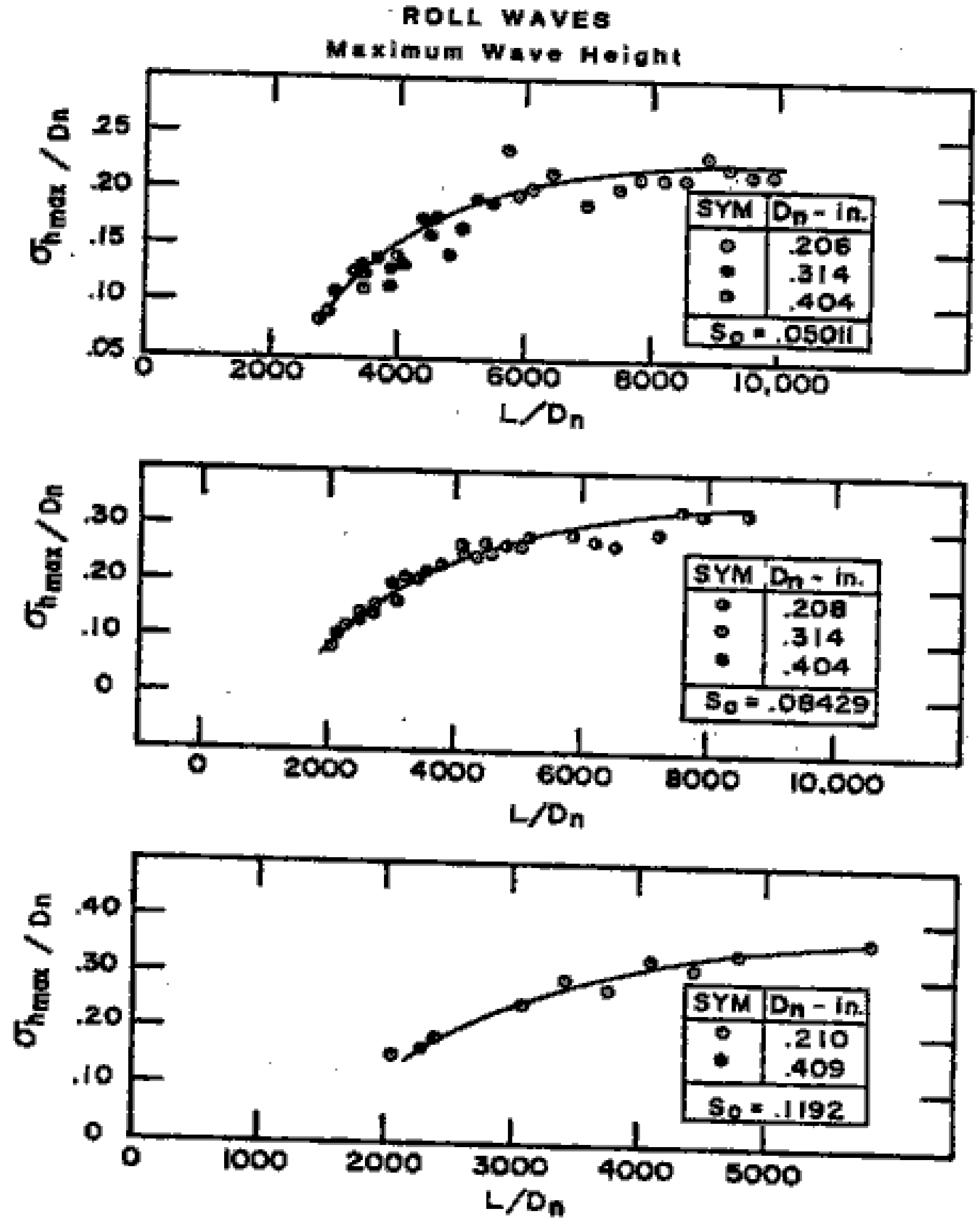


PLATE 22.3 C-5

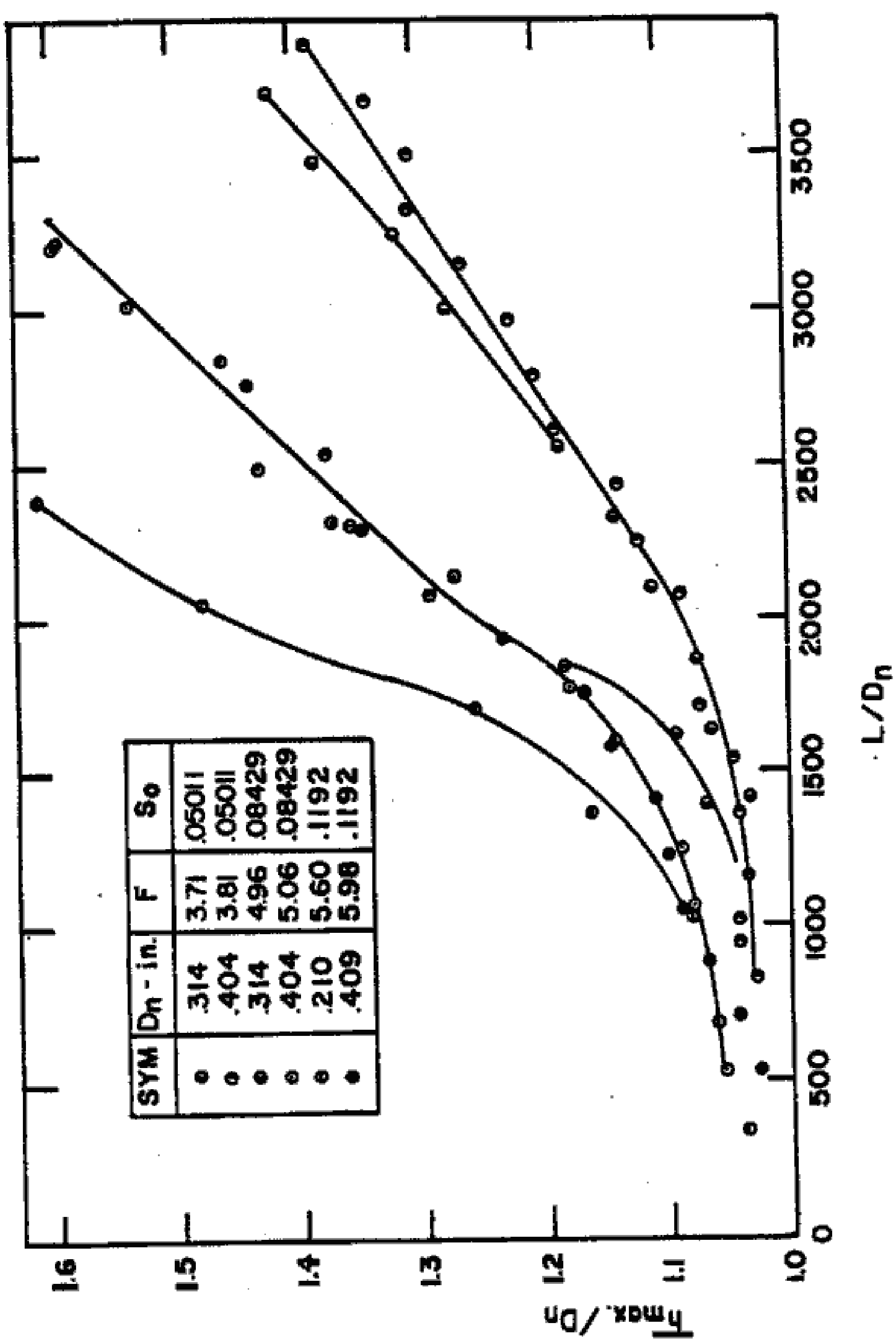


Standard deviation of the maximum depth, $\sigma_{h_{max}}$.

PLATE 22.3 C-6

ROLL WAVES

Maximum Wave Height



Average maximum depth $\overline{h}_{max.}$

PLATE 22.3 C-7

c. Maximum Sidewall Slopes (Freeboard Area)

The following sidewall slopes are generally the maximum values used for channels on at least one side of the concrete lined channel.

Lining Material	Maximum Slope
Soil Cement	2:1
Portland Cement Concrete	Vertical (Trapezoidal 2:1)
Grouted Rock Rip-Rap	2:1
Dumped Rock Rip-Rap	2:1
Earth Lined	6:1
Grass Lined (sodded)	6:1
Gravel Mulch	6:1

d. Channel Maintenance and Access Road

A maintenance and access road having a minimum of 12 feet top width shall be provided on both sides of improved channels. The roads should be sloped away from the channel, and roadway runoff carried in a controlled manner to the channel. In some cases the City Engineer/SSCAFCA may require additional width. Channel maintenance and access roads shall be surfaced with gravel base course. The thickness of said base course shall be 6 inches.

Turnouts will be provided at no more than ½ mile intervals and turnarounds must be provided at all access road dead ends.

Ingress and egress from public right-of-way and/or easements to the channel maintenance and access roads must be provided.

e. Channel Access Ramps

Channel access ramps for vehicular use will be provided as necessary for complete access to the channel throughout its entire length with the maximum length of channel between ramps being one-half mile.

Ramps shall be constructed of 8" thick reinforced concrete and will not have slopes greater than 10% and ramps shall not enter the channel at angles greater than 15% from a line parallel to the channel centerline.

Ramps may be constructed on one side of the channel and must be approved by SSCAFCA. The maintenance and access road on the “ramp” side shall be offset around the ramp to provide for continuity of the road full length of the channel.

The downhill direction of the ramp should be oriented downstream.

f. Street Crossings

Street crossing or other drainage structures over the concrete lined channel should be of the all weather type, i.e., bridges or concrete box culverts. Crossing structures should conform to the channel shape in order that they disturb the flow as little as possible.

It is preferred that the channel section be continuous through crossing structures. However, when this is not practicable, hydraulic disturbance shall be minimized, and crossing structures should be suitably isolated from the channel lining with appropriate joints.

Street crossing structures shall be capable of passing the l00 year frequency design storm flow.

Channel lining transitions at bridges and box culverts should conform to the provisions for transitions hereinafter provided. Drainage structures having a minimum clear height of 8 feet and being of sufficient width to pass maintenance vehicles may result in minimizing the number of required channel access ramps. Unless otherwise specifically authorized by the City Engineer/SSCAFCA, all crossing structures must have at least 8.0 feet of clear height.

g. Subdrainage

Concrete lined channels to be constructed in areas where the ground water table is greater than two feet below the channel invert, weep holes or other subdrainage systems are not required.

Areas where the ground water table is within two feet or less of the channel bottom, there shall be provided, special subdrainage systems as necessary to relieve water pressures from behind the channel lining.

D. Storm Inlets

D.1. DESIGN Q

The Design Q for storm inlet design should be determined based on the following procedures.

a. Outline the drainage area on a map with an appropriate scale.

b. Outline the drainage area tributary to each proposed storm inlet, designating this area with the corresponding subarea number and with a letter (2A, 2B, 2C, etc.). Drainage areas should be differentiated by color or line type.

c. Calculate the tributary area in acres for each storm inlet or battery of storm inlets.

d. Assuming satisfactory drainage area relationships, the storm inlet design Q will be calculated as follows:

$$Q_{DES} = \frac{Q_P}{A_T} A$$

where A = Area in acres tributary to storm inlet
A_T = Total area in acres of the appropriate subarea
Q_P = Peak Q from appropriate subarea, in c.f.s.

In cases where the main line design Qs are reduced because of a restricted outlet, the storm inlet design Qs must be reduced by the same percentage.

If, during the design of a project, it is determined that the proposed storm inlet interception points will change the interception points assumed in the main line hydrology, then the main line Qs should be adjusted accordingly.

2. Required Data and Calculations

a. Street Flow Carrying Capacity

Submitted data should include complete cross sections between property lines of streets at the proposed storm inlet and of any streets which control the flow of water to the pertinent locations. Street cross sections should indicate the following:

(1) Dimensions from the street center line to the top of curb and property line.

(2) Gutter slope upstream of each storm inlet.

(3) Elevations for the top of curb, flow line, property line and street crown at each storm inlet center line.

(4) Curb batter.

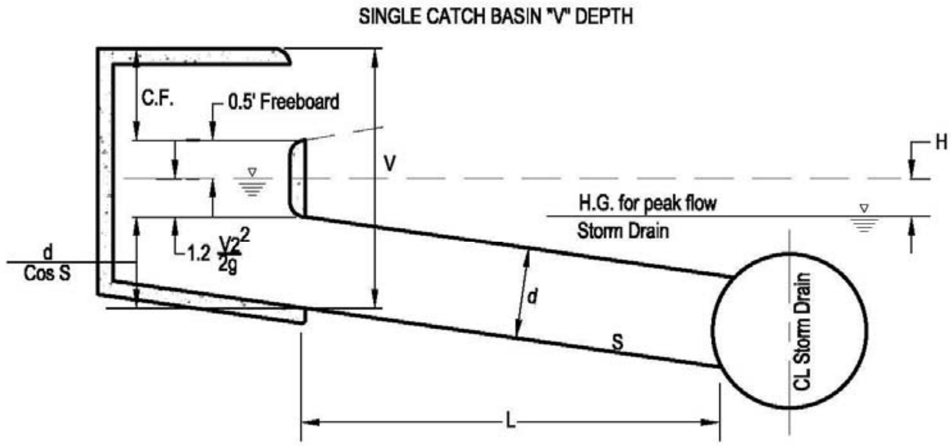
Please refer to Plates 22.3 D-1 to 22.3 D-4 inclusive, for nomographs giving street capacities for some typical street sections. These nomographs have been developed for 8” curb heights. Be aware that the City of Rio Rancho standard height is 6”.

b. Storm Inlet Size and Type

Size and type of storm inlet should be determined by physical requirements and by inlet flow capacities given in Plates 22.3 D-5 to 22.3 D-7, inclusive. Criteria used, if other than those recommended in this section, must be cited and accompanied by appropriate calculations.

c. Connector Pipe and "V" Depth Calculation

(1) Single Storm Inlet



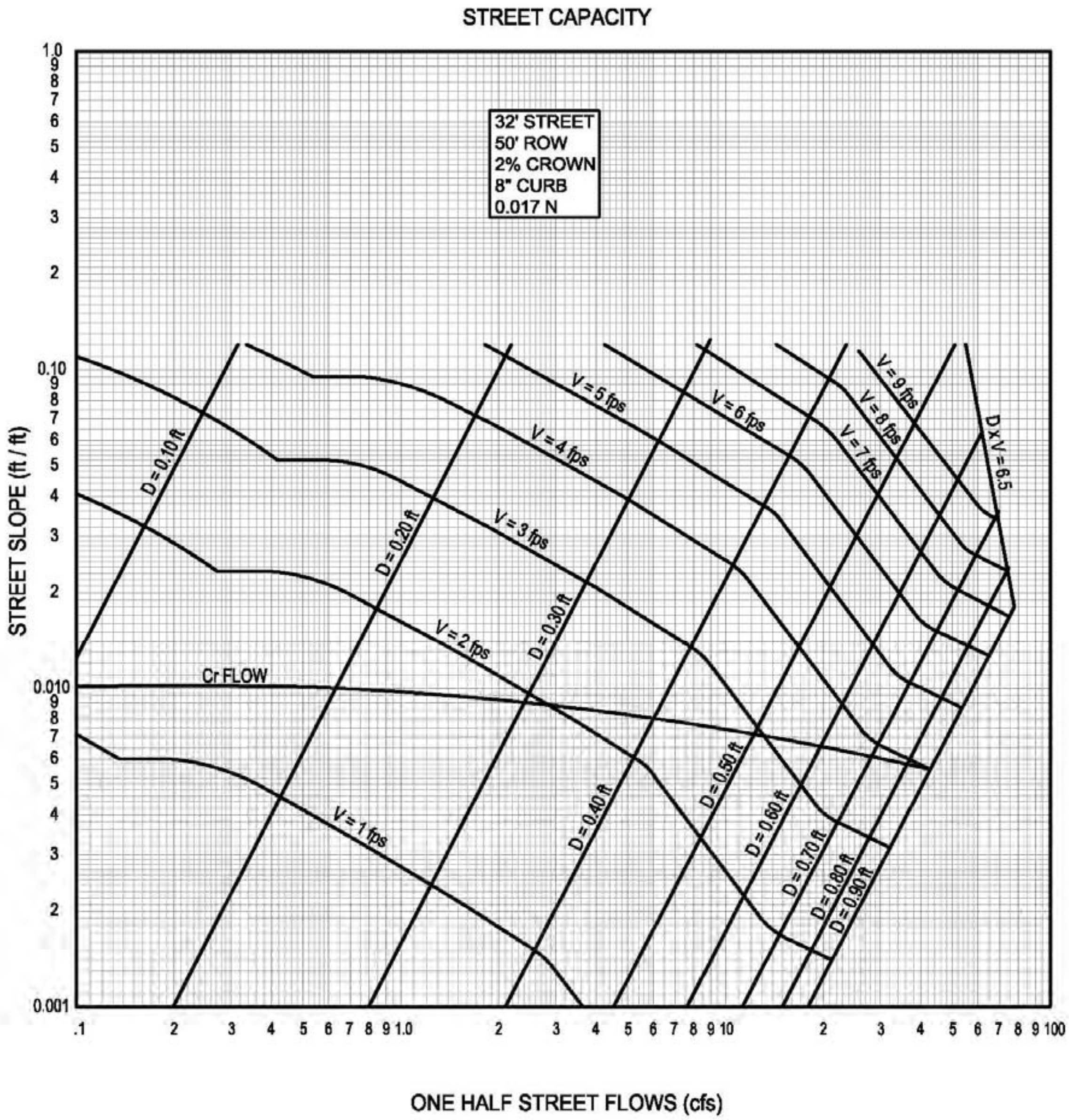

STREET CAPACITY
32' STREET
50' ROW
2% CROWN
8" CURB
0.017 N
STREET SLOPE (ft / ft)
ONE HALF STREET FLOWS (cfs)
V = 1 fps
V = 2 fps
V = 3 fps
V = 4 fps
V = 5 fps
V = 6 fps
V = 7 fps
V = 8 fps
V = 9 fps
D x V = 6.5
D = 0.10 ft
D = 0.20 ft
D = 0.30 ft
D = 0.40 ft
D = 0.50 ft
D = 0.60 ft
D = 0.70 ft
D = 0.80 ft
D = 0.90 ft
Cr FLOW

PLATE 22.3 D-1

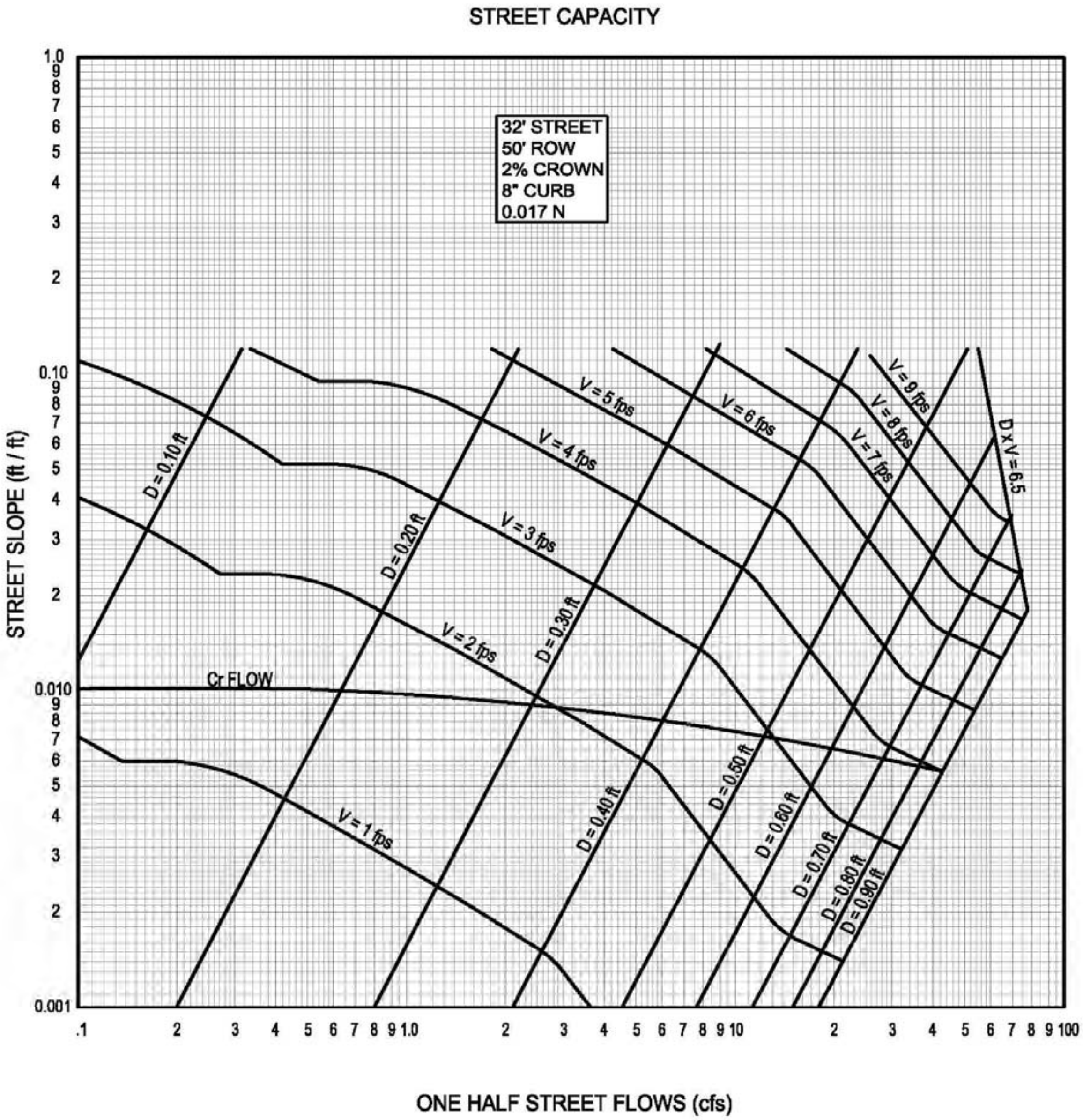

STREET CAPACITY
32' STREET
50' ROW
2% CROWN
8" CURB
0.017 N
STREET SLOPE (ft / ft)
ONE HALF STREET FLOWS (cfs)
V = 1 fps
V = 2 fps
V = 3 fps
V = 4 fps
V = 5 fps
V = 6 fps
V = 7 fps
V = 8 fps
V = 9 fps
D x V = 6.5
Cr FLOW
D = 0.10 ft
D = 0.20 ft
D = 0.30 ft
D = 0.40 ft
D = 0.50 ft
D = 0.60 ft
D = 0.70 ft
D = 0.80 ft
D = 0.90 ft
1.0
0.10
0.010
0.001
.1
1.0
10
100

PLATE 22.3 D-2

STREET CAPACITY

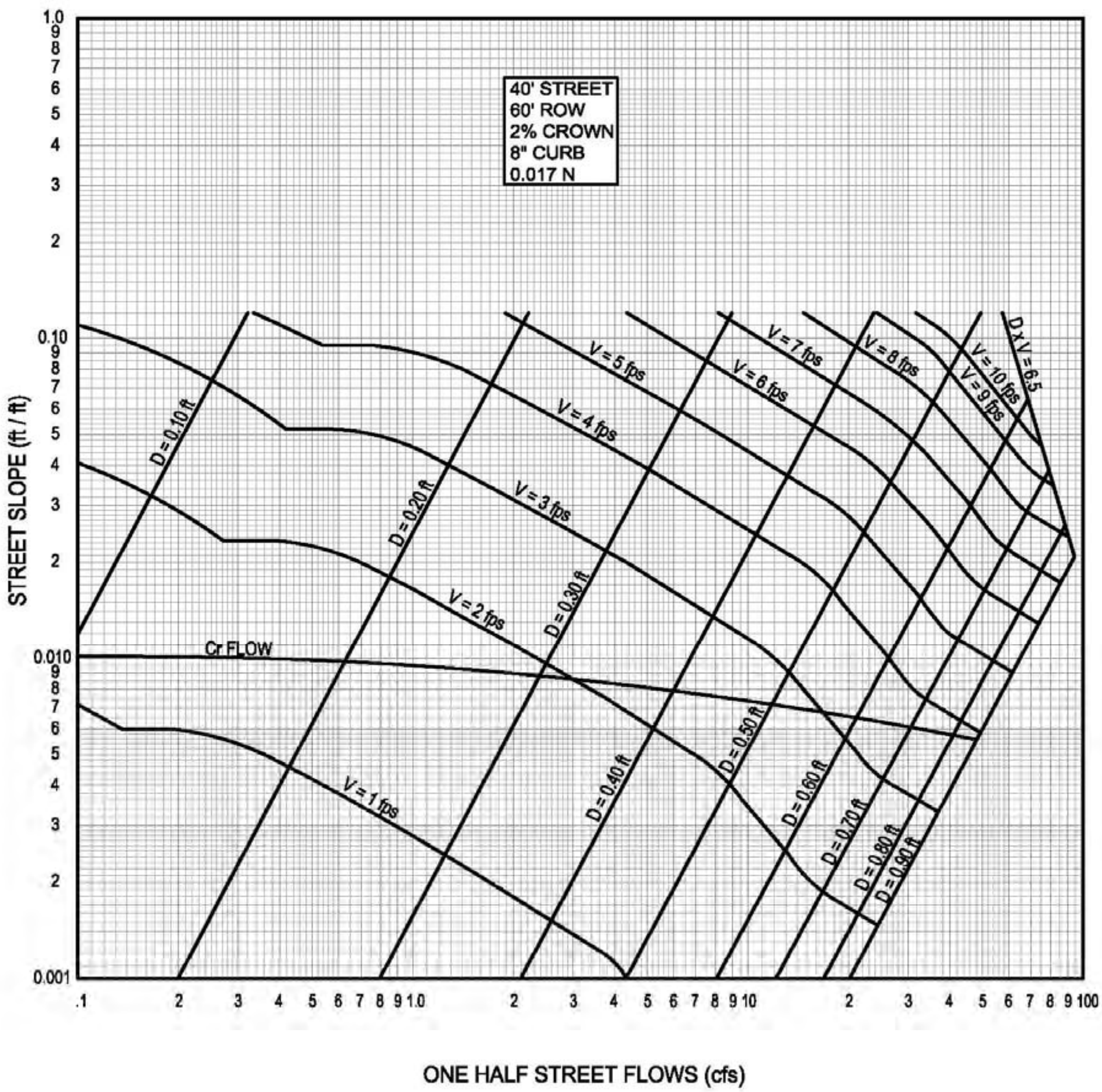


PLATE 22.3 D-3

STREET CAPACITY

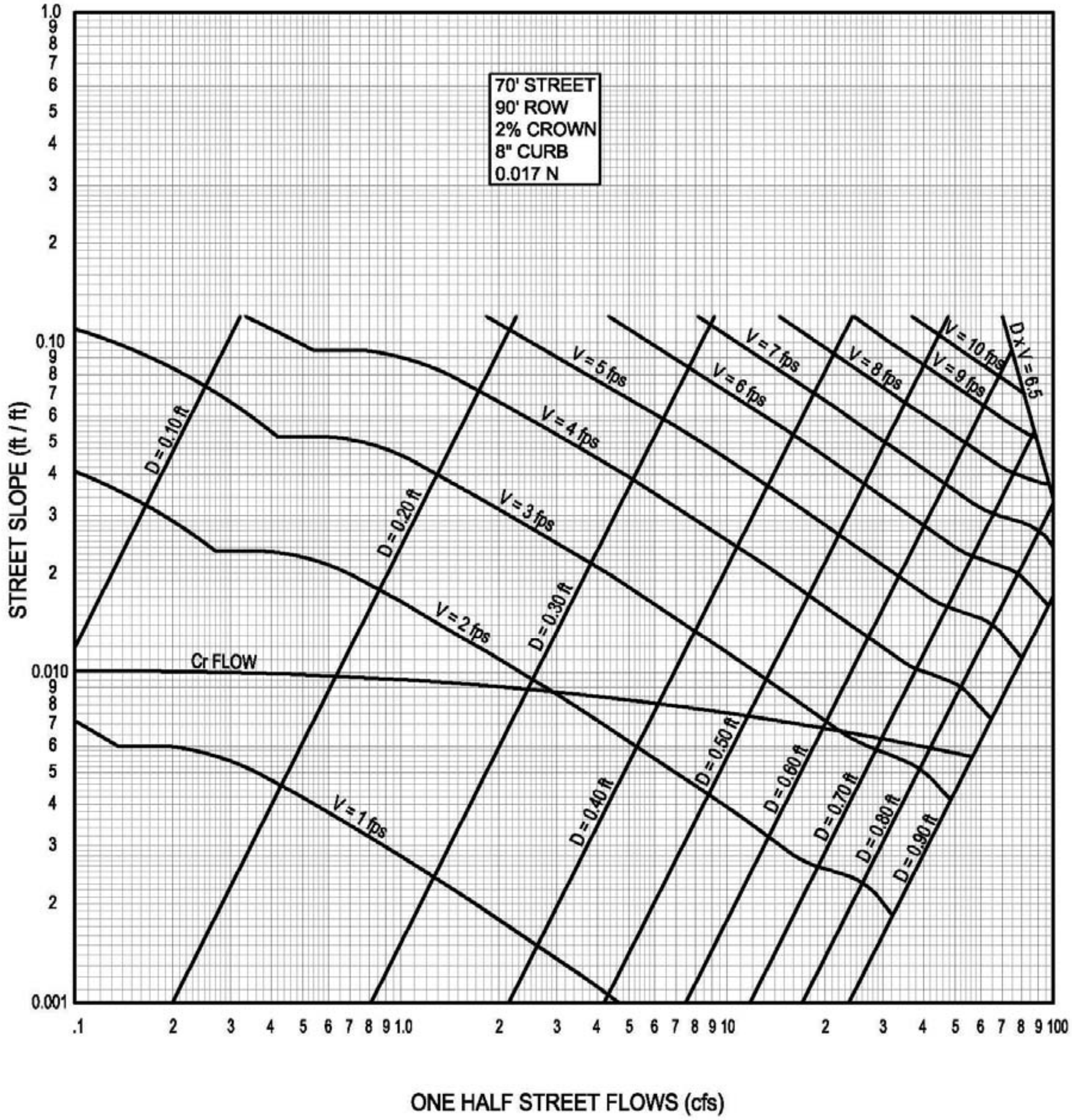


PLATE 22.3 D-4

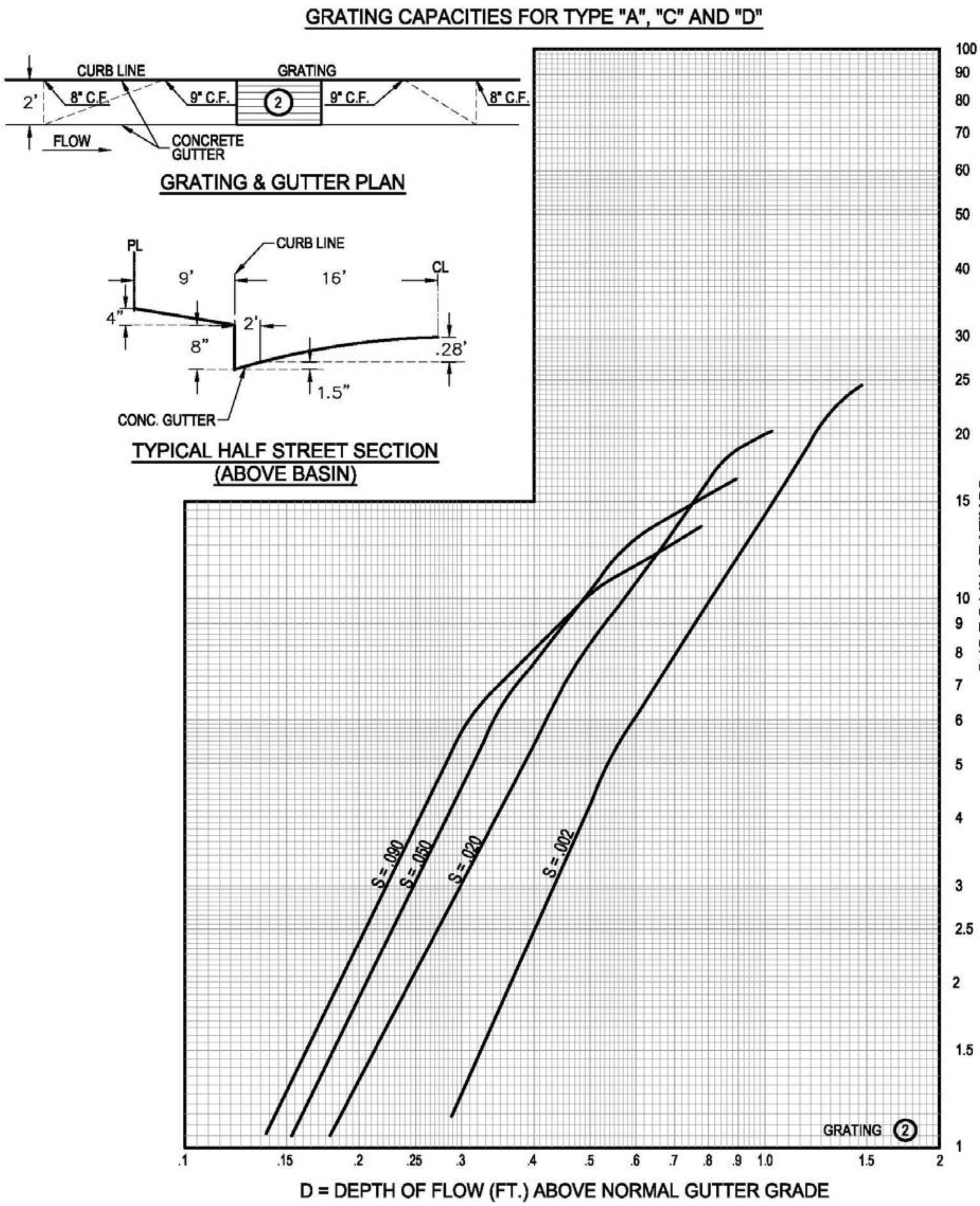

GRATING CAPACITIES FOR TYPE "A", "C" AND "D"
CURB LINE
GRATING
2'
8" C.F.
9" C.F.
2
9" C.F.
8" C.F.
FLOW
CONCRETE GUTTER
GRATING & GUTTER PLAN
PL
CURB LINE
9'
16'
CL
4"
2'
8"
.28'
1.5"
CONC. GUTTER
TYPICAL HALF STREET SECTION (ABOVE BASIN)
S = .090
S = .050
S = .020
S = .002
GRATING 2
Q (C.F.S.) IN GRATINGS
D = DEPTH OF FLOW (FT.) ABOVE NORMAL GUTTER GRADE

PLATE 22.3 D-5

GRATING CAPACITIES FOR TYPE DOUBLE "C" AND "D"

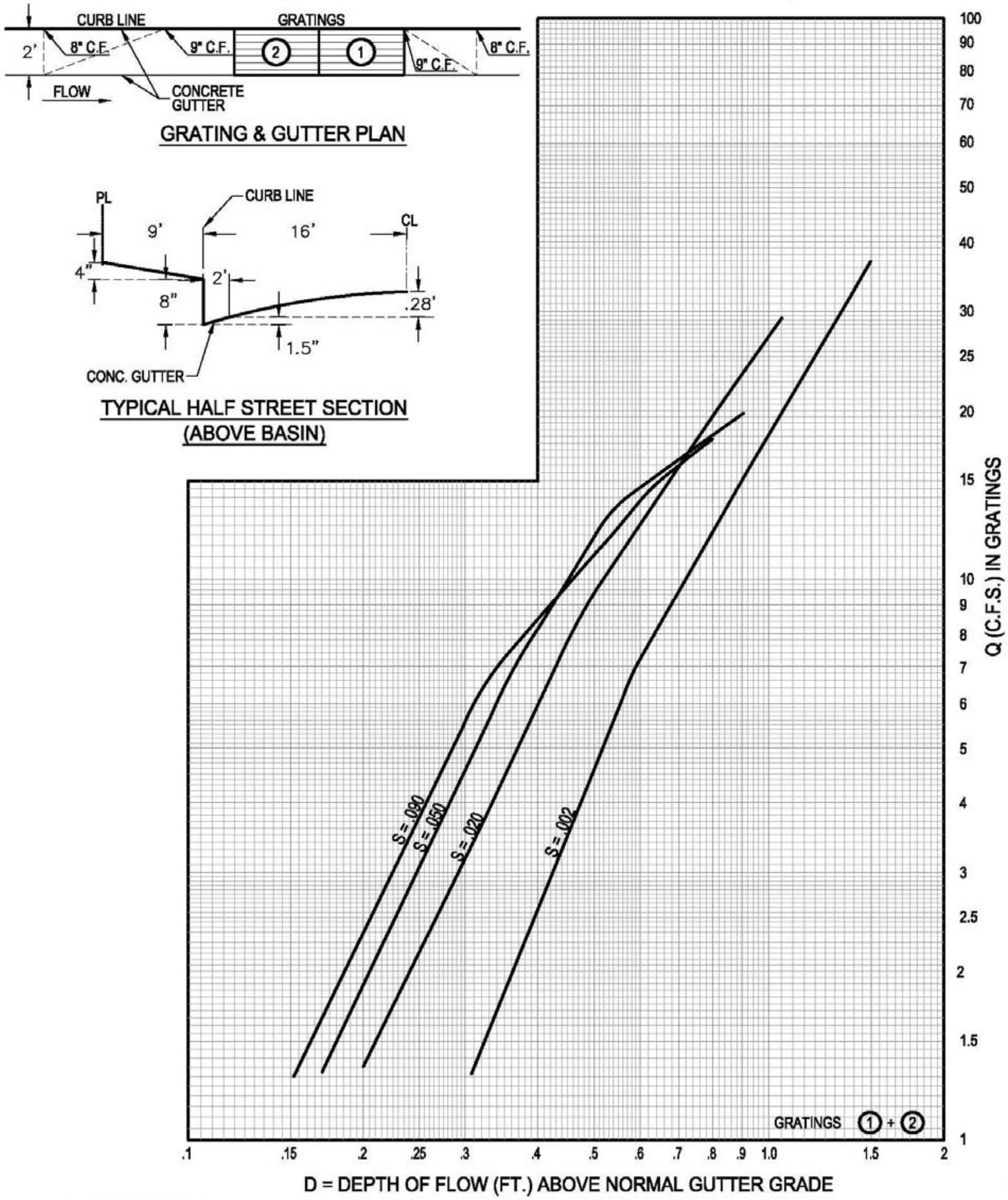


PLATE 22.3 D-6

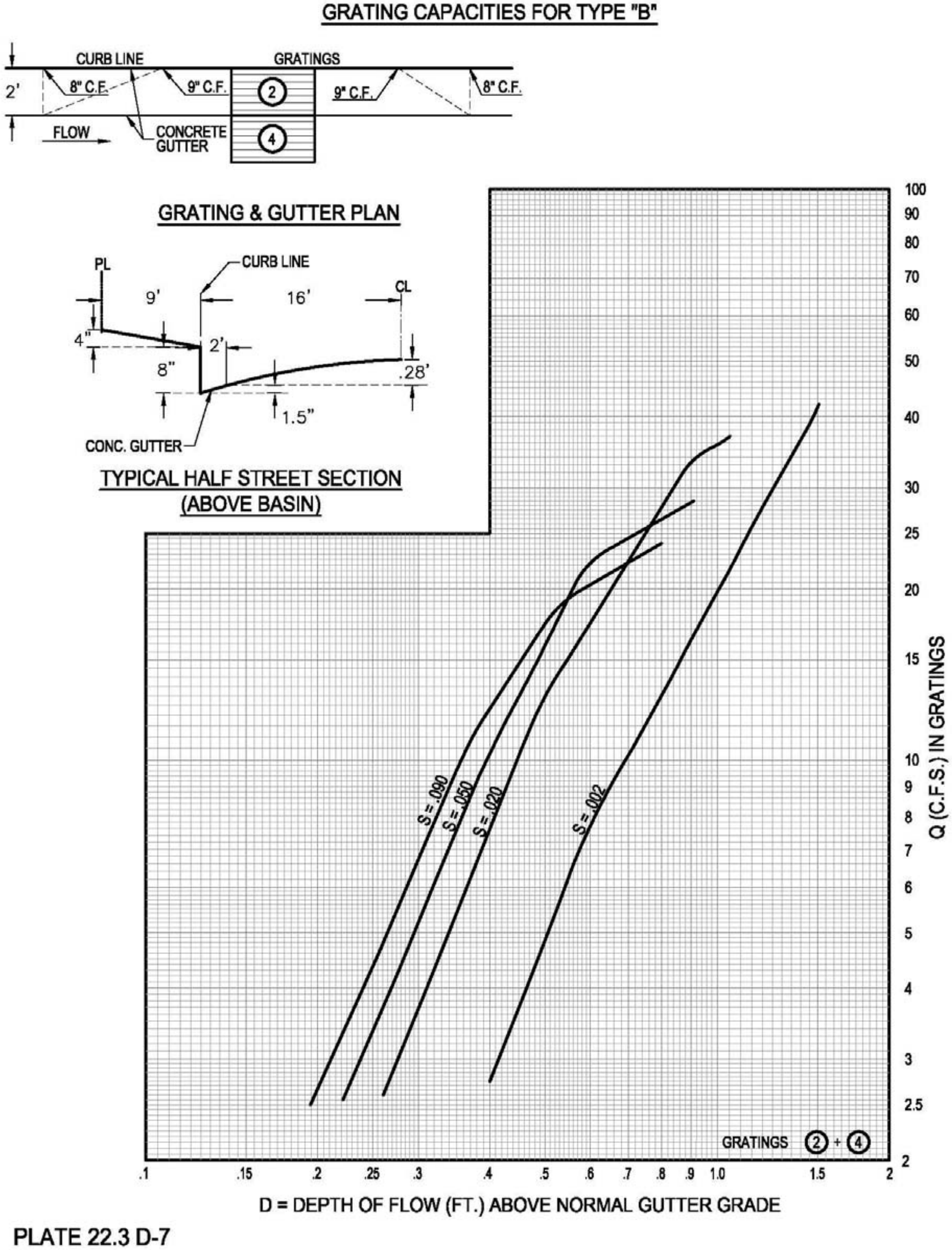

GRATING CAPACITIES FOR TYPE "B"
CURB LINE
GRATINGS
2'
8" C.F.
9" C.F.
2
9" C.F.
8" C.F.
FLOW
CONCRETE GUTTER
4
GRATING & GUTTER PLAN
PL
CURB LINE
9'
16'
CL
4"
2'
8"
.28'
1.5"
CONC. GUTTER
TYPICAL HALF STREET SECTION
(ABOVE BASIN)
S = .090
S = .050
S = .020
S = .002
GRATINGS 2 + 4
Q (C.F.S.) IN GRATINGS
100
90
80
70
60
50
40
30
25
20
15
10
9
8
7
6
5
4
3
2.5
2
.1
.15
.2
.25
.3
.4
.5
.6
.7
.8
.9
1.0
1.5
2
D = DEPTH OF FLOW (FT.) ABOVE NORMAL GUTTER GRADE

PLATE 22.3 D-7

Given the available head (H), the required connector pipe size can be determined from culvert equations, such as those given in King & Brater, Handbook of Hydraulics, Section Four, 5th Edition. Plate 22.3 D-8 can be used for a nomographic solution of a culvert equation for culverts flowing full.

The minimum storm inlet "V" depth should be determined as follows:

$$V = C.F. + 0.5 + 1.2\ \frac{V^2}{2g} + \frac{d}{\text{Cos } S}$$

where

- V = Depth of the storm inlet , or "V" depth, measured in feet from the invert of the connector pipe to the top of the curb.
- C.F = Vertical dimension of the curb face at the storm inlet opening, in feet.
- v = Average velocity of flow in the connector pipe, in feet per second, assuming a full pipe section.
- d = Diameter of connector pipe, in feet.
- S = Slope of connector pipe.

The term $1.2(V^2/2g)$ includes an entrance loss of .2 of the velocity head.

Assuming a curb face at the storm inlet opening of 10 inches, which is the value normally used, and Cos S = 1, the above equation may be simplified to the following:

$$V = 1.33 + 1.2\ \frac{V^2}{2g} + d$$

Please refer to Plate 22.3 D-9 for a graphical solution to the above equation for curb faces of 10 inches.

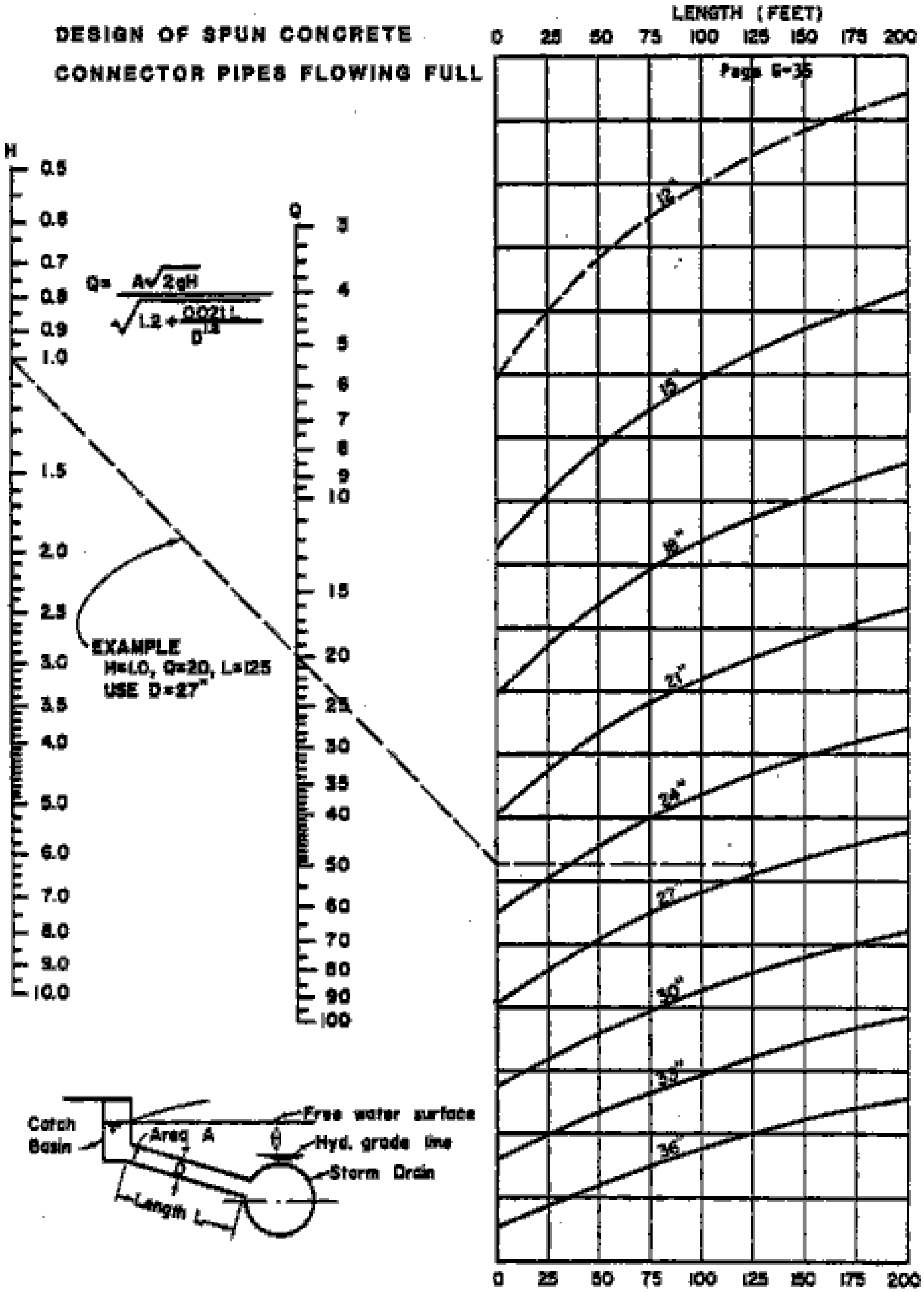

DESIGN OF SPUN CONCRETE
CONNECTOR PIPES FLOWING FULL
LENGTH (FEET)
Page G-35
EXAMPLE
H=1.0, Q=20, L=125
USE D=27"
Catch Basin
Free water surface
Area A
Hyd. grade line
Storm Drain
Length L

PLATE 22.3 D-8

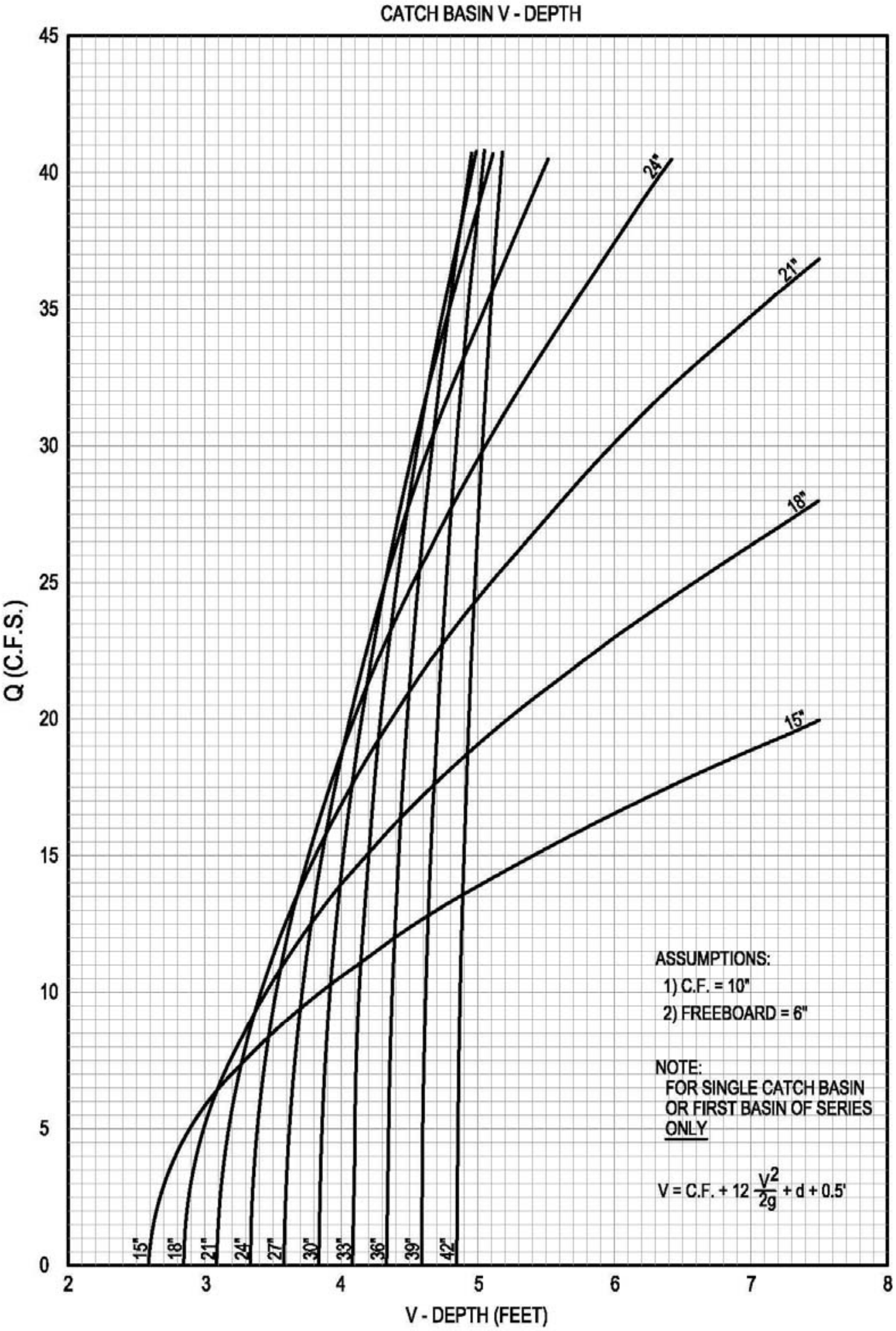

CATCH BASIN V - DEPTH
Q (C.F.S.)
V - DEPTH (FEET)
45
40
35
30
25
20
15
10
5
0
2
3
4
5
6
7
8
24"
21"
18"
15"
15"
18"
21"
24"
27"
30"
33"
36"
39"
42"
ASSUMPTIONS:
1) C.F. = 10"
2) FREEBOARD = 6"
NOTE:
FOR SINGLE CATCH BASIN
OR FIRST BASIN OF SERIES
ONLY
V = C.F. + 12 V2/2g + d + 0.5'

PLATE 22.3 D-9

STORM INLET **CALCULATION SHEET**

Sht___ of___

PROJECT ____________________

DESIGN FREQUENCY ____________________

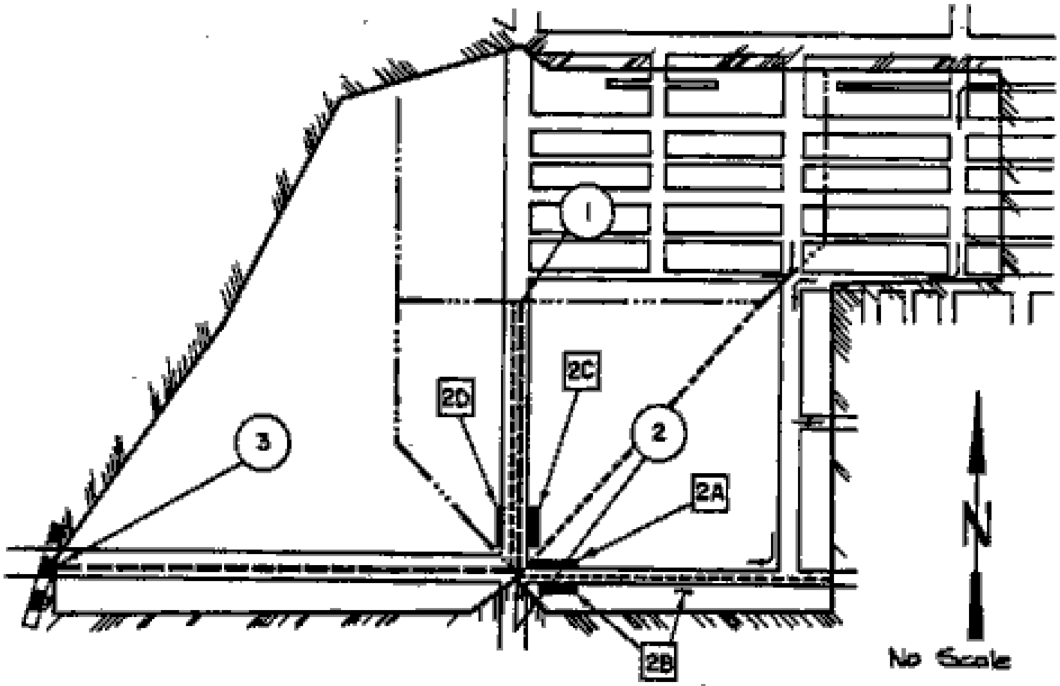
CALCULATED BY ____________

DATE ____________________

FLOW DIAGRAM (Indicate street slopes)	Sym.	Drain. Area	Q		Cap. of ½ Street	Gutter "d"	C.B.		Connector Pipe			V
			Total	Inter.			No.	Size	Head	L	Dia.	Depth

PLATE 22.3 D-10

EXAMPLE CATCH BASIN HYDROLOGY PROBLEM



LEGEND

- Major Drainage Area Boundary
- Mainline Sub-Drainage Area Boundaries
- Catch Basin Sub-Drainage Area Boundaries
- Flow Path
- Mainline
- Outlet
- Catch Basins
- ◯ Mainline Sub-Drainage Area Numbers
- ☐ Catch Basin Sub-Drainage Area/ Numbers

MAINLINE HYDROLOGY DATA **

Reach or Sub-Area	Area (Acres)		Q (c.f.s.)	
	Sub-Area	Total	Sub-Area(2)	Reach
①	45		70	
①-②		45		70
②	70		105	
②-③		115		160
③	50		75	
③-Outlet		165		220

** Provided by L.A.C.F.C.D. Hydraulic Div.

CATCH BASIN HYDROLOGY *

For Mainline Sub Drainage Area No. 2

$A_T = 70$ Acres $\quad Q_P = 105$ c.f.s. $\quad Q_P/A_T = \frac{1.5 \text{ c.f.s.}}{\text{Acre}}$

C.B. Sub Drain Area #	A (Acres)		Q_P/A_T (c.f.s./Acre)		Q_{des} (c.f.s.)
2A	40	x	1.5	=	60
2B	5	x	1.5	=	7.5
2C	15	x	1.5	=	22.5
2D	10	x	1.5	=	15

$$Q_{des} = \frac{Q_P}{A_T} A$$

PLATE 22.3 D-11

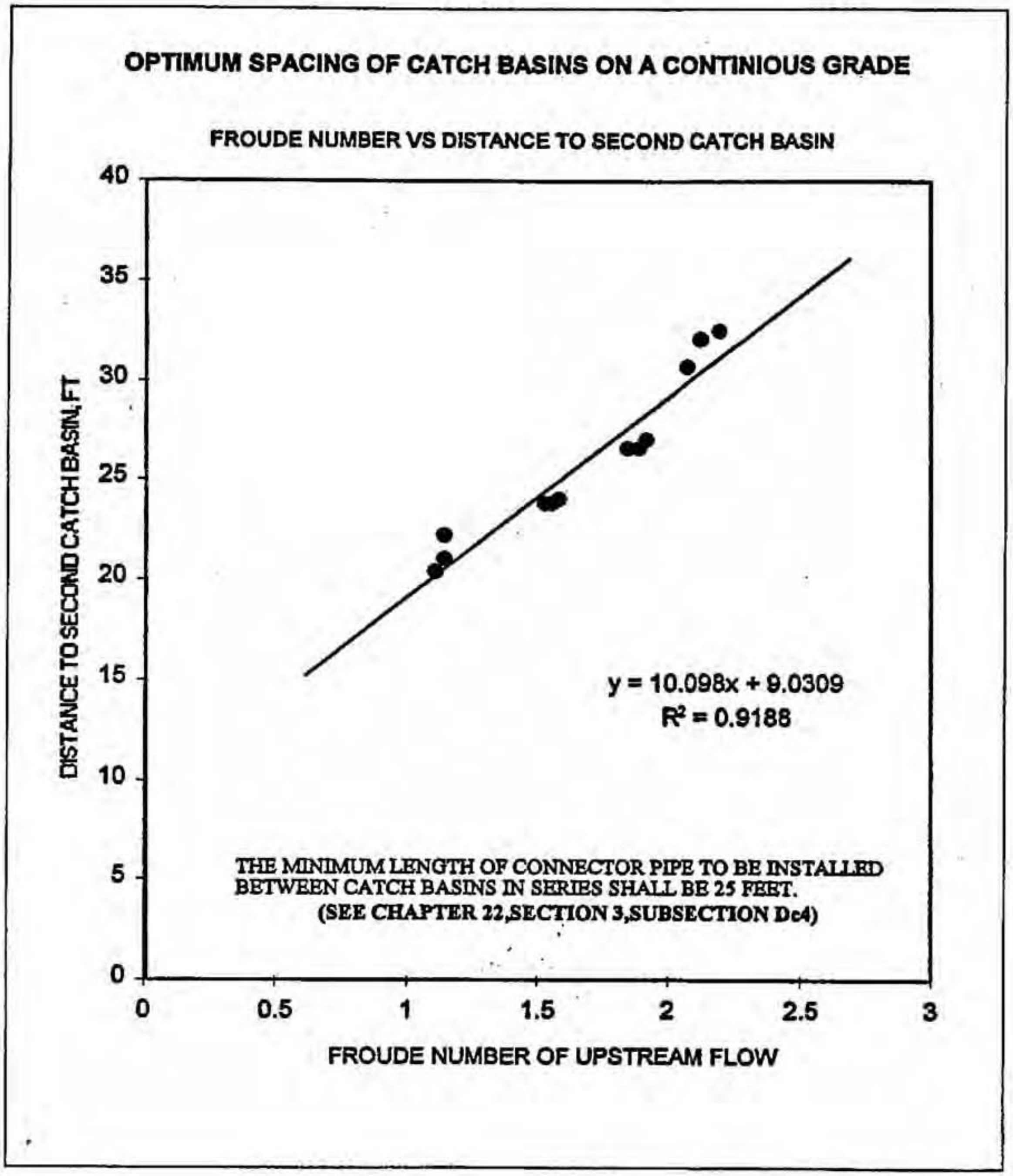
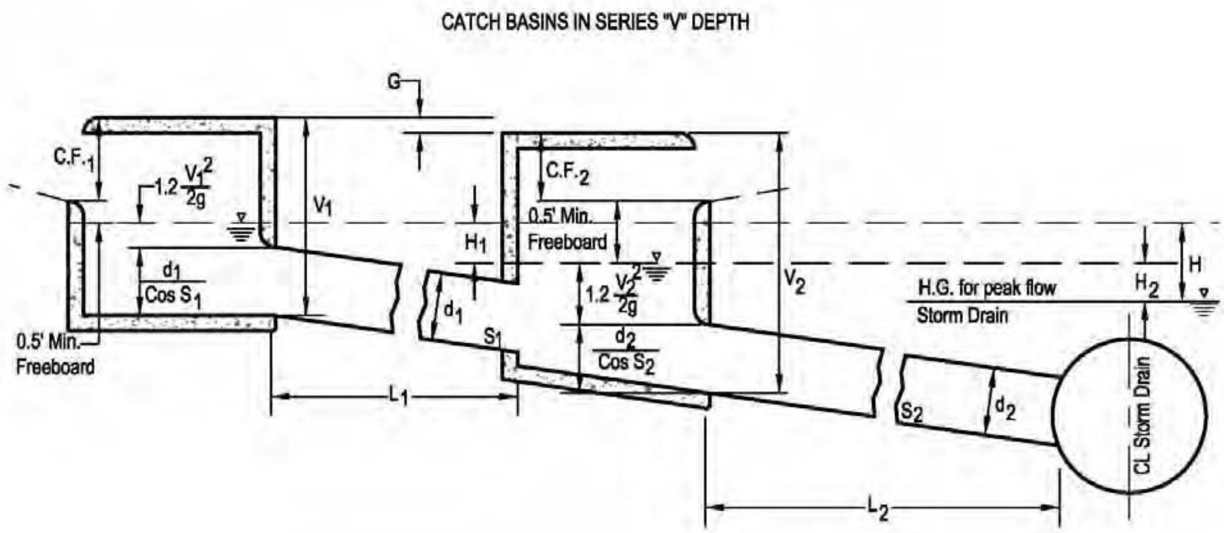


PLATE 22.3 D-12

d. Storm Inlets in Series



Select a connector pipe size for each storm inlet, and determine the related head loss (H_1, H_2) by means of a culvert equation, or by Plate 22.3 D-9. The sum of head losses in the series should not exceed the available head, i.e.,

$$H_1 + H_2 + \ldots\ldots\ldots\ldots + H_n \leq H$$

The minimum storm inlet "V" depths are determined in the following manner:

(1) The first storm inlet "V" depth is calculated as for a single storm inlet:

$$V_1 = 1.33 + 1.2 \frac{V_1^2}{2g} + d_1$$

(2) The second storm inlet "V" depth is determined as follows:

$$V_2 = C.F._1 + 0.5 + H_1 + 1.2 \frac{V_2^2}{2g} + \frac{d_2}{\text{Cos } S_2} - G$$

Assuming again that C.F. = 0.83 and Cos S_2 = 1,

$$V_2 = 1.33 + H_1 + 1.2 \frac{V_2^2}{2g} + d_2 - G$$

(3) The freeboard provided for the second storm inlet generally should not be less than 0.5 feet and shall be checked as follows:

$$FB_2 = V_2 - \frac{d_2}{\text{Cos } S_2} - 1.2 \frac{V_2^2}{2g} - C.F._2$$

If $C.F._2 = 0.83$ and $\text{Cos } S_2 = 1$,

$$FB_2 = V_2 - d_2 - 1.2 \frac{V_2^2}{2g} - 0.83$$

Where especially "tight" conditions prevail, the 0.5 feet freeboard requirement referred to above may be omitted. In such cases the difference between the gutter elevation and the hydraulic grade line elevation of the main line will be accepted as the available head.

(4) Connector pipes between storm inlets in series are be checked for adverse slope by the following relationship:

$$V_2 - 0.5 > V_1 - G$$

The figure of 0.5 shown above is the standard 6-inch cross slope of the storm inlet floors.

3. Other Criteria

a. General

(1) Existing drainage systems which are not required to carry any portion of the design Q of a proposed system may be designated to be abandoned in place upon completion of the proposed drain. Such existing drainage systems should not be sealed or removed before completion of the proposed system, if needed to carry off storm water during the construction period. It is the designer's responsibility to ascertain the necessity of maintaining existing drainage systems in place.

Existing street or sidewalk culverts may be designated to have the interfering portions removed and the inlets sealed, or the culverts may be kept in operation and connected to the storm drain or to the back of a proposed storm inlet. If the culvert is to be connected, a structural detail should be provided. Refer to the City Engineer/SSCAFCA for instructions.

Existing street or sidewalk culverts that do not interfere with construction should be maintained in place.

(2) Storm inlets will be located within street rights-of-way unless otherwise approved by the City Engineer/SSCAFCA. All storm inlets which must be located outside street property lines in order to intercept storm waters under existing conditions are

considered "must" storm inlets. Right-of-way or an easement for such storm inlets must be acquired. Storm inlets may be located outside dedicated streets to accommodate future street widenings and should be designed to intercept storm water under existing conditions.

Storm inlets to be constructed off the paved portion of the roadway but within the street property lines must be made operable by grading the roadway to permit storm water to flow to the basin. Street remodeling of this nature will be performed during construction.

(3) If a project is to have one or more cutoff points in phased construction, each cutoff point should have a battery of storm inlets at the upstream terminus sufficient to collect the flow carrying capacity of the storm drain at that point. Each battery of storm inlets should be designed with sufficient data regarding types and sizes of storm inlets, connector pipe sizes and D-loads, "V" depths, local depressions, and whatever other information may be necessary to construct the system.

(4) Sump designs for storm inlets should normally be limited to local streets and only those situations where terrain or grading considerations warrant their use. When specifying a sump storm inlet(s) the designer shall ensure that surrounding properties are protected from the occurrence of system clogging by demonstrating that one of the following emergency backup conditions exist:

1) The design storm peak flow rate will release to either a public R.O.W. or public easement without rising above any adjacent structure pad elevations; or

2) Sufficient storage is available within a combination of public R.O.W., public easement, and nonstructurally occupied private properties to hold 100% of the design event volume, without inflicting damage to structures, until such time as the underground system can be unclogged.

When relying on public easements across private property to achieve either objective, the easement language creating the encumbrance shall specify that said easement is a surface flowage easement and no structural improvements which would interfere with conveyance or storage of water shall be allowed. Any surface modification within the flowage easement will require an encroachment agreement from the City.

b. Storm Inlets

The selection of type, number, and spacing of storm inlets should be based on Plates 22.3 D-l through 22.3 and the following instructions. Be aware that the City of Rio Rancho standard street curb heights are 6” and this may require design and construction adjustments.

City standard storm inlets "Type A, B and C" are combination inlet(s) with both curb opening and grading. Storm inlet "Type D" is a grating only inlet. Basin gratings tend to accumulate debris and clog. The curb opening both limits debris accumulation and offsets lost capacity due to clogging of the grating. Except for certain valley applications, combination basins should be used. Due to main line clogging, grating only basins should be used in valley applications where main line pipe diameters are 24" or less or where quarter full pipe velocities are less than 2.5 f.p.s.

"Type A" storm inlets should be used for single inlet applications and as the first inlet in a battery of inlets. The "Type A" basin performs the function of sweeping debris of the street upstream of the grating and minimizing clogging. "Type A" inlets are used with standard curb and gutter.

"Type B" storm inlets are generally placed downstream of and/or in conjunction with "Type A" storm inlets on streets other than arterials and collectors. This type storm inlet has potential to collect substantial runoff when the grating is clean. If "Type B" basins are used alone, without a "Type A" within 150 feet upstream, the capacity shown in Plate 22.3 D-7 should be reduced 15% due to clogging. "Type B" storm inlets are used with standard curb and gutter.

"Type C" storm inlets are generally placed downstream of and/or in conjunction with "Type A" storm inlets. If "Type C" storm inlets are used without a "Type A" within 150 feet upstream, the capacity shown in Plates 22.3 D-5 and 22.3 D-6 should be reduced 15% for clogging. "Type C" storm inlets are used with standard curb and gutter.

"Type D" storm inlets are generally used on streets with slope greater than 5%, in driveways and in certain valley areas as described above. "Type D" storm inlets can be used with either standard curb and gutter or with mountable curb.

The number of storm inlets to be connected in series should not exceed two. If the connection of more than two storm inlets in series is unavoidable, consideration should be given to designing a lateral drain.

c. Connector Pipe

(1) The minimum diameter of connector pipe is 18 inches.

(2) The horizontal alignment of connector pipes must not contain angle points or bends, unless approved by the City Engineer/SSCAFCA.

(3) Connections at manholes or junction structures are preferred.

(4) The storm inlet spacing shall be a minimum of 25 feet between curb transitions.

(5) Storm inlet connector pipes shall outlet at the downstream end of the storm inlets, unless prevented by field conditions. Downstream, in this paragraph, refers to the directions of the gutter slope at the storm inlet in question.

(6) Where feasible, connector pipes should be located so as to avoid, as much as possible, cutting into existing cross gutters and spandrels.

(7) The conversions of type A's, B's or C's to D's storm inlets will not be permitted. If the storm inlet is in conflict with a driveway, the storm inlet will be removed and replaced with another inlet outside of the driveway. To avoid conflicts with driveways, the engineer should identify the proposed driveways on the grading plan when storm inlets front the lots.

E. Street Hydraulics

l. A secondary use of the street network is the conveyance of storm runoff. This secondary use must always be subsidiary to the primary function of streets which is the safe conveyance of people and vehicles. The goals of street hydraulic design are therefore:

   a. To provide an economical means of transporting storm runoff.

   b. To ensure that the safety and convenience of the public are preserved.

   c. To prevent storm runoff, once collected by the street system, from leaving the street right-of-way except at specially designated locations.

2. Street hydraulic design criteria are as follows:

   a. Manning's roughness coefficient is 0.017.

   b. Conjugate and/or sequent depth in the event of the 100-year design discharge may not exceed curb height and shall be contained within the street right-of-way.

   c. Flow depths in the event of the 10-year design discharge may not exceed 0.33 feet in any collector or arterial street. One lane free of flowing or standing water in each traffic direction must be preserved on arterial streets.

   d. The product of depth times velocity shall not exceed 6.5 in any location in any street in the event of a 10-year design storm (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet.)

3. For streets with more than two driving lanes in each direction:

   a. The product of depth times velocity may not exceed 6.5 at any location in any street in the event of a 10-year design storm (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet).

b. Inverted crown streets are prohibited unless prior authorization provided to and approved by SSCAFCA.

c. The assumption of equal flow distribution between gutters on undivided streets and between street sections on divided streets is only valid where its validity can be demonstrated.

4. Plates 22.3 D-1 through 22.3 D-4 may be used where applicable in the hydraulic design of streets. T-intersections, radical slope changes and intersections are potential locations for hydraulic jumps when upstream slopes are steeper than critical slope.

   When conditions indicate that a hydraulic jump or that the effects of superelevation will allow runoff to exceed street hydraulic design criteria, provisions must be made for treatment of the problem. The warping of street sections and the construction of deflector walls for these purposes is prohibited unless specifically authorized by the City Engineer/SSCAFCA.

5. Intersections and other radical changes in street cross section and slope require special consideration whenever the flow depth/street slope relationship results in flows occurring in the supercritical flow regime. The critical slope line shown on the street rating curves is used to determine on which side of critical depth the flow occurs and if slope or cross section changes will allow the flow to cross through critical depth from supercritical.

   If flow is likely to cross into the subcritical flow range, then Plate 22.3 E-1, "Tail Water vs. Froude Number" is used to determine the height and Plate 22.3 E-2, "Length of Jump vs. Froude Number" figure is used to determine jump length. The height of jump should not exceed curb height and shall be contained within the street right-of-way.

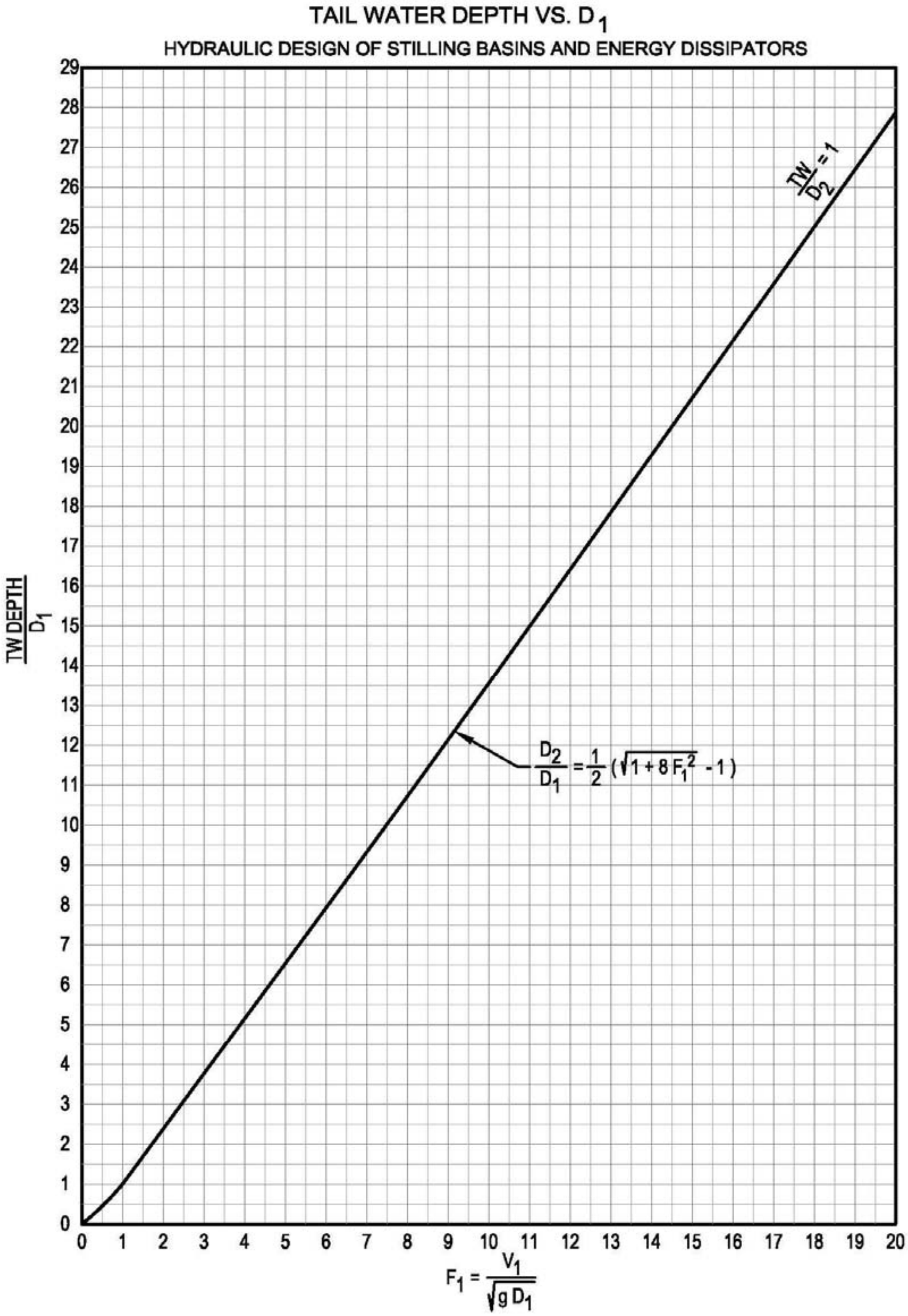


FIGURE5. - RATIO OF TAIL WATER DEPTH TO D_1 (BASIN I)

PLATE 22.3 E-1

LENGTH OF JUMP IN TERMS OF D_1

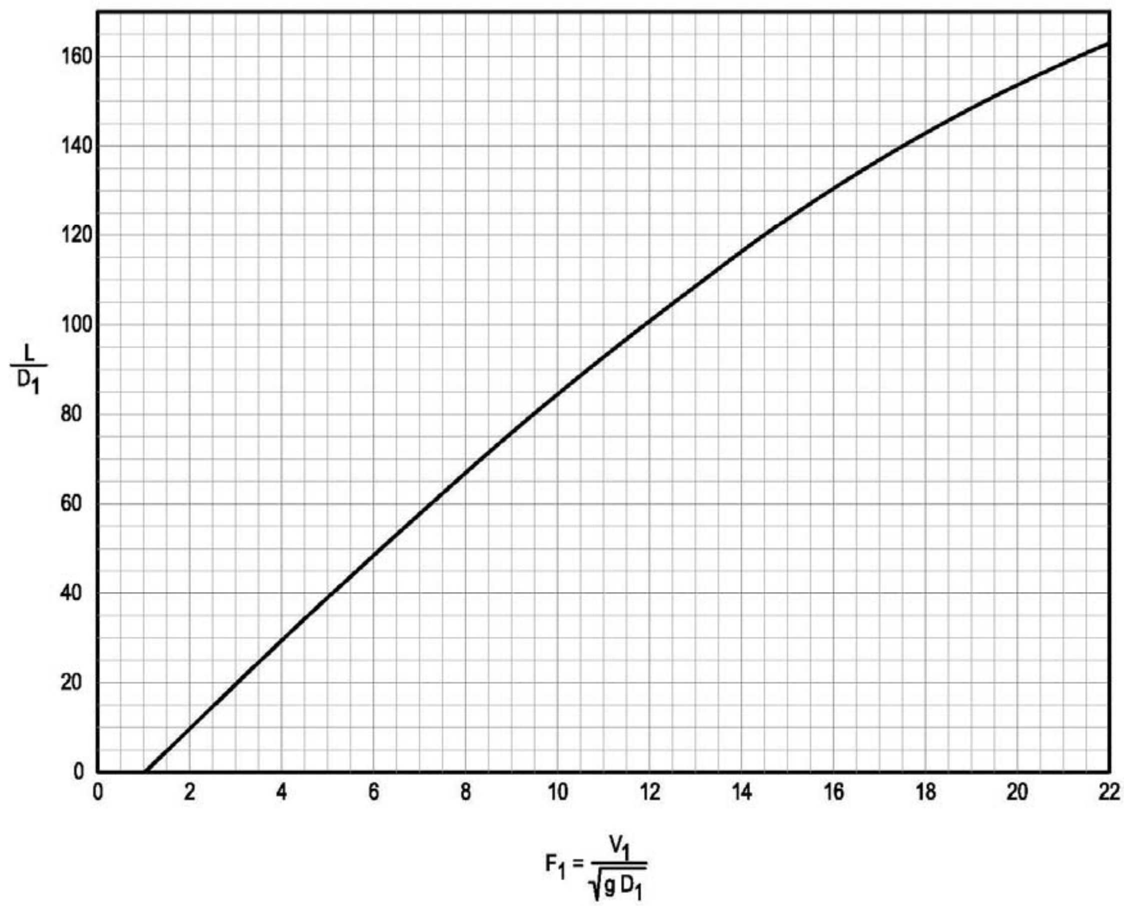


FIGURE 6. - LENGTH OF JUMP IN TERMS OF D_1 (BASIN I)

PLATE 22.3 E-2

F. Berms and Levees

All levees and berms constructed for drainage or flood control purposes and which are required to contain or convey 50 cfs or more in the event of the 100 year design discharge must conform to the following guidelines:

1. Cross Section

   a. Unarmored faces of berms and levees must have side slopes not steeper than 6:1 (horizontal to vertical).

   b. Rock rip-rapped faces of berms and levees must have side slopes not steeper than 3:1 (horizontal to vertical)

   c. Concrete faced berms and levees will have side slopes of 2:1 (horizontal to vertical)

   d. Berms and levees less than 4.0 feet in height must have a minimum top width of 8.0 feet.

   e. Berms and levees 4.0 feet high and greater must have a minimum top width of 12 feet.

   f. All berms and levees must be provided with a structural keyway with bottom width equal to the top width and depth equal to at least one half the height, but not less than 3.0 feet and side slopes not steeper than 2:1 (horizontal to vertical)

2. Certification

   All levees and berms shall be inspected during construction and certified by a New Mexico Professional Engineer as to their substantial compliance to the plans and specifications. Certified as-built drawings, accompanied by daily inspection reports, shall also be provided.

3. Berm or Levee

   Any berm or levee whose purpose is to divert or convey runoff in a major arroyo shall be specially designed on a case-by-case basis and shall meet or exceed the guidelines listed herein.

4. Freeboard

   Berms and levees must be provided with freeboard for the 100-year design flow based on the following guidelines:

   a. For flow depths less than 3.0 feet and not involving a major arroyo; minimum freeboard is 2.0 feet.

b. For flow depths 3.0 feet and greater and, involving a major arroyo; minimum freeboard is 3.0 feet.

c. If the berm or levee structure is necessary to protect existing property or structures from a FEMA flood plain, FEMA criterion must be complied with in the design and construction of the structure.

5. Bank Protection

All berms and levees expected to convey or divert 50 cfs or more in the event of the 100-year design discharge must be provided with bank protection according to the following guidelines:

a. Bank protection must be provided wherever design velocities exceed 3 feet/sec.

b. Bank protection must be provided on the outside of curves from the beginning of curvature, through the curve and for a distance equal to 5 times the flow velocity in feet downstream from the point of tangency.

c. When required, bank protection must be provided to two feet above the design flow depth plus additional depth as required (e.g. superelevation, waves at confluences, hydraulic jumps, etc.).

d. Bank protection must extend downward on a projection of the bank slope, to a minimum depth equal to 1.5 times the design flow depth but never less than 3.0 feet. Bank protection for major arroyos shall be accompanied by a City Engineer/SSCAFCA approved sediment transport analysis.

NOTE: *Berms, dams, levees, and diversions of certain magnitudes and nature may fall within the jurisdiction of the Office of the State Engineer. The design professional is expected to be aware of and comply with regulations promulgated by that jurisdiction.*

G. Miscellaneous Hydraulic Calculations

1. Hydraulic Jump

a. Location

If the water surface from a downstream control is computed until critical depth is reached, and similarly the water surface from an upstream control is computed until critical depth is reached, a hydraulic jump will occur between these controls and the top of the jump will be located at the point where pressure plus momentum, calculated for upper and lower stages, are equal.

b. Length

The length of a jump is defined as the distance between the point where roller turbulence begins and water becomes white and foamy due to air entrainment, and the point downstream where no return flow is observable.

(1) For rectangular channels, the length of jump (L) for the range of Froude Numbers between two and twenty, based on flow depth, is given by the following equation:

$L = 6.9\ (D_2 - D_1)$

where D_1 and D_2 are the sequent depths.

(2) For trapezoidal channels, the length of jump (L) is given by the following equation:

$$L = 5D_2\left(1+4\sqrt{\frac{t_2-t_1}{t_1}}\right)$$

where t_1 = width of water before jump

t_2 = width of water after jump

Side Slope	$L/(D_2-D_1)$
2:1	44.2
1:1	33.5
1/2:1	22.9
Vertical	6.9

2. Trashrack Head Loss

The head loss through a trashrack is commonly determined from the following equation:

$h_{TR} = K_{TR}\ (V_n/2g)$

$K_{TR} = 1.45 - 0.45\ (A_n/A_g) - (A_n/A_g)^2$

where K_{TR} = Trashrack coefficient

A_n = Net area through bars, in ft.^{2}

A_g = Gross area of trashrack and supports (water area without trashrack in place), in ft.^{2}

V_n = Average velocity through the rack openings (A/A_n), f.p.s.

For maximum head loss, assume that the rack is clogged, thereby reducing the value of A_n by 50%.

3. Side Channel Weirs:

The Los Angeles District Corps of Engineers, as mentioned in Section C-2.5, has developed a side channel spillway inlet. The City or SSCAFCA may require this type of structure for drains outletting into their facilities. The Corps' procedure for designing a side channel spillway is as follows:

a. Set the top of that part of the main channel wall at the location of the proposed spillway about 6 inches above the computed water surface level in the main channel.

b. Determine the length of spillway (L) required to discharge the design inflow of the side inlet by the following equation, in which the maximum value of H is not greater than one and one-half feet.

$$L = \frac{Q}{CH^{3/2}}$$

where: Q = discharge of side inlet, in c.f.s.

C = weir coefficient

H = depth of water over the crest of the side inlet in feet

c. Determine the depth of flow in the approach side channel at the upstream end of the spillway.

d. Set the side channel invert elevation at the upstream end of the spillway at an elevation below the spillway crest a distance equal to the water depth as determined in c., above, minus the assumed head on the spillway.

e. Set the side channel invert slope equal to the spillway and the main channel water-surface slopes.

f. By trial, determine the width of the side channel required to maintain a constant depth of flow at several points downstream from the upstream end of the spillway. The discharge at each of these points is assumed to be the difference between the initial discharge less the amount spilled over that part of the spillway as computed by CLH3/2, in which C is

3.087 and H is equal to the critical depth over the crest (neglecting the velocity of approach).

g. Plot the widths thus determined for the side channel on the channel plan and approximate a straight or curved line through them to locate the point of intersection of this line and the main channel wall.

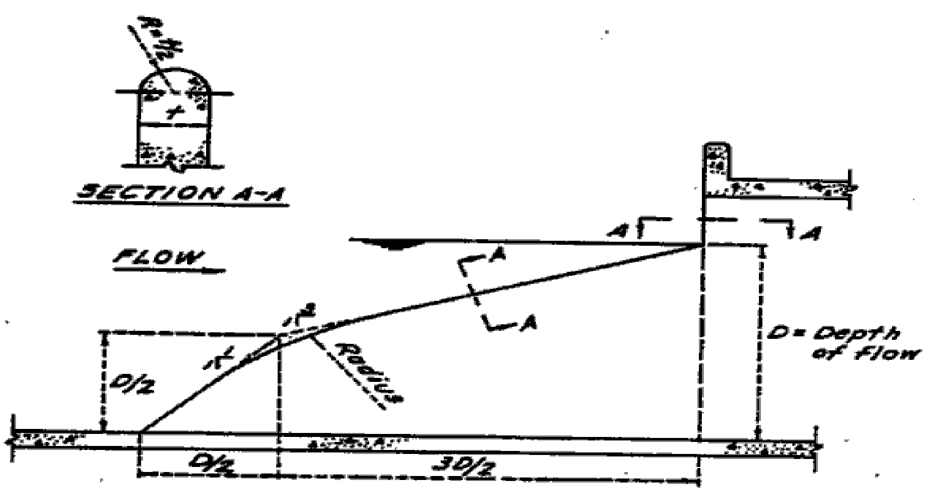
h. If the length between the assumed point at the upstream end of the spillway and this intersection point is equal to the length determined in b., above, the angle at the intersection indicates the required convergence for the side channel.

i. From the final layout determine the width and recompute the water surface in the side channel for the final design. The discharge over each portion of the spillway is calculated by using the average head between the two sections considered.

4. *Pier Extensions:*

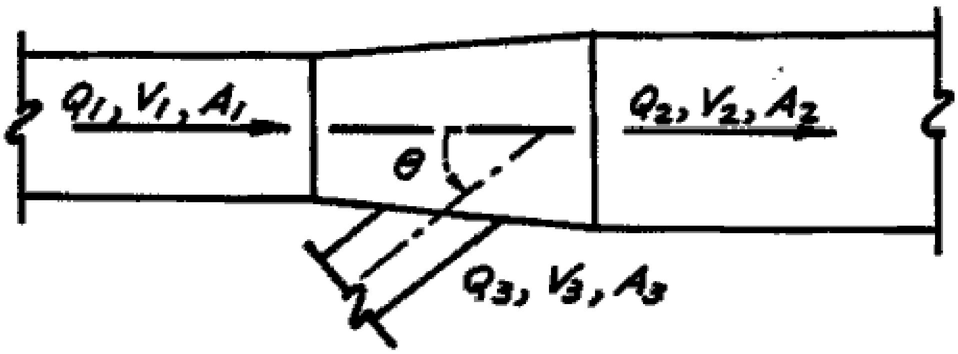
Pier extensions of a streamlined nature should be used when heavy debris flow is anticipated.

In supercritical flow the addition of a specified width to account for the assumed amount of debris may result in impractical and costly structures. In lieu of assuming additional pier width for debris, the use of streamline pier extensions should be investigated. Unless an unusual quantity of debris is anticipated, it can be assumed that the major portion of the debris will not cling to the pier extension. Pier extensions should be designed using the criteria indicated in the figure below.



5. Junctions

a. Thompson Equation
The Thompson Equation for junctions is described by the following:



$$\Delta y \cdot A_{avg.} = \frac{Q_2 V_2 - Q_1 V_1 - Q_3 V_3 \cos\theta}{g}$$

Where Δy = difference in hydraulic gradient for the two end sections, in feet
A_{avg} = average area, in feet^2 = $1/6\,(A_1 + 4A_m + A_2)$ or, for practical use $1/2\,(A_1 + A_2)$
A_m = mean area of flow, in feet^2

The above equation is applicable only to prismoidal and circular conduits or channels. The friction force may be considered negligible or can be calculated and taken into account. It is recommended that the Thompson equation not be used when an open channel changes side slope going through a junction.

For details of the above method, refer to the Los Angeles County Flood Control District Hydraulic Design Manual, March 1982.

In the following compilations:

a. "w", the unit weight of water, has been omitted since it appears in all terms.

b. The assumptions are made that the cosines of the invert slopes equal unity and that the tangents and sines of the friction slopes are equal.

<u>The general equilibrium equation for all cases is:</u>

$$P_2 + M_2 = P_1 + M_1 + M_3\cos\theta + P_i + P_w - P_f$$

where P_1 = hydrostatic pressure on section 1

P_2 = hydrostatic pressure on section 2

P_i = horizontal component of hydrostatic pressure on invert

P_w = axial component of hydrostatic pressure on walls

P_f = retardation force of friction (S_1 and S_2 are friction slopes - see Kings Hdbk.)

M_1 = momentum of moving mass of water entering junction at section 1

M_2 = momentum of moving mass of water leaving junction of section 2

M = axial component of momentum of the moving mass of water entering the junction at section 3

$P = A\overline{y}$

$\overline{y}$ = distance to centroid from water surface

$$M = \frac{Q^2}{g * A}$$

CASE 1. OPEN TRAPEZOIDAL CHANNEL

$$M_1 = \frac{Q_1^2}{(b_1 + z_1 D_1)\, g D_1}$$

$$M_2 = \frac{Q_2^2}{(b_2 + z_2 D_2)\, g D_2}$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{A_3 g} (\cos\theta) \qquad \text{where } A_3 = \text{water area at section 3}$$

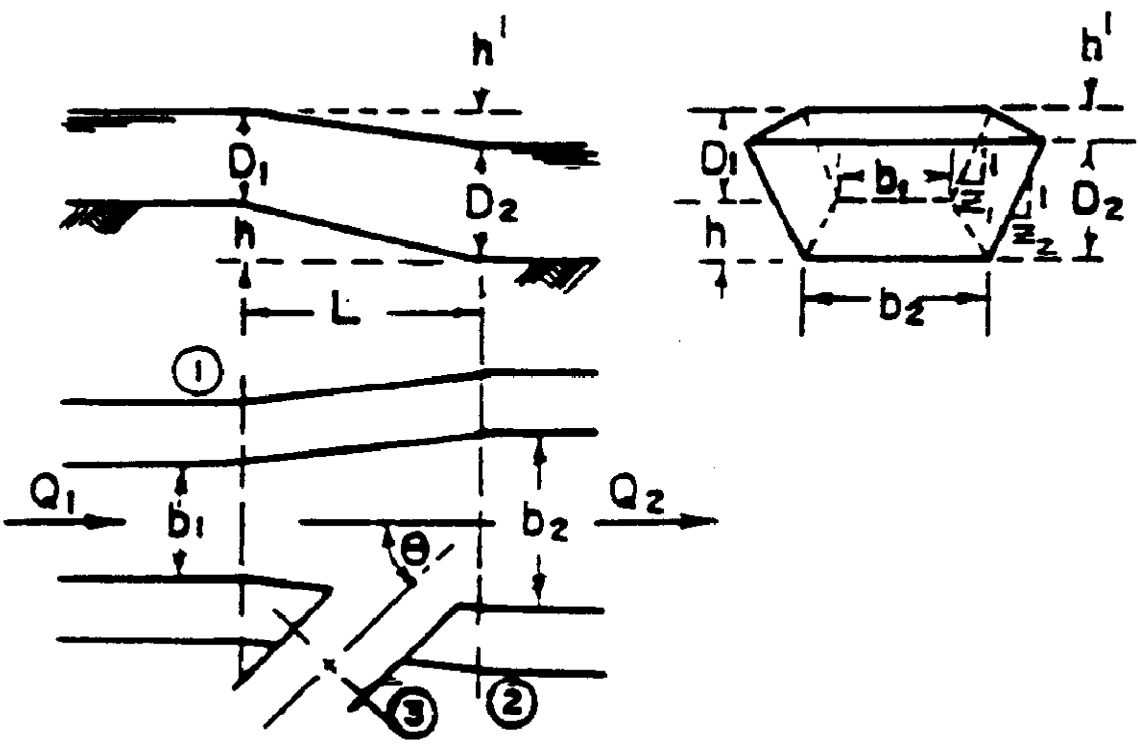
$$P_1 = \frac{D_1^{\,2}}{6} \cdot (3b_1 + 2z_1 D_1)$$

$$P_2 = \frac{D_2^{\,2}}{6} \cdot (3b_2 + 2z_2 D_2)$$

$$P_i = \left(\frac{b_1 + b_2}{2}\right) h \left[D_1 + \frac{(D_2 - D_1)(b_1 + 2b_2)}{3(b_1 + b_2)} \right]$$

$$P_w = \frac{D_1 + D_2}{4} \left[\frac{b_1 + b_2}{2} (D_1 - D_2) + h' (z_1 D_1 + z_2 D_2) + (b_2 + z_2 D_2) D_2 - (b_1 + z_1 D_1) D_1 \right]$$

$$P_f = \frac{L(s_1 + s_2)}{4} \left[(b_1 + z_1 D_1) D_1 + (b_2 + z_2 D_2) D_2 \right]$$



CASE 2. OPEN RECTANGULAR CHANNEL

$$M_1 = \frac{Q_1^2}{b_1 D_1 g}$$

$$M_2 = \frac{Q_2^2}{b_2 D_2 g}$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{A_3\, g}\left(\cos\theta\right)$$

$$P_1 = \frac{b_1 D_1^2}{2}$$

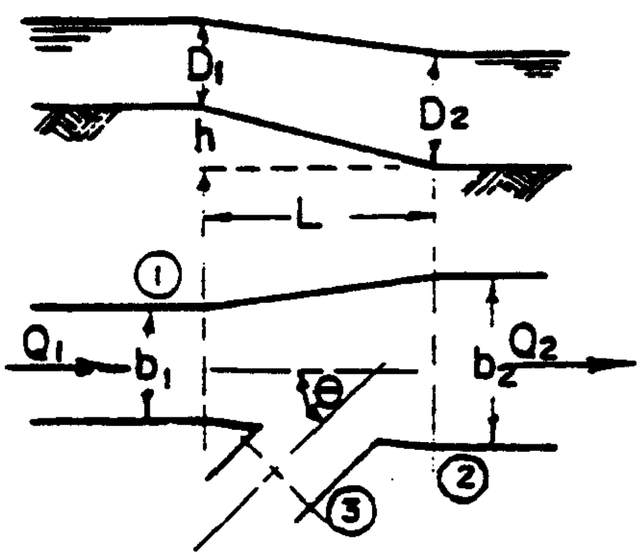
Where A_3 = water area at section 3

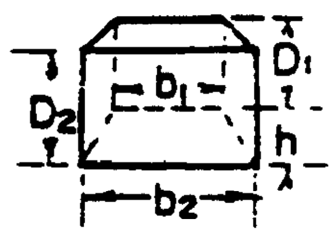
$$P_2 = \frac{b_2 D_2^2}{2}$$

$$P_i = \left(\frac{b_1 + b_2}{2}\right) h \left[D_1 + \frac{(D_2 - D_1)\,(b_1 + 2b_2)}{3\,(b_1 + b_2)}\right]$$

$$P_w = \frac{D_1 + D_2}{4}\;(b_2 - b_1)\left[D_1 + \frac{(D_2 - D_1)\,(D_1 + 2D_2)}{3\,(D_1 + D_2)}\right]$$

$$P_f = \frac{L(s_1 + s_2)}{4} \cdot (b_1 D_1 + b_2 D_2)$$





CASE 3. RECTANGULAR BOX UNDER PRESSURE

$$M_1 = \frac{Q_1^2}{b_1 d_1 g}$$

Where A_3 = water area at section 3

$$M_2 = \frac{Q_2^2}{b_2 d_2 g}$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{A_3\, g}(\cos\theta)$$

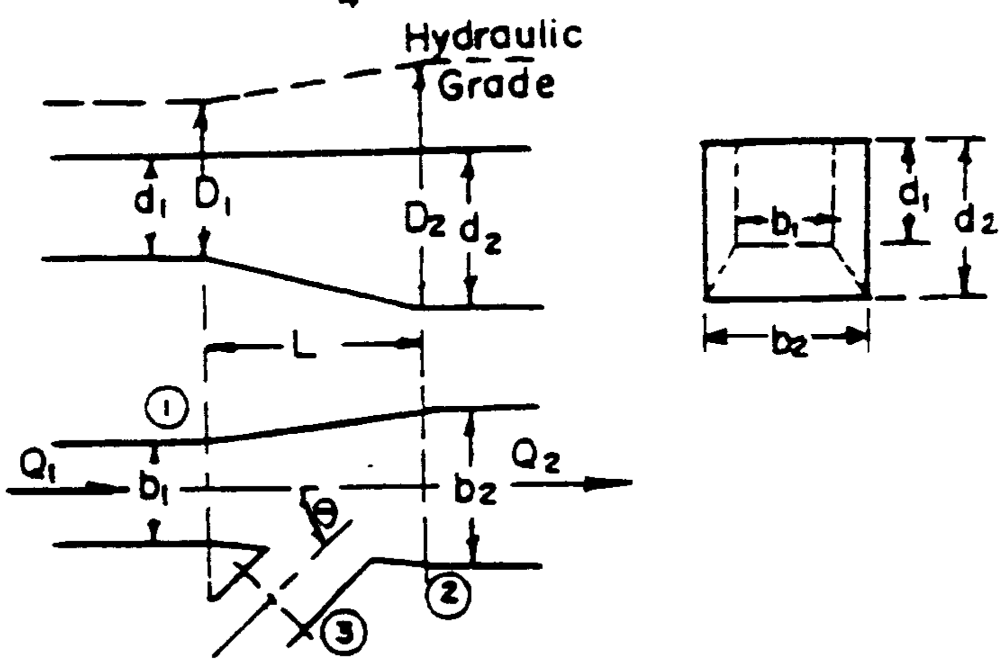
$$P_1 = b_1 d_1\left(D_1 - \frac{d_1}{2}\right)$$

$$P_2 = b_2 d_2\left(D_2 - \frac{d_2}{2}\right)$$

$$P_i = \frac{b_1 + b_2}{2}\,(d_2 - d_1)\left[D_1 + \frac{(D_2 - D_1)\,(b_1 + 2b_2)}{3(b_1 + b_2)}\right]$$

$$P_w = \frac{d_1 + d_2}{4}\,(b_2 - b_1)\left[D_1 + D_2 - \frac{d_1 + d_2}{2}\right]$$

$$P_f = \frac{L(s_1 + s_2)}{4}\,(b_1 d_1 + b_2 d_2), \ldots \text{where } s = \left[\frac{Qn(b+d)^{2/3}}{.936(bd)^{5/3}}\right]^2$$



CASE 4. CIRCULAR CONDUIT UNDER PRESSURE, PIPE INLET

$$M_1 = \frac{Q_1^2}{25.2\, d_1^2}$$

$$M_2 = \frac{Q_2^2}{25.2\, d_2^2}$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{25.2\, d_3^2} (\cos\theta)$$

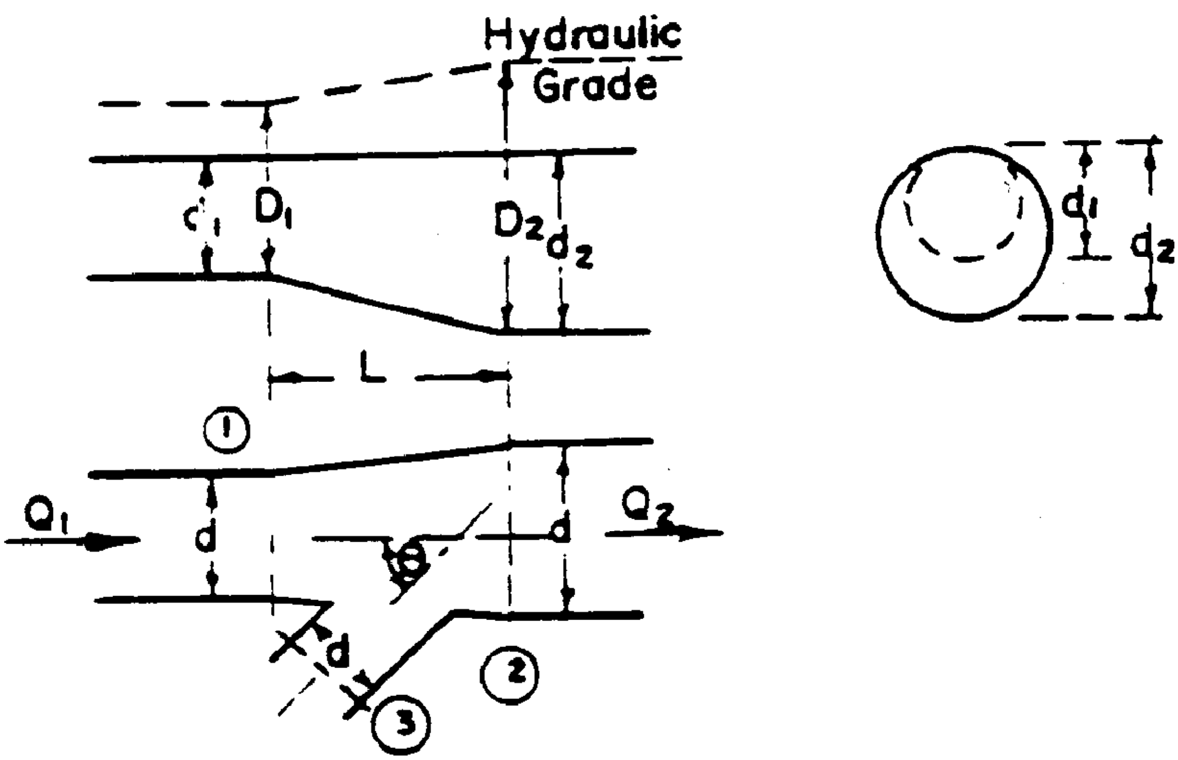
$$P_1 = .784\, d_1^2 \left(D_1 - \frac{d_1}{2}\right)$$

$$P_2 = .784\, d_2^2 \left(D_2 - \frac{d_2}{2}\right)$$

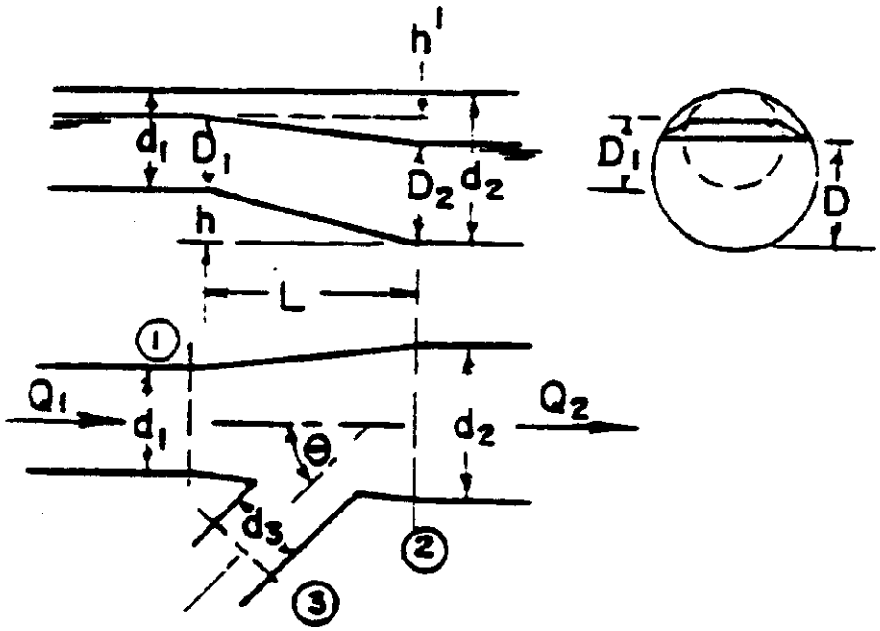
$$P_i = 0$$

$$P_w = .392\left[(d_2^3 - d_1^3) + (d_2^2 - d_1^2)(D_1 + D_2 - d_1 - d_2)\right]$$

$$P_f = .196\, L(s_1 + s_2)\,(d_1^2 + d_2^2), \ldots\ \text{where } s = \left(\frac{Qn}{.463\, d^{8/3}}\right)^2$$



CASE 5. CIRCULAR CONDUIT FLOWING PARTIALLY FULL, PIPE INLET



$$M_1 = K_1 \left(\frac{Q_1}{d_1}\right)^2$$

$$M_2 = K_2 \left(\frac{Q_2}{d_2}\right)^2$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{25.2\, d_3^2} (\cos\theta)$$

$$P_1 = C_1\, d_1^3$$

$$P_2 = C_2\, d_2^3$$

$$P_i = o$$

* $$P_w = A_2\bar{y}_2 - A_1\bar{y}_1 + \frac{h'}{2} (A_2 + A_1) + \frac{(h')^2}{12} (T_2 - T_1)$$

$$P_f = \frac{L(s_1 + s_2)}{4} (A_1 + A_2)$$

*WHERE $h' = h + D_1 - D_2$, THE TERM $\frac{(h')^2}{12}(T_2 - T_1)$ IS USUALLY NEGLIGIBLE.

Los Angeles County Flood Control District

FACTORS FOR CIRCULAR CONDUITS FLOWING PARTLY FULL

D = depth of water
d = diameter of conduit

Tabulated Values

K = momentum / $(Q/d)^2$ C = Pressure / d^3 F = Velocity Head / $(Q/d^2)^2$

D/d	K	C	F	D/d	K	C	F	D/d	K	C	F	D/d	K	C	F
.00	∞	.0000	∞	.25	.2026	.0157	0.659	.50	.0792	.0833	.1007	.75	.0492	.2121	.0389
.01	23.919	.0000	9188.	.26	.1916	.0173	0.589	.51	.0773	.0873	.0958	.76	.0485	.2185	.0379
.02	8.403	.0000	1134.	.27	.1817	.0190	0.530	.52	.0753	.0914	.0912	.77	.0479	.2249	.0369
.03	4.507	.0001	326.	.28	.1727	.0207	0.479	.53	.0736	.0956	.0869	.78	.0473	.2314	.0359
.04	2.961	.0002	140.9	.29	.1645	.0226	0.435	.54	.0719	.0998	.0829	.79	.0467	.2380	.0351
.05	2.115	.0003	71.9	.30	.1569	.0255	0.395	.55	.0703	.1042	.0793	.80	.0462	.2447	.0342
.06	1.620	.0005	42.1	.31	.1493	.0266	0.361	.56	.0687	.1087	.0758	.81	.0456	.2515	.0334
.07	1.285	.0007	26.5	.32	.1435	.0287	0.331	.57	.0672	.1133	.0726	.82	.0451	.2584	.0327
.08	1.058	.0010	17.97	.33	.1376	.0309	0.304	.58	.0658	.1179	.0696	.83	.0446	.2653	.0320
.09	0.888	.0013	12.68	.34	.1320	.0332	0.280	.59	.0645	.1227	.0668	.84	.0441	.2723	.0313
.10	0.760	.0017	9.28	.35	.1269	.0356	0.259	.60	.0632	.1276	.0641	.85	.0437	.2794	.0307
.11	0.662	.0021	7.03	.36	.1221	.0381	0.240	.61	.0620	.1326	.0617	.86	.0433	.2865	.0301
.12	0.582	.0026	5.45	.37	.1177	.0407	0.222	.62	.0608	.1376	.0594	.87	.0429	.2938	.0295
.13	0.518	.0032	4.31	.38	.1135	.0434	0.207	.63	.0597	.1428	.0572	.88	.0425	.3011	.0290
.14	0.466	.0038	3.48	.39	.1096	.0462	.1931	.64	.0586	.1481	.0551	.89	.0421	.3084	.0285
.15	0.421	.0045	2.84	.40	.1060	.0491	.1804	.65	.0575	.1534	.0532	.90	.0418	.3158	.0280
.16	0.383	.0053	2.36	.41	.1026	.0520	.1689	.66	.0565	.1589	.0514	.91	.0414	.3233	.0276
.17	0.351	.0061	1.982	.42	.0993	.0551	.1585	.67	.0559	.1644	.0496	.92	.0411	.3308	.0272
.18	0.324	.0070	1.681	.43	.0963	.0583	.1489	.68	.0547	.1700	.0480	.93	.0408	.3384	.0266
.19	0.299	.0080	1.438	.44	.0934	.0616	.1402	.69	.0538	.1758	.0465	.94	.0406	.3460	.0265
.20	0.278	.0091	1.242	.45	.0907	.0650	.1321	.70	.0530	.1816	.0450	.95	.0403	.3537	.0261
.21	0.259	.0103	1.080	.46	.0882	.0684	.1248	.71	.0521	.1875	.0437	.96	.0401	.3615	.0259
.22	0.243	.0115	0.946	.47	.0857	.0720	.1180	.72	.0514	.1935	.0424	.97	.0399	.3692	.0256
.23	0.228	.0128	0.833	.48	.0834	.0757	.1118	.73	.0506	.1996	.0411	.98	.0398	.3770	.0254
.24	0.215	.0143	0.740	.49	.0813	.0795	.1060	.74	.0499	.2058	.0400	.99	.0397	.3848	.0253
												1.00	.0396	.3927	.0252

F - 03

b. Oblique Waves

When evaluating junctions, refer to EM 1110-2-1601, Hydraulic Design of Flood Control Channels, 1994 edition, U.S. Army Corps of Engineers for the appropriate methodology. One of the methods from this reference, Plate B-54, is included on the following page.

c. Example Problems

For Implementation of the Thompson equation, please refer to example problems 1 through 6 at the end of this section.

For additional examples and information about junction evaluation, refer to the Los Angeles County Flood Control District Hydraulic Design Manual, March 1982, or to the Office Standard No. 115, Hydraulic Analysis of Junctions, 1968 edition, Storm Drain Design Division, Bureau of Engineering, City of Los Angeles.

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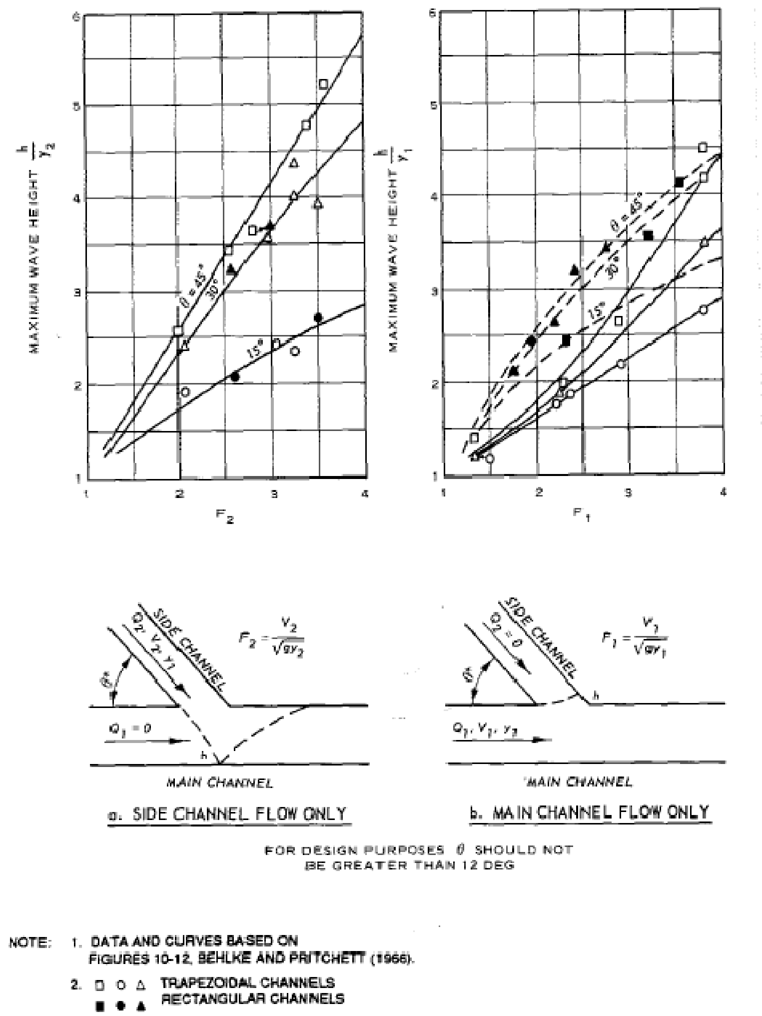
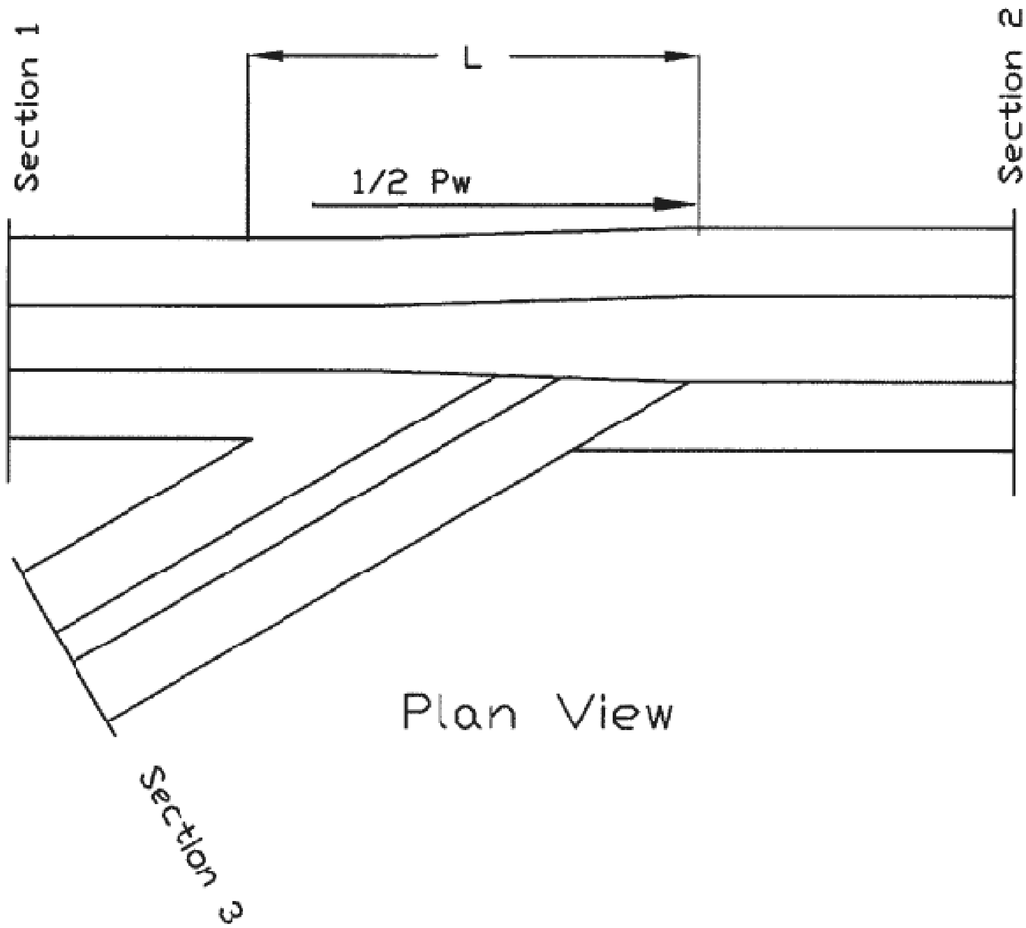


MAXIMUM WAVE HEIGHT
CHANNEL JUNCTION

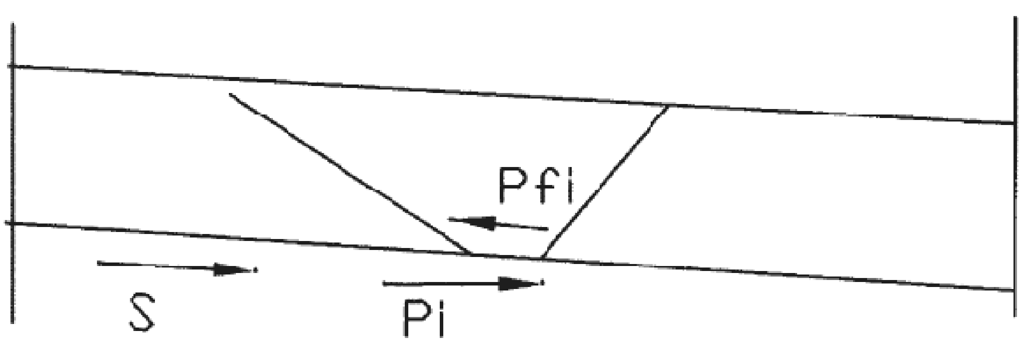
Plate B-54

*

Example Problems 1 - 3
Trapezoidal Junction
With Trapezoidal Side Inlet



Plan View



Profile View

Example Problem # 1

Junction Side flow Less than 10%

Supercritical Trapezoidal Channel with Trapezoidal Side Inlet

Stable Junction

Upsteam Channel Section 1			Downstream Channel Section 2			Side Channel Outlet Section 3		
Q1=	1,994.0	cfs	Q2=	2,144.0	cfs	Q3=	150	cfs
b1=	15.0	ft.	b2=	15.0	ft.	B3=	5	ft.
SS =	2.0	H:V	SS=	2.0	H:V	SS=	2	H:V
Dn=	3.5	ft.				Dn=	2	ft.
Dc=	6.2	ft.				Angle=	30	deg.
S1=	2.0%		S2=	2%		S inJunc.	2.0%	
D1=	3.5	ft.	D2 =	?	ft.	L=	50	ft.
Fr. 1	2.8					Area=	18	sq. ft.
						Fr.3	1.2	
						Chan S=	0.5%	

Eq. $P2 + M2 = P1 + M1 + M3\cos\Theta + Pw + Pi - Pf$

Rearrange and place P2 and M2 on right side to solve near zero difference.

Pi = Longitudinal Component of invert pressure, cubic feet.

Pw = Longitudinal Component of wall pressure, cubic feet.

Fr = Froude Number

Assume the slope through junction

F2 = M2 + P2

F1 = M1 + P1

Pi = Longitudinal Component of invert pressure, cubic feet.

Pw = Longitudinal Component of wall pressure, cubic feet.

Pf = loss due to friction, cubic feet.

Iterate for D2

Trial No.	D2 ft.	F1 cu. ft.	F2 cu. ft.	Pw cu. ft.	M3CosΘ cu. ft.	Pi cu. ft.	Fr. 2	Pf cu. ft.	Convergence Difference	
1	4.0	1,724.1	1714.4	28.1	33.6	56.3	2.4	84.5	43	
2	3.9	1,724.1	1759.1	27.4	33.6	55.5	2.5	83.0	-1	Convergence
3	3.8	1,724.1	1807.2	26.6	33.6	54.8	2.6	81.4	-49	
4	3.7	1,724.1	1858.9	25.9	33.6	54.0	2.7	79.9	-101	

Check Oblique Wave potential due to Side Junction Angle using EM 1110-2-1601, Plate B-54.

With Main Channel Flow & Side Flow Zero, oblique wave potential is 2.7*3.9 = 10.5'

For design, address wall height and freeboard or try different geometry to reduce wave ht.

With Main Flow zero & full side channel flow, minimal oblique wave potential due to low Froude Number

Example Problem # 2

Side flow Greater than 10% (approx. 20%)

Supercritical Trapezoidal Channel with Trap Side Inlet

Unstable, Junction creates hydraulic jump

Upsteam Channel Section 1			Downstream Channel Section 2			Side Channel Outlet Section 3		
Q1=	1,994.0	cfs	Q2=	2,394.0	cfs	Q3=	400	cfs
b1=	15.0	ft.	b2=	15.0	ft.	B3=	20	ft.
SS =	2.0	H:V	SS=	2.0	H:V	SS=	2	H:V
Dn=	5.1	ft.				Dn=	2.8	ft.
Dc=	6.2	ft.				Angle=	45	degrees
S1=	0.5%		S2=	0.5%		S inJunc.	0.5%	
D1=	5.1	ft.	D2 =	?	ft.	L=	50	ft.
Fr. 1	1.4					Area=	29.7	sq. ft.
						Fr.3	1.9	
						Chan S=	1.0%	

Eq. P2 + M2 = P1 + M1 + M3cosΘ + Pw + Pi - Pf

Rearrange and place P2 and M2 on right side to solve near zero difference.

Fr. = Froude Number

Assume the slope through junction

F2 = M2 + P2

F1 = M1 + P1

Pi = Longitudinal Component of invert pressure, cubic feet.

Pw = Longitudinal Component of wall pressure, cubic feet.

Pf = loss due to friction, cubic feet.

Iterate for D2

Trial No.	D2 ft.	F1 cu. ft.	F2 cu. ft.	Pw cu. ft.	M3CosΘ cu. ft.	Pi cu. ft.	Fr. 2	Pf cu. ft.	Convergence Difference	
1	5.5	1,244.3	1582.5	22.5	118.3	19.9	1.50	33.9	-211.5	
2	6.0	1,244.3	1512.7	55.4	118.3	20.8	1.28	36.3	-110.2	
3	6.5	1,244.3	1477.9	94.2	118.3	21.8	1.10	38.8	-38.2	
4	6.9	1,244.3	1471.8	129.6	118.3	22.5	0.98	40.9	2.0	Unstable

Check Oblique Wave potential due to Side Angle using EM 1110-2-1601

Convergence at 6.9' however Froude No. is 1 which is in unstable zone. No solution possible.

Example Problem # 3

Side flow Greater than 10% (approx. 30%)

Supercritical Trapezoidal Channel with Trap Side Inlet

Adjust Example 2 to avoid hydraulic jump

Upsteam Channel Section 1			Downstream Channel Section 2			Side Channel Outlet Section 3		
Q1=	1,994.0	cfs	Q2=	2,394.0	cfs	Q3=	400	cfs
b1=	15.0	ft.	b2=	20.0	ft.	B3=	20	ft.
SS =	2.0	H:V	SS=	2.0	H:V	SS=	2	H:V
Dn=	5.1	ft.	Dn=		ft.	Dn=	2.8	ft.
Dc=	6.2	ft.	Dc=		ft.	Angle=	12	deg.
S1=	0.5%		S2=	1%		S inJunc.	2.0%	
D1=	5.1	ft.	D2 =	?	ft.	L=	50	ft.
Fr. 1	1.4					Area=	29.7	sq. ft.
						Fr.3	1.9	
						Chan S=	1.0%	

Eq. P2 + M2 = P1 + M1 + M3cosΘ + Pw + Pi - Pf

Rearrange and place P2 and M2 on right side to solve near zero difference.

Fr. = Froude Number

Assume the slope through junction

F2 = M2 + P2

F1 = M1 + P1

Pi = Longitudinal Component of invert pressure, cubic feet.

Pw = Longitudinal Component of wall pressure, cubic feet.

Pf = loss due to friction, cubic feet.

Iterate for D2

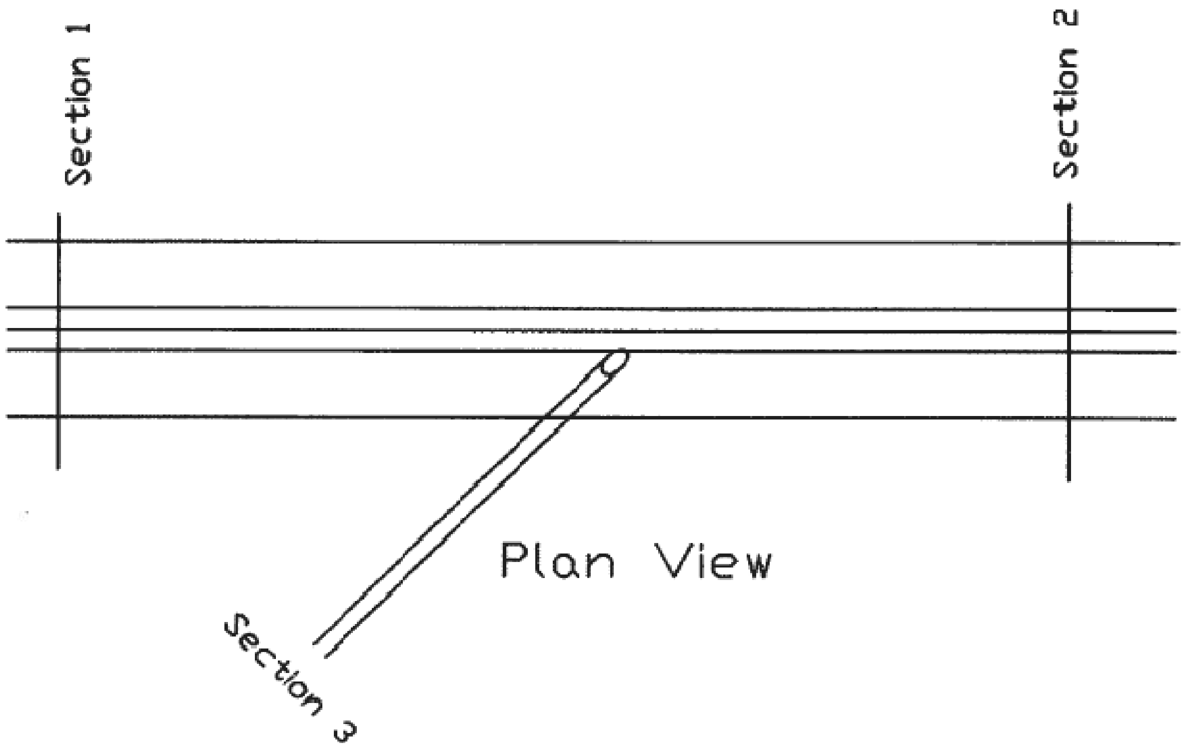
Trial No.	D2 ft.	F1 cu. ft.	F2 cu. ft.	Pw cu. ft.	M3CosΘ cu. ft.	Pi cu. ft.	Fr. 2	Pf cu. ft.	Convergence Difference	
1	5.1	1,244.3	1504.2	117.0	163.6	89.3	1.4	53.0	57.1	
2	4.9	1,244.3	1537.5	112.5	163.6	87.4	1.5	51.5	18.9	
3	4.8	1,244.3	1556.9	110.3	163.6	86.5	1.6	50.7	-2.9	Converges

Check Oblique Wave potential due to Side Angle using EM 1110-2-1601

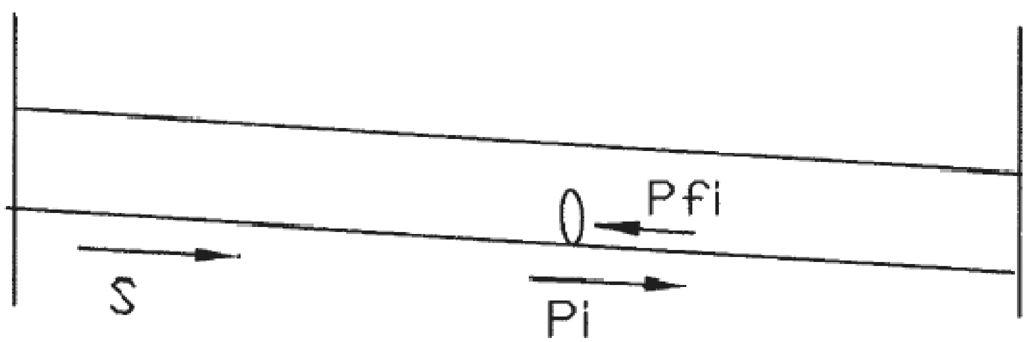
With Side Flow zero oblique wave potential is 1.2*4.8 = 6.1'

With Main Flow zero oblique wave potential is 1.8*2.8 = 5.0'

Example Problems 4 - 6
Trapezoidal Junction
With Pipe Side Inlet



Plan View



Profile View

Example Problem # 4

Side flow Greater less than 10%

Supercritical Trapezoidal Channel with Pipe Inlet

Stable Junction

Upsteam Channel Section 1			**Downstream Channel Section 2**			**Pipe Outlet Section 3**		
Q1=	1,340.0	cfs	Q2=	1,470.0	cfs	Q3=	130	cfs
b1=	10.0	ft.	b2=	10.0	ft.	Dia.=	48	in.
SS =	2.0	H:V	SS=	2.0	H:V			
Dn=	3.6	ft.						
Dc=	5.6	ft.				Angle=	45	deg.
S1=	1.5%		S2=	2%		S inJunc.	0.5%	
D1=	3.6	ft.	D2 =	?	ft.	L=	12	ft.
Fr. 1	2.4					Area=	12.57	sq. ft.
						Pipe S=	0.8%	

Eq. P2 + M2 = P1 + M1 + M3cosΘ + Pw + Pi - Pf

Rearrange and place P2 and M2 on right side to solve near zero difference.

Fr. = Froude Number

Pipe flowing full

F2 = M2 + P2

F1 = M1 + P1

Pi = Longitudinal Component of invert pressure, cubic feet.

Pw = Longitudinal Component of wall pressure, cubic feet.

Pf = loss due to friction, cubic feet.

Iterate for D2

Trial No.	D2 ft.	F1 cu. ft.	F2 cu. ft.	Pw cu. ft.	M3CosΘ cu. ft.	Pi cu. ft.	Fr. 2	Pf cu. ft.	Convergence Difference	
1	3.5	996.5	1217.7	1.5	29.5	3.2	2.7	16.5	-203	
2	4.0	996.5	1054.7	11.6	29.5	3.4	2.1	18.1	-32	
3	4.1	996.5	1029.3	14.8	29.5	3.5	2.0	18.4	-3	Convergence

Check Oblique Wave potential due to Side Angle using EM 1110-2-1601 - Plate B-54

With Side Flow = 0, oblique wave ht. potential is 4.1*1.7 = 7' (conservative as Plate does not exactly apply).

Example Problem # 5

Side flow Greater than 10% (approx. 30%)

Supercritical Trapezoidal Channel with Pipe Inlet

Unstable Junction

Upsteam Channel			Downstream Channel			Pipe Outlet		
Section 1			Section 2			Section 3		
Q1=	1,340.0	cfs	Q2=	1,589.0	cfs	Q3=	249	cfs
b1=	10.0	ft.	b2=	10.0	ft.	Dia.=	48	in.
SS =	2.0	H:V	SS=	2.0	H:V			
Dn=	4.8	ft.	Dn=		ft.			
Dc=	5.7	ft.	Dc=		ft.	Angle=	45	deg.
S1=	0.5%		S2=	0.5%		S inJunc.	0.5%	
D1=	4.8	ft.	D2 =	?	ft.	L=	12	ft.
Fr. 1	1.4					Area=	12.57	sq. ft.
						Chan S=	3.0%	

Eq. P2 + M2 = P1 + M1 + M3cosΘ + Pw + Pi - Pf

Rearrance and place P2 and M2 on right side to solve near zero difference.

Fr. = Froude Number

Pipe flowing full.

F2 = M2 + P2

F1 = M1 + P1

Pi = Longitudinal Component of invert pressure, cubic feet.

Pw = Longitudinal Component of wall pressure, cubic feet.

Pf = loss due to friction, cubic feet.

Iterate for D2

Trial No.	D2 ft.	F1 cu. ft.	F2 cu. ft.	Pw cu. ft.	M3CosΘ cu. ft.	Pi cu. ft.	Fr. 2	Pf cu. ft.	Convergence Difference
1	5.0	781.7	992.5	0.0	108.3	4.4	1.5	7.3	-105
2	5.5	781.7	941.1	0.0	108.3	4.6	1.2	7.8	-54
3	6.0	781.7	918.0	0.0	108.3	4.9	1.0	8.4	-32
4	6.5	781.7	918.8	0.0	108.3	5.1	0.9	9.0	-33

No convergence - hydraulic jump will occur - junction will not remian supercritical

Example Problem # 6

Side flow Greater than 10% (approx. 20%)

Supercritical Trapezoidal Channel with Pipe Side Inlet

Example Problem #5 Revised for Stability

Upsteam Channel			**Downstream Channel**			**Pipe Outlet**		
Section 1			**Section 2**			**Section 3**		
Q1=	1,340.0	cfs	Q2=	1,589.0	cfs	Q3=	249	cfs
b1=	10.0	ft.	b2=	10.0	ft.	Dia.=	48	in.
SS =	2.0	H:V	SS=	2.0	H:V			
Dn=	4.8	ft.						
Dc=	6.0	ft.				Angle=	30	deg.
S1=	0.5%		S2=	0.5%		S inJunc.	5.0%	
D1=	5.1	ft.	D2 =	?	ft.	L=	20	ft.
Fr. 1	1.4					Area=	12.57	sq. ft.
						Pipe S=	3.0%	

Eq. P2 + M2 = P1 + M1 + M3cosΘ + Pw + Pi - Pf

Rearrange and place P2 and M2 on right side to solve near zero difference.

Fr. = Froude Number

Pipe flowing full

F2 = M2 + P2

F1 = M1 + P1

Pi = Longitudinal Component of invert pressure, cubic feet.

Pw = Longitudinal Component of wall pressure, cubic feet.

Pf = loss due to friction, cubic feet.

Iterate for D2

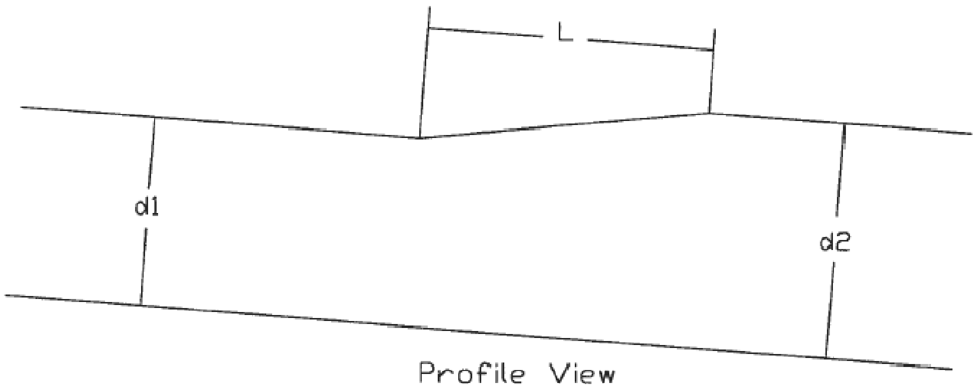
Trial No.	D2 ft.	F1 cu. ft.	F2 cu. ft.	Pw cu. ft.	M3CosΘ cu. ft.	Pi cu. ft.	Fr. 2	Pf cu. ft.	Convergence Difference	
1	4.8	759.8	1022.4	49.0	132.7	74.3	1.6	12.3	-19	
2	4.9	759.8	1006.7	19.4	132.7	75.0	1.5	10.1	-30	
3	5.0	759.8	992.5	22.7	132.7	75.8	1.5	10.2	-12	
4	5.1	759.8	979.6	26.0	132.7	76.5	1.4	10.3	5	Convergence

Check Oblique Wave potential due to Side Angle using EM 1110-2-1601, Plate B-54

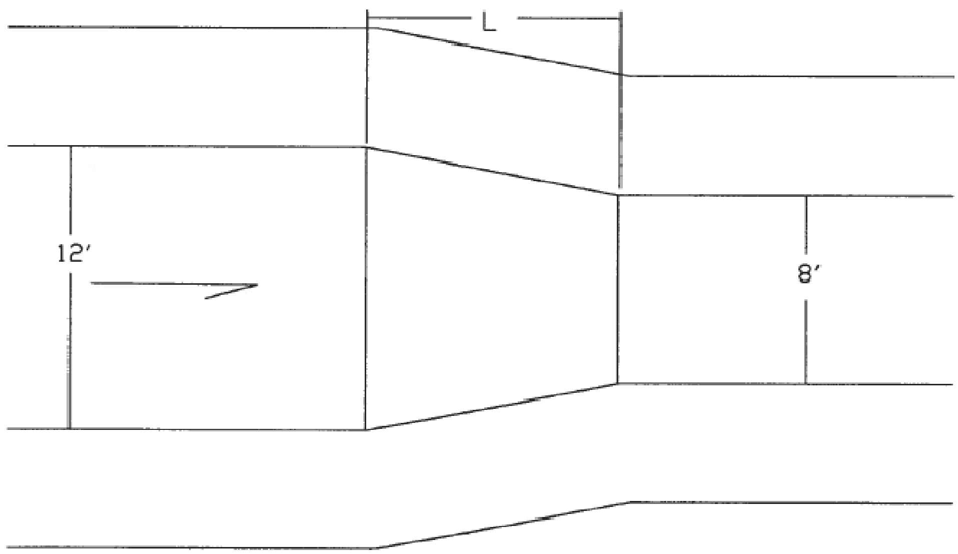
With Side Flow = 0, oblique wave ht. potential is 4.1*1.7 = 7' (conservative as Plate does not exactly apply).

With Main Flow zero, oblique wave potential 3.1*2.3 = 7.1' (estimate based on equivalent channel)

Example Problem 7
Supercritical Flow Transition



Profile View



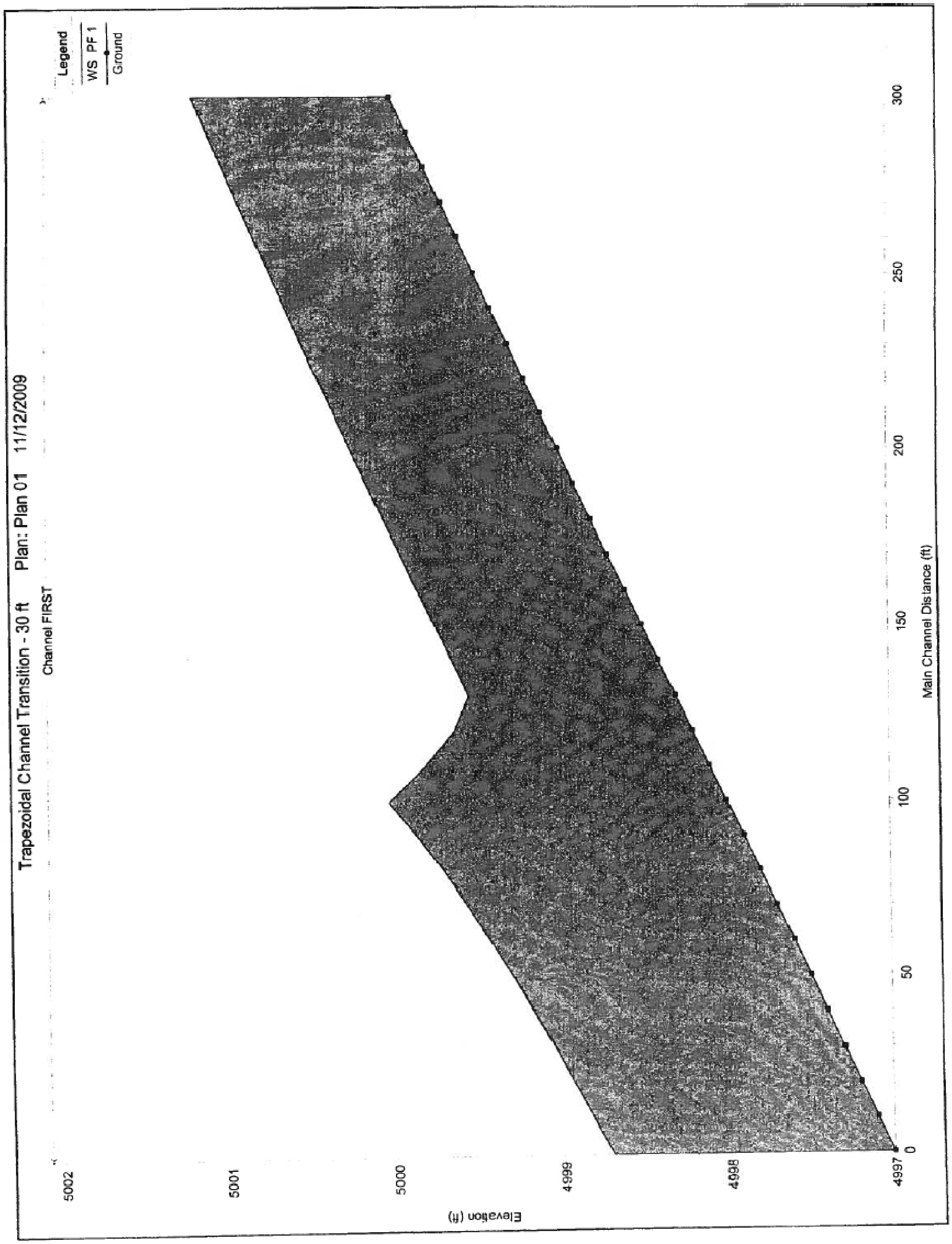
Plan View

Q = 200 cfs
Slope = 1 %

2:1 side slopes
n= 0.015

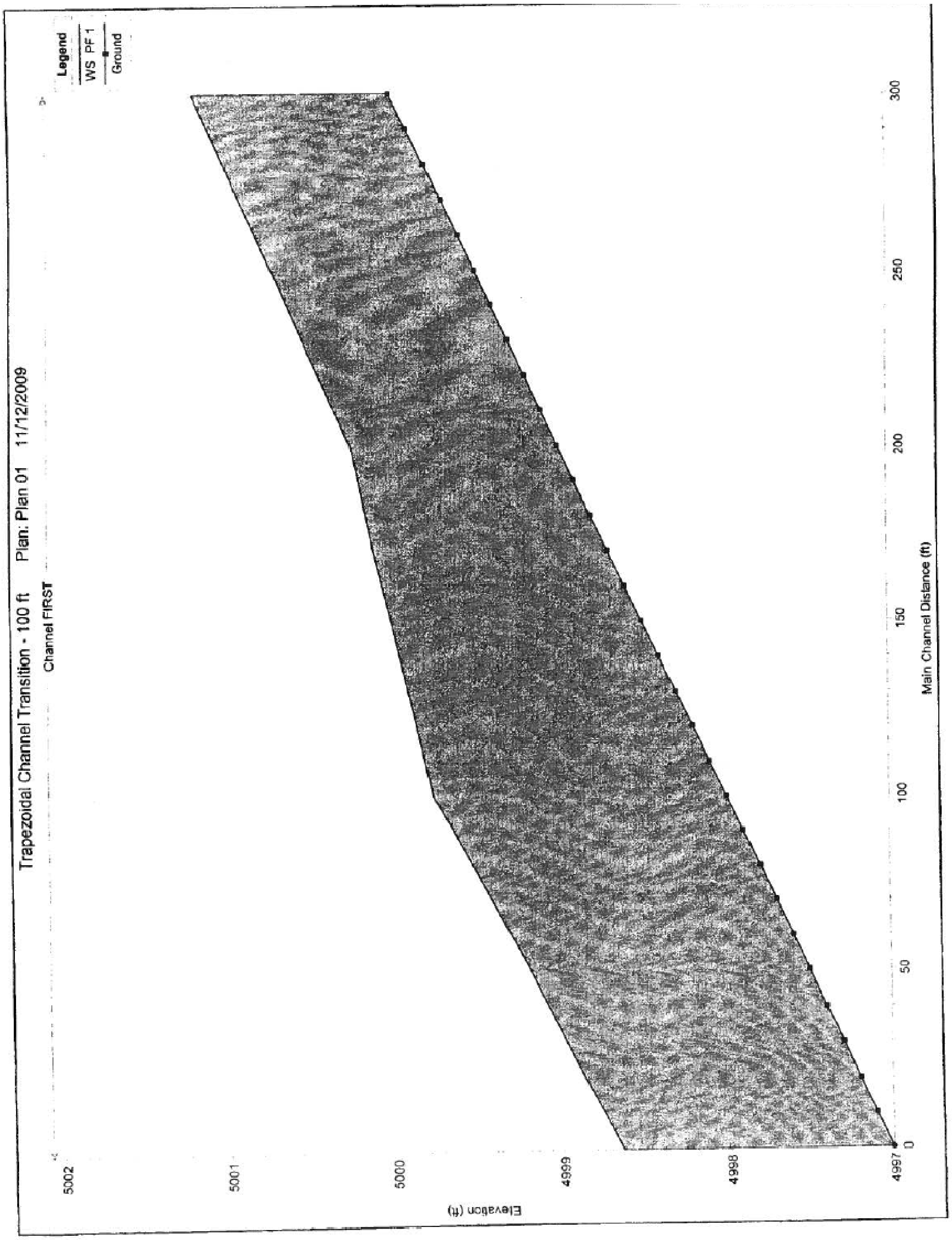
Determine the minimum length L to ensure that a hydraulic jump does not occur through the transition and that the transition angle is small enough so that oblique waves are negligible.

Example 7										
HEC-RAS Run										
Bottom Width Transition 12' to 8'										
30' Transition from Sta. 200 to Sta. 230										
River Sta	**Q Total**	**Min Ch El**	**W.S. Elev**	**Crit W.S.**	**E.G. Elev**	**E.G. Slope**	**Vel Chnl**	**Flow Area**	**Top Width**	**Froude**
	(cfs)	**(ft)**	**(ft)**	**(ft)**	**(ft)**	**(ft/ft)**	**(ft/s)**	**(sq ft)**	**(ft)**	**No.**
400	200	5000.00	5001.18	5001.88	5003.49	0.01	12.64	16.99	16.73	2.05
390	200	4999.90	5001.10	5001.78	5003.35	0.01	12.49	17.19	16.78	2.01
380	200	4999.80	5001.00	5001.68	5003.22	0.01	12.38	17.36	16.82	1.99
370	200	4999.70	5000.91	5001.58	5003.09	0.01	12.28	17.51	16.85	1.97
360	200	4999.60	5000.82	5001.48	5002.98	0.01	12.23	17.59	16.87	1.95
350	200	4999.50	5000.72	5001.38	5002.86	0.01	12.17	17.67	16.89	1.94
340	200	4999.40	5000.63	5001.28	5002.75	0.01	12.13	17.74	16.91	1.93
330	200	4999.30	5000.53	5001.18	5002.64	0.01	12.08	17.81	16.93	1.92
320	200	4999.20	5000.44	5001.08	5002.53	0.01	12.04	17.88	16.94	1.91
310	200	4999.10	5000.34	5000.98	5002.42	0.01	12.00	17.93	16.96	1.9
300	200	4999.00	5000.24	5000.88	5002.31	0.01	11.97	17.98	16.97	1.89
290	200	4998.90	5000.14	5000.78	5002.21	0.01	11.97	17.98	16.97	1.89
280	200	4998.80	5000.04	5000.68	5002.11	0.01	11.97	17.98	16.97	1.89
270	200	4998.70	4999.94	5000.58	5002.01	0.01	11.97	17.98	16.97	1.89
260	200	4998.60	4999.84	5000.48	5001.91	0.01	11.97	17.98	16.97	1.89
250	200	4998.50	4999.74	5000.38	5001.81	0.01	11.97	17.98	16.97	1.89
240	200	4998.40	4999.64	5000.28	5001.71	0.01	11.97	17.98	16.97	1.89
230	200	4998.30	4999.54	5000.18	5001.61	0.01	11.97	17.98	16.97	1.89
220	200	4998.20	4999.63	5000.19	5001.44	0.01	11.30	19.40	16.40	1.66
210	200	4998.10	4999.81	5000.23	5001.28	0.01	10.35	21.73	16.16	1.4
200	200	4998.00	5000.03	5000.28	5001.22	0.00	9.51	24.45	16.11	1.18
190	200	4997.90	4999.85	5000.18	5001.17	0.00	9.97	23.22	15.80	1.26
180	200	4997.80	4999.70	5000.08	5001.11	0.00	10.32	22.37	15.59	1.32
170	200	4997.70	4999.56	4999.98	5001.06	0.01	10.60	21.73	15.42	1.37
160	200	4997.60	4999.42	4999.88	5000.99	0.01	10.84	21.20	15.29	1.42
150	200	4997.50	4999.29	4999.78	5000.93	0.01	11.06	20.75	15.17	1.46
140	200	4997.40	4999.17	4999.68	5000.87	0.01	11.25	20.37	15.06	1.49
130	200	4997.30	4999.04	4999.58	5000.80	0.01	11.44	19.99	14.96	1.53
120	200	4997.20	4998.93	4999.48	5000.73	0.01	11.55	19.79	14.91	1.55
110	200	4997.10	4998.81	4999.38	5000.65	0.01	11.66	19.59	14.86	1.57
100	200	4997.00	4998.70	4999.28	5000.57	0.01	11.77	19.40	14.80	1.59



Trapezoidal Channel Transition - 30 ft Plan: Plan 01 11/12/2009
Channel FIRST
Legend
WS PF 1
Ground
Elevation (ft)
5002
5001
5000
4999
4998
4997
Main Channel Distance (ft)
0
50
100
150
200
250
300

Example 7										
HEC-RAS Run										
Trapazoid, 2:1 SS, Bottom Width Transition 12' to 8'										
100' Transition from Sta. 200 to Sta. 300										
River Sta	**Q Total**	**Min Ch El**	**W.S. Elev**	**Crit W.S.**	**E.G. Elev**	**E.G. Slope**	**Vel Chnl**	**Flow Area**	**Top Width**	**Froude**
	(cfs)	**(ft)**	**(ft)**	**(ft)**	**(ft)**	**(ft/ft)**	**(ft/s)**	**(sq ft)**	**(ft)**	**No.**
400	200	5000.00	5001.18	5001.88	5003.49	0.01	12.64	16.99	16.73	2.05
390	200	4999.90	5001.10	5001.78	5003.35	0.01	12.49	17.19	16.78	2.01
380	200	4999.80	5001.00	5001.68	5003.22	0.01	12.38	17.36	16.82	1.99
370	200	4999.70	5000.91	5001.58	5003.09	0.01	12.28	17.51	16.85	1.97
360	200	4999.60	5000.82	5001.48	5002.98	0.01	12.23	17.59	16.87	1.95
350	200	4999.50	5000.72	5001.38	5002.86	0.01	12.17	17.67	16.89	1.94
340	200	4999.40	5000.63	5001.28	5002.75	0.01	12.13	17.74	16.91	1.93
330	200	4999.30	5000.53	5001.18	5002.64	0.01	12.08	17.81	16.93	1.92
320	200	4999.20	5000.44	5001.08	5002.53	0.01	12.04	17.88	16.94	1.91
310	200	4999.10	5000.34	5000.98	5002.42	0.01	12.00	17.93	16.96	1.90
300	200	4999.00	5000.24	5000.88	5002.31	0.01	11.97	17.98	16.97	1.89
290	200	4998.90	5000.20	5000.81	5002.18	0.01	11.76	18.40	16.79	1.82
280	200	4998.80	5000.15	5000.75	5002.07	0.01	11.59	18.77	16.60	1.76
270	200	4998.70	5000.11	5000.68	5001.96	0.01	11.44	19.13	16.42	1.70
260	200	4998.60	5000.05	5000.62	5001.88	0.01	11.38	19.33	16.21	1.66
250	200	4998.50	5000.00	5000.56	5001.80	0.01	11.33	19.53	16.01	1.63
240	200	4998.40	4999.95	5000.50	5001.72	0.01	11.28	19.74	15.81	1.59
230	200	4998.30	4999.90	5000.44	5001.66	0.01	11.28	19.86	15.61	1.57
220	200	4998.20	4999.85	5000.38	5001.59	0.01	11.28	20.00	15.41	1.55
210	200	4998.10	4999.81	5000.33	5001.53	0.01	11.28	20.14	15.22	1.52
200	200	4998.00	4999.76	5000.28	5001.48	0.01	11.30	20.28	15.04	1.50
190	200	4997.90	4999.64	5000.18	5001.41	0.01	11.48	19.93	14.95	1.53
180	200	4997.80	4999.52	5000.08	5001.33	0.01	11.58	19.73	14.89	1.56
170	200	4997.70	4999.41	4999.98	5001.25	0.01	11.69	19.53	14.84	1.58
160	200	4997.60	4999.30	4999.88	5001.18	0.01	11.79	19.35	14.79	1.59
150	200	4997.50	4999.19	4999.78	5001.10	0.01	11.89	19.18	14.75	1.61
140	200	4997.40	4999.08	4999.68	5001.02	0.01	11.98	19.02	14.70	1.63
130	200	4997.30	4998.97	4999.58	5000.94	0.01	12.07	18.87	14.66	1.65
120	200	4997.20	4998.86	4999.48	5000.86	0.01	12.15	18.74	14.63	1.66
110	200	4997.10	4998.75	4999.38	5000.77	0.01	12.23	18.61	14.59	1.68
100	200	4997.00	4998.64	4999.28	5000.69	0.01	12.30	18.49	14.56	1.69
Continue iteration to determine the minimum transition length in accordance with Section 3.C.1										



Trapezoidal Channel Transition - 100 ft Plan: Plan 01 11/12/2009
Channel FIRST
Legend
WS PF 1
Ground
Elevation (ft)
5002
5001
5000
4999
4998
4997
Main Channel Distance (ft)
0
50
100
150
200
250
300

H. Sediment Transport and Channel Stability

Moving water has the ability to transport sediment. The amount of sediment per unit of water that can be transported is related to flow depth, velocity, temperature, vertical and horizontal channel alignment, the amount of sediment available, the size and density of the sediment available and many other minor but sometimes important parameters. A channel's stability can be defined in terms of its ability to function properly during flood event without serious aggradation and/or degradation and that its continued operation can be relied upon without extraordinary maintenance and repairs. While channel stability problems are largely associated with earth and flexibly lined channels, concrete lined, supercritical channels are not immune. Any time a downstream channel reach has a lower sediment capacity than some upstream reach, there is a potential for sediment accumulation. The following worksheets can be used to make qualitative determinations with regard to channel stability.

Detailed qualitative analyses must be performed for any design requiring construction in a major arroyo. Methods found in items C.7 and C.8 in the Bibliography at the end of Section 22.3 shall be used in sediment transport analyses.

CHANNEL STABILITY WORK SHEET INSTRUCTIONS

A stable earth-lined channel is defined for the purposes of design as one in which neither degradation or aggradation is occurring at such a rate that it causes a continuous and serious maintenance problem. Channel degradation can cause extensive damage to bridges and other crossing structures due to the undermining of their foundations. Channel aggradation on the other hand results in reduced channel and crossing structure capacities and, therefore, in increased frequency of flooding.

CHANNEL STABILITY WORKSHEET - A

The Proposed Development or Land Use Change Will Affect:	In the Following Way:		
	No Change	Increase	Decrease
Flow Rates..........................	____________	________	________
Flow Velocities	____________	________	________
Flow Frequencies	____________	________	________
Flow Duration	____________	________	________
Flow Depth.........................	____________	________	________
Sediment Reaching the Channel.........................	____________	________	________
Sediment Particle Size........	____________	________	________
Streambed Material Size	____________	________	________
Channel Vegetation			

CHANNEL STABILITY WORKSHEET - B

An Increase or Decrease in:	Will Have the Following Effect in the Channel	
	Increase	Decrease
Flow Rate	Degradation	Aggradation
Flow	Degradation	Aggradation
Flow Frequency.	Degradation	Aggradation
Flow Duration	Degradation	Aggradation
Flow Depth	Degradation	Aggradation
Sediment Reaching the Channel	Aggradation	Degradation
Sediment Particle Size	Aggradation	Degradation
Streambed Material Size	Aggradation	Degradation
Channel Vegetation	Aggradation	Degradation

1. Channels

a. Earthwork

The following shall be compacted to at least 90% of maximum density as determined by ASTM D-1557 (modified Proctor):

(1) The 12 inches of subgrade immediately beneath concrete lining (both channel bottom and side slopes).

(2) Top 12 inches of maintenance road (either as subgrade or finished roadway if unsurfaced).

(3) Top 12 inches of earth surface within 10 feet of concrete channel lip. It is particularly important to compact earth immediately adjacent to concrete lip. This area is sometimes overlooked when forms are removed.

(4) All fill material.

b. Concrete

(1) All concrete channels shall be continuously reinforced.

(2) All exposed concrete drainage structures shall be tinted with San Diego Buff or a color approved by the City Engineer/SSCAFCA.

(3) Materials

(a) Cement type: ILA or I-IILA

(b) Minimum cement content: 5.5 sacks/c.y.

(c) Maximum water-cement ratio: 0.53 (6 gals. per sack)

(d) Maximum aggregate size: 1 ½ inches

(e) Air content range: 4-7%

(f) Maximum slump: 3 inches

(g) Minimum compressive strength (f_c): 3500 psi @ 28 days

(h) Class F Fly ash meeting the requirements of ASTM C618 shall be proportioned in the mix at a 1:4 ratio of fly ash to cement weight.

(i) Steel reinforcement shall be a minimum of grade 60 deformed bars. Wire mesh shall not be used, however welded wire mats are allowed.

(4) Lining Section

(a) Bottom width - 10 feet minimum

(b) Side Slopes - 1 vertical to 2 horizontal slope, or flatter

(c) Concrete lining thickness

All concrete lining shall have a minimum thickness of 8 inches.

The lining shall be thickened to 10 inches on the channel bottom and lower 18 inches of the side slope when design velocity exceeds 25 feet per second. This will provide an additional top two inches of sacrificial concrete. Steel placement shall be based upon the standard 8” thickness as measured from the bottom of the concrete lining.

(d) Concrete Finish

The surface of the concrete lining shall be provided with a tined finish. Pneumatically applied “shotcrete” is an acceptable concrete lining alternative and does not require a timed finish, but it must be preapproved by SSCAFCA. Precautions shall be taken to guard against excessive working or wetting of finish.

(e) Concrete Curing

All concrete shall be cured by the application of liquid membrane-forming curing compound (white pigmented) immediately upon completion of the concrete finish.

(f) Steps

Ladder-type steps shall be installed at locations suitable for rescue operations along the channel but not farther than 700 ft. apart on both sides of the channel. Bottom rung shall be placed approximately 12 inches vertically above channel invert.

(5) Joints

(a) Insofar as feasible, channels shall be continuously reinforced without transverse joints. However, expansion joints may be installed where new concrete lining is connected to a rigid structure or to existing concrete lining which is not continuously reinforced.

(b) The preferred design avoids longitudinal joints. However, if included, longitudinal joints should be on side slope at least one foot vertically above channel invert.

(c) All joints shall be designed to prevent differential displacement and shall be watertight.

(d) Construction joints are required at the end of a day's run, where lining thickness changes.

(6) Reinforcing Steel for Continuously Reinforced Channels

(a) Ratio of longitudinal steel area to concrete area not including additional thickness of sacrificial concrete

$$A_{s_{\text{long}}} \Big/ A_{c_{\text{long.}}} \geq .005$$

(b) Ratio of transverse steel area to concrete area not including additional thickness of sacrificial concrete

$$A_{s\,\text{transv.}} \,/\, A_{c\,\text{transv.}} \geq .0025$$

Note: *In (a) and (b) above A_s = crossectional area of steel in the direction indicated; Ac = crossectional area of concrete in the direction indicated. Longitudinal = long.; transverse = transv.*

(c) Steel Placement: Temperature and shrinkage steel shall be placed so as to be in the top of the middle third of the slab, but at least 3" from the bottom of the slab. Longitudinal steel shall be on tip of the transverse steel. (NOTE: Inspectors must insure this requirement is not violated by contractors during pouring operations.)

2. Earthwork for Levees and Berms

All earthfill berms and levees shall be constructed of high quality fill material free of debris, organic matter, frozen matter and stone larger than 6 inches in any dimension. The key trench shall be scarified to a depth of 6 inches to ensure bonding with the fill material. Lifts shall not exceed 12 inches of loose material before compaction. The material in each lift shall contain optimum moisture content (-1% to +3%) and shall be compacted to at least 90% and not more than 95% of maximum density as determined by ASTM D 1557 or as recommended by a geotechnical engineer and accepted by the City Engineer/SSCAFCA. Proper bonding between lifts shall be guaranteed by scarifying each lift after compaction to a depth of at least 3 inches.

Levees and berms intended to provide flood protection for properties and structures shall comply with all FEMA requirements for removal from a 100 year floodplain. A minimum 3' freeboard above the high water elevation is required on all levees and berms.

BIBLIOGRAPHY

22.3 Hydraulics

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B. Closed Conduits

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C. Channels

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2. U.S. Army Corps of Engineers:- Hydraulic Design of Flood Control Channels EM 1110-2-1601,. Office of the Chief of Engineers, Washington, D.C. 20314, 1970

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6. U.S.D.A Soil Conservation Service: Planning and Design of Open Channels, Technical Release No. 25, Washington, D.C., October, 1971

7. U.S.D.A Soil Conservation Service:. Sedimentation, National Engineering Handbook, Section 3, Chapter 4, Washington, D.C., 1971

8. Simons, Li and Associates: Design Guidelines and Criteria - Channels and Hydraulic Structures on Sandy Soil, P.O. Box 1816 Ft. Collins, Colorado, 80522, 1981

9. Los Angeles County Flood Control Authority, Design Manual Hydraulic P.O. Box 2418 Los Angeles, California 90054 Rev. 1973.

10. Albuquerque Metropolitan Arroyo Flood Control Authority Draft Design guide for Trapezoidal Concrete Flood Control Channels, Rev. April, 1982.

D. Storm Inlets

1. Los Angeles County Flood Control Authority, Design Manual - Hydraulic P.O. Box 2418 Los Angeles, California 90054 Rev. 1972.

E. Street Hydraulics

1. See Reference C-1

2. See Reference C-4

F. Berms and Levees

1. See Reference C-6

2. See Reference C-7

3. See Reference C-8

G. Sediment & Erosion Control

1. SSCAFCA's Sediment and Erosion Design Guide prepared by Mussetter Engineering, Inc. dated November 2008. Available at www.sscafca.com.