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SUSITNA HYDROELECTRIC PROJECT

DRAINAGE STRUCTURE AND WATERWAY DESIGN GUIDELINES

Report by
Harza-Ebasco Susitna Joint Venture

Prepared for
Alaska Power Authority

Draft Report
September 1984

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DRAINAGE STRUCTURE AND WATERWAY DESIGN GUIDELINES

1.0 INTRODUCTION

1.1 SETTING

This manual is intended to be used by design engineers during the preparation of contract plans and specifications for Alaska Power Authority projects. The guidelines in the manual incorporate State-of-the-Art Engineering Practices and procedures, and also incorporate Alaska Department of Fish and Game 1981 proposed habitat regulations where appropriate. Although the manual is organized in such a way that it is applicable to any Alaska Power Authority project in general, nevertheless it contains much detailed information which can be used directly in the preparation of contract documents for any specific project.

1.2 SCOPE

The purpose of these best practices guidelines is to establish the proper procedures for drainage structures and waterways required for implementation of a major construction project using the Susitna Project as an example.

Drainage structures considered in these guidelines will consist of culverts, waterways, and the waterways beneath bridges which are required to implement temporary or permanent access to project features. The drainage structures and pertinent waterway work will be classified by the type of fish utilization that occurs in the watercourse where the project feature is proposed. These types are:

Type A: Watercourse that is used by anadromous fish during any period of the year.

Type B: Watercourse that is utilized by resident fish during any period of the year.

Type C: Watercourse that has no history of being used by anadromous or resident fish.

Waterway work for Type A and B watercourses will be limited to the necessary adjustments in the watercourse at the inlet and outlet of the drainage structure to assure efficient hydraulic conditions, fish movement, and to preclude deleterious sediment transport or deposition in or around the drainage structure. Waterway work for Type C watercourses can in some cases be more extensive in that a collector system may be required to channel surface runoff to the watercourse in question. A typical example of this is an interceptor ditch along a roadway or waterway work associated with diverting the watercourse during the construction of the drainage structure.

The watercourse work will be divided into two distinct stages. Stage I will be the field investigations and design. During this stage, site specific investigation results and design criteria will be presented to secure the necessary permits from the Alaska Department of Fish and Game and any other permits ie., CIRI, State of Federal. Stage II would be the construction of the drainage structure of waterway.

1.3 PROJECT DESCRIPTION (Example)

1.3.1 General

The Susitna Hydroelectric Project, located on the Susitna River approximately 60 miles northeast of Anchorage, will consist of two dams and reservoirs and ancillary features necessary for field investigation, design, site access, construction, operation and transmission of energy to the centers of Anchorage and Fairbanks. The Federal Energy Regulatory Commission License Application Project No. 7114-000 as accepted July 29, 1983 included both Watana and Devil Canyon. Watana is located at river mile 184 on the Susitna River and is to be initiated first. Devil Canyon, located at river mile 152 is scheduled to begin seven years after commencement of Watana Construction.

1.3.2 Watana Dam and Reservoir

1.3.2.1 Dam Site The Watana Dam, Figure A-1 Appendix A, will create a reservoir approximately 48 miles long, with a surface area of 38,000 acres, and a gross storage capacity of 9,500 acre-feet at Elevation 2185, the normal maximum operating level.

The dam will contain a rolled embankment, earth and rock fill structure, with a central impervious core. The design crest elevation of the dam will be 2205, which results in a maximum height of 885 feet above the foundation and a crest length of 4,100 feet. During construction, the river will be diverted through two concrete-lined diversion tunnels, each 36 feet in diameter and 4,100 feet long, on the north bank of the river.

The power intake will also be located on the north bank with a bi-level approach channel excavated in rock, serving it and the spillway. Turbine discharge will flow through six draft tube tunnels to surge chambers downstream from the powerhouse. The surge chambers will discharge to the river through two concrete-lined tailrace tunnels. The spillway located to the north of the power intake will consist of an upstream ogee control structure with three radial gates and an inclined concrete chute and flip bucket.

Emergency release facilities will be located in one of the diversion tunnels to allow lowering of the reservoir over a period of time to permit emergency inspection or repair of impoundment structures.

1.3.2.2 Transmission Facilities. The project transmission facilities will deliver power from the Susitna River basin generating plants to the major load centers at Anchorage and Fairbnks in an economical and reliable manner. The facilities will consist of overhead transmission lines, underwater cables, switchyards, substations, a load dispatch center, and a communications system.

1.3.2.3 Access Plan. Access to the project site (See Figure A-3, Appendix A) will connect with the existing Alaska Railroad at Cantwell where a railhead and storage facility occupying 40 acres will be constructed.

1.3.2.4 Site Facilities. Included among the site facilities will be a combination camp and village that will be constructed and maintained at the project site. The camp/village will be a largely self-sufficient community housing 3300 people during construction of the project.

Permanent facilities required will include a permanent town or small community for approximately 130 staff members and their families. Other permanent facilities will include a maintenance building for use during subsequent operation of the power plant.

A plan showing the location of the camp/village and the permanent town is shown on Figure A-1, Appendix A.

1.3.3 Devil Canyon Dam and Reservoir

1.3.3.1 Dam Site. The Devil Canyon dam and surrounding area in relation to main access facilities and camp facilities are shown on Figure A-4, Appendix A.

The Devil Canyon Dam will form a reservoir approximately 26 miles long with a surface area of 7,800 acres and a gross storage capacity of 1,000,000 acre-feet at Elevation 1455, the normal maximum operating level.

The dam will be a thin-arch concrete structure with a crest elevation of 1463 (not including a three-foot parapet) which results in a maximum height of 646 feet. An earth-and rockfill saddle dam will provide closure to the south bank. The saddle dam will be a central core type earth and over fill similar in cross section to the Watana Dam. The dam will have a nominal crest elevation of 1469. The maximum height above foundation level of the dam is approximately 245 feet.

During construction, the river will be diverted by means of a single 30-foot diameter concrete-lined diversion tunnel on the south bank of the river.

A power intake on the north bank will consist of an approach channel excavated in rock leading to a reinforced concrete gate structure. The turbines will discharge to the river by means of a single 38-foot diameter trailrace tunnel leading from a surge chamber downstream from the powerhouse cavern.

Outlet facilities consisting of seven individual outlet conduits will be located in the lower part of the main dam. The spillway will also be located on the north bank. As at Watana, this spillway will consist of an upstream ogee control structure with three radial gates and an inclined concrete chute and flip bucket.

1.3.3.2 Transmission Facilities.

Transmission lines will be built between the Devil Canyon switchyard and the Gold Creek switching station.

1.3.3.3 Access Plan. Access to the Devil Canyon site (See Figure A-3, Appendix) will consist primarily of a railroad extension from the existing Alaska Railroad at Gold Creek to a railhead and storage facility adjacent to the Devil Canyon camp area.

1.3.3.4 Site Facilities. The construction of the Devil Canyon Dam will require various facilities to support the construction activities throughout the entire construction period.

As described for Watana, a construction camp will be constructed and maintained at the project site. The camp/village will provide housing and living facilities for 1,800 people during construction. Electric power will be provided from Watana. A plan showing the location of the camp/village is shown on Figure A-4, Appendix A.

1.4 STAGE I PRE-PROJECT FIELD INVESTIGATIONS

During the feasibility and licensing phase of the Susitna Project investigations in the areas described in section 1.3 "Project Description", will be necessary to design and construct the Project. This time frame is defined herein as Stage I and involves activities in the engineering and environmental sciences. All such activities would be conducted under the applicable technical and regulatory permits required by Federal, State, and/or local authorities.

1.4.1 Engineering Activities

The principal activity during this stage would be subsurface exploration for the major project features at Watana including improvements to the present camp site which could also include the development of a temporary airstrip.

The geotechnical explorations will include damsite subsurface drilling, monitoring, dozer or backhoe excavation of inspection trenches, geophysical surveys and investigation of quarry materials and borrow materials. The above activities (except for quarry and borrow area development) can be accomplished with light equipment, helicopter transport or with special ground transport equipment. Whereas quarry and borrow area development may require heavier equipment and access crossing natural waterways or earthwork which could impede drainage courses. In these instances drainage structures will be designed and constructed using the criteria established in Section 2.0 and 3.0 of these guidelines.

The temporary air field will require the design and construction of a drainage collector system and perhaps drainage structures to insure that surface runoff will reach the existing drainage features. Criteria for this design is presented in sections 2.0 and 3.0.

The preceeding is an estimate of what will be the Stage I engineering work requiring drainage and waterway design.

1.4.2 Environmental Study Activities

Environmental science activities will consist of aquatic, terrestrial, and cultural resource field investigations.

Environmental science activities involve most areas of the Susitna Project. The biological studies encompass both aquatic and terrestrial programs. The aquatic studies are concentrated on the mainstem Susitna River from the Oshetna River (the upstream boundary of the reservoir impoundment) to Cook Inlet. In addition, tributaries within this reach, lakes within the proposed impoundment areas and streams along the proposed access road are being studied. The primary terrestrial study area includes that portion of the Susitna Basin that lies within about 15 miles of the Susitna River from Gold Creek to the Oshetna River mouth. In addition, studies are being conducted within the Susitna River floodplain between Gold Creek and Cook Inlet. Cultural resource studies would be conducted primarily in the vicinity of the impoundment areas, along the access roads, railway and transmission line routes.

These activities will not involve ground distrubance nor require culvert or bridges; hence waterways and drainage courses will not-be affected.

1.5 STAGE II PROJECT CONSTRUCTION

The Alaska Power Authority and their engineering consultant will prepare engineering design memoranda, construction drawings and specifications for the features described in 1.3.2 Watana Dam and Reservoir. In project features requiring drainage structures or waterways, the technical criteria presented in these guidelines will be incorporated, and used in the design memoranda, and construction contract documents.

2.0 FLOW DETERMINATION

2.1 GENERAL

In this section, the methodologies for determining the flow in a waterway for a specified recurrence frequency are discussed.

2.2 GAGED WATERCOURSES

The U.S. Geological Survey, in cooperation with the Alaska Department of Transportation and Public Facilities and other State and Federal Agencies, maintains a network of stream gaging stations and crest gages throughout the State of Alaska. The data obtained from these programs is published in Water Resources Data for Alaska, Part 1, Surface Water Records. The U.S. Geological Survey has published computer print-outs of frequency-discharge curves for all stations with satisfactory length of record. Data obtained from the stream gaging program has been used to formulate a report that presents regional flood frequency curves for most sites in Alaska. The publication contains Magnitude and Frequency of Floods in Alaska South of the Yukon River, Geological Survey Circular 493.

In the case where a site is being investigated on a waterway that has a gage and historical records of flow, the drainage area above the site will be compared with that above the gage to determine if there is compatibility in the factors that affect runoff for the two areas. Factors that affect runoff can be grouped into two major categories; climatic, which for the purposes of these guidelines may have little or no incidence, and physiographic. Climatic factors mainly include the effects of rain, temperature, and evapotranspiration, all of which exhibit seasonal changes in accordance with the climatic environment. Physiographic factors may be further classified into two kinds: basin characteristics and channel characteristics. Basin characteristics include such factors as size, shape, drainage area, permeability and capacity of groundwater reservoirs, presence of lakes and swamps, land use, etc. Channel characteristics are related mostly to hydraulic properties of the channel which govern the movement and configuration of flood waves and develop the storage capacity. It should be noted that the above classification of factors are interdependent to a certain extent. For clarity, the following is a list of the major factors:

Meteorologic factors

1) Rainfall
   a) Intensity
   b) Duration
   c) Time distribution
   d) Areal distribution
   e) Frequency
   f) Geographic location
2) Snow
3) Temperature
4) Evapotranspiration

Physiographic factors

1) Basin Characteristics
   a) Geometric factors
      1. Drainage area
      2. Shape
      3. Slope
      4. Stream density
      5. Mean Elevation
   b) Physical factors
      1. Land use or cover
      2. Surface infiltration condition
      3. Soil type
      4. Geological condition, such as the permeability and capacity of groundwater reservoir
      5. Topographical condition, such as the presence of lakes, swamps, and glaciers.

2) Channel characteristics

   a) Carrying capacity, considering size and shape of cross section, slope, and roughness

   b) Storage capacity

If there is no significant difference in these factors for both drainage areas above the site or above the existing gage, the flow at the site can be computed for the specified frequency by multiplying the gaged flow by the ratio of the squares of drainage areas.

$$Q_s = Q_g \frac{(A_s)^{1/2}}{(A_g)^{1/2}}$$

Q_s = Site flow

Q_g = Gage flow

A_s = Site drainage area

A_g = Gaged drainage area

In cases where compatibility in the factors cannot be readily ascertained, the staff hydrologist should be consulted or the flow determination be made using the methodology for ungaged watercourses outlined in the succeeding paragraph.

2.3 UNGAGED WATERCOURSES

2.3.1 General

The relation between rainfall and peak runoff has been represented by many empirical and semiempirical formulas. The rational formula which will be used in these guidelines can be taken as representative of these formulas. The rational formula is:

$$Q = CIA$$

where Q is the peak discharge in cubic feet per second (cfs), C a runoff coefficient dependent on the physiographic conditions of the drainage area, the average rainfall intensity (I) in inches per hour and A is the drainage area in acres.

In using the rational formula it is assumed that the maximum rate of flow, due to a rainfall intensity over the drainage area, is produced by that rainfall intensity being maintained for a time equal to the period of concentration of flow at the point under consideration (T_c).

The elements involved in runoff are far more complicated than the rational formula indicates. In larger drainage areas the temporary storage of storm water in overland travel toward stream channels and in these channels themselves accounts for a considerable reduction in the peak discharge rate. It is for this reason the Alaska Department of Highways recommends that use of this method be restricted to drainage areas less than 200 acres unless no other method is available to estimate discharges.

The remainder of this section will be dedicated to quantifying the parameters used in the rational formula.

2.3.2 Runoff Coefficient

The rational formula runoff coefficient (C) is the ratio of runoff to the average rate of rainfall at an average intensity when all the drainage area is contributing. Since this is the only manipulative parameter in the rational formula, judgement in its selection should reflect the physiographic factors listed in paragraph 2.2.

Table 2.3.1 presents values of relative imperviousness for various surfaces. In Table 2.3.2 the runoff coefficient C can be determined by weighting physiographic factors (watershed characteristics) and summing them.

Table 2.3.1 Values of Relative Imperviousness

Type of Surface	Factor C
For all watertight roof surfaces	0.75 to 0.95
For asphalt runway pavements	0.80 to 0.95
For concrete runway pavements	0.70 to 0.90
For gravel or macadam pavements	0.35 to 0.70
*For impervious soils (heavy)	0.40 to 0.65
*For impervious soils, with turf	0.30 to 0.55
*For slightly pervious soils	0.15 to 0.40
*For slightly pervious soils, with turf	0.10 to 0.30
*For moderately pervious soils	0.05 to 0.20
*For moderately pervious soils, with turf	0.00 to 0.10

*For slopes from 1% to 2%

To account for antecedent precipitation conditions, as reflected by the frequency of the selected rainfall intensity, a correction factor C_a should be multiplied with the runoff coefficient C. Values of C_a for various recurrence intervals are listed below:

Recurrence Interval (Years)	C_a
2 to 10	1.0
25	1.1
50	1.2
100	1.25

In no case should the product C x C_a exceed 1.

2.3.3 Drainage Area

The drainage area, in acres, which contributes to the site for which the discharge is to be determined, can be calculated from a topographic map or

Table 2.3.2 RUNOFF COEFFICIENTS "C"

Designation of Watershed Characteristics	Runoff Producing Characteristics of Watershed With Corresponding Weights			
	Extreme 75 to 100%	High 50 to 75%	Normal 30 to 50%	Low 25 to 30%
Relief	(40) Steep, rugged terrain, with average slopes generally above 30%.	(30) Hilly, with average slopes of 10 to 30%.	(20) Rolling, with average slopes of 5 to 10%.	(10) Relatively flat land, with average slopes of 0 to 5%.
Soil	(20) No effective soil cover; either rock or thin soil mantle of negligible infiltration capacity.	(15) Slow to take up water; clay or other soil low infiltration capacity.	(10) Normal; deep permeable soils.	(5) High; sands, loamy sands and other loose, open soils.
Vegetal cover	(20) No effective plant cover; bare or very sparse cover.	(15) Poor to fair; clean-cultivated crops or poor natural cover; less than 10% of drainage area under good cover.	(10) Fair to good; about 50% of drainage area in good grassland, woodland, or equivalent cover; not more than 50% of area in clean-cultivated crops.	(5) Good to excellent; about 90% of drainage area in good grassland, woodland, or equivalent cover.
Surface Storage	(20) Negligible, surface depressions few and shallow; drainageways steep and small; no ponds or marshes.	(15) Low; well-defined system of small drainageways; no ponds or marshes.	(10) Normal; considerable surface depression storage; drainage system similar to that of typical prairie lands; lakes, ponds, and marshes less than 2% of drainage area.	(5) High; surface depression storage high; drainage system not sharply defined; large floodplain storage or a large number of lakes, ponds, or marshes.

from measurements taken in the field. If the former is used, a site visit should be programmed to gather information to be used in determining the runoff coefficient C and the parameters that will affect the value of the selected rainfall intensity. Also, the site visit, literature review and discussions with fisheries resource managers should be used to ascertain the type of water course. (see 1.2 Scope).

2.3.4 Rainfall Intensity

Rainfall intensity, for the drainage area in question can be estimated for specific recurrence intervals (frequency) from the isohyetal maps in Appendix B. The average rainfall I used in the rational formula depends upon size and shape of the drainage area, the land slope, type of surface, whether flow is overland or channelized as well as the rainfall intensity.

The former factors are instrumental in determining the time of concentration (T_c) for the drainage area.

The theory underlying the development of the rational formula is that the maximum discharge at any point in a drainage system occurs when:

1. The entire area tributary to the point is contributing to the flow.

2. The average rainfall intensity producing such flow is based upon the rainfall which can be expected to fall in the time required for water to flow from the most remote point of the area to the point being investigated. The "most remote point" is the point from which the time of flow is greatest. It may not be at the greatest linear distance from the point under investigation.

Nomographs for the determination of time of concentration T_c for small and large drainage areas are presented in Figures 2.3.1 and 2.3.2 respectively. These nomographs utilize the length of travel (L) in feet and the difference in elevation (H) in feet between the beginning and end point.

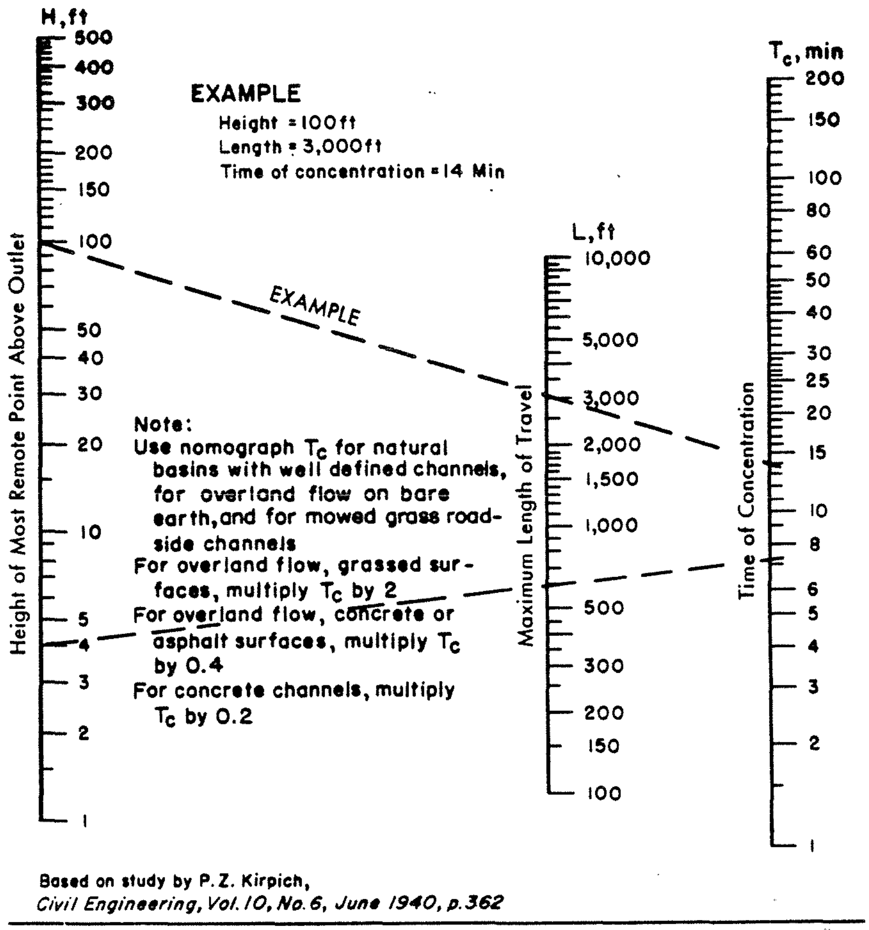


Fig. 2.3.1 Tc Nomograph for Small Watersheds

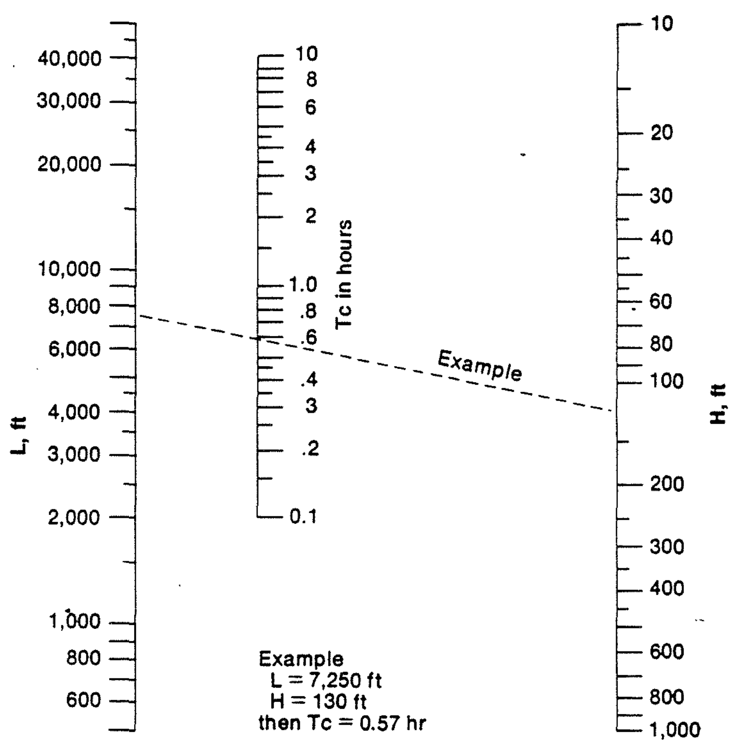


Fig. 2.3.2 Tc Nomograph for Large Watersheds

The time of concentration of the preceeding nomographs may be calculated by the following equation:

$$T_c = .0078 \left(\frac{L}{\sqrt{H/L}} \right)^{0.77}$$

where T_c is in minutes. The T_c calculated by the preceeding methods assumes a natural drainage basin with well defined channels, for overland flow on bare earth, and for mowed grass road side channels. If the overland flow is on grassed surfaces multiply the T_c by 2. For overland flow on concrete or asphalt surfaces multiply T_c by 0.4. For concrete channels, multiply T_c by 0.2.

Alternatively travel times for overland flow in watersheds with a variety of land covers can be calculated by the Uplands Method. (See Figure 2.3.3). The individual times are calculated from the velocity for each ground cover slope, subarea and the summation of the time giving the time of concentration T_c.

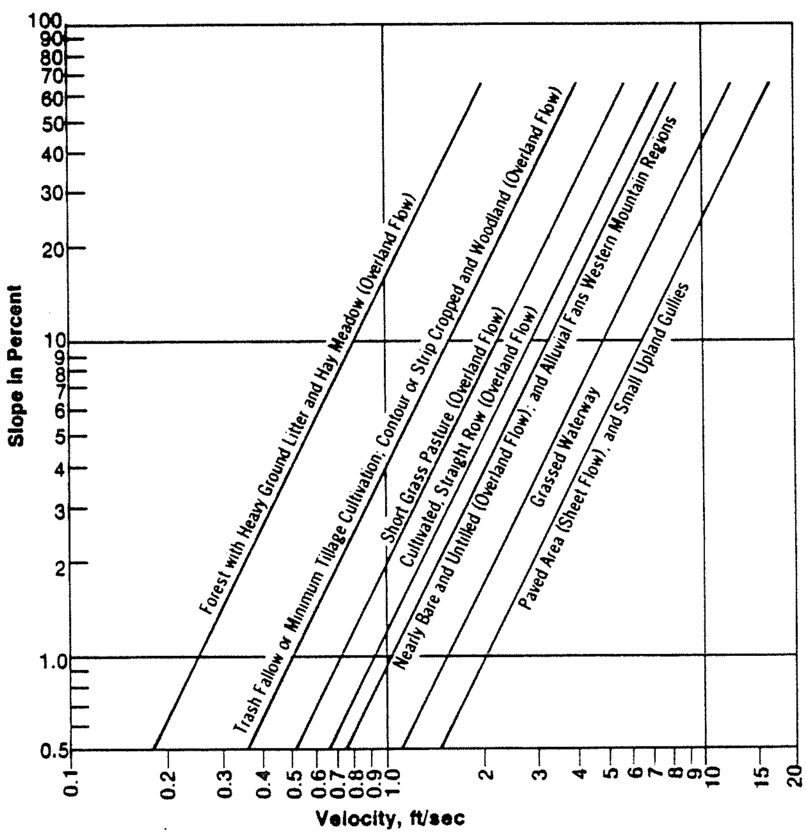


Figure 2.3.3 Upland Method Velocity Determination

With the time of concentration calculated and the rainfall intensity for the area selected (Appendix B) the average rainfall intensity for the drainage area may be determined using Figure 2.3.4. The curve for the selected one hour rainfall is followed to the right or left until reaching the calculated time of concentration and the average rainfall intensity (I) can be determined.

2.4 EXAMPLE PEAK DISCHARGE DETERMINATION

As an example of the use of the Rational Method, a hypothetical drainage area and its characteristics are used.

A drainage area of 40 acres with a distance from its most remote point being measured as 2500 feet of which 500 feet is overland flow in forests with heavy ground litter having an average slope of 5%. The remaining 2000 feet can be classified as in a natural basin with well defined channels with a drop in elevation of 150 feet.

Watershed characteristics:	Reference Table 2.3.2
Relief; Flat to rolling land average slope approximately 7 percent	0.15
Soil; Medium soil permeabilities	0.18
Vegetal cover; 25% of the area under good cover	0.13
Surface Storage; Well defined system of drainage ways on 50% of area, negligible on remainder	0.17
Sum C =	0.63

It is required to determine the runoff for recurrence periods of 2, 10 and 50 years for a location 100 miles north of Anchorage. Referring to Appendix

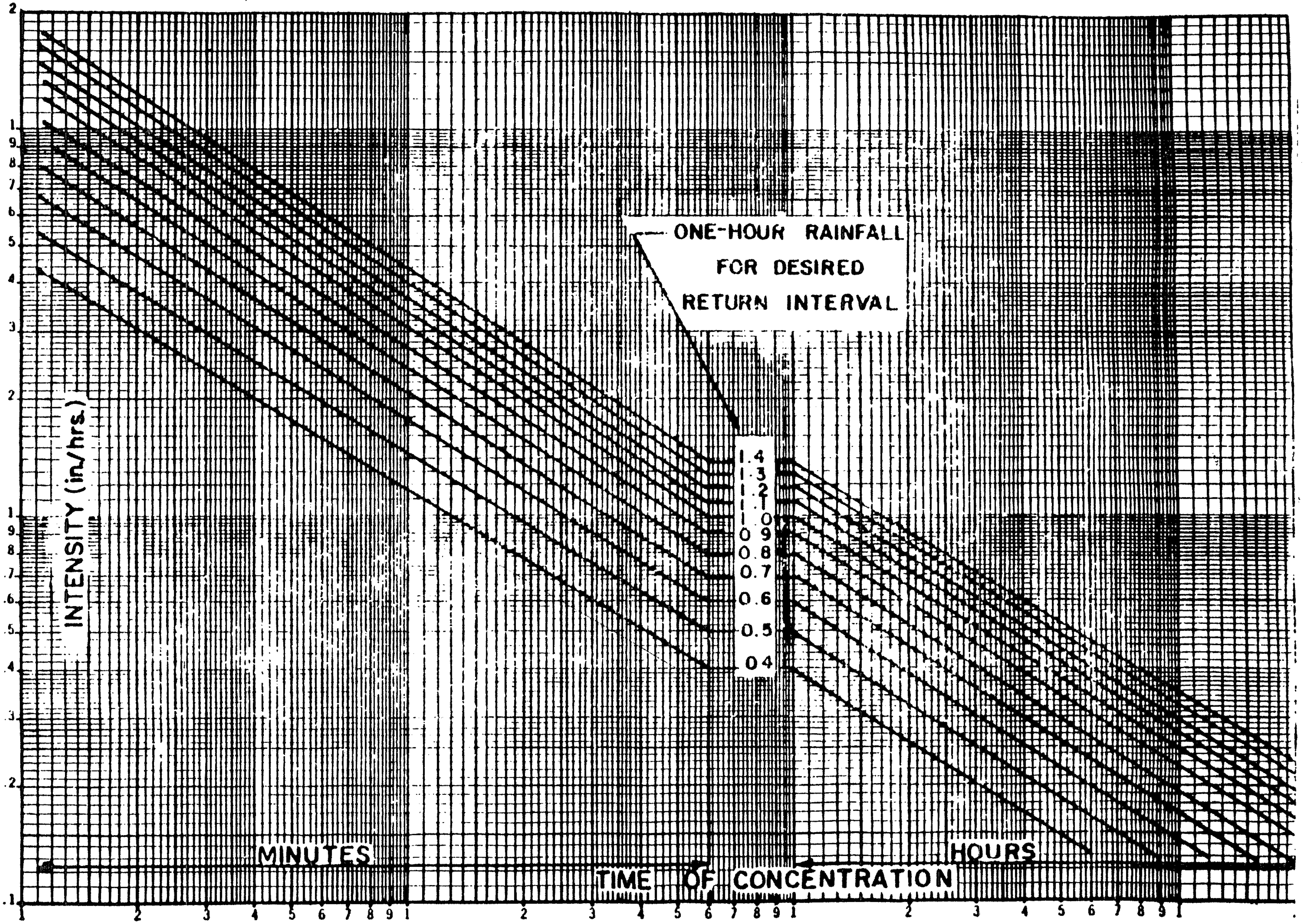

ONE-HOUR RAINFALL FOR DESIRED RETURN INTERVAL
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
INTENSITY (in/hrs.)
MINUTES
HOURS
TIME OF CONCENTRATION
2
1.
9
8
7
6
5
4
3
2
1
.9
.8
.7
.6
.5
.4
.3
.2
.1
1 2 3 4 5 6 7 8 9 1 2 3 4 5 6 7 8 9 1 2 3 4 5 6 7 8 9 1

Fig. 2.3.4

B Figures B2, B4 and B6 we estimate 0.4, 0.6 and 0.7 inches per hour respectively.

Time of concentration T_c)

Overland flow Figure 2.3.3 yields a velocity of 0.6 ft/sec.

The Remaining route of 2000 feet and 150 feet of drop from Figure 2.3.1 give 9 minutes.

Time of concentration

$$T_c = 9 + \frac{500 \text{ ft}}{0.6 \text{ ft/sec } 60 \text{ sec}} = 22.9 \text{ minutes}$$

Average Rainfall Intensity (I)

Referring to Figure 2.3.4 with the given rainfall intensities the average rainfall intensities (I) can be derived as follows:

Frequency	One hour Rainfall	Average Rainfall Intensity (I)
2 year	0.4 in.	0.72 in.
10 year	0.6 in.	1.25 in.
50 year	0.7 in.	1.48 in.

Runoff Calculations

Two year frequency

$$Q = CIA = 0.63 \times 0.72 \times 40 = 18.14 \text{ ft}^3/\text{sec}$$

Ten year frequency

$$Q = CIA = 0.63 \times 1.25 \times 40 = 31.50 \text{ ft}^3/\text{sec}$$

Fifty year frequency

$$Q = C\ C_a{*}\ I\ A = 0.63 \times 1.2 \times 1.48 \times 40 = 44.76 \text{ ft}^3/\text{sec}$$

*Antecedent precipitation correction factor see paragraph 2.3.2.

3.0 HYDRAULIC DESIGN

3.1 INTRODUCTION

An effective drainage structure and waterway design process involves many factors, principal of which are hydraulic performance, structural adequacy and overall construction and maintenance costs. The design process will include an assessment by a fisheries biologist to determine whether the water course is a fish stream, Type A or Type B, (see section 1.2-Scope).

A fish stream is defined as any water flow that is accessible to fish and capable of supporting aquatic life. This would include, but is not limited to, all Alaska Department of Fish and Game designated streams and all their tributaries up to impassable natural barriers. Type A freshwater systems above blockages may also support resident fish stocks. Evaluation and recommendations will be made by a fisheries biologist during site location to determine the presence of fish stocks.

If the waterway is classified as either Type A or Type B the following criteria should be included in the design process.

3.2 FISH PASSAGE PROBLEMS

The efficient passage of fish through a drainage structure requires close attention to the resolution of three problems:

1. Excessive water velocity
2. Inadequate water depth

3. Excessive hydraulic jump

3.2.1 Excessive Water Velocity

Excessive water velocities can block fish movement simply by exceeding the swimming ability of fish. Swimming ability varies with species, size and age of fish, and length of drainage structure (culvert). Studies of fish movement have provided the information presented on Table 3.2.1.

Slope is the most important factor determining velocity in culverts. Slopes steeper than 0.5 percent (1/2 foot drop in 100 feet) generally create excessive velocities for fish passage.

3.2.2 Inadequate Water Depth

Fish require sufficient water depth to attain maximum swimming abilities. The depth required is directly related to fish size with larger fish requiring deeper water. When insufficient depths are encountered, fish are unable to produce full propulsion.

Causes of inadequate depth. The two most frequently encountered reasons for insufficient water depth are steep slope and a wide, flat channel bottom (no low flow channel).

a. All other factors being constant, the steeper the slope of a structure the shallower the water depth.

b. All other factors being constant, the wider the structure bottom the shallower the water depth.

Table 3.2.1

AVERAGE CROSS SECTIONAL VELOCITIES IN FEET/SECOND MEASURED[1/]
AT THE OUTLET OF THE CULVERT

Length of Culvert in Feet	Group I Upstream migrant salmon fry and fingerlings when upsteam migration takes place at mean annual flood	Group II Adult and juvenile slow swimmers: grayling, longnose suckers, whitefish, burbot, sheefish, Northern pike, Dolly Varden/Arctic Char, upstream migrant salmon fry and fingerlings when migration not at mean annual flood	Group III Adult moderate swimmers: pink salmon, chum salmon, rainbow trout, cutthroat trout	Group IV Adult high performance swimmers: king salmon, coho salmon, sockeye salmon, steelhead
30	1.0	4.6	6.8	9.9
40	1.0	3.8	5.8	8.5
50	1.0	3.2	5.0	7.5
60	0.9	2.8	4.6	6.6
70	0.8	2.6	4.2	6.0
80	0.8	2.3	3.9	5.5
90	0.7	2.1	3.7	5.1
100	0.7	2.0	3.4	4.8
150	0.5	1.8	2.8	3.7
200	0.5	1.8	2.4	3.1
200	0.5	1.8	2.4	3.0

1/ Title 5 Fish and Game Part 6 Protection of Fish and Game Habitat Chapter 95 - Alaska Department of Fish and Game

Minimum water depths required for instream movement of juveniles will vary with species and size of fish present. Generally, 0.2 foot (2.4 inches) is sufficient for passage. Minimum water depths for adult fish are:1/

King Salmon-0.8 feet
Other salmon and trout over 20 inches-0.6 feet
Trout under 20 inches-0.4 foot

3.2.3 Excessive Hydraulic Jump

The two basic causes for a hydraulic jump at the downstream end of a structure are bed scour and slope of structure placement.

a. Degradation of the streambed below the structure can result in lowering the water surface below the downstream end of the structure. This occurs most frequently in steep gradient streams with erodible bottom materials. Degradation of a receiving waterway can create a hydraulic jump at a downstream end of the structure.
tributary.

b. Placement of a flat sloped structure on a steep sloped waterway builds in a jump.

3.2.4 Guidelines for Structures

Location: The guidelines for locating structures for fish passage are also coincidental with those for hydraulic design.

1. There should not be a sudden increase in velocity immediately above, below, or at the crossing.

2. Structures should not be located on a sharp bend in the stream channel.

1/ Lauman, J.E. Salmonid Passage at Stream - Road Crossings: A Report with Department Standards for Passage of Salmonids. 1976 Department of Fish and Wildlife Portland Oregon.

3. Structures should be designed to fit the stream channel alignment. They should not necessitate a channel change to fit a particular cross ing design.

3.3 DRAINAGE STRUCTURE DESIGN CRITERIA

All drainage structures in waterways in which fish are known to frequent (Type A or B) shall be designed in accordance with the following criteria:

Water-course Type	Flood Frequency	
A	2 year	Maximum velocity per Table 3.2.1 group and twice the depth of flow per paragraph 3.2.2
B	10 years	No static head at culvert entrance
C	50 years*	Allowable pondage at site

* In the case that the drainage structure is at a primary road or railway the flood frequency is to be 100 years.

Drainage structures in waterways where there are no anadromous fish will be designed for criteria B and C above. Drainage structures that are classified as temporary, meaning that they will be removed and the habitat rehabilitated within a 10 year period will be designed for the preceeding criteria except that the flood frequency of criteria C will be 25 years.

Drainage structures in fishery streams shall be placed with the waterway substrate in its invert. In the case of culverts, at least one fifth of the diameter of each round culvert and at least 6 inches of the height of each elliptical or arch type culvert is to be set below the stream bed at both the inlet and outlet of the culvert. The above is not applicable to bottom less arch type culverts. In the case of a rock substrate, a request for variance should be submitted to the Alaska Department of Fish and Game (ADF&G) for approval.

A drainage structure design data sheet, tabulating information for each site, prepared by a fisheries biologist and a design engineer will be submitted to ADF&G for approval and approved prior to undertaking any construction. (See end of 3.4.3.8, Inlets and Culvert Capacity.)

The drainage structure design will require the following conditions to be adhered to during its construction.

a. All bank cuts, slopes, fills and exposed earth work attributable to installation in a waterway must be stabilized to prevent erosion during and after construction.

b. The width and depth of the temporary diverison channel must equal or exceed 75 percent of the width and the depth, respectively, of that portion of the waterway which is covered by ordinary high water at the diversion site, unless a lesser width or depth is specified by the ADF&G on the permit for activities undertaken during periods of lower flow;

c. During excavation or construction, the temporary diversion channel must be isolated from water of the waterway, to be diverted, by natural plugs left in place at the upstream and downstream ends of the diversion channel.

d. The diversion channel must be constructed so that the bed and banks will not significantly erode at expected flows.

e. Diversion of water flow into the temporary diversion channel must be conducted by first removing the downstream plug then removing the upstream plug, then closing the upstream end and then the downstream end, respectively, of the natural channel of the diverted waterway.

f. Rediversion of flow into the natural stream must be conducted by removing the downstream plug from the natural channel and then the upstream plug, then closing the upstream end and then the downstream end, respectively, of the diversion channel.

g. After use, the diversion channel and the natural waterway must be stabilized and rehabilitated as may be specified by permit conditions.

3.4 WATERWAY HYDRAULICS

3.4.1 General

A field inspection is basic to the design of diversion channels, culverts, and bridge encroachment into waterways, all of which encompass the drainage structures to which these guidelines are addressed.

For the design of drainage structures, the hydraulic condition of the prepared structure will be similar to the natural waterway upstream and downstream of the proposed structure site must be known. The parameters for a typical section will be measured in the field. During this inspection a check should be made of downstream controls. At times the tailwater is controlled by a downstream obstruction or by water stages in another waterway.

3.4.2 Waterways

This section describes the techniques for investigation of the waterway on which a drainage structure is to be constructed and the construction activities for a new waterway such as a temporary diverison channel. Hydraulic investigation and design of waterways will be based upon Manning's formula for uniform flow unless existing site conditions indicate that flows will be non uniform. A full treatment of this subject may be found in treatment of this subject may be found in Open-Channel Hydraulics by Ven Te Chow, Mc Graw Hill 1959.

The Manning formula:

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

Where: V is the mean velocity in fps;
R is the hydraulic radius ft;
S is the slope of the waterway, and
n is the coefficient of roughness, specifically known as Manning's n

The discharge in the waterway may be determined by multiplying by "A" the area of the water prism in the formula.

a. Waterway Investigation

A hydraulic rating curve of the waterway should be determined by measuring the waterway cross section between highwater marks on both sides of the waterway. If these marks are not visible a high water level should be estimated. Figure 3.4.1 is an example of a waterway cross section measurement.

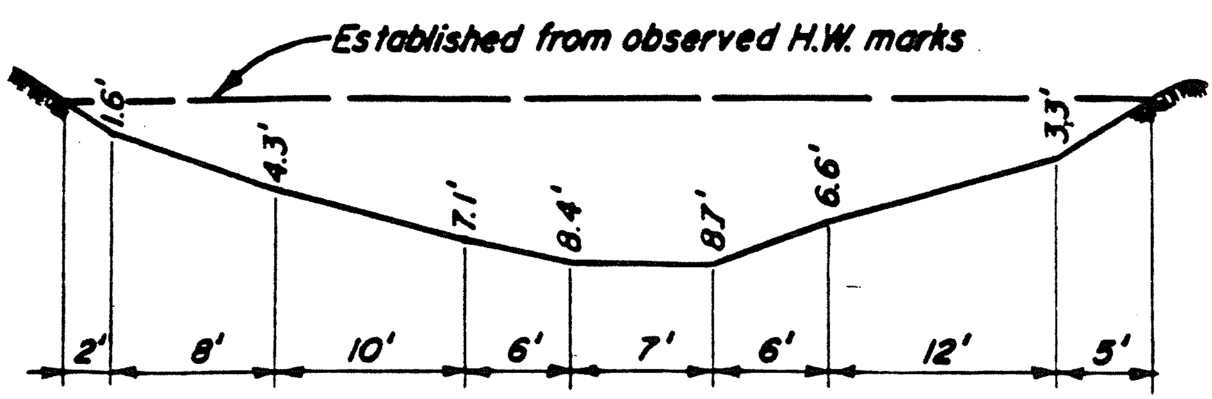


Figure 3.4.1 Waterway Cross Section Measurement

From the cross section the area and wetted perimeter should be calculated for at least 3 levels, or more if the waterway is deep, including the maximum level.

From the measured slope of the waterway and a determination of waterway roughness n, the discharges for the selected levels (depth of flows) can be calculated using Manning's formula. The n values for typical channel conditions are presented in Table 3.4.1 and a method used by the U.S. Soil Conservation Service for computing an n value taking into consideration factors that affect n is presented in Table 3.4.2.

b. Waterway Design

The required capacity of the waterway should be determined by the method indicated in Section 2.0-Flow Determination. If the waterway is to be designed for fish passage, the group (Table 3.2.1) and the minimum depth of flow for instream movement (paragraph 3.2.2) snould be determined.

Table 3.4.1 Typical Channel Roughness Coefficients[1/]

Value of n	Channel Condition
0.016-0.017	Smoothest natural earth channels, free from growth, with straigth alignment.
0.020	Smooth natural earth channel, free from growth, little curvature.
0.0225	Small earth channels in good condition, or large earth channels with some growth on banks or scattered cobbles in bed.
0.030	Earth channels with considerable growth. Natural streams with good alignment, fairly constant section. Large floodway channels, well maintained.
0.035	Earth channels considerably covered with small growth. Cleared but not continuously maintained floodways.
0.040-0.050	Mountain streams in clean loose cobbles. Rivers with variable section and some vegetation growing in banks. Earth channels with thick aquatic growths.
0.060-0.075	Rivers with fairly straight alignment and cross section, badly obstructed by small trees, very little underbrush or aquatic growth.
0.100	Rivers with irregular alignment and cross section, moderately obstructed by small trees and underbrush. Rivers with fairly regular alignment and cross section, heavily obstructed by small trees and underbrush.
0.125	Rivers with irregular alignment and cross section, covered with growth of virgin timber and occasional dense patches of bushes and small trees, some logs and dead fallen rees.
0.150-0.200	Rivers with very irregular alignment and cross section, many roots, trees, bushes, large logs, and other drift on bottom, trees continually falling into channel due to bank caving.

1/ Design of Small Dams, U.S. Bureau of Reclamation, 1977.

Table 3.4.2 Channel Roughness Determination[1/]

Steps

1. Assume basin n
2. Select modifying n for roughness or degree of irregularity
3. Select modifying n for variation in size and shape of cross section
4. Select modifying n for obstructions such as debris deposits, stumps, exposed and fallen logs
5. Select modifying n for vegetation
6. Select modifying n for meandering
7. Add items 1 through 6

Aids in Selecting Various n Values

1. Recommended basic in values
   - Channels in earth------------0.010
   - Channels in rock-------------0.015
   - Channels in fine gravel------------0.014
   - Channels in coarse gravel----------0.028

2. Recommended modifying n value for degree of irregularity
   - Smooth-----------------------0.000
   - Minor------------------------0.005
   - Moderate----------------------------0.010
   - Severe------------------------------0.020

3. Recommended modifying n value for changes in size and shape of cross section
   - Gradual----------------------0.000
   - Occasional-------------------0.005
   - Frequent------------------0.010 to 0.015

4. Recommended modifying n value for obstruction such as debris, roots, etc.
   - Negligible effect------------0.000
   - Minor effect-----------------0.010
   - Appreciable effect-----------------0.030
   - Severe effect----------------------0.060

5. Recommended modifying n values for vegetation
   - Low effect----------0.005 to 0.010
   - Medium effect-------0.010 to 0.025
   - High effect---------------0.025 to 0.050
   - Very high effect----------0.050 to 0.100

6. Recommended modifying n value for channel meander

   L_s=Straight length of reach L_m=Meander length of reach

L_m/L_s	n
1.0-1.2	0.000
1.2-1.5	0.15 times n_s
>1.5	0.30 times n_s

where n_s=items 1+2+3+4+5

1/ Design of Small Dams, U.S. Bureau of Reclamation, 1977.

The design of a stable channel is accomplished by trial and error. It is reasonable to expect a channel to suffer some damage during a 50-year flood event, but one would desire a stable channel for the 10-year flood event. Therefore as a trial starting point, the channel section will be designed for maximum discharge with a velocity approximately 20% higher than the velocity that would be permissible in the channel during the 10-year flood event.

Two methods will be presented for channel design; the Permissible Velocity Method and the Tractive Force Method. Examples of their use will also be presented.

3.4.2.1. Permissible Non-erodible Velocity Method

The maximum permissible velocity, or non-erodible velocity is the greatest mean velocity that will not cause erosion of the channel body. In general, old and well-seasoned channels will stand much higher velocities than new ones, because the old channel bed is usually better stabilized, particularly with the deposition of colloidal matter. When other conditions are the same, a deeper channel will convey water at a higher mean velocity without erosion than a shallower one.

Table 3.4.3. lists the maximum permissible velocity for channels with erodible linings based on uniform flow in continuously wet, aged channels.

Table 3.4.3*

RECOMMENDED PERMISSIBLE VELOCITIES (ft./sec.) FOR UNLINED CHANNELS

Type of Material for Excavated Section	Clear Water	Silt - Carrying Water
Fine Sand (non colloidal)	1.5	2.5
Sandy Loam (non colloidal)	1.7	2.5
Silt Loam (non colloidal)	2.0	3.0
Ordinary Firm Loam	2.5	3.5
Volcanic Ash	2.5	3.5
Fine Gravel	2.5	4.0
Stiff Clay (colloidal)	3.7	5.0
Graded Material:		
Loam to Gravel	3.7	5.0
Silt to Gravel	4.0	5.5
Gravel	5.0	6.0
Coarse Gravel	5.5	6.5
Gravel to Cobbles (<6")	6.0	7.0
Gravel to Cobbles (>6")	7.0	8.0
Shales and Hardpans	7.0	8.0

* State of California, Dept. of Public Works, Division of Highways, "Planning Manual of Instructions, Part 7, Design," 1963.

Using permissible velocity as a criterion, the design procedure for an unlined channel section, assumed to be trapezoidal, is as follows:

1. For the given kind of material forming the channel body, estimate the roughness coefficient n, side slope z, and the maximum permissible velocity, V (Table 3.4.3).

2. Compute the hydraulic radius R by use of the Manning formula.

3. Compute the water area required by the given discharge and permissible velocity, i.e.: $A = Q/(1.2V)$.

4. Compute the wetted perimeter, $P = A/R$.

5. Solve simultaneously for b and y (base and depth of flow).

6. With the given section, by iteration, calculate with varying depths of flow, the depth and velocity for the 10-year flood discharge. Check if velocity is equal or less than the permissible. If not, change slope if possible or lower velocity and repeat 1.

7. For fish streams, repeat 6 for the 2-year flood discharge to check if velocity is equal to or less than permissible fish passage velocity for the designated group (Table 3.2.1) and the depth of flow is at least 50% greater than that indicated in paragraph 3.2.2 for the fish type. If the above are not met, a further channel revision may be required necessitating recalculation beginning with 1 or the incorporation of a low flow section in the invert of the channel.

A calculation example follows in steps 1 through 6

Compute the bottom width and depth of flow of a trapezoid channel laid on a slope of .0016 and carrying a design discharge of 400 cfs. The channel is to be excavated in earth containing non-colloidal gravelly silt.

Solution:

For the given conditions, the following are estimated: n = 0.025, side slope z = 2:1, and maximum permissible velocity = 3.75 x 1.2 = 4.5 fps.

1. Using the Manning Formula, solve for R

$$4.5 = \frac{1.49}{0.025} R^{2/3} (.0016)^{1/2}$$

R = 2.60 ft
Then A = 400/4.5 = 88.8 ft^2, and P = A/R = 88.8/2.60 = 34.2

2. $A = (b + zy)y = (b + 2y)y = 88.8 \text{ ft}^2$
and $P = b + 2\ (1 + z^2)^{1/2}y = [b + 2(5)^{1/2}y] = 34.2$ ft.

3. Solving the two equations simultaneously:
$(b + 2y)y = 88.8$
$(b + 4.47y) = 34.2$
$88.8 - 2y^2 = 34.2y - 4.47\ y^2$
$2.47y^2 - 34.2y + 88.8 = 0$
$y = 3.46$ ft
$b = 18.7$ ft

3.4.2.2 Tractive Force Method

The tractive force method takes into account physical factors of bed material, channel section, depth of flow and velocity. This method will be confined to non cohesive materials for which the permissible tractive force is related to particle size and shape, and sediment load in the water. The tractive force is the unit force tending to cause erosion of the material forming the channel. Figure 3.4.2 shows curves for recommended values of permissible unit tractive force for particles up to about 4 inches in diameter. For coarser material, the permissible tractive force in psf is equal to 0.4 times the diameter in inches as shown in Figure 3.4.3. The diameter is that of a particle of equivalent spherical volume. The curves in Figures 3.4.2 and 3.4.3 are based on particle sizes of which 25% by weight are larger.

The limiting condition for permissible tractive force is governed by the particles on the sides rather than those on the bottom of the channel. The resistance of the material on the sides is reduced by the sliding force down the sides due to gravity. The effect of side slopes is expressed as factor K, which is the ratio of the tractive force required to initiate motion of a particle on the sloping sides to that on a level bottom. The equation is:

$$K = \left(1 - \frac{\sin^2 \emptyset}{\sin^2 \emptyset} \right)^{1/2}$$

ϕ = side slope angle

θ = angle of repose of the material which varies with particle size and shape as shown in Figure 3.4.4.

The solution of this equation is given in Figure 3.4.5.

The formula for maximum tractive force is:

T_0 = 62.4 RS

S = energy gradient in ft/ft (channel slope for uniform flow)

R = hydraulic radius

In a wide open channel, the hydraulic radius is approximately equal to the depth of flow y; hence, T_0 = 62.4 yS.

Channels in fine material less than 5 mm in diameter are designed by using the recommended values of tractive force ploted in Figure 3.4.2. In this case, "d" is the mean diameter for which 50% by weight are larger. The sliding effect of the particles down the channel sides due to their own weight is neglected.

An example using values for; a 10-year flood design, a trapezoidal channel laid on a slope of .0016, and carrying a discharge of 400 cfs. The channel is to be excavated in earth containing noncolloidal coarse gravels and pebbles, 25% of which is 1.25 in or over in diameter. Manning's n = 0.025.

For trapezoidal channels, the maximum unit tractive force on the sloping sides is usually less than that on the bottom (Figure 3.46); hence, the side force is the controlling value in the analysis. The design of the channel should therefore include: (a) the proportioning of the section dimensions for the maximum unit tractive force on the sides and (b) checking the proportioned dimensions for the maximum unit tractive force on the bottom.

a. <u>Proportioning the Section Dimensions:</u>

1. Assuming side slopes of 2:1 and a b/y ratio =5, the maximum unit tractive force on the sloping sides (Figures 3.4.6) is .775 x 62.4 yS = .775 x 62.4 x .0016y = 0.078y psf.

2. Considering a very rounded material 1.25 in. in diameter, the angle of repose (Figure 3.4.4) is θ = 33.5. With θ = 33.5 and SS = 2.1, the permissible tractive force ratio on the sloping sides (Figure 3.4.5) is K = 0.6. For a size of 1.25 in., the permissible tractive force on a level bottom is T = 0.4 x 1.25 = 0.5 psf (this can also be obtained from Figure 3.4.2) and the permissible tractive force on the sides is equal to 0.6 x 0.5 = 0.3 psf.

3. For a state of impending motion of the particles on side slopes, .078y = 0.3 or y = 3.88 ft. Accordingly, the bottom width b = 5 x 3.85 = 19.3 ft. For this trapezoidal section, A = 104 sq ft and R = 2.85.

4. By the Manning equation $Q = \frac{1.486}{n} AR^{2/3}S^{1/2}$

   $= \frac{1.486}{.025} (104)\ (2.85)^{2/3}\ (.0016)^{1/2} = 491$ cfs

   Further computation will show that for a side slope of 2:1 and b/y ratio of 4.1, b = 15.8 ft., Q = 425 cfs, which is close to the design discharge.

b. <u>Checking the proportioned dimensions:</u>

With SS = 2:1 and b/y = 4.1, the maximum unit tractive force on the channel bottom (Figure 3.4.6) is 0.97 x 62.4 x 3.85 x .0016 = 0.374 psf < 0.5 psf (permissible tractive force on the bottom).

5. Determining maximum flow conditions: with base width and side slopes determined, the depth of flow required for the maximum flow conditions can be determined using the Manning formula.

6. For fish streams, repeat paragraph 5 for the 2-year flood discharge to check if velocity is equal to or less than permissible fish passage velocity for the designated group (Table 3.2.1) and the depth of flow is at least 50% greater than that indicated in paragraph 3.2.2 for the fish type. If the above are not met, a further channel revision may be required, necessitating recalculation beginning with 1 or the incorporation of a low flow section in the invert of the channel.

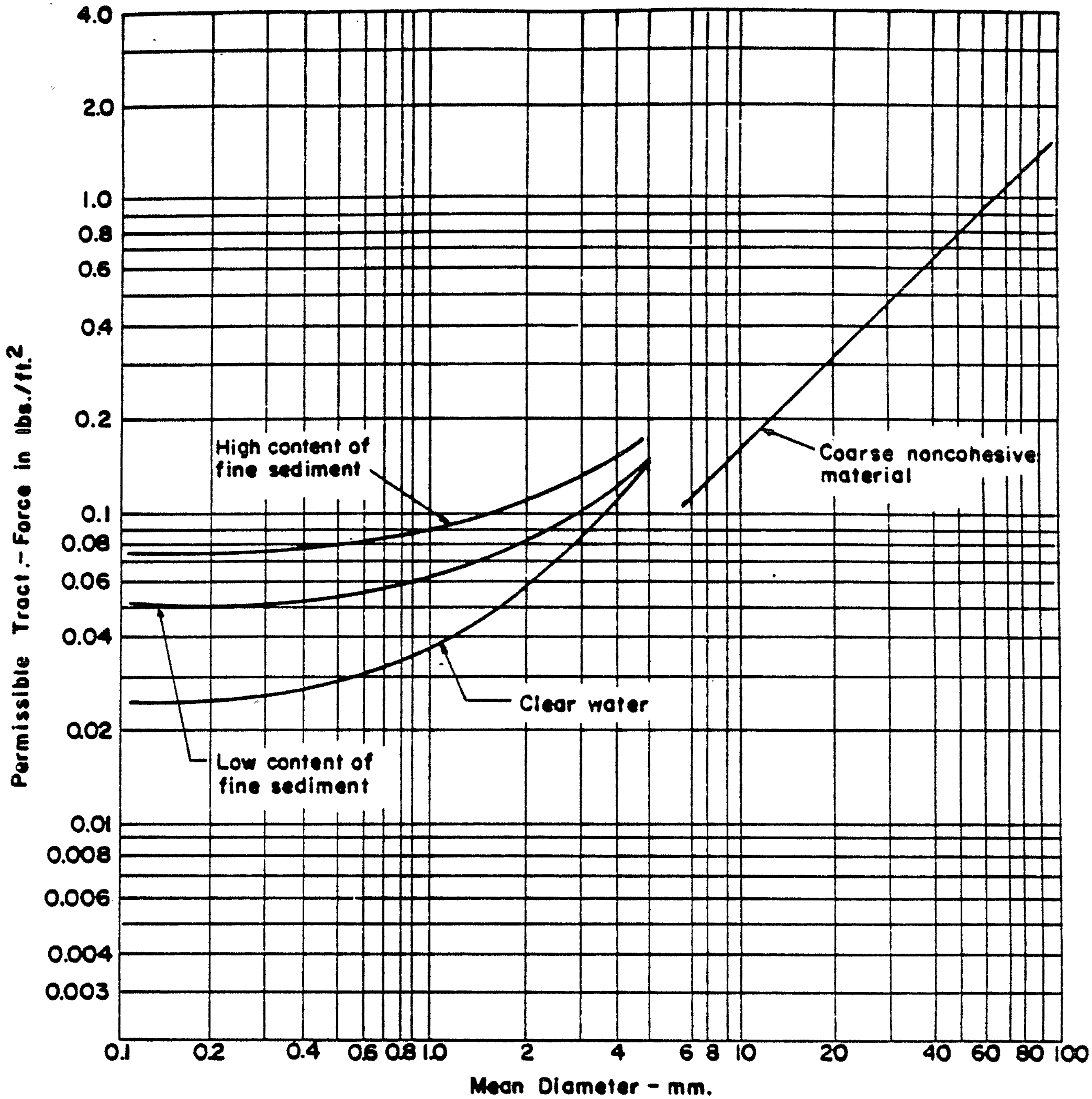


Figure 3.4.2

RECOMMEND PERMISSIBLE UNIT TRACTIVE FORCE

FOR CANALS IN NONCOHESIVE MATERIAL

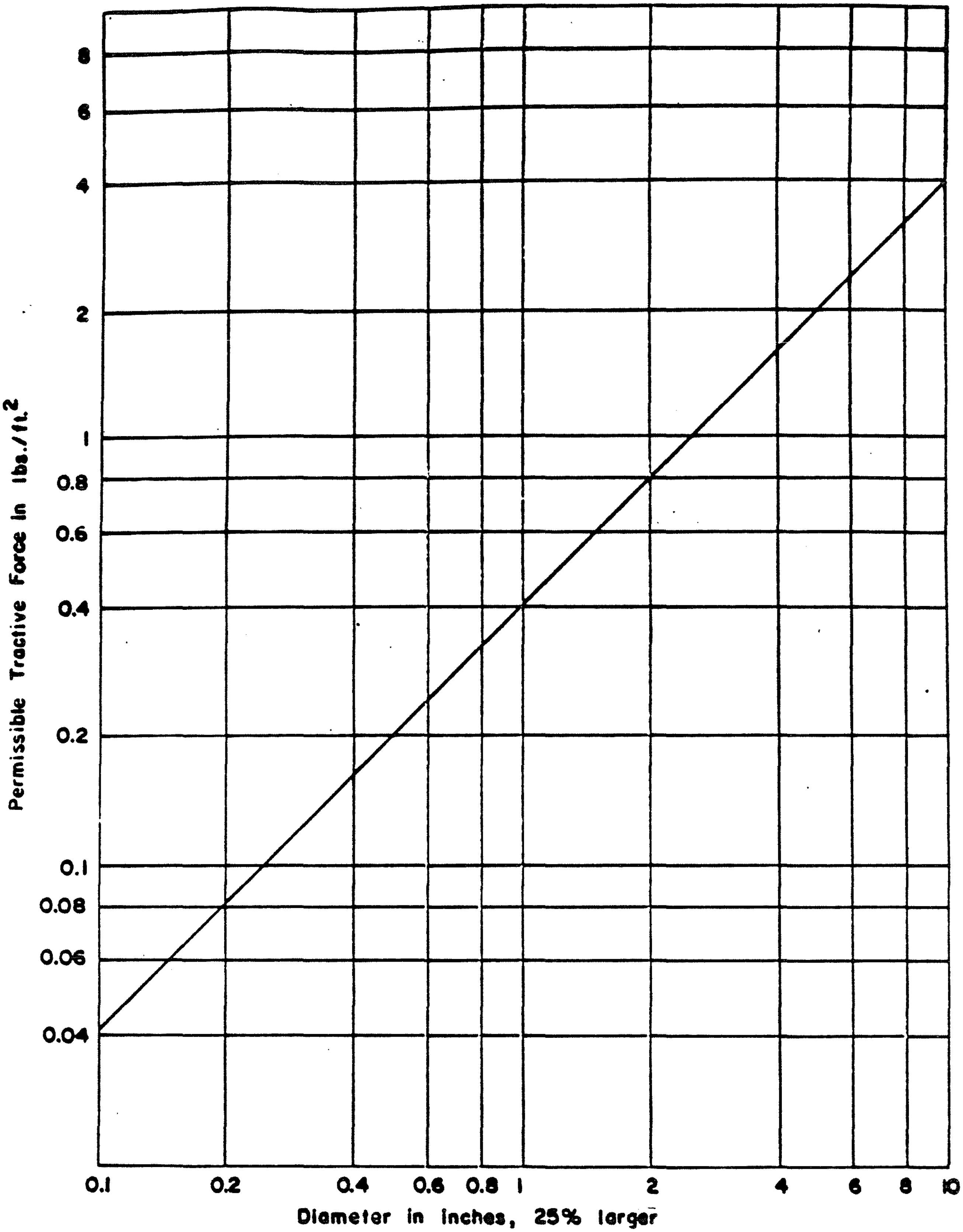


Figure 3.4.3

PERMISSIBLE TRACTIVE FORCE FOR COARSE NONCOHESIVE MATERIAL

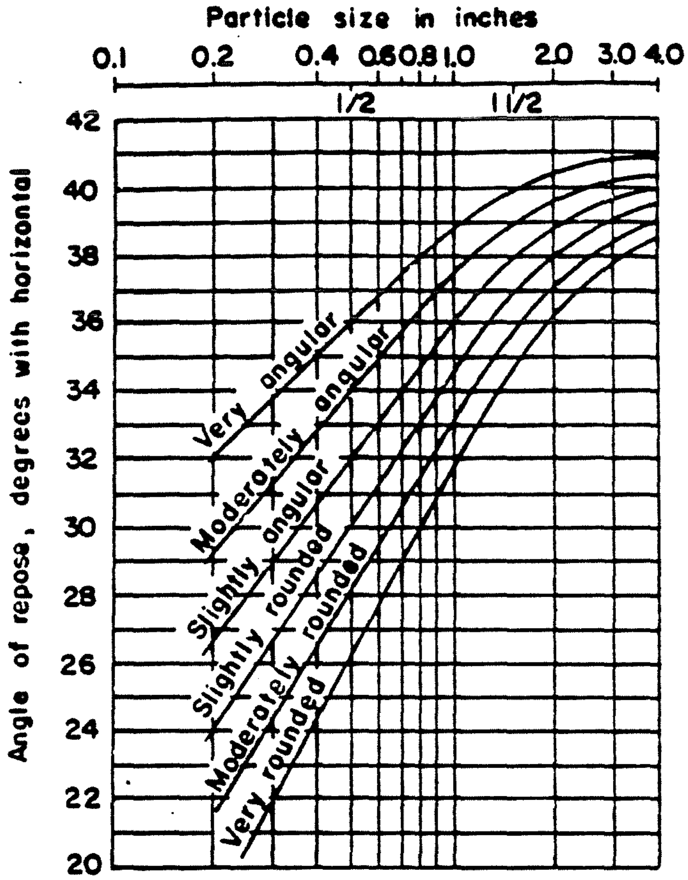


Figure 3.4.4

ANGLES OF REPOSE OF
NONCOHESIVE MATERIAL

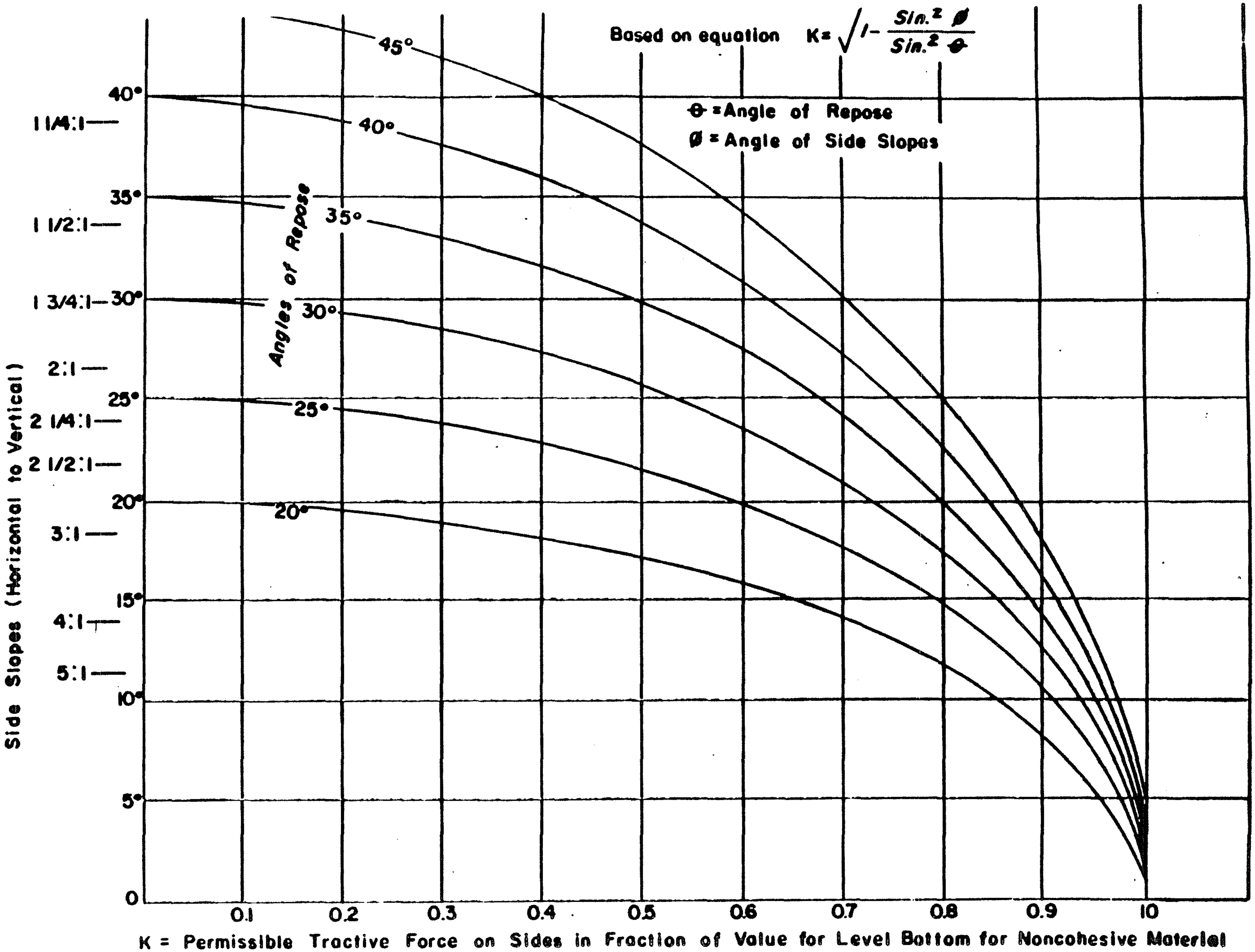
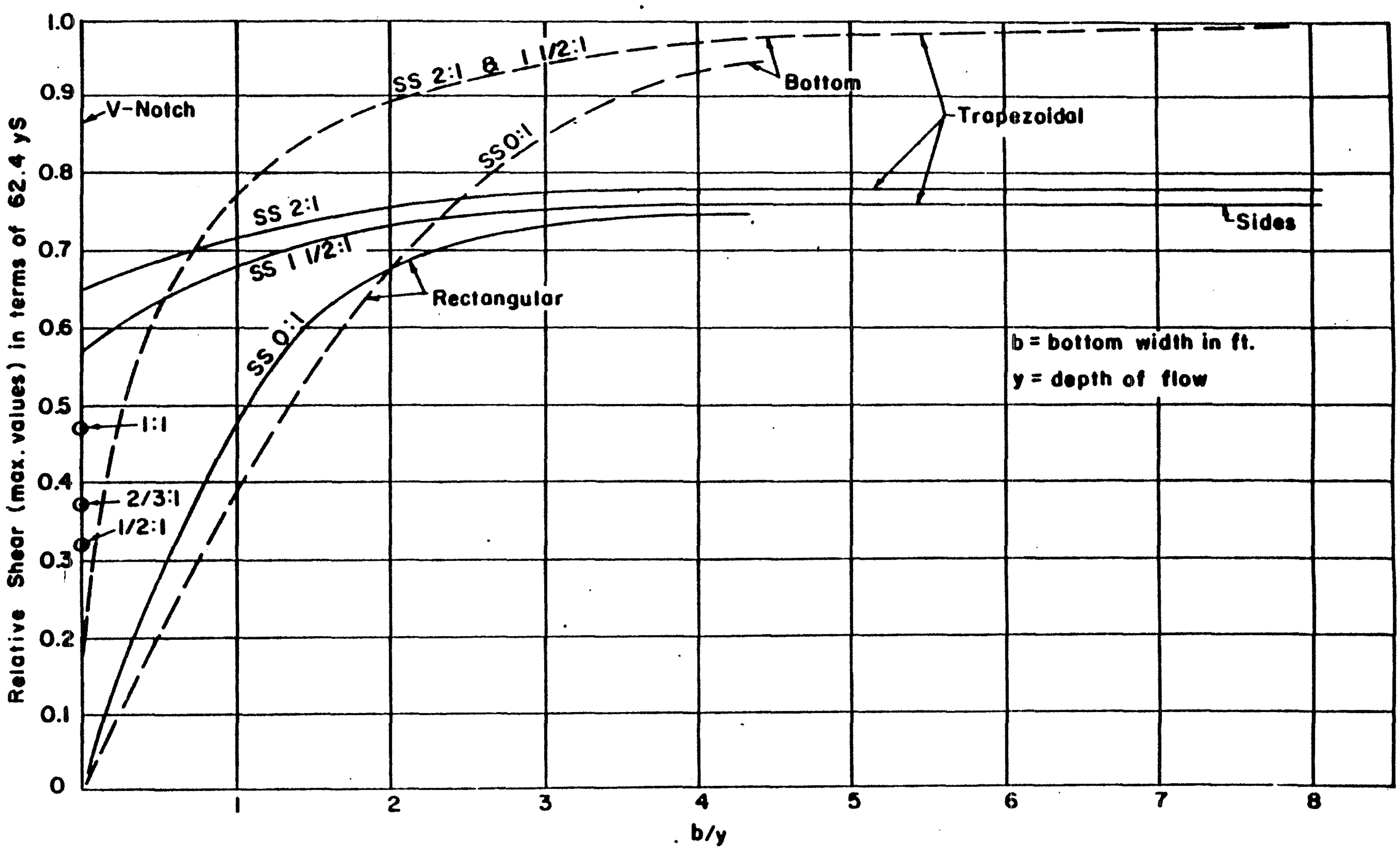


Figure 3.4.5
TRACTIVE FORCE RATIO K VS SIDE SLOPE



THE MAXIMUM TRACTIVE FORCE ON BED AND SIDES

Figure 3.4.6
THE MAXIMUM TRACTIVE FORCE ON BED AND SIDES

3.4.3 Culverts

3.4.3.1 Fish Passing Requirements. The following presentation on culvert design is essentially a repitition of the Hydraulic Engineering Circular No. 5 pepared by the Bureau of Public Roads, U.S. Department of Commerce. As such the design criteria established are for the design of highway culverts and includes no provisions of fish passage criteria. Therefore, this paragraph will ammend the following in the instances that the culvert is to be placed on a waterway that has been established to have resident fish or used by anadromous fish.

In Sections 3.2, Fish Passage Problems and 3.3 Drainage Structure Design Criteria, the basic requirements were presented for the successful design of a culvert for passing fish. They were:

1. Velocity requirement per fish group (Specified in Table 3.2.1)

2. Place invert below waterway bed by at least 0.2 diameter

3. Maintain depth of flow requirement for fish type per paragraph 3.2.2 Inadequate Water Depth.

It can be shown that concrete culvert characteristics with full flow, when the lower 20% of the diameter is filled with the streambed substrate, are modified as follows:

Area reduced by 14.5%
Hydraulic radius reduced by 11%
Average roughness coefficient n
increased by 30%

These changes in parameters will reduce the culvert capacity by about 39%. Therefore the selection of the culvert size as presented in the following text will require a correction. This correction is achieved by increasing the design discharge (full pipe flow only) by 63% before starting the design procedure indicated in 3.4.3.11 Outlet Control Nomographs.

For low flow design, as in the case of the 2 year flood, the culvert will flow partially full and the discharge depth for runoff discharge can be computed taking into consideration the culvert section with fill material using Manning's formula. The hydraulic radius is accounted for by weighting the perimeter with the n's of the culvert and the substrate material as per the following equation.

$$n = \frac{(P_c n_c + P_s n_s^{1.5})^{2/3}}{(P_c + P_s)^{2/3}}$$

"Important"

Per the preceeding, culverts meeting the requirements prescribed herein should be designed for a maximum capacity equivalent to: 1.63 x the calculated design discharge.

3.4.3.2 Scope of Guidelines. The following text contains a brief discussion of the hydraulics of conventional culverts and charts for selecting a culvert size for a given set of conditions. Instructions for using the charts are provided. Some approximations are made in the hydraulic design procedure for simplicity. These approximations are discussed at appropriate points throughout the text.

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

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

It is possible by involved hydraulic computations to determine the probable type of flow under which a culvert will operate for a given set of conditions. The need for making these computations may be avoided, however, by computing headwater depths from the charts in this circular for both inlet control and outlet control and then using the higher value to indicate the type of control and to determine the headwater depth. This method of determining the type of control is accurate except for a few cases where the headwater is approximately the same for both types of control.

Both inlet control and outlet control types of flow are discussed briefly in the following paragraphs and procedures for the use of the charts are given.

3.4.3.4 Culverts Flowing With Inlet Control. Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater (HW) and the entrance geometry, including the barrel shape and cross-sectional area, and the type of inlet edge. Sketches of inlet-control flow for both unsubmerged and submerged projecting entrances are shown in sections A and B of Figure 3.4.7. Section C shows a mitered entrance flowing under a submerged condition with inlet control.

In inlet control the roughness and length of the culvert barrel and outlet conditions (including depth of tailwater) are not factors in determining culvert capacity. An increase in barrel slope reduces headwater to a small degree and any correction for slope can be neglected for conventional or commonly used culverts flowing with inlet control.

In all culvert design, headwater or depth of ponding at the entrance to a culvert is an important factor in culvert capacity. The headwater depth (or headwater HW) is the vertical distance from the culvert invert at the entrance to the energy line of the headwater pool (depth + velocity head). Because of the low velocities in most entrance pools and the difficulty in determining the velocity head for all flows, the water surface and the energy line at the entrance are assumed to be coincident, thus the headwater depths given by the inlet control charts in this circular can be higher than will occur in some installations. For the purposes of measuring headwater, the culvert invert at the entrance is the low point in the culvert opening at the beginning of the net cross-section of the culvert barrel. (Refer to paragraph 3.4.3.1).

Headwater-discharge relationships for the various types of circular and pipe-arch culverts flowing with inlet control are based on laboratory research with models and verified in some instances by prototype tests. This

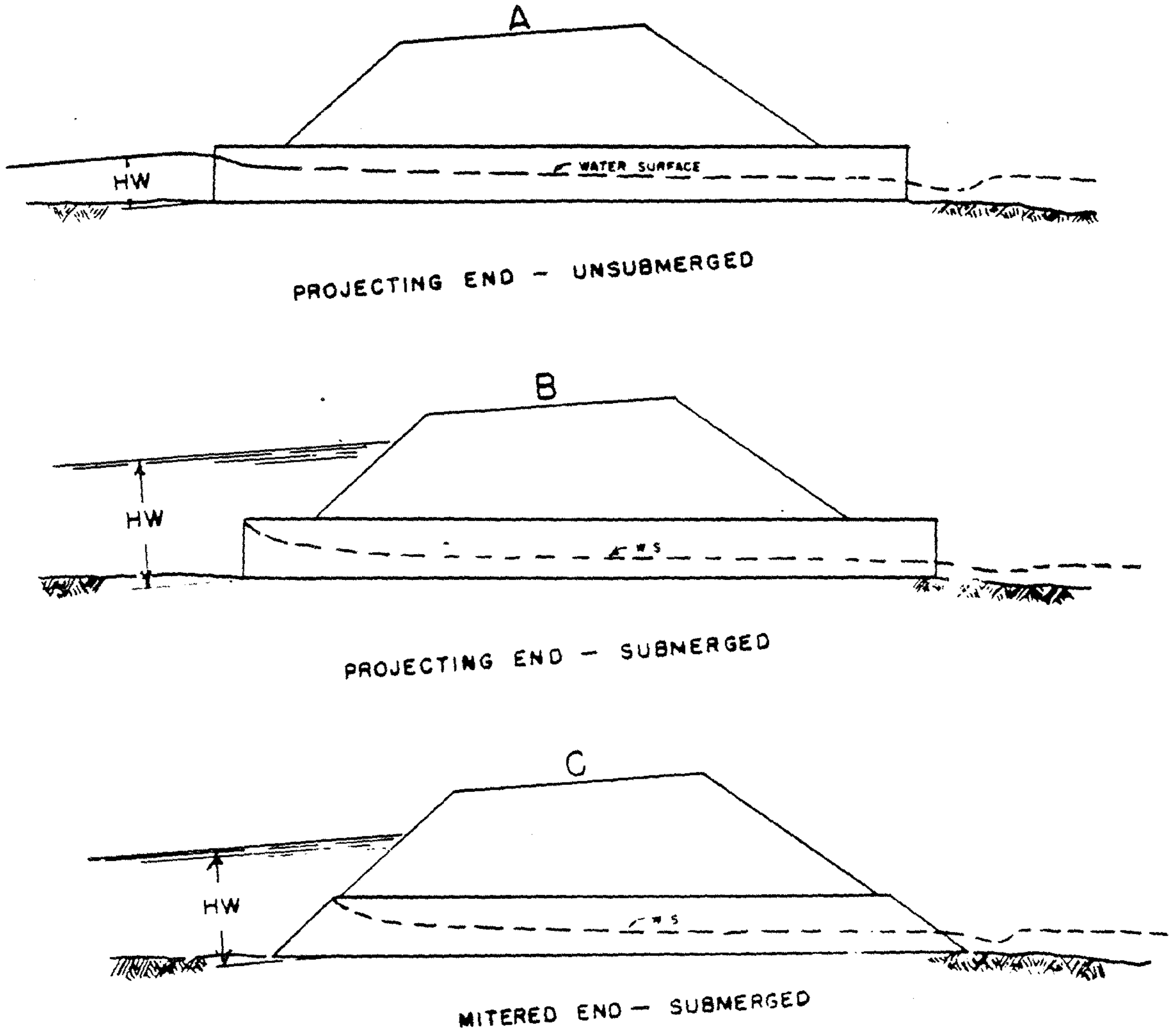


Figure 3.4.7
INLET CONTROL

research is reported in National Bureau of Standards Report No. 4444[1] entitled "Hydraulic Characteristics of Commonly Used Pipe Entrances", by John L. French and "Hydraulics of Conventional Highway Culverts", by H. G. Bossy[2]. Experimental data for box culverts with headwalls and wingwalls were obtained from an unpublished report of the U.S. Geological Survey.

These research data were analyzed and nomographs for determining culvert capacity for inlet control were developed by the Division of Hydraulic Research, Bureau of Public Roads. These nomographs, Charts 1 through 6, give headwater-discharge relationships for most conventional culverts flowing with inlet control through a range of headwater depths and discharges. Chart No. 7 is included to stress the importance of improving the inlets of culverts flowing with inlet control.

3.4.3.5 Culverts Flowing With Outlet Control. Culverts flowing with outlet control can flow with the culvert barrel full or part full for part of the barrel length or for all of it, (see Figure 3.4.8). If the entire cross section of the barrel is filled with water for the total length of the barrel, the culvert is said to be in full flow or flowing full, Sections A and B. Two other common types of outlet-control flow are shown in Sections C and D. The procedures given in this text provide methods for the accurate determination of headwater depth for the flow conditions shown in Sections

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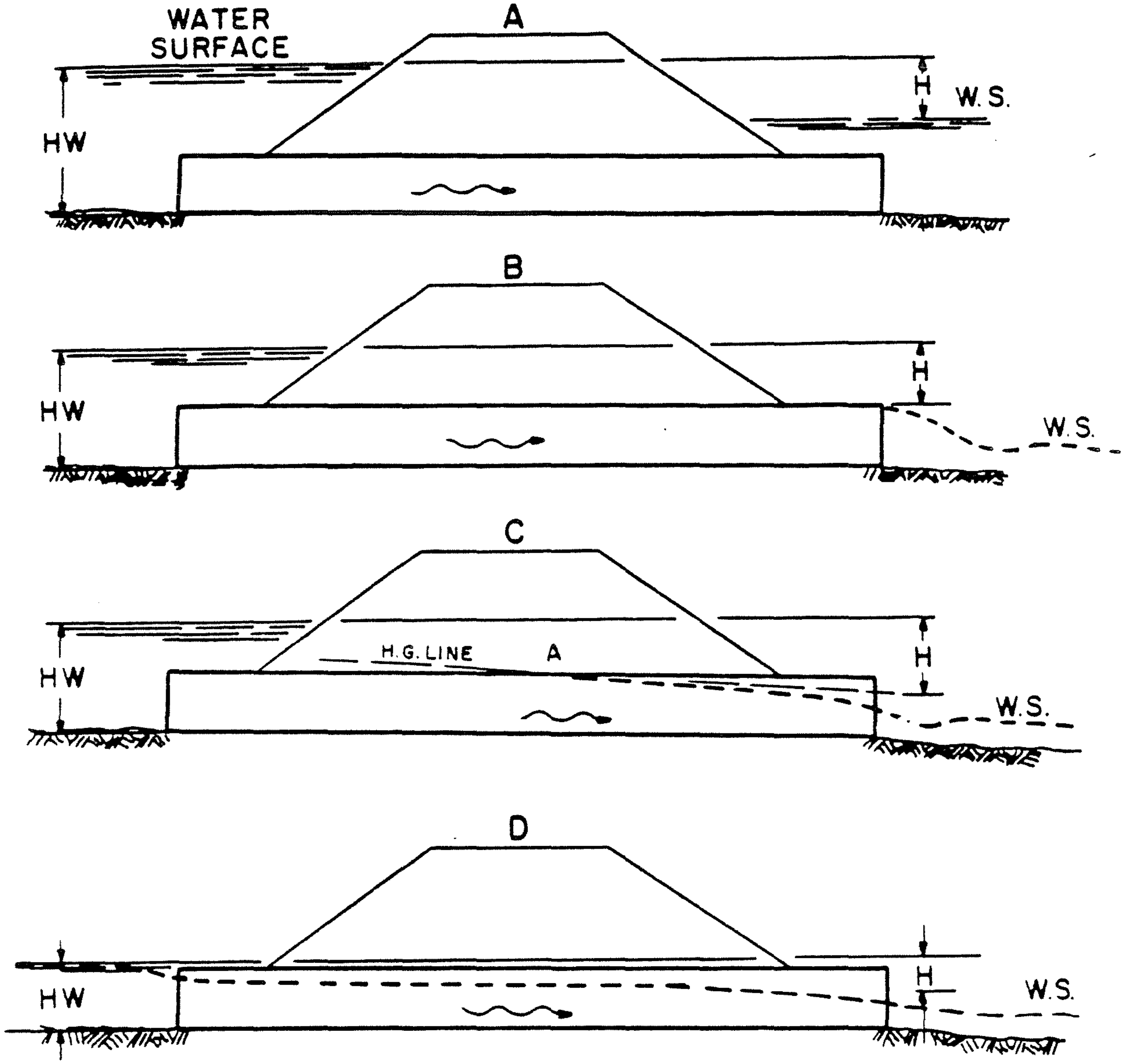
1/ Available from Division of Hydraulic Research, Bureau of Public Roads.

2/ Presented at the Tenth National Conference, Hydraulics Division, ASCE., August 1961. Available on loan from Division of Hydraulic Research, Bureau of Public Roads.



OUTLET CONTROL

Figure 3.4.8

A, B and C. The method given for the part full flow condition, Section D, gives a solution for headwater depth that decreases in accuracy as the headwater decreases.

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

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

The velocity head H_v equals $V^2/2g$, where V is the mean or average velocity in the culvert barrel. (The mean velocity is the discharge Q, in cfs, divided by the cross-sectional area A, in square feet, of the barrel.).

The entrance loss H_e depends upon the geometry of the inlet edge. This loss is expressed as a coeffcient k_e times the barrel velocity head or $H_e = k_e\ V^2/2g$. The entrance loss coefficients k_e for various types of entrances when the flow is in outlet control are given in Table 3.4.4.

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

$$H = \frac{29n^2L}{R^{1.33}} \frac{V^2}{2g}$$

Where: n = Manning's roughness coefficient

L = length of culvert barrel (ft)

V = mean velocity of flow in culvert barrel (ft/sec)

g = acceleration of gravity, 32.2 (ft/sec^2)

R = hydraulic radius or A/P (ft)

Where: A = area of flow for full cross-section (sq ft)

P = wetted perimeter (ft)

Substituting in equation 1 and simplifying, we get for <u>full flow</u>

$$H = \left(1 + k_e + \frac{29n^2L}{R^{1.33}}\right)\frac{V^2}{2g}$$

Table 3.4.4 Entrance Loss Coefficients

Coefficient k_e to apply to velocity head $V^2/2g$ for determination of head loss at entrance to a structure, such as a culvert or conduit, operting full or partly full with control at the outlet.

$$\text{Entrance head loss He} = k_e \frac{V^2}{2g}$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius - 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls	
Square-edge	0.5
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7

*Note: "End-section conforming to fill slope", made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These later sections can be designed using the information given in 3.4.3.8 Inlets and Culvert Capacity.

Figure 3.4.9 shows the terms of equation 2, the energy line, the hydraulic grade line and the headwater depth, HW. The energy line represents the total energy at any point along the culvert barrel. The hydraulic grade line, sometimes called the pressure line, is defined by the elevations to which water would rise in small vertical pipes attached to the culvert wall along its length. The energy line and the pressure line are parallel over the length of the barrel except in the immediate vicinity of the inlet where the flow contracts and re-expands. The difference in elevation between these two lines is the velocity head, $V^2/2g$.

The expression for H is derived by equating the total energy upstream from the culvert entrance to the energy just inside the culvert outlet with consideration of all the major losses in energy. By referring to Figure 3.4.9 and using the culvert invert at the outlet as a datum, we get:

$$d_1 + \frac{V_1}{2g} + LS_0 = d_2 + H_v + H_e + H_f$$

Where: d_1 and d_2 = depths of flow as shown in Fig. 3.4.9

$\frac{V1}{2g}$ = velocity hed in entrance pool

LS_0 = length of culvert times barrel slope

Then:
$$d_1 + \frac{V_1^2}{2g} + LS_0 - d_2 = H_v + H_e + H_f$$

And:
$$H = d_1 + \frac{V_1^2}{2g} + LS_0 - d_2 = H_v + H_e + H_f$$

From the development of this energy equation and Figure 3.4.9, head H is the difference between the elevations of the hydraulic grade line at he outlet and the energy line at the inlet. Since the velocity head in the entrance pool is usually small under ponded conditions, the water surface or headwater pool elevation can be assumed to equal the elevation of the energy line. Thus headwater elevations and headwater depths, as computed by the methods given in this text, for outlet control, can be higher than might occur in some installations. Headwater depth is the vertical distance from the culvert invert at the entrance to the wate surface, assuming the water surface (hydraulic grade line) and the energy line to be coincident, $d_1 + \frac{V_1^2}{2g}$ in Figure 3.4.9.

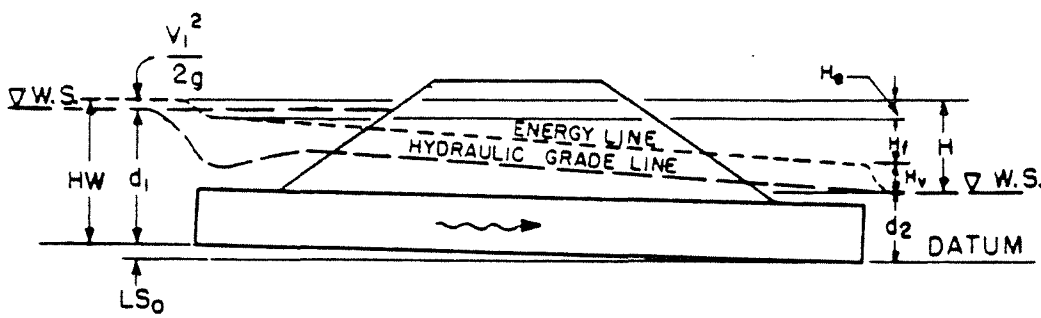


Figure. 3.4.9
CULVERT HYDRAULICS DIAGRAM

Equation 2 can be solved for H readily by the use of the full-flow nomographs, Charts 8 through 14. Each nomograph is drawn for a particular barrel shape and material and a single value of n as noted on the respective charts. these nomographs can be used for other values of n by modifying the culvert length for the use of the full-flow monographs as directed in 3.4.3.11 Outlet Control Nomographs.

In culvert design, the depth of headwater HW or the elevation of the ponded water surface is usually desired. Finding the value of H from the nomographs or by equation 2 is only part of the solution for this headwater depth or elevation. In this case of Figure 3.4.8 Section A or Figure 3.4.9 where the outlet is totally submerged, the headwater pool elevation (assumed to be the same elevation as the energy line) is found by adding H to the elevation of the tailwater. The headwater depth is the difference in elevations of the pool surface and the culvert invert at the entrance.

When the tailwater is below the crown of the culvert, the submerged condition discussed above no longer exists and the determination of headwater is somewhat more difficult. In discussing outlet-control flow for this condition, tailwater will be assumed to be so low that it has no effect on the culvert flow. (The effect of tailwater will be discussed later.) The common types of flow for the low tailwater condition are shown in Sections B, C and D of Figure 3.4.8. Each of these flow conditions are dependent on the amount of discharge and the shape of the culvert cross-section. Each condition will be discussed separately.

Full flow at the outlet, Section B of Figure 3.4.8 will occur only with the higher rates of discharge. Charts 15 through 20 are provided to aid in determining this full flow condition. The curves shown on these charts give the depth of flow at the outlet control. This depth is called critical depth d_c. When the discharge is sufficient to give a critical depth equal to the crown of the culvert barrel, full flow exists at the outlet as in Section B of Figure 3.4.8. The hydraulic grade line will pass through the crown of the culvert at the outlet for all discharges greater than the discharge causing critical depth to reach the crown of the culvert. Head H can be measured from the crown of the culvert in computing the water surface elevation of the headwater pool.

When critical depth falls below the crown of the culvert at the outlet, the water surface drops as shown in either Sections C or D, depending again on the discharge. To accurately determine headwater for these conditions, computations for locating a backwater curve are usually required. These backwater computations are tedious and time consuming an they should be avoided if possible. Fortunately, headwater for the flow condition shown in Section C can be solved by using the nomographs and the instructions given in this text.

For the condition shown in Section C, the culvert must flow full for part of its length. The hydraulic grade line for the portion of the length in full flow will pass through a point where the water breaks with the top of the culvert as represented by point A in Section C. Backwater computations show that the hydraulic grade line if extended as a straight line will cut the plane of the outlet cross section at a point above critical depth (water surface). This depth is at a height approximately equal to one haif the distance between critical depth and the crown of the culvert. The elevaton of this point can be used as an equivalent hydraulic grade line and H, as determined by equation 2 or the nomographs, can be added to this elevation to find the water surface elevation of tne headwater pool.

The full flow condition for part of the barrel length, Section C, will exist when the headwater depth HW, as computed from the above headwater pool elevation, is equal to or greater than the quantity:

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

Where V is the mean velocity for the net cross section of the barrel; K_e, the entrance loss coefficient; and D, the inside height of the culvert. If the headwater is less than the above value, a free water surface, Figure 2D will extend through the culvert barrel.

The part full flow condition of Section D must be solved by a backwwater computation if accurate headwater depths are desired. Details for making this computation are not given in this text. Instead the solution used is the same as that given for the flow condition of Section C, with the reservation that headwater depths become less accurate as the discharge for a particular culvert decreases. Generally, for design purposes, this method is satisfactory for headwater depths above 0.75D, where D is the heignt of the culvert barrel. Culvert capacity charts found in Hydraulic Enginering Circular No. 10 give a more accurate and easy solution for this free surface flow condition.

Headwater depth HW can be expressed by a common equation for all outlet-control conditions, including all depths of tailwater. This is accomplished by designating the vertical dimension from the culvert invert at the outlet to the elevation from which H is measured as h_o. The headwater depth HW elevation is:

$$HW = H + h_o - LS_o$$

All the terms in the equation are in feet. H is comptued by equation 2 or found from the full-flow nomographs. L is the length of culvert in feet and S_o the barrel slope in feet per feet. The distance h_o is discussed in the following paragraphs for the various conditions of outlet-control flow. Headwater HW is the distance in feet from the invert of the culvert at the inlet to the water surface of the headwater pool.

When the elevation of the water surfce in the outlet channel is equal to or above the elevation of the top of the culvert opening at the outlet, Section A, h_o is equal to the tailwater depth. Tailwater depth TW is the distance in feet from the culvert invert at the outlet to the water surface in the outlet channel. The relationship of HW to the other terms in equation 3 is illustrated in Figure 3.4.10.

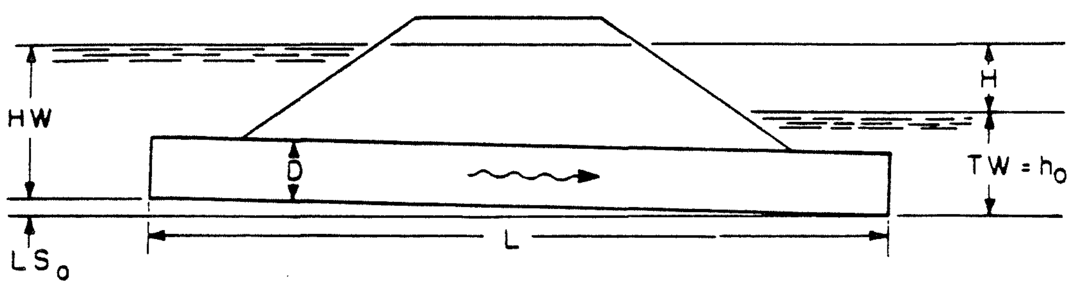


Figure 3.4.10

CULVERT OUTLET SUBMERGED

If the tailwater elevation is below the top of the culvert opening at the outlet, Sections B, C and D of Figure 3.4.8, h_o is more difficult to determine. The discharge, size and shape of culvert, and the TW must be considered. In these cases, h_o is the greater of two values (1) TW depth as defined above or (2) $(d_c + D) \div 2$. The latter dimension is the distance to the equivalent hydraulic grade line discussed previously. In this fraction d_c is the critical depth, as read from Charts 15 through 20 and D is the culvert height. The value of d_c can never exceed D, making the upper limit of this fraction equal to D. Where TW is the greater of these two values, critical depth is submerged sufficiently to make TW effective in increasing the headwater. The sketch in Figure 3.4.11 shows the terms of equation 3 for this low tailwater condition. Figure 3.4.11 is drawn similar to Section C of Figure 3.4.8, but a change in discharge can change the water surface profile to that of Section B or D.

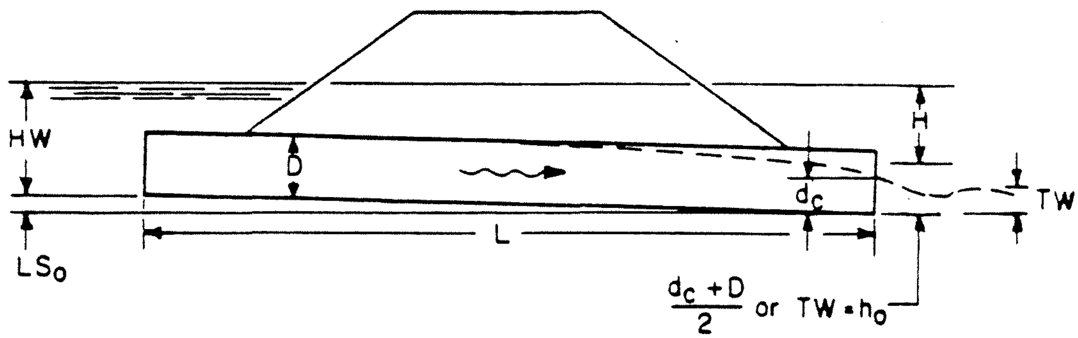


Figure 3.4.11

CULVERT OUTLET LOW TAILWATER

3.4.3.6 Computing Depth of Tailwater. In culverts flowing with outlet control, tailwater can be an important factor in computing both the headwater depth and the hydraulic capacity of a culvert. Thus, in many culvert designs, it becomes necessary to determine tailwater depth in the outlet channel.

Much engineering judgment and experience is needed to evaluate possible tailwater conditions during floods. As has been mentioned previously, a field inspection should be made to check on downstream controls and to determine water stages. Often times tailwater is controlled by a downstream obstruction or by water stages in another stream. Fortunately, most natural channels are wide compared to the culvert and the depth of water in the natural channel is considerably less than critical depth, thus the tailwater is ineffective and channel depth computations are not always warranted.

An approximation of the depth of flow in a natural stream (outlet channel) can be made by using Manning's formula if the channel is reasonably uniform in cross section, slope and roughness. Values of n for natural streams for use in Manning's have been presented in Tables 3.4.1 and 3.4.2. If the water surface in the outlet channel is established by downstream controls, other means must be found to determine the tailwater elevation. Sometimes this necessitates a study of the stage-discharge relationship of another stream into which the stream in question flows or the utilization of data on reservoir elevations if one of the dams is involved.

<u>3.4.3.7 Velocity of Culvert Flow</u>. A culvert, because of its hydraulic characteristics, increases the velocity of flow over that in the natural channel. High velocities are most damaging just downstream from the culvert - outlet and the erosion potential at this point is a feature to be considered in culvert design.

Energy dissipators for channel flow have been investigated in the laboratory and many have been constructed, especially in irrigation channels. Designs for highway use have been developed and constructed at culvert outlets. All energy dissipators add to the cost of a culvert, therefore, they should be used only to prevent or to correct a serious erosion problem (see Reference 5).

The judgment of engineers working in the particular area is required to determine the need for energy dissipators at culvert outlets. As an aid in evaluating this need, culvert outlet velocities should be computed. These computed velocities can be compared with outlet velocities of alternate culvert designs, existing culverts in the area, or the natural stream velocities. In many streams the maximum velocity in the main channel is considerably higher than the mean velocity for the whole channel cross-section. Culvert outlet velocities should be compared with maximum stream velocities in determining the need for channel protection. A change in will of culvert does not change outlet velocities appreciably in most cases.

Outlet velocities for culverts flowing with inlet control may be approximated by computing the mean velocity for the culvert cross section using Manning's formula:

$$V = \frac{1.49}{n} R^{2/3} S_o^{1/2}$$

Since the depth of flow is not known, the use of tables or charts is recommended in solving this equation[3/]. The outlet velocity as computed by this method will usually be high because the normal depth, assumed in using Manning's formula is seldom reached in the relatively short length of the average culvert. Also, the shape of the outlet channel, including aprons and wingwalls, have much to do with changing the velocity occuring at the end of the culvert barrel. Tailwater is not considered effective in reducing outlet velocities for most inlet control conditions.

In outlet control, the average outlet velocity will be the discharge divided by the cros-sectional area of flow at the outlet. This flow area can be either that corresponding to critical depth, tailwater depth (if below the crown of the culvert) or the net cross section of the culvert barrel.

3/ See References.

3.4.3.8 Inlets and Culvert Capacity. Inlet shape, edge geometry and skew of the entrance affects culvert capacity. Both the shape and edge geometry have been investigated by recent research but the effect of skew for various flow conditions has not been examined. Results show that the inlet edge geometry is particularly important to culvet performance in inlet-control flow. A comparison of several types of commonly used inlets can be made by referring to Charts 2 and 5. The type of inlet has some effect on capacity in outlet control but generally the edge geometry is less important than in inlet control.

As shown by the inlet control nomograph on Chart 5, the capacity of a thin edge projecting metal pipe can be increased by incorporating the thin edge in a headwall. The capacity of the same thin edged pipe can be further increased if the entrance is rounded, bevelled or tapered by the addition of an attachment or the building of these shapes into a headwall. A sketch on the nomograph, Chart 7 shows the dimensions of two possible bevels. Although nomographs have not been prepared for other barrel shapes, the capacity of box culverts can be increased at little cost by incorporating a bevel into the headwall. In computing headwater depths for outlet control, when the above bevel is used, k_e equals 0.25 for corrugated metal barrels and 0.2 for concrete barrels.

DRAINAGE STRUCTURE DESIGN DATA SHEET 1

Location: Township ______________ Range ______________

Section ______________ Meridian ______________

Project Feature: (Project access road, material site access road, etc.)

Station: ____________

Type of Water Course A B C -

User Fish Group (A & B Type Watercourse only) I II III IV

Drainage Area: ____________ acres

Q_2: ______________ cfs Q_{design}: ________________ cfs

Frequency of Q_{design}: ________________ years

Watercourse Area for Q_2: __________ ft^2 Gradient ____________ ft/ft

Watercourse depth of flow for Q_2: _____________ ft

Classify channel substate: ______________________________________

____

Channel configuration: Braided Meandering Straight

Other: (describe) ____________________________

Culvert Type: ____________________ Other ____________________

Size: ____________________ ____________________

Slope: ___________________ ft/ft Length: _________________ ft

V, Q_2: ___________________ ft/sec V, Q_{design}: ______________ ft/sec

HW/D, Q_2: ___________________ % HW/D Q_{design}: _____________ %

Attested to by:

_________________________________ _____________________________

Fisheries Biologist Design Engineer

A849/DATA-SH.1

PROJECT: ______________

DESIGNER: ______________

DATE: ______________

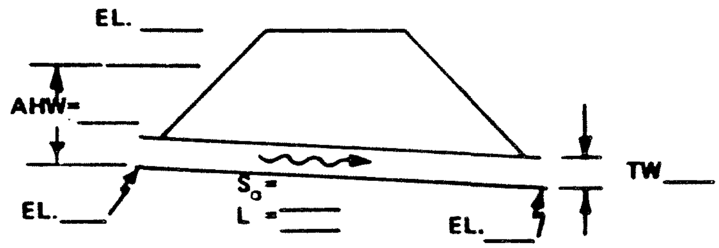
HYDROLOGIC AND CHANNEL INFORMATION

Q_1 = ________ TW_1 = ________

Q_2 = ________ TW_2 = ________

SKETCH

STATION: ______________



ALLOWABLE OUTLET VELOCITY = ________

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROL HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				

SUMMARY & RECOMMENDATIONS:

Drainage Structure Design Data Sheet 2

3.4.3.9 Procedure for Selectin of Culvert Size

Step 1: List design data. Drainage Structure Design Data Sheets 1 and 2 are provided for this.

a. Design discharge Q, in cfs., for required periods (i.e. Q_25 or Q_50 etc).

b. Approximate length L of culvert, in feet.

c. Slope of culvert. (If grade is given in percent, convert to slope in ft. per ft.).

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

e. Mean and maximum flood velocities in natural stream.

f. Type of culvert for first trial selection, including barrel material, barrel cross-sectional shape and entrance type.

Step 2: Determine the first trial size culvert.

Since the procedure given is one of trial and error, the intitial trial size can be determined in several ways:

a. By arbitrary selection.

b. By using an approximating equation such as $\frac{Q}{10} = A$ from which the trial culvert dimensions are determined.

c. By using inlet control nomographs (Charts 1-7) for the culvert type selected. If this method is used, an $\frac{HW}{D}$ must be assumed, say $\frac{ZH_W}{D} = 1.5$, and the given Q, a trial size is determined.

If any trial size is too large in dimension of limited height of embankment or availability of size, multiple culverts may be sued by dividing the discharge equally between the number of barrels used. Raising the embankment height or the use of pipe arch and box culverts with width greater than height should be considered. Final selection should be based on an economic analysis.

Step 3: Find headwater depth for trial size culvert.

a. Assuming INLET CONTROL

(1) Using the trial size from step 2, find the headwater depth HW by use of the appropriate inlet control nomograph (Charts 1-7). Tailwater TW conditions are to be neglected in this determination. HW in this case is found by multiplying $\frac{HW}{D}$ obtained from the nomographs by the height of culvert D.

(2) If HW is greater of less than allowable, try another trial size until HW is acceptable for inlet control before computing HW for outlet control.

b. ASSUMING OUTLET CONTROL

(1) Approximate the depth of tailwater TW, in feet, above the invert at the outlet for the design flood condition in the outlet channel. (See general discussion on tailwater, 3.4.3.6).

(2) For tailwater TW elevation equal to or greater than the top of the culvert at the outlet set h_o equal to TW and find HW by the following equation (equation 3).

$$HW = H + h_o - LS_o$$

where

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

H = head loss in feet as determined from the appropriate nomograph (Charts 8-14).

h = vertical distance in feet from culvert invert at outlet to the hydraulic grade line (In this case h_o equals TW, measured in feet above the culbert invert).

S_o = slope of barrel in ft./ft.

L = culvert length in ft.

(3) For tailwater TW elevations <u>less than</u> the top of the culvert at the outlet, find headwater HW by equation 3 as in b (2) above except that

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

where

d_c = critical depth in ft. (Charts 15 through 20 Note: d_c cannot exceed D

D = height of culvert opening in ft.

Note: Headwater depth determined in b (3) becomes increasingly less accurate as the headwater computed by this method falls below the value $D + (1 + K_e)\frac{V^2}{2g}$. (See discussions under 3.4.3.5 Culvert Flowing Full with Outlet Control.

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

d. If outlet control governs and the HW is higher than is acceptable, select a larger trial size and find HW as instructed under Step 3b. (Inlet control need not be checked, since the smaller size was satisfactory for this control as described under Step 3a).

Step 4: Try a culvert of another type or shape and determine size and HW by the above procedure.

Step 5: Compute outlet velocities for size and types to be considered in selection and determine need for channel protection.

a. If outlet control governs in Step 3c above, outlet velocity equals $\frac{Q}{Ao}$, where A≡/Zo is the cross-sectional area of flow in the culvert barrel at the outlet. If d_c or TW is less than the height of the culvert barrel use A_o corresponding to d_c or TW depth, whichever gives the greater area of flow. A_o should not exceed the total cross-sectional area A of the culvert barrel.

b. If inlet control governs in step 3c, outlet velocity can be assumed to equal mean velocity in open-channel flow in the barrel as computed by Manning's equation for the rate of flow, barrel size, roughness and slope of culvert selected.

Note: Charts and tables are helpful in computing outlet velocities. (See References)

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

3.4.3.10 Inlet-Control Nomographs

Charts 1 through 7

Instructions for Use

1. To determine Headwater (HW), given Q, and size and type of culvert.

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

   b. If $\frac{H^{ZW}}{D}$ scale marked (1) represents entrance type used, read $_{D}$n HZW o scale (1). If another of the three entrance types listed on the nomograph is used, extend the point of intersection in (a) horizontally to scale (2) or (3) and read $_{D}$. Compute HW by multiplying $\frac{ZHZW}{D}$ by D.

2. To determine discharge (Q) per barrel given HW, and size and type of culvert.

   a. Compute $\frac{HW}{D}$ for given conditions

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

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

   d. If $\frac{HW}{B}$ is read in (c) multiply by B (span of culvert) to find Q.

3. To determine culvert size, given Q, allowable HW and type of culvert.

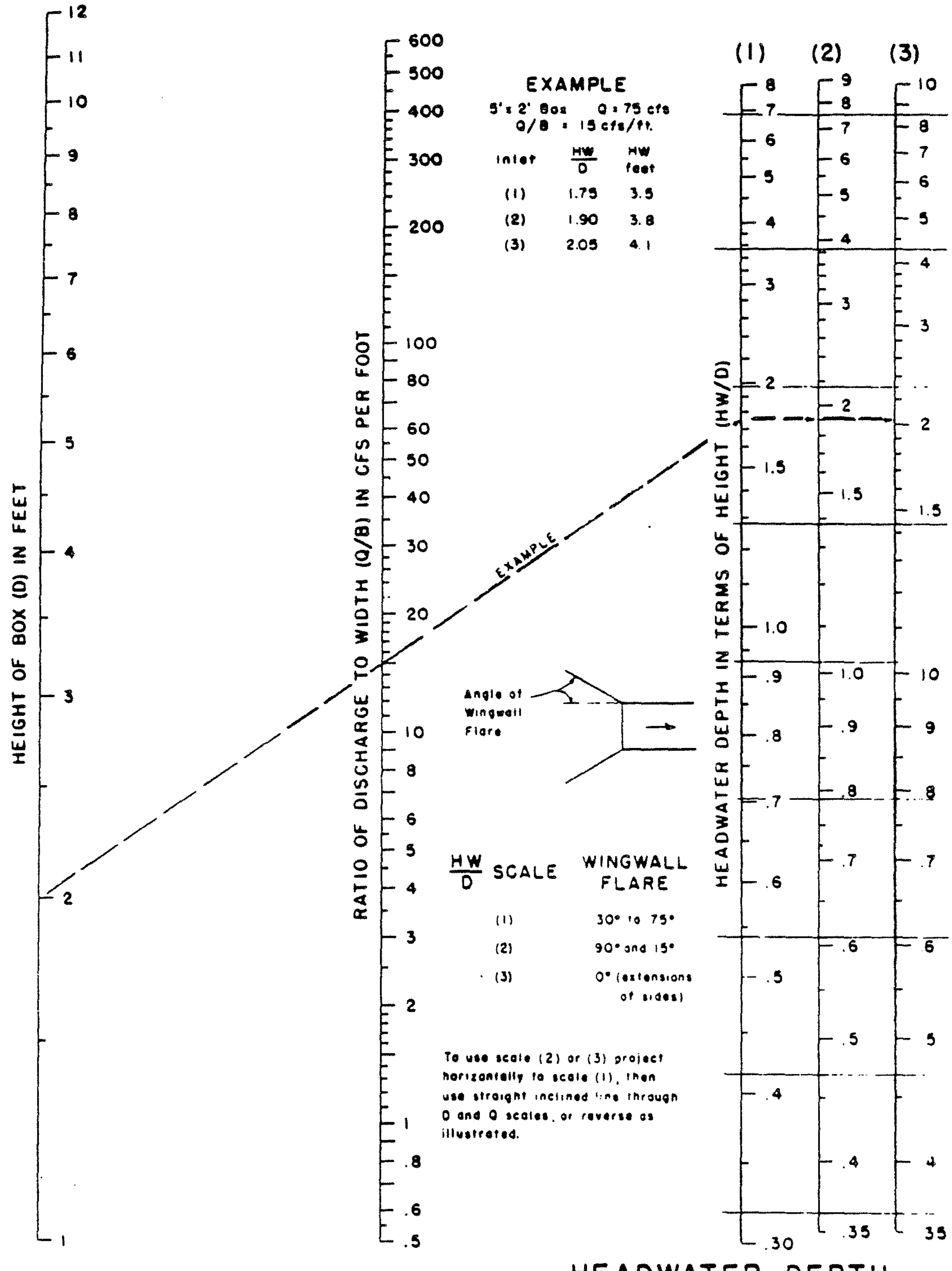
a. Using a trial size, compute $\frac{HW}{}$

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

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

d. If D is not originally assumed, repeat procedure with a new D.

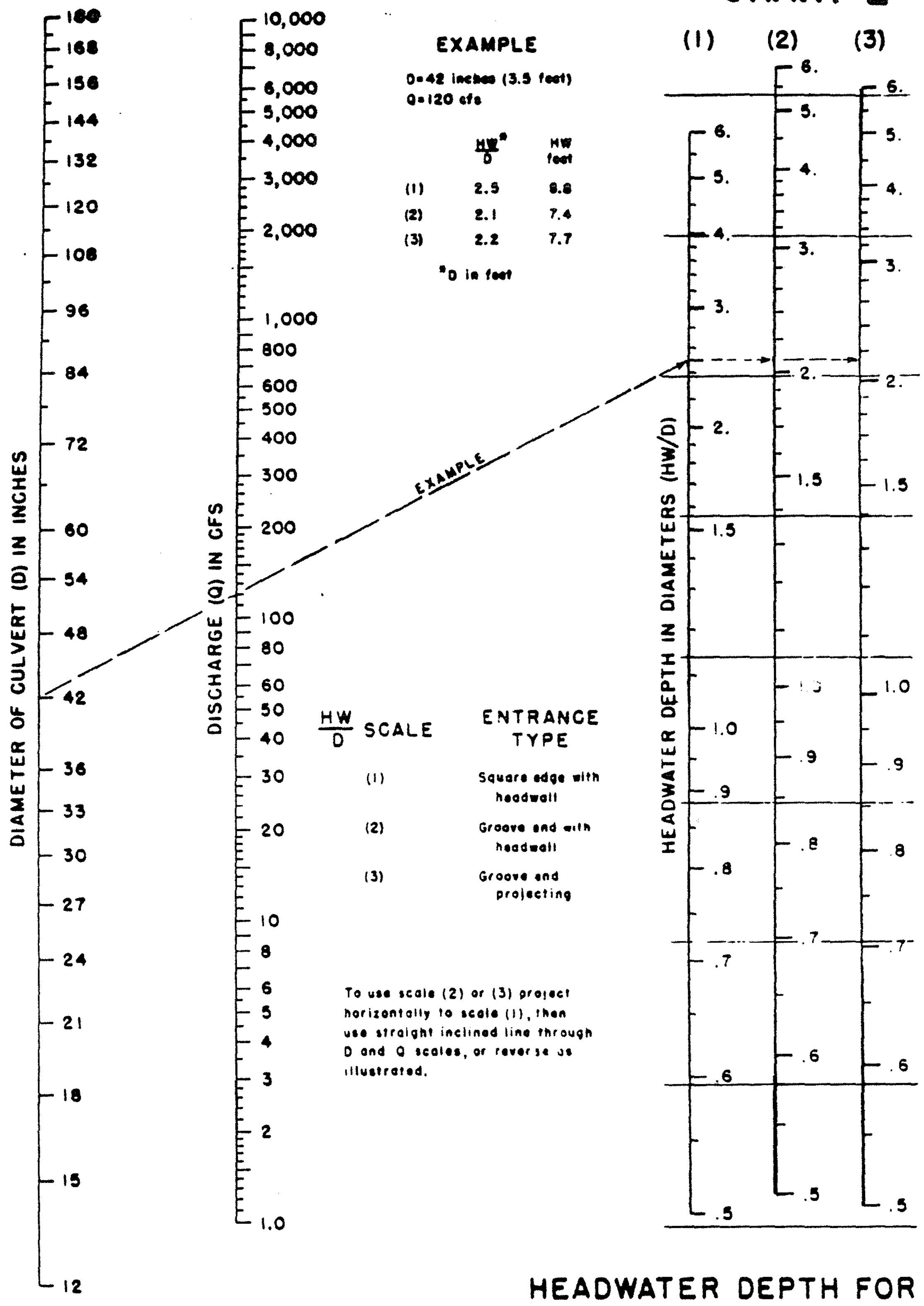
CHART I



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN 1963

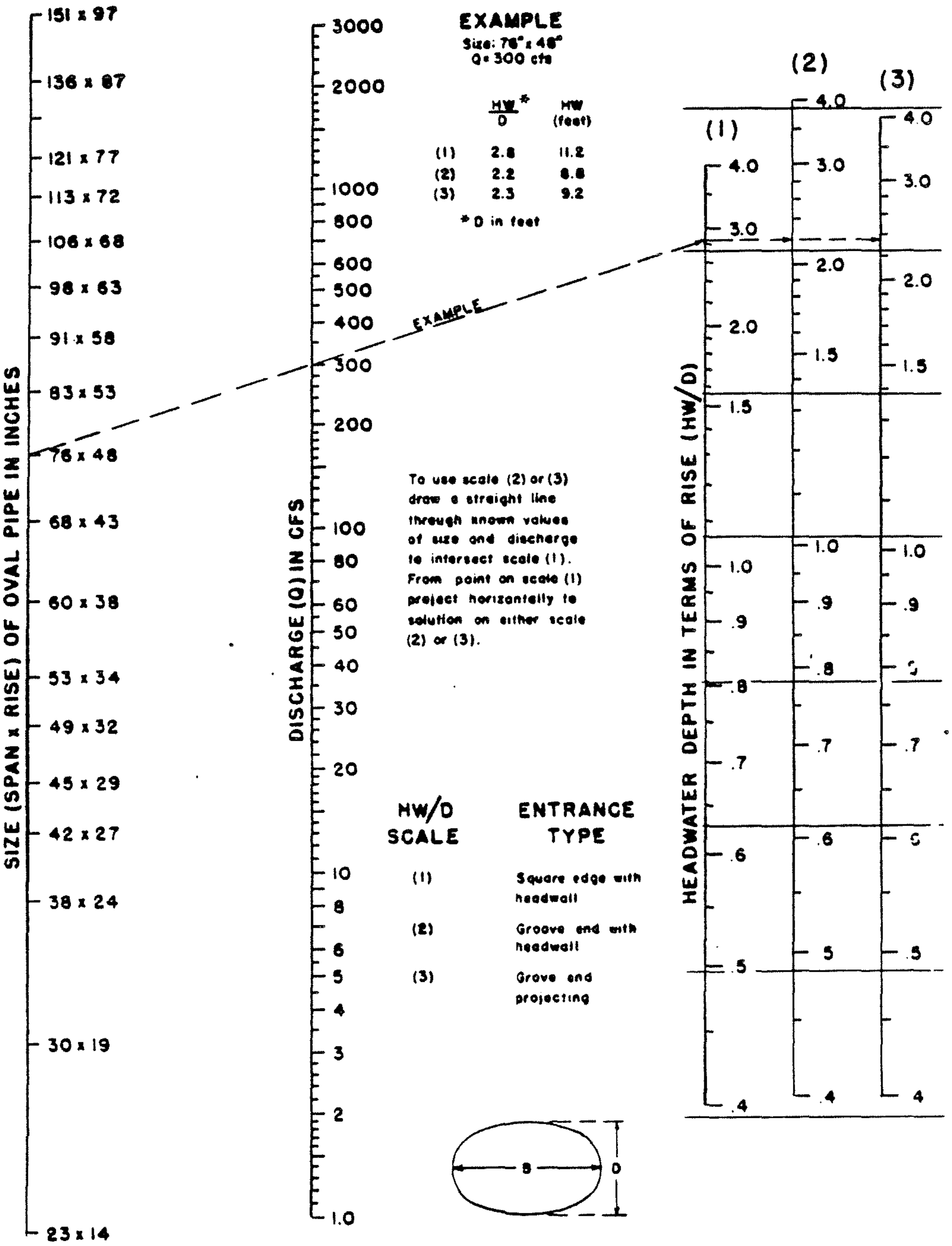
CHART 2



HEADWATER SCALES 2&3
REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

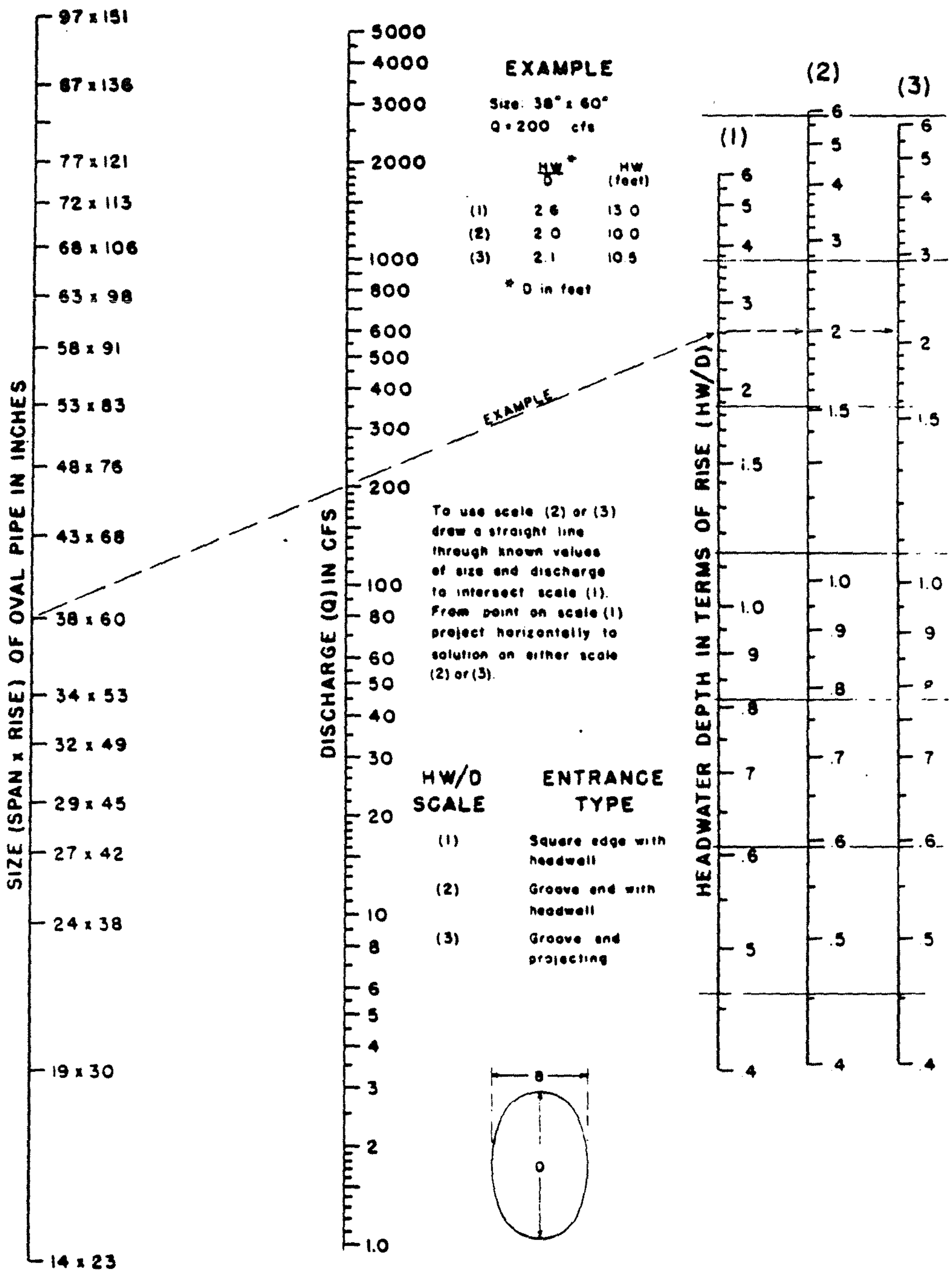
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

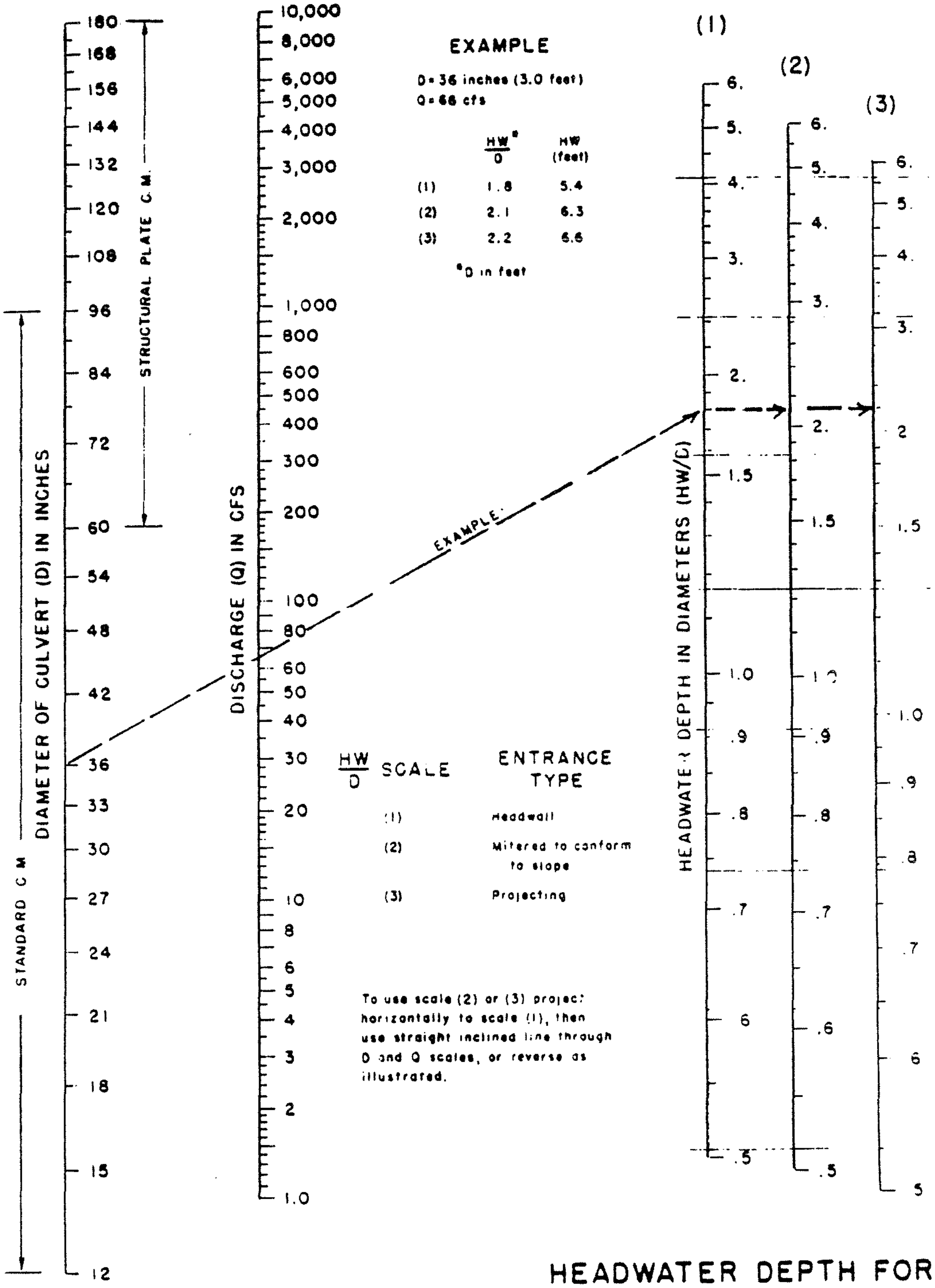
CHART 4



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

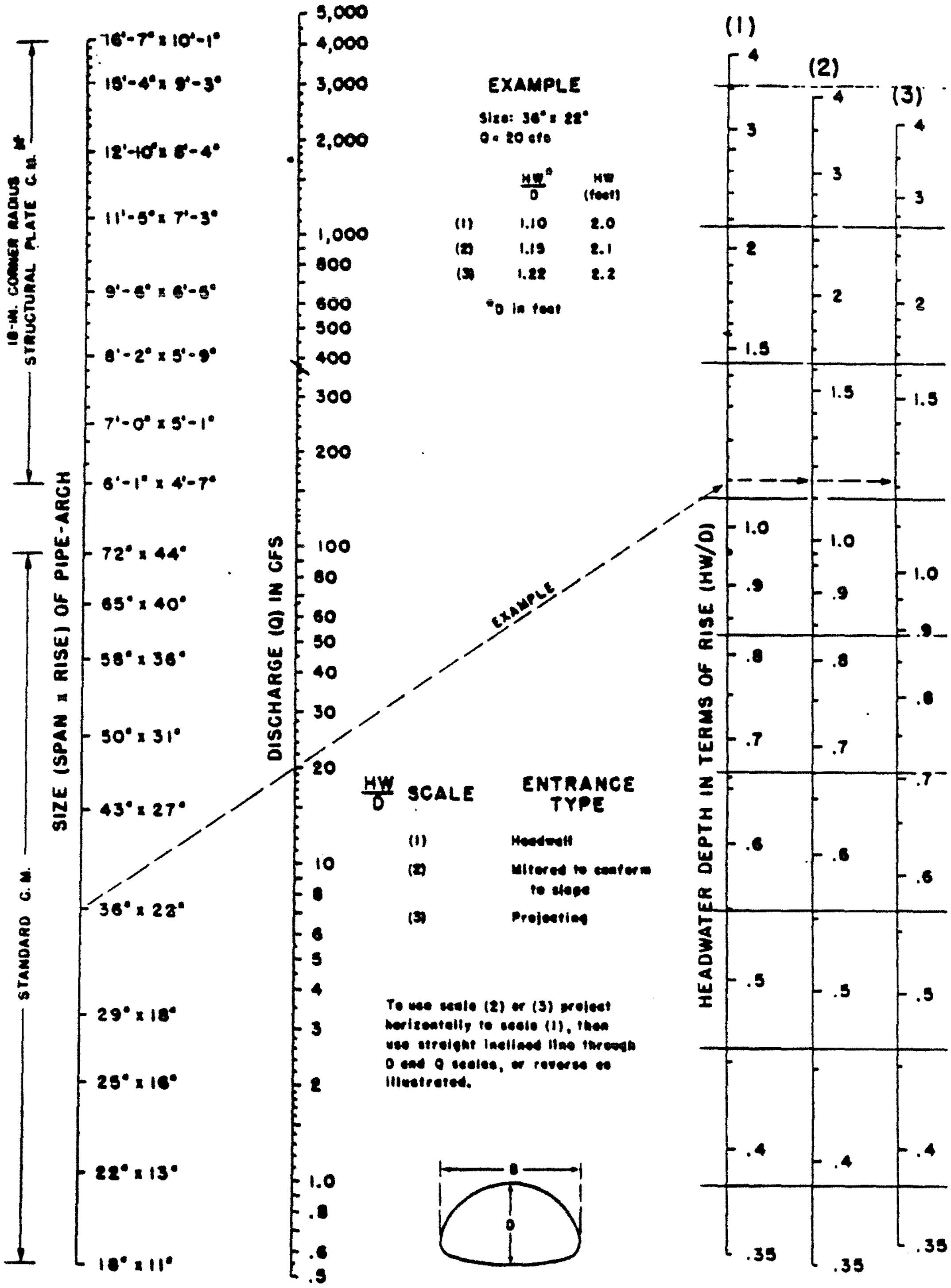
CHART 5



HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN 1963

CHART 6

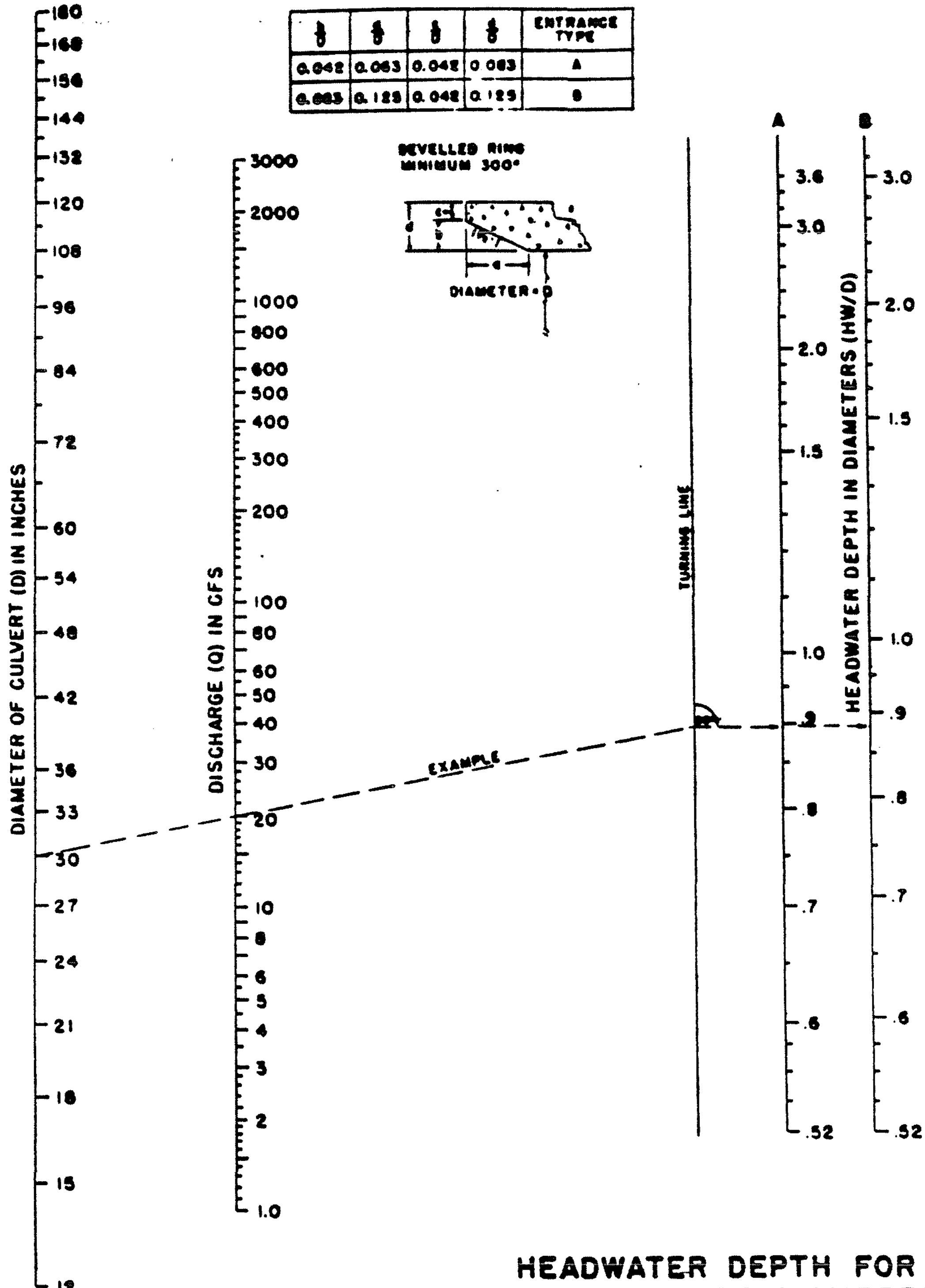


*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

BUREAU OF PUBLIC ROADS JAN. 1963

HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL

CHART 7



BUREAU OF PUBLIC ROADS
MARCH 1964

HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELLED RING INLET CONTROL

3.4.3.11 Outlet - Control Nomographs

Charts 8 through 14

Instruction for Use

Outlet control nomographs solve equation 2, paragraph 3.4.3.5, for head H when the culvert barrel flows full for its entire length. They are also used to determine head H for some part-full flow conditions with outlet control. These nomographs do not give a complete solution for finding headwater HW, since they only give H in equation 3, $HW = H + h_o - LS_o$. (See discussion for 3.4.3.5 Culverts Flowing with Outlet Control).

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

   a. Locate appropriate nomograph for type of culvert selected. Find k_e for entrance type in Table 3.4.4.

   b. Begin nomograph solution by locating starting point on length scale. To locate the proper starting point on the length scales follow instructions below:

      (1) If the n value of the nomograph corresponds to that of the culvert being used, select the length curve for the proper k_e and locate the starting point at the given culvert length. If a k_e curve is not shown for the selected k_e, see (2) below. If the n value for the culvert selected differs from that of the nomograph, see (3) below.

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

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

$$L_1 = L \frac{n_1}{n}$$ See instruction 2 for n values.

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

d. Pivot the straightedgeon this point on the turning line and connect given discharge rate. Read head in feet on the head (H) scale. For values beyond the limit of the chart scales, find H by solving equation 2.

2. Values of n for commonly used culvert materials.

<u>Concrete</u>

Pipe	Boxes
0.012	0.012

Corrugated Metal

	Small Corrugations (2 2/3" x 1/2")	Medium Corrugations (3" x 1")	large Corrugations (6" x 2")
Unpaved	0.024	0.027	Varies*
25% paved	0.021	0.023	0.026
Fully paved	0.012	0.012	0.012

*Variation in n with diameter shown on charts. The various n values have been incorporated into the nomographs and no adjustment for culvert length is required as instructed in 1b (3).

3. To use the box culvert nomograph, chart 8, for full-flow for other than square boxes.

   a. Compute cross-sectional area of the rectanglar box.

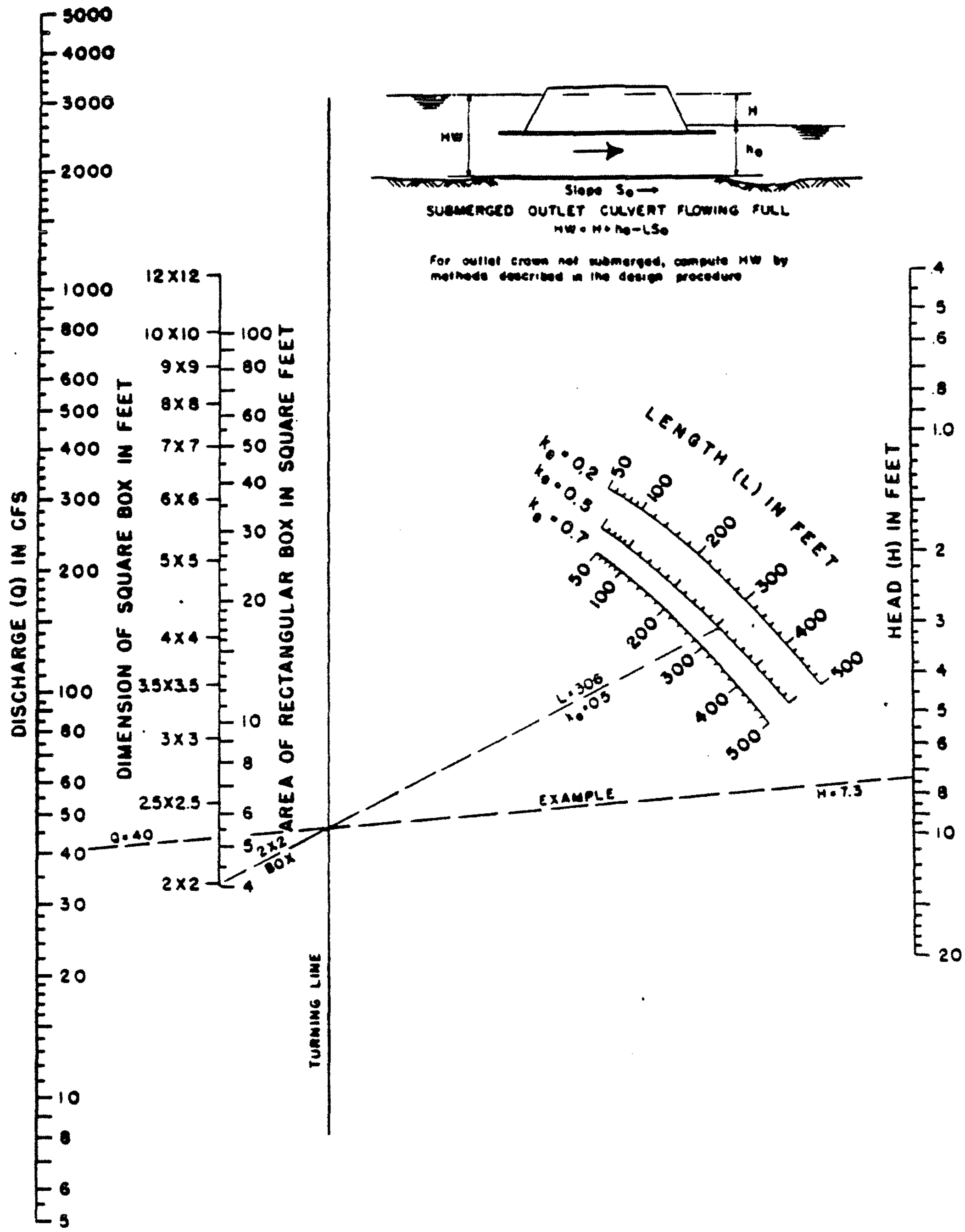
   b. Connect proper point (see instruction 1) on length scale to barrel area [4]/ and mark point on turning line.

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

---

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

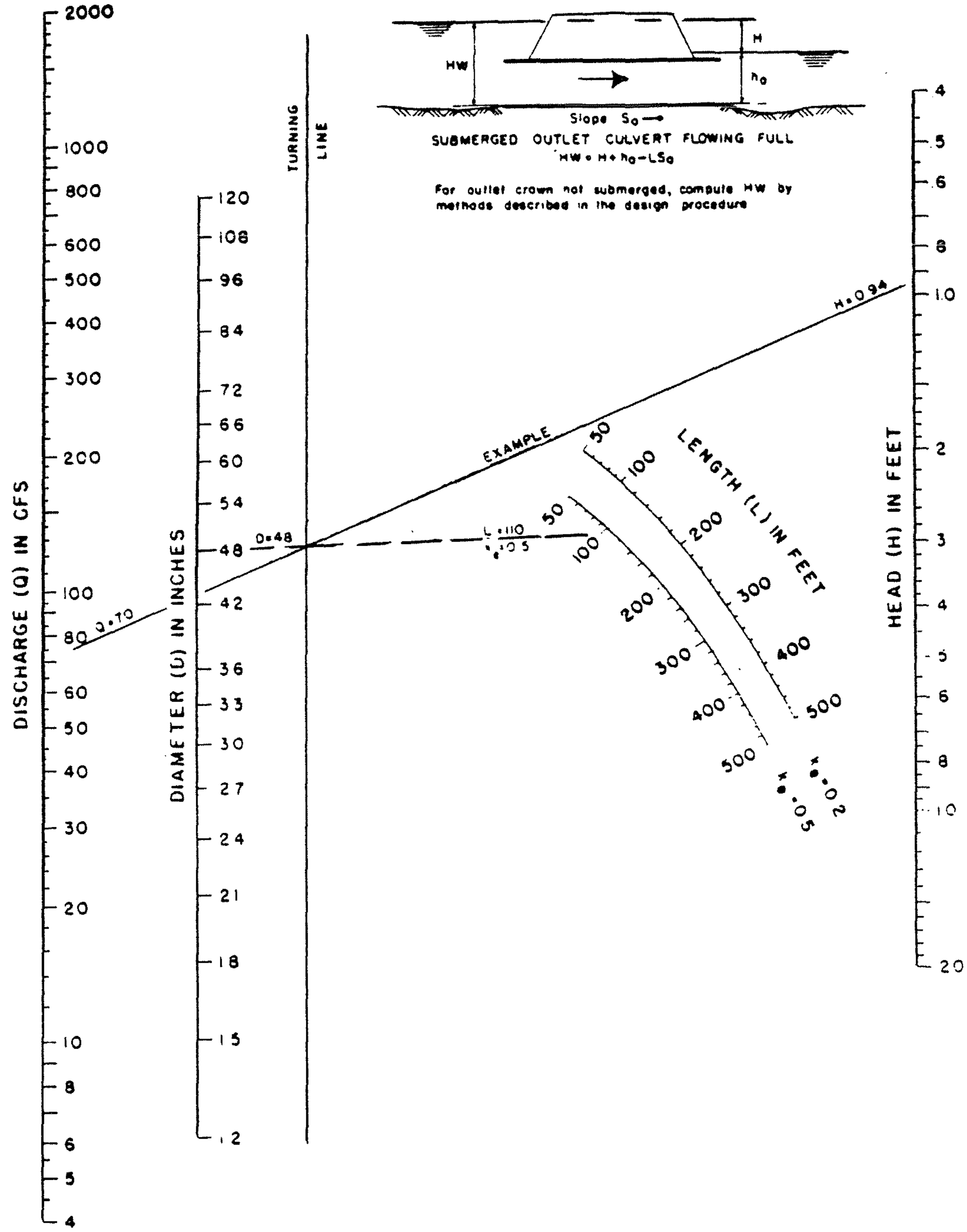
CHART 8



HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 9



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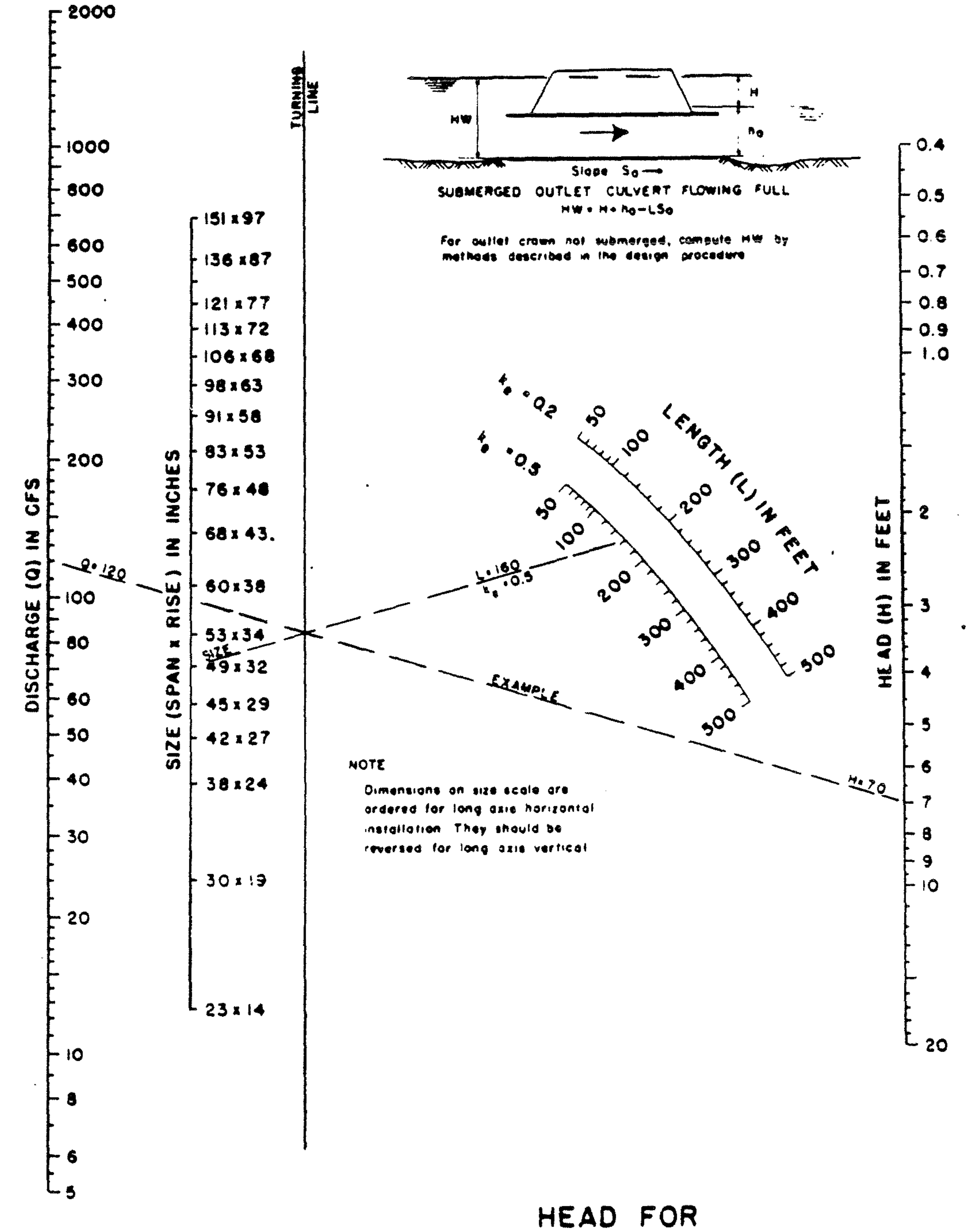
HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

CHART 10



HEAD FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL OR VERTICAL
FLOWING FULL
n = 0.012

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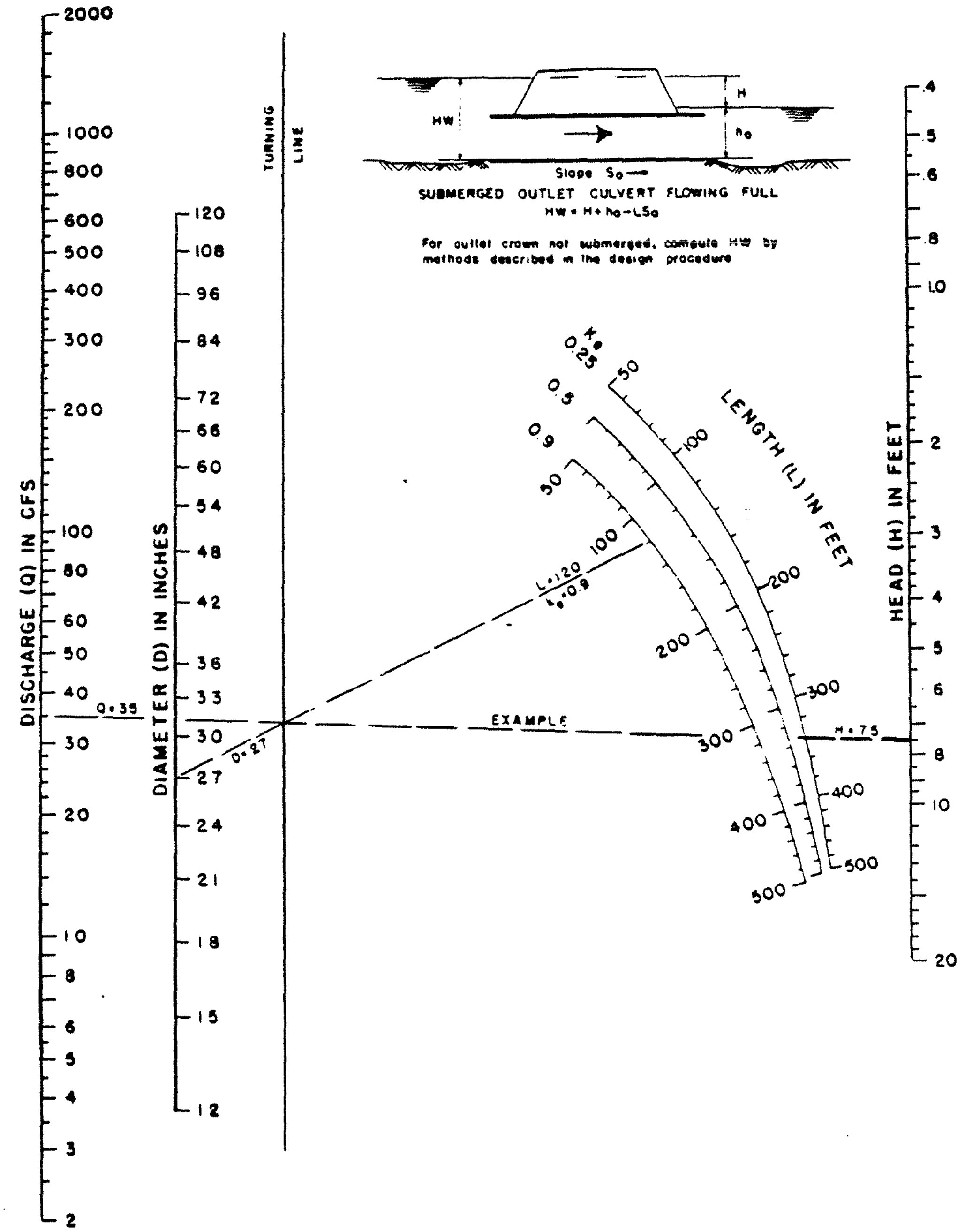
CHART 11



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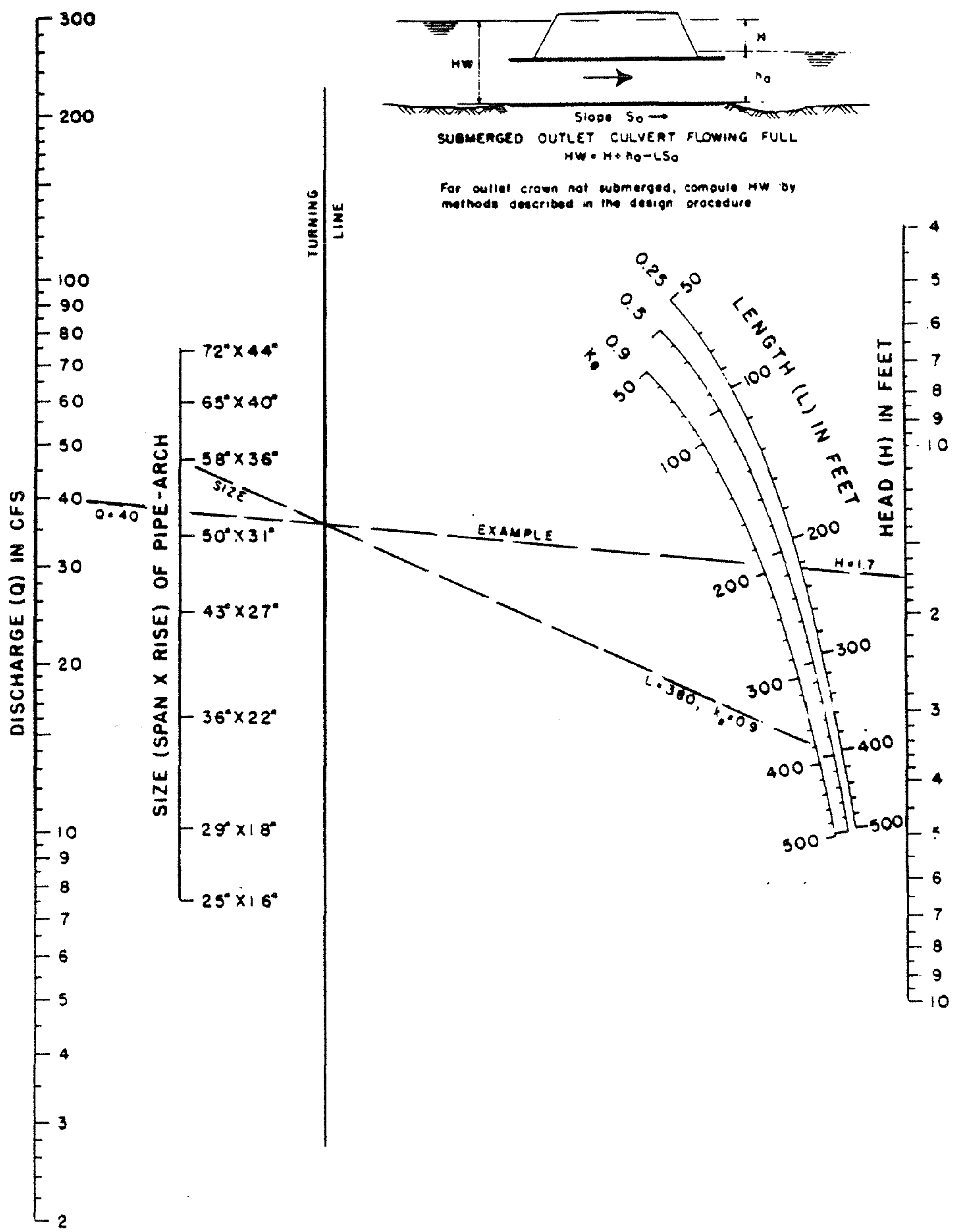
HEAD FOR STANDARD C. M. PIPE CULVERTS FLOWING FULL $n = 0.024$

CHART 12



HEAD FOR
STANDARD C. M. PIPE-ARCH CULVERTS
FLOWING FULL
n=0.024

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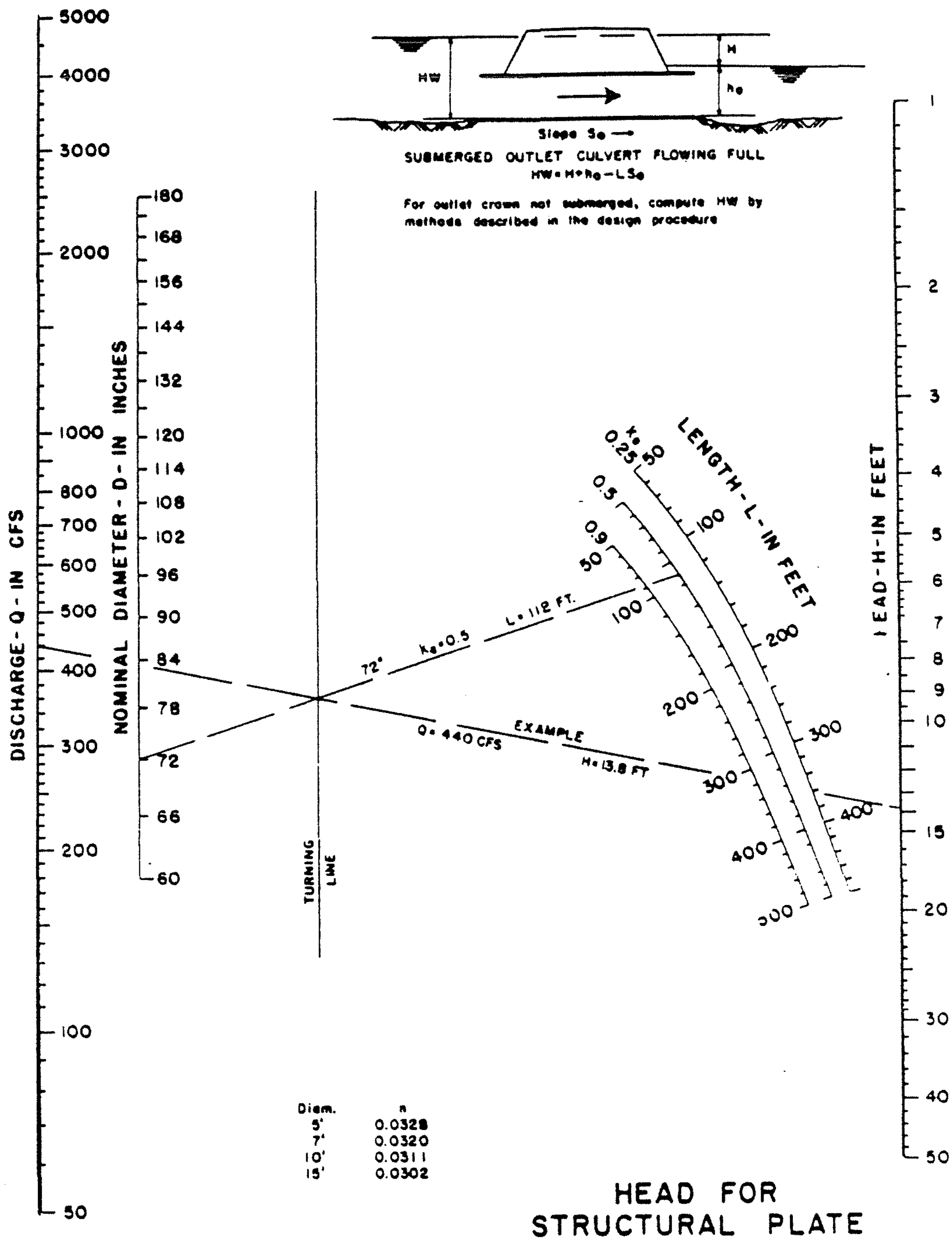
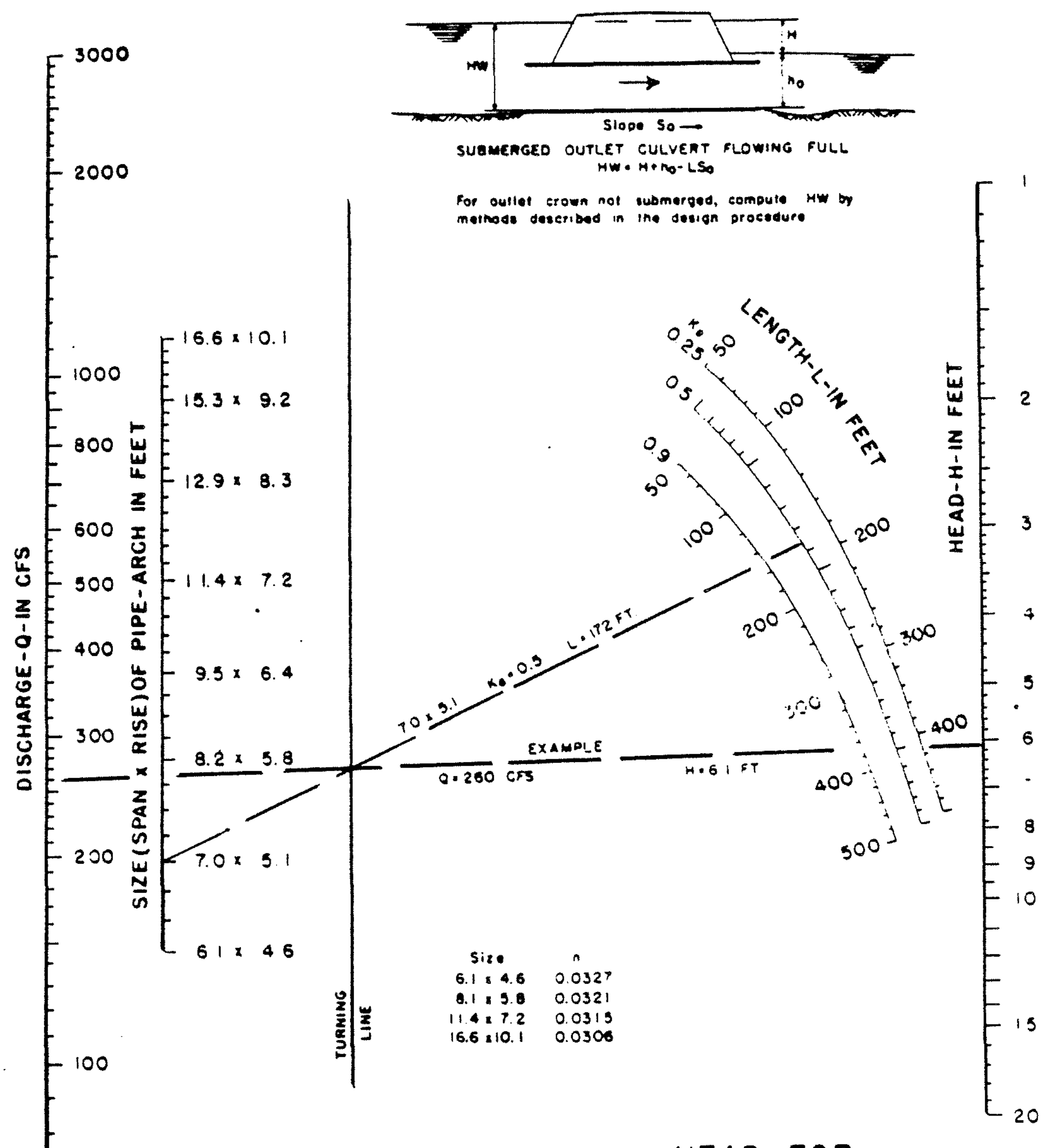


HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL
$n = 0.0328$ TO 0.0302

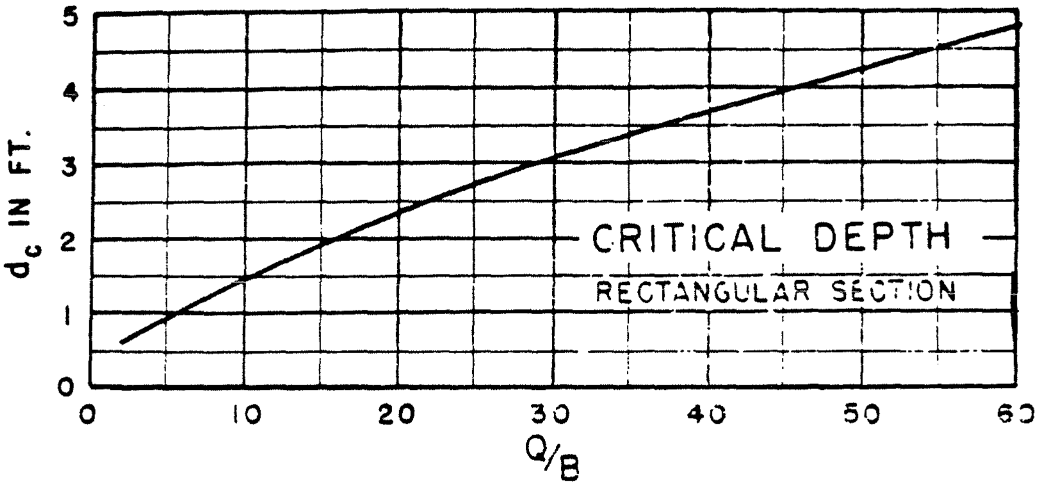
BUREAU OF PUBLIC ROADS JAN 1963

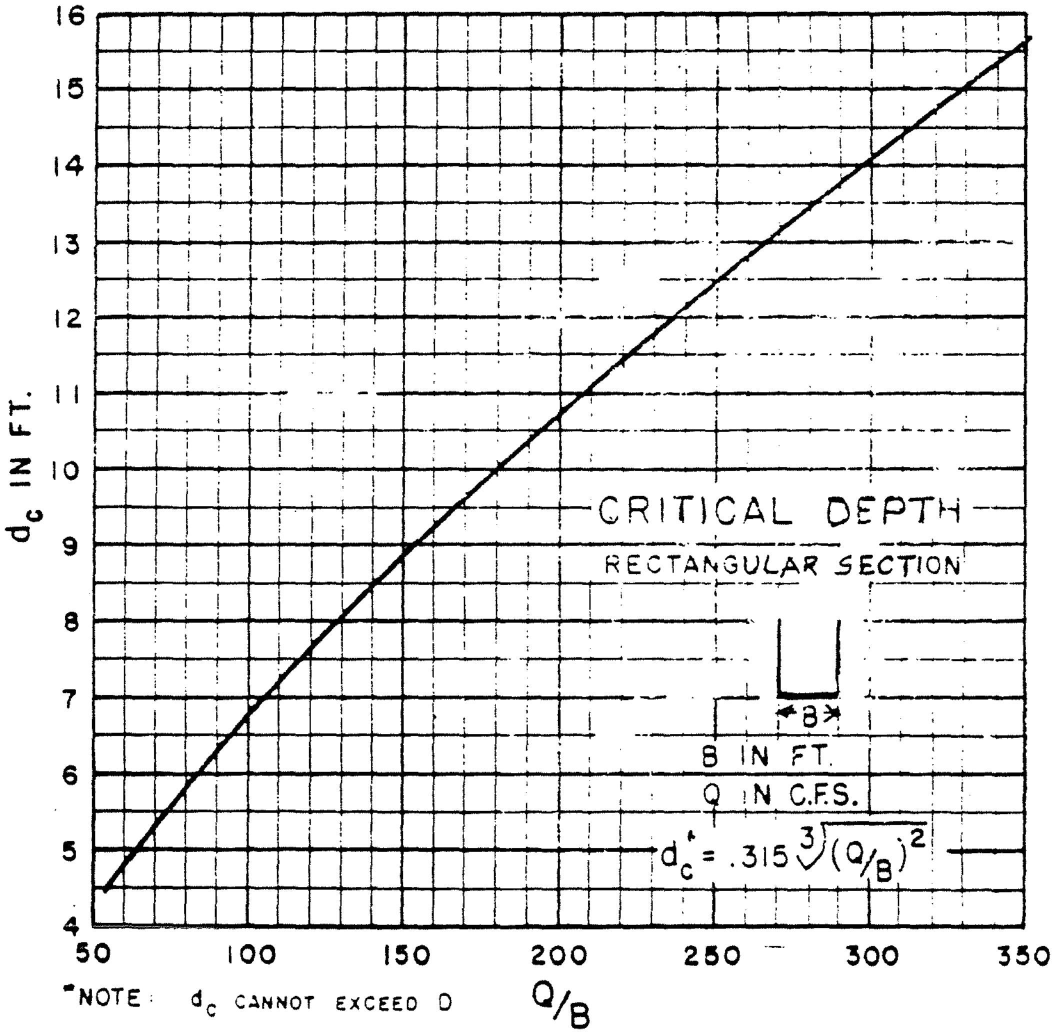


HEAD FOR
STRUCTURAL PLATE
CORRUGATED METAL
PIPE ARCH CULVERTS
18 IN. CORNER RADIUS
FLOWING FULL
n = 0.0327 TO 0.0306

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 15





*NOTE: d_c CANNOT EXCEED D

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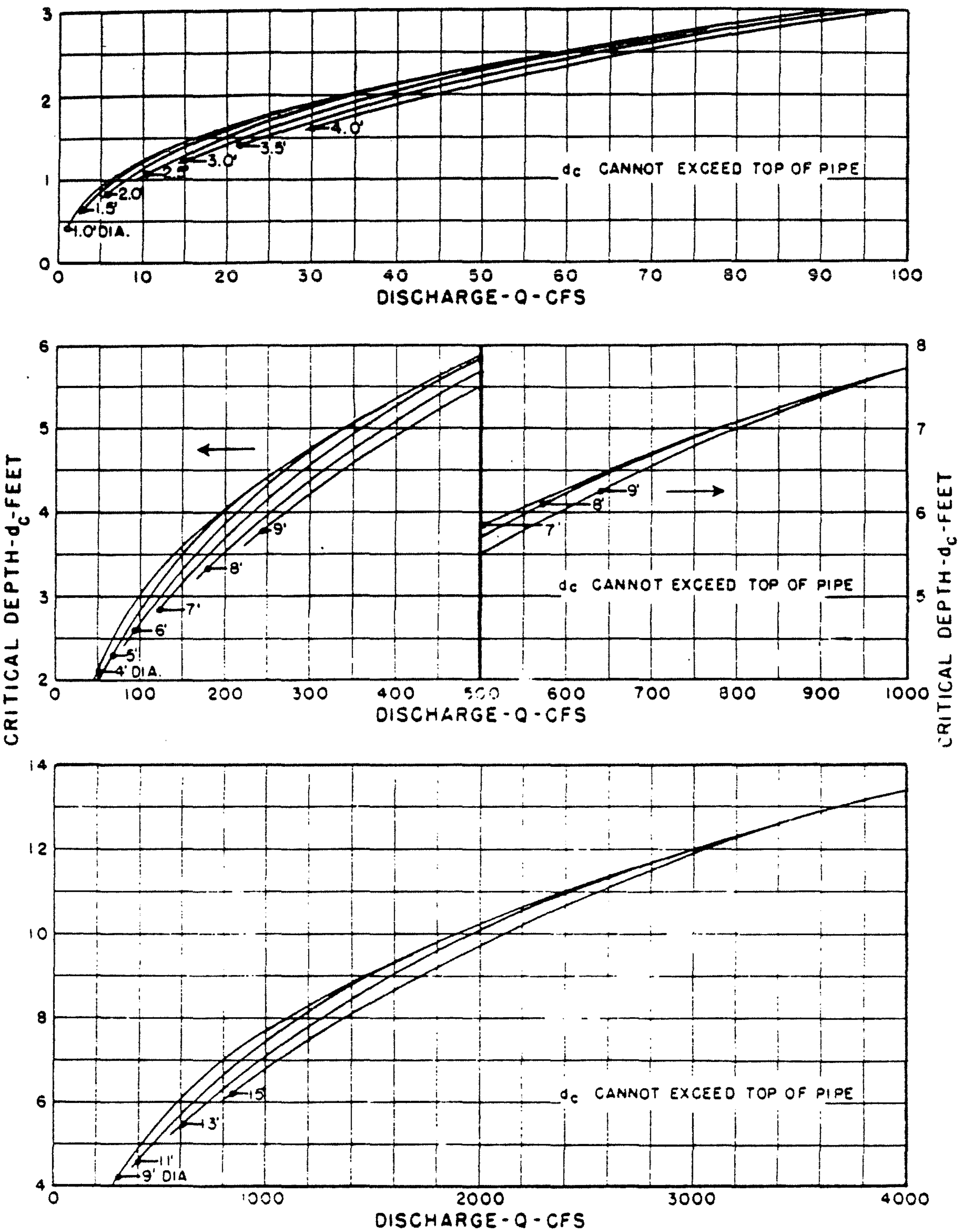
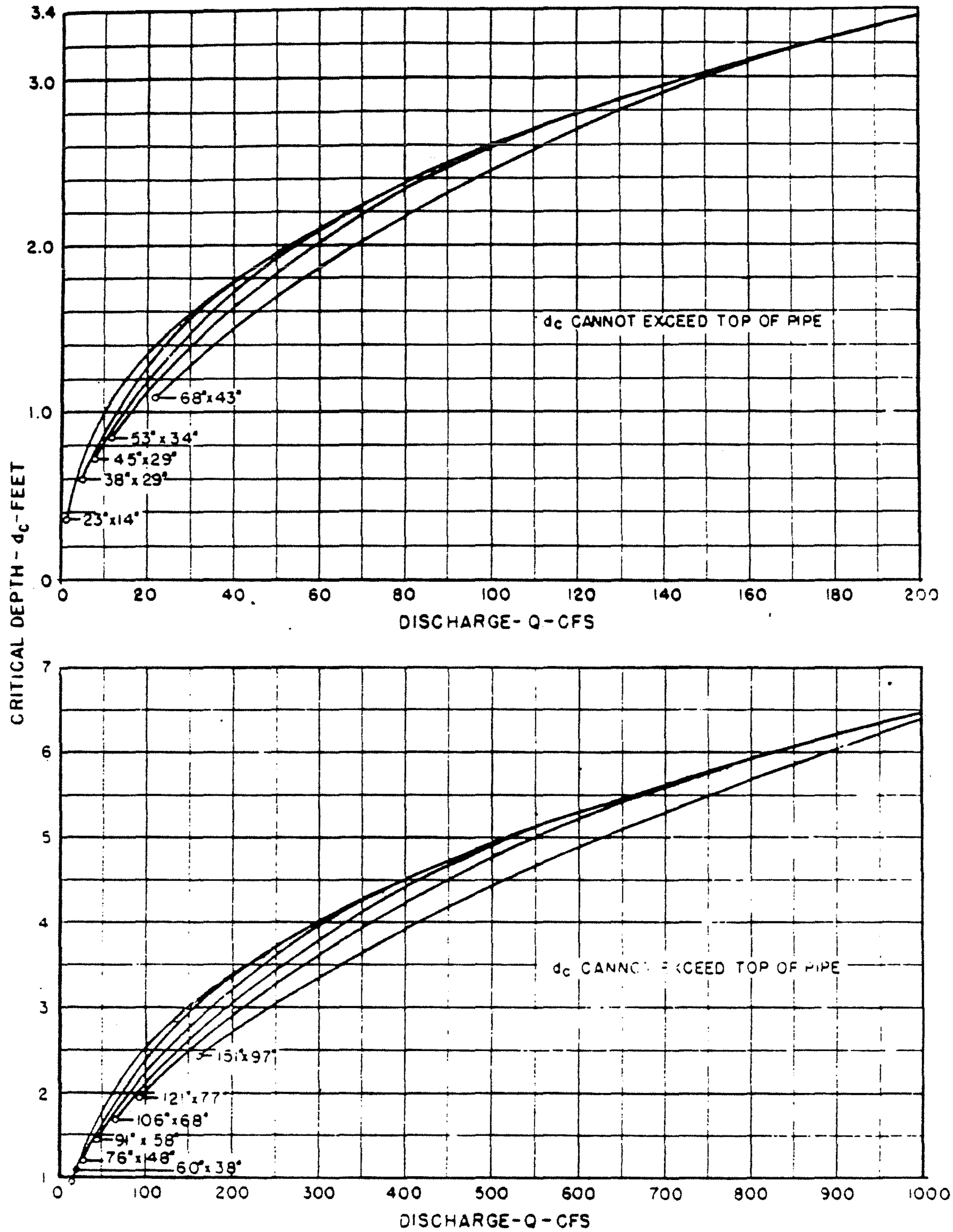


BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

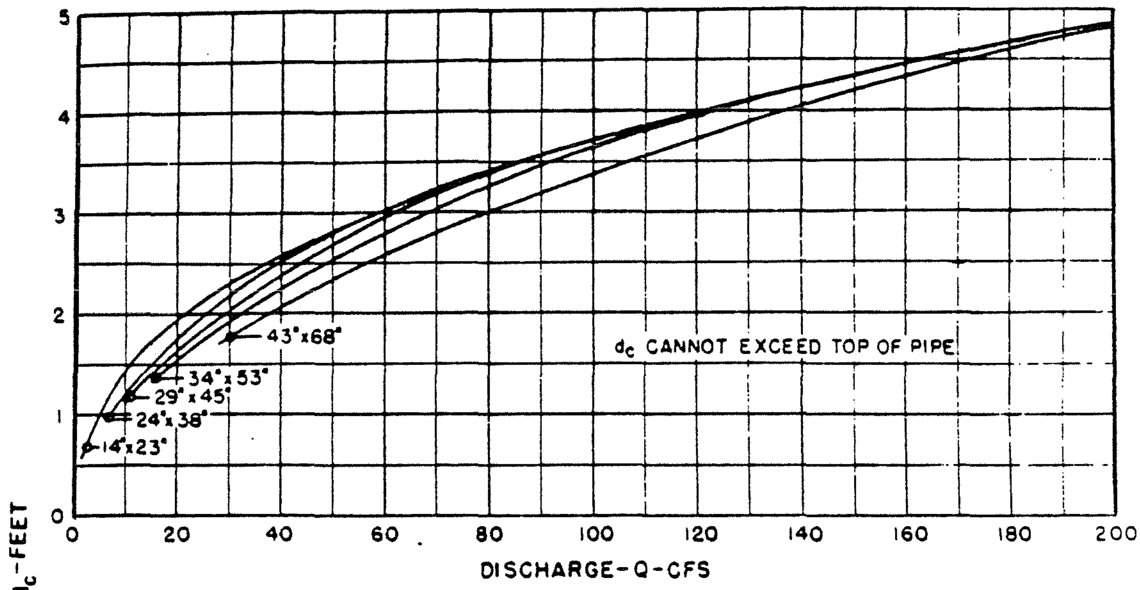
CHART 17

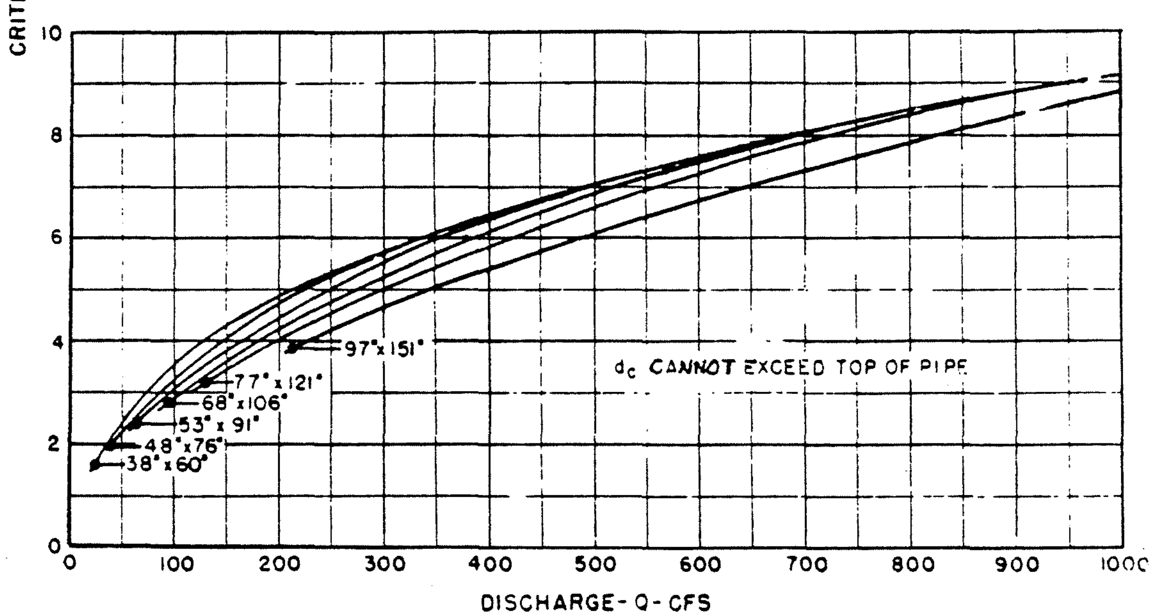


BUREAU OF PUBLIC ROADS

JAN. 1964

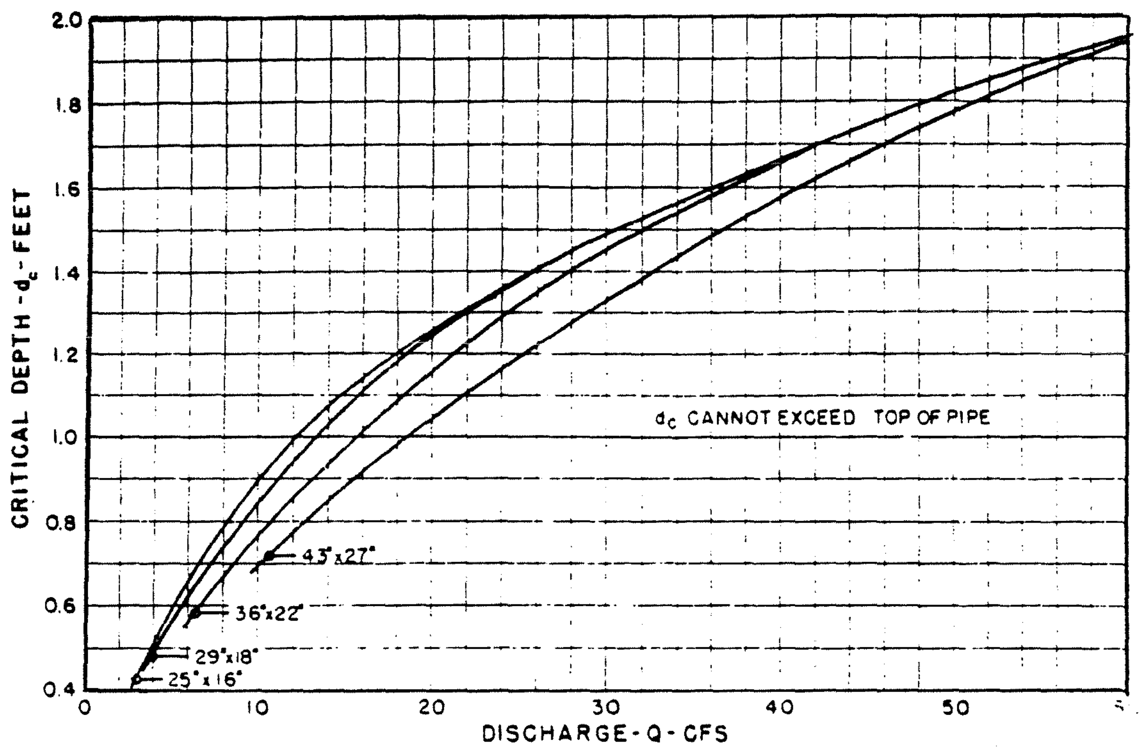
CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL

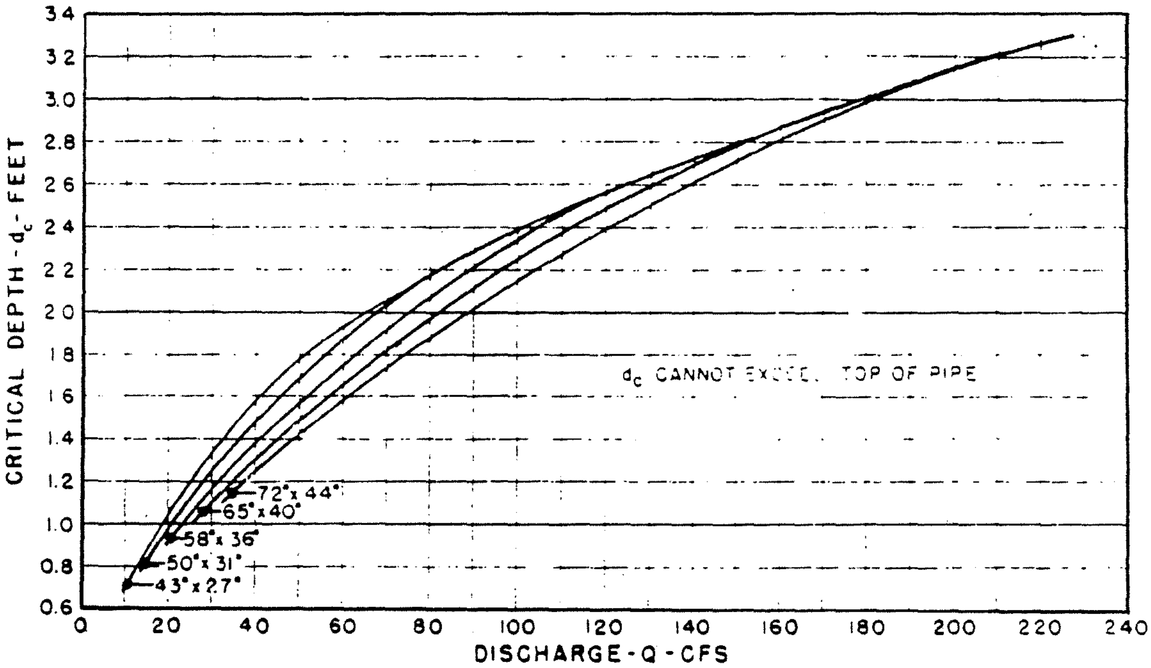




BUREAU OF PUBLIC ROADS
JAN. 1964

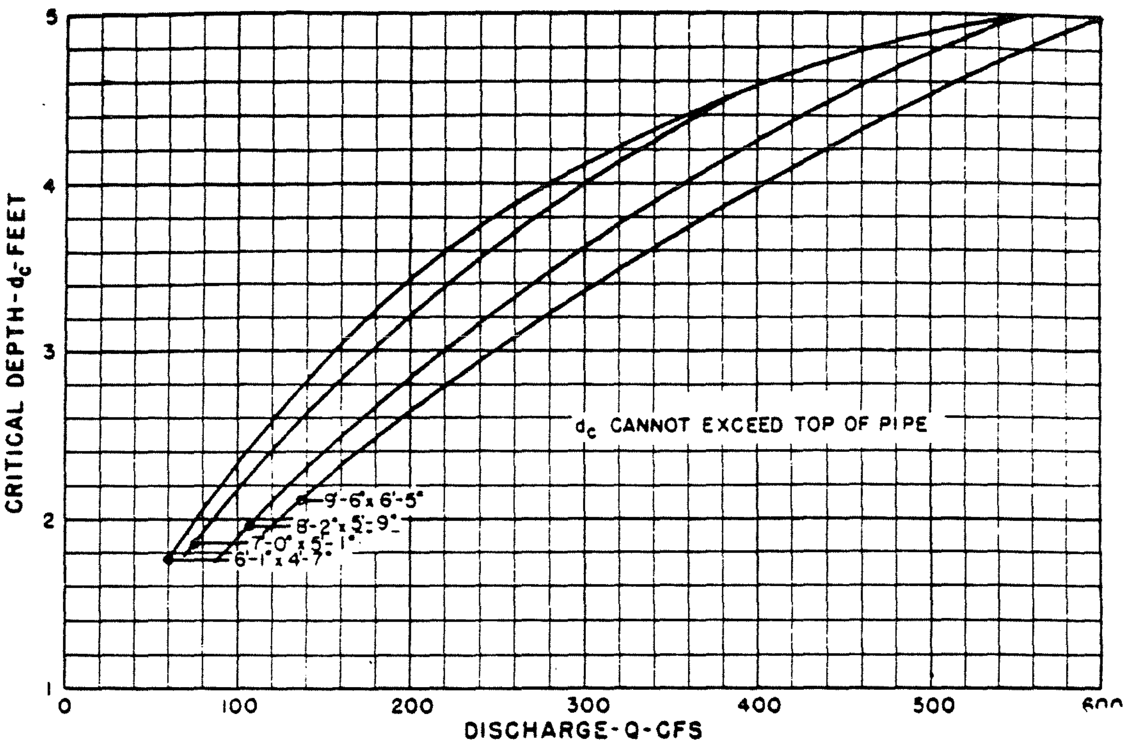
CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS VERTICAL

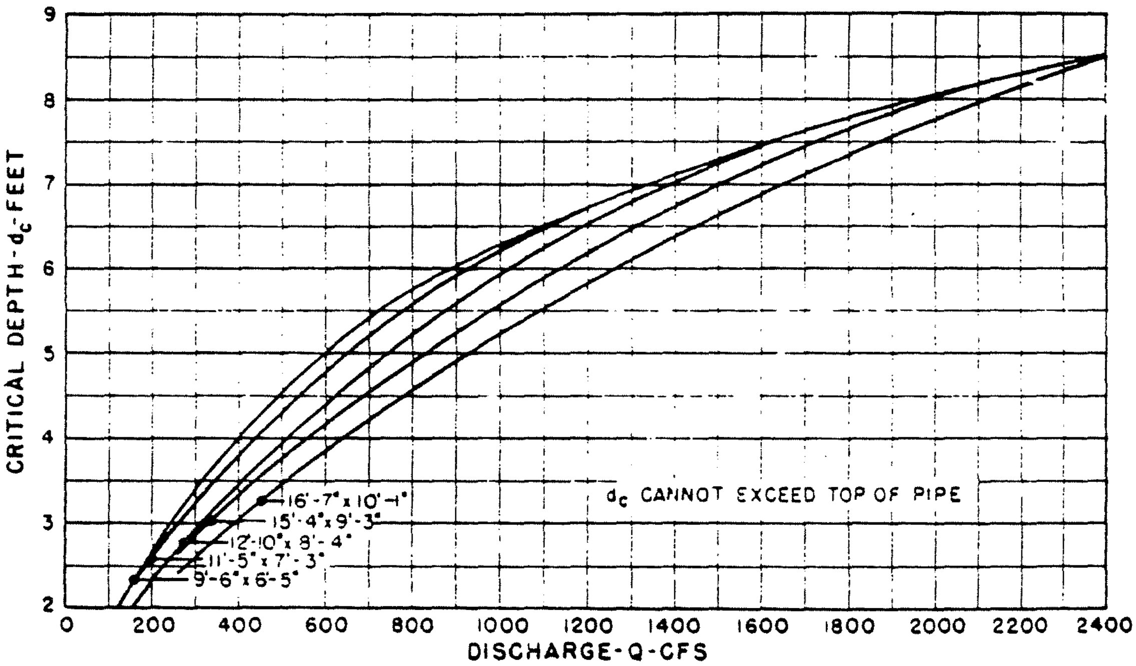




BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
STANDARD C.M. PIPE-ARCH





BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
STRUCTURAL PLATE
C.M. PIPE-ARCH
18 INCH CORNER RADIUS

3.4.3.12 Performance Curves. The principal disadvantage in using nomographs for the selection of culvert sizes is that it requires the trial and error method described in the text. Some engineers who limit their selection to a relatively small number of types of culverts would find it advantageous to prepare performance curves such as shown in Figure 3.4.12. These curves are applicable through a range of headwaters and discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes.

Figure 3.4.12 is plotted from the data shown in the following tabulations. These data were obtained from the nomographs contained in the text. (Computer programs are available from Public Roads for making these computatios). The first tabulation is for the inlet-control curve on Figure 3.4.12, and the second tabulation is for the outlet-control curves.

Data for Inlet-Control Curve

HW* / D	Q* (Read)	HW = HW/D X 4
.5	21 c.f.s.	2.0 ft.
.6	29	2.4
.7	37	2.8
.8	46	3.2
.9	56	3.6
1.0	65	4.0
1.1	74	4.4
1.3	90	5.2
1.5	102	6.0
1.7	112	6.8
2.0	126	8.0
2.5	145	10.0
3.0	165	12.0

* From Chart 5 Projecting Inlet (3)

DATA FOR OUTLET-CONTROL CURVES

Q	d_c	$\frac{d_c+D}{2}$	H	HW for Various S_o				
(Assume)	Chart 16	(Compute)	Chart 11 *	%	.5%	1%	1.5%	2.0%
20 cfs	1.3 ft.	2.6 ft.	.2 ft.	2.8 ft.	-	-	-	-
40	1.9	3.0	.8	3.8	2.8	1.8	.8	-
60	2.3	3.2	1.9	5.1	4.1	3.1	2.1	1.1
80	2.7	3.4	3.3	6.7	5.7	4.7	3.7	2.7
100	3.1	3.6	5.2	8.8	7.8	6.8	5.8	4.8
120	3.3	3.6	7.5	11.1	10.1	9.1	8.1	7.1
140	3.5	3.8	10.2	14.0	13.0	12.0	11.0	10.0
160	3.7	3.8	13.6	17.4	16.4	15.4	14.4	13.4

$$HW = H + h_o - LS_o \qquad \text{where } h_o = \frac{d_c + D}{2}$$

* From Chart II - or by Equation 2.

The curves plotted apply only to the type and length of culvert shown. Culverts placed on grades steeper than about 2.5 percent will operate on the inlet control curve for the headwater-discharge range of this plot. If a free outfall condition does not exist a correction for tailwater should be made an instructed in Step 3b, 3.4.3.9 Procedure for Selection of Culvert Size.

HYDRAULIC PERFORMANCE CURVES
FOR
48-INCH C.M. PIPE CULVERT
WITH
PROJECTING INLET

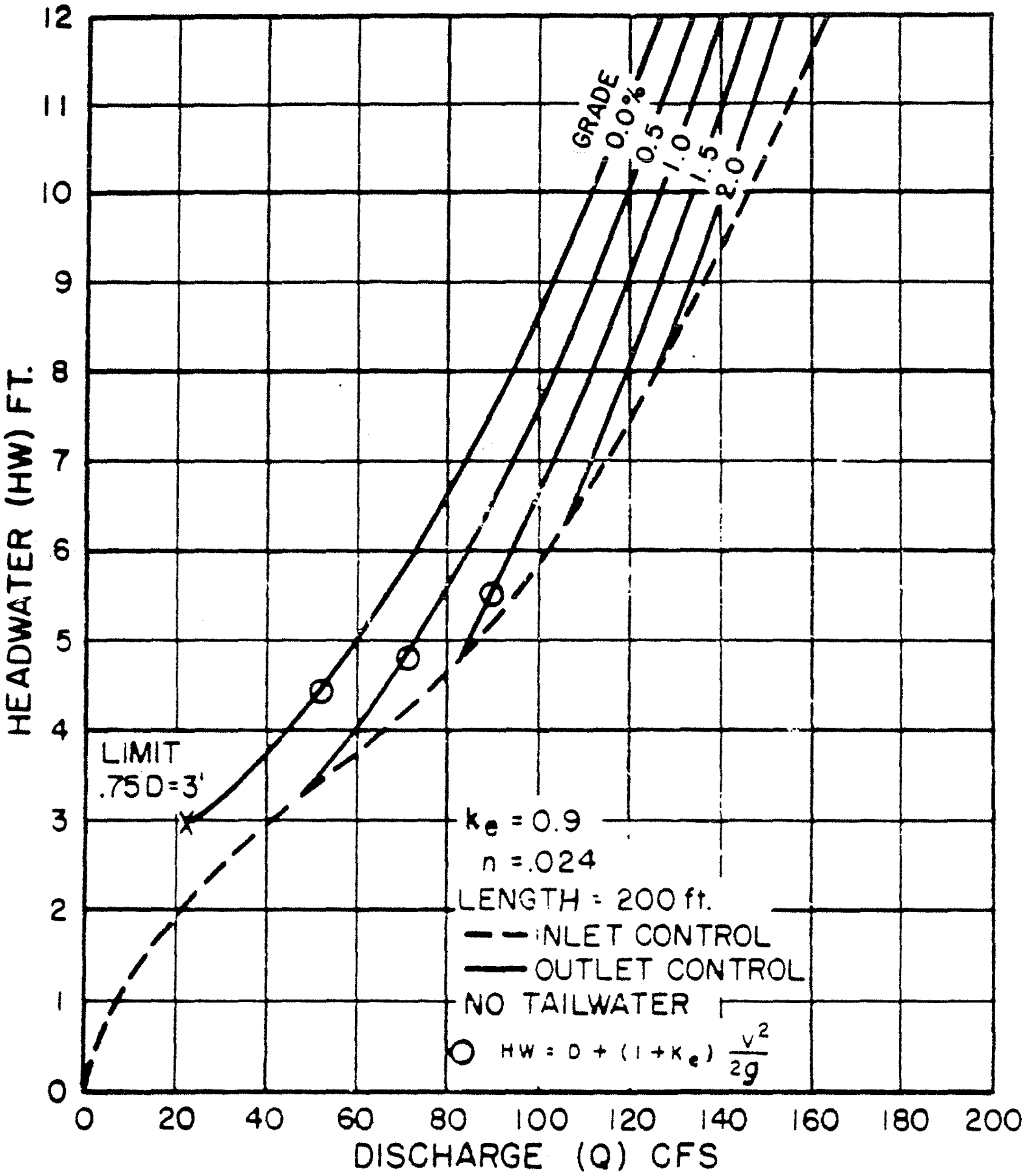


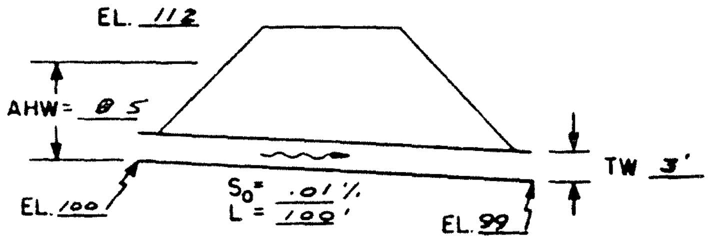
Figure 3.4.12

PROJECT: 142 B

DESIGNER: L. D. K

DATE: 2-18-64

HYDROLOGIC AND CHANNEL INFORMATION	SKETCH
Q_1 = 160 cfs : Q_{50} TW_1 = 5.0'	STATION : 321+14
Q_2 = ____ TW_2 = ____	
(Q_1 = DESIGN DISCHARGE, SAY Q_{25} ; Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})	



MEAN STREAM VELOCITY = 8'/sec
MAX. STREAM VELOCITY = 10'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	INLET CONT. HW	OUTLET CONTROL $HW = H + h_o - LS_o$: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
CMP (C.T.) Headwall	160	Assume 54"	1.56	7.6												Hw less than 8.5' · Try 48"
"	160	48"	2.25	9.0	.5	8.5	3.7	3.8	3	3.8	1.0	11.1	11.1	13.2'/sec		Hw high Try 54"
"	160	54"	1.56	7.0	.5	4.7	3.6	4.1	3	4.1	1.0	7.8	7.8	11.1'/sec		velocity at d_c size o.k.
Concrete (C.T.) Sq. Edge - Hdwl	160	48"	2.35	9.4	.5	4.7	3.7	3.8	3	3.8	1.0	7.5	9.4	14'/sec		Hw high Try 54"
"	160	54"	1.6	7.2	.5	2.9	3.6	4.1	3	4.1	1.0	6.0	7.2	14.7'/sec		Hw OK Vel > CMP Try 48 G. Edge
Concrete (C.T.) Groove end - HDWL	160	48	.95	7.8	.2	1.0	3.7	3.8	3	3.8	1.0	6.8	7.8	14.0'/sec		Hw OK Vel. high

SUMMARY & RECOMMENDATIONS:

PROJECT: E 14-2 (3)

DESIGNER: F.P.R.

DATE: 2-20-64

HYDROLOGIC AND CHANNEL INFORMATION

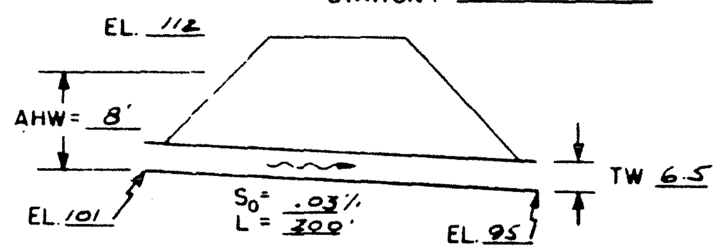
Q_1 = 400 cfs Q_{50} TW_1 = 6.5'

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: 8+61



MEAN STREAM VELOCITY = 8'/sec

MAX. STREAM VELOCITY = 12'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	INLET CONT. HW	OUTLET CONTROL $HW = H + h_o - LS_o$: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
Concrete (Cir) Gr End Proj	400		Assume 1.5			Find D:		78"								HW = 9.1 Too high Try 84"
"	400	84"	1.18	8.3												HW High Try 90"
"	400	90"	1.05	7.9	.2	1.9	5.2	6.3	6.5	6.5	6.0	2.4	7.9	28'/sec I.C. 10'/sec O.C.		If Too large Try 2 pipes
Same type 2 pipes	200	54"	1.85	8.3												Too small
"	200	60"	1.38	6.9	.2	3.4	4.0	4.5	6.5	6.5	6.0	3.9	6.9	23'/sec I.C. 10'/sec O.C.		Use See Comments
Cir CMP Bevel B (chart 7)	200	60"	1.34	6.7	.25	2	4.0	4.5	6.5	6.5	6.0	6.7	6.7	14'/sec I.C. 10'/sec O.C.		Use Bevel A can be used here
		Might try single concrete oval or metal arches														

SUMMARY & RECOMMENDATIONS:

PROJECT: I 85-2

DESIGNER: L. A. H.

DATE: 2-23-64

HYDROLOGIC AND CHANNEL INFORMATION

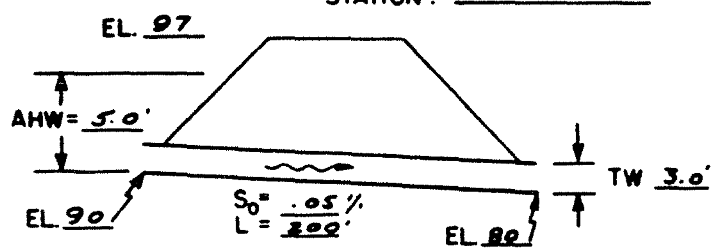
Q_1 = 120 cfs. : Q_{25} TW$_1$ = 3.0'

Q_2 = ______ TW$_2$ = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION : 314+10



MEAN STREAM VELOCITY = 12'/sec

MAX. STREAM VELOCITY = 15'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. HW/D	INLET CONT. HW	OUTLET CONTROL HW=H+h_o−LS_o: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
CMP (Cir.) Mitered	120	Assume 54"	1.25	5.6'								Outlet control does not govern		Not computed - Take Channel		HW high Try 60"
"	120	60"	.97	4.9	.7	2.5	3.0	4.0	3.0	4.0	10.0		4.9			Need more cover - Try arch
CMP Arch Mitered	120	72"x 44"	1.24	4.6	.7	3.4	2.4	3.0	3.0	3.0	10.0		4.6			Check Box Culvert
Concrete Box 30° W.W.	120	4'x 4'	1.23	4.9	.4	2.0	3.1	3.5	3.0	3.5	10.0		4.9			
Concrete Oval Gr. End Proj.	120	60"x 38"	1.51	4.8	.2	2.9	2.7	2.9	3.0	3.0	10.0		4.8			
Concrete Cir Groove End Proj	120	54"	1.11	5.0	.2	1.7	3.1	3.8	3.0	3.8	10.0		5.0			

SUMMARY & RECOMMENDATIONS:

IN-PLACE COST, AVAILABILITY, LOCATION, COVER REQUIREMENTS, ETC., SHOULD BE CONSIDERED BY THE DESIGNER IN SELECTING CULVERT. CM PIPE ARCH CULVERTS OR CONCRETE OVAL PIPES MIGHT BE A SOLUTION WHERE COVER IS LIMITED.

3.4.4 Bridges

3.4.4.1 General. This paragraph will present the design criteria applicable to the locating of the bridge abutments and substructure, which constrict the waterway, to insure hydraulic conditions for safe a structure and the efficient passage of fish in watercourses that have been classified Type A or B. The criteria for fish presented in Table 3.2.1 and a depth of flow 50% greater than that specified in 3.2.2 Inadequate Water Depth shall be required during the two year flood.

The criteria for design discharge determination shall be in accordance with the applicable portions of 3.3 Drainage Structure Design Criteria. The watercourse bed stability at the critical section should be investigated by either of the methods indicated in 3.4.2.1 Permissible Velocity Method or 3.4.2.2 Tractive Force Method.

3.4.4.2 Hydraulics of Constrictions in Watercourses. When an area constriction is introduced to an otherwise uniform, friction-controlled flow in prismatic channel of mild slope a backwater profile is first developed upstream from the constriction. Please refer to Figure 3.4.13.

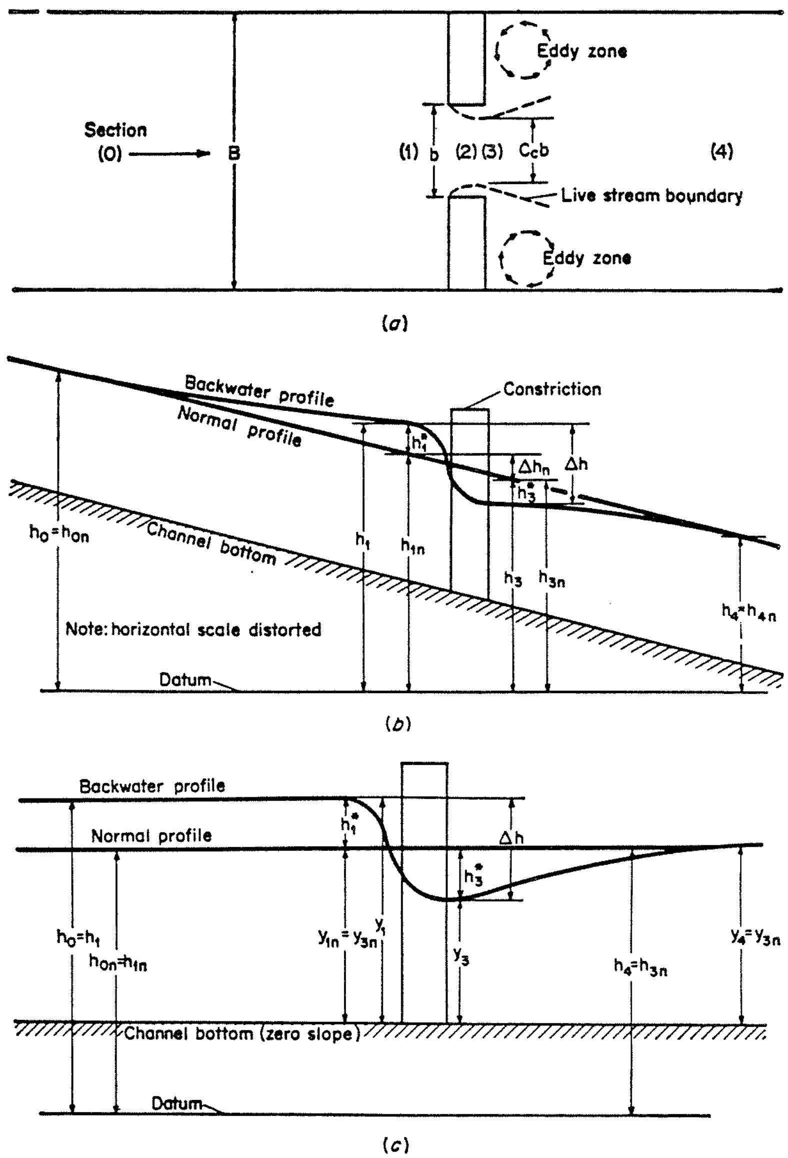
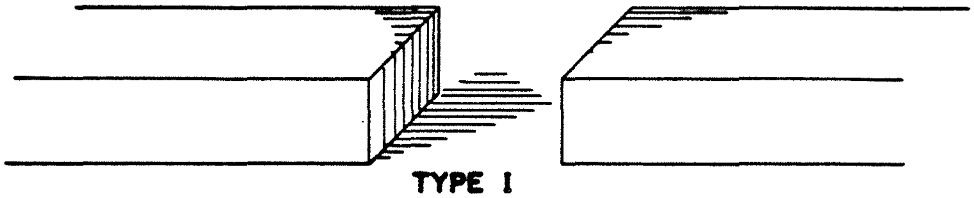
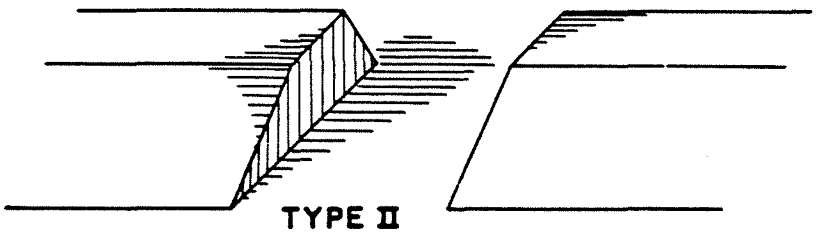


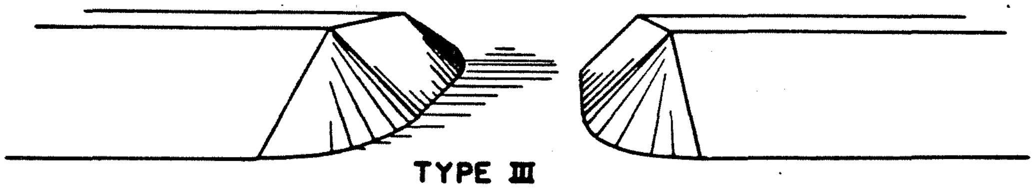
Figure 3.4.13. Definition of flow through constriction. (a) Plan; (b) elevation; (c) elevation, adapted to assumption of zero friction loss.



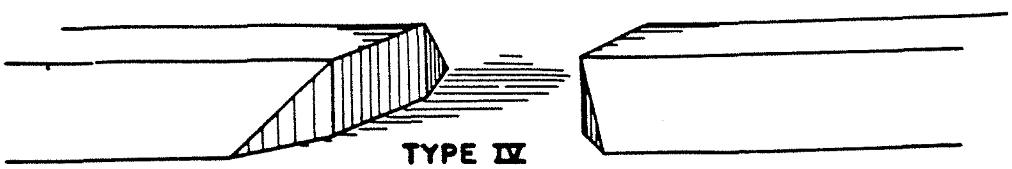
Type I opening, vertical embankment, vertical abutment



Type II opening, embankment and abutment slope.



Type III opening, embankment and abutment slope/



Type IV opening, embankment slope and vertical abutment with wing walls.

Figure 3.4.14 Constriction Types

The upstream end point of the backwater curve is assumed to be at section 0. Near the constriction at section 1 the central body of water begins to accelerate. An adequate approximation for the location of Section 1 may be taken at a point one opening width b from the center of the opening.

At the constriction, the flow is rapidly varied, characterized by marked acceleration in directions both normal and parallel to the streamlines. The longitudinal water surface drops rapidly in this region. Within the constriction, the live stream contracts to a width somewhat less than the nominal width of the opening, and the spaces between the live stream and the constriction boundaries are separation zones occupied by eddying water. As the water passes through the contraction, the contracted stream reaches a minimum width at Section 2, which corresponds to the vena contracta in an orifice flow. After the vena contracta, the live stream begins to expand until it reaches downstream Section 4, where the uniform-flow regime is reestablished in the full-width channel. Between Sections 3 and 4, the flow is gradually varied. Over the whole reach from Sections 0 to 4 encompassed by the backwater effect of the constriction, the total energy loss is the same as that for uniform flow.

The equation for the discharge through the constricted Section 3 is

1 Eq. 1:

$$Q = CA_3 \left[2g\left(h - h_f + \frac{V_1^2}{2g}\right)\right]^{1/2}$$

A_3 = area water prism at Section 3
V_1 = average water velocity at Section 1
h_f = hydraulic friction loss between Sections 1 and 3
C = is an overall coefficient of discharge

The overall coefficient of discharge C is calculated by first determining the C', the coefficient of discharge standard value whicn is a function of the physical type of abutment configuration along the flow lines of the constriction. The types are shown in Figure 3.4.14. The coefficient C' are dependent upon two factors; m the precent of channel contraction and L/b the ratio of the width of the abutment parallel to flow and the width of the constricted opening.

The value of M may be calculated by:

Eq. 2:
$$m = [1 - \frac{K_c}{K_l + K_r + K_c}] \; 100\%$$

where K refers to the conveyance capacity

Eq. 3:
$$K = \frac{1.486}{n} r^{2/3} \; A$$

and subscripts L, r and c refer to the Sections at l to the right of, left of, and the constricted section.

The overall coefficient of discharge C is now determined by adjusting C' for the effects of secondary variable by multiplying C' by the appropriate correction factors k.

A listing of these corrections follows:

k_F = a coefficient that adjusts C' for the influence of a nonstandard value of F

k = a coefficient that adjusts C' for the influence of angularity of flow

k_θ = a coefficient that adjusts C' for the influence of angle of wing walls

k_e = a coefficient that adjusts C' for the influence of eccentricity of constriction

k_j = a coefficient that adjusts C' for the influence of piers and piles

k_r = a coefficient that adjusts C' for the influence caused by rounding entrance corner of abutment for vertical-faced constrictions

k_t = a coefficient that adjusts C' for the influence of submergence of bridge members

k_w = a coefficient that adjusts C' for the influence of length of wing walls

k_x = a coefficient that adjusts C' for the influence of the ratio of distances x/b (See Fig. 3.4.19c and Fig. 3.4.20c)

k_y = a coefficient that adjusts for the influence of ratio of depth of water width to opening,

$$\frac{y_a + y_b}{2b}$$

The C' values and the correction factors can be obtained from the Figures 3.4.15 to 3.4.23 at the end of this Section.

It is possible that certain combinations of the empirical coefficients applied to C' may appear to yield a value of C greater than 1.0. In such cases, however, a value of C = 1.0 should be used.

Referring to Figure 3.4.13 in designing bridge opening for the maximum discharge we are concerned with backwater profile or the surcharge above the normal profile and the depression in the normal profile at Section 3. The interest in the former is to see whether overtopping of the banks occurs and

TABLE 3.4.5

Key to Tables for C' and k Values for Each Constriction Type

Type	C'	k_F	k_ϕ	k_θ	k_e	k_j	k_r	k_t	k_w	k_x	k_y
Type I	3.4.15 A	3.4.15B B	3.4.16 D		3.4.23 A	3.4.23 C,D	3.4.15 C	3.4.23 B	3.4.15 A,B&C		
Type II SS=1:1	3.4.17 A		3.4.17 C		3.4.23 A	3.4.23 C,D		3.4.23 B			3.4.18 B
Type II SS=2:1	3.4.18 A		3.4.18 C		3.4.23 A	3.4.23 C,D		3.4.23 B			3.4.18 B
Type III SS=1:1	3.4.19 A		3.4.19 B		3.4.23 A	3.4.23 C,D		3.4.23 B		3.4.19 C	
Typed III SS=2:1	3.4.20 A		3.4.20 B		3.4.23 A	3.4.23 C,D		3.4.23 B		3.4.20 C	
Type IV SS=1:1	3.4.21 A		3.4.21 B	3.4.21 C	3.4.23 A	3.4.23 C,D		3.4.23 B			
Type IV SS=2:1	3.4.22 A	3.4.22 C	3.4.22 B	3.4.22 D	3.4.23 A	3.4.23 C,D		3.4.23 B			

SS = Sideslope

the latter to determine water velocity at Section 3. By adjusting Eq. 1 we obtain a relationship for h.

Eq. 4

$$h = \frac{V_3^2}{2gC^2} - \frac{V_1^2}{2g} + h_F$$

where h_F is the friction loss between Sections 1 and 3 and may be calculated by:

Eq. 5

$$h_F = b \left(\frac{Q}{\sqrt{K_1 K_3}} \right)^2 + L \left(\frac{Q}{K_3} \right)^2$$

where; b = the distance section 1 is upstream of the constriction, generally equal to the breadth of the constriction

L = the length of the constriction

K_1 & K_3 = are the total conveyances of Sections 1 and 3 respectively

In Figure 3.4.13 the increase h_1* in water surface from the normal stage to the backwater stage at Section 1 is known as the backwater of the constriction. The distance h is the difference in water-surface elevation between Sections 1 and 3. The ratio h_1*/ h is called the backwater ratio, which is known to be a function of the channel roughness, percentage of channel contraction, and constriction geometry. A laboratory investigation (Reference 7) was made on the backwater effect due to vertical-faced constrictions with square-edged abutments. Data plotted in Figure 3.4.24 indicates the relationship among backwater ratio, Manning's n, and contraction ratio m. It can be seen that the channel roughness is relatively unimportant as a factor in determining the backwater ratio. In fact, the limit of change in the backwater ratio due to roughness is practically reached at an n of about 0.050. The previously cited laboratory investigation also reveals that the influence of cross-sectional shape on backwater ratio is included in the contraction ratio.

The backwater ratio in Figure 3.4.24 is for constriction of basic type, that is, for a vertical-faced constriction with square abutments. The backwater ratio for other types of constriction may be obtained by multiplying the backwater ratio by an adjustment factor k_a. This factor has been found to be a function of the contraction ratio m and the ratio C/C_{basic} C_{basic} and C are, respectively, the discharge coefficients for the basic type and for other types of constriction that can be determined by the method described in the preceding text. The value of C_{basic} can be obtained directly from Figure 3.4.15 a and b. Based on experimental data, the relationship among k_a, m, and C/C_{basic} is shown in Figure 3.4.25.

3.4.4.3 Procedure for Design of Bridge Waterway. The first step is to list design data. Drainage Structure Design Data Sheet 1 is provided for this.

a. Design discharge Q in cfs, for required periods (i.e. Q_{50} or Q_{100} etc.)

b. Establish constriction type, (I, II III or IV) breadth of constriction, length of constriction and the constriction centerline relative principal channel water prism. In watercourses with fish, Type A or B see 1.2 Scope, it is preferable that the abutments or piers fall outside of the two year flood channel. This will avoid the need to undertake an analysis for the two year flood.

During the field investigation the watercourse should be surveyed at Section 1 (a distance upstream of the proposed constriction equal to the breadth of the constriction) and Section 3 (at the downstream end of the constriction), refer to 3.4.2 Waterways. The field investigation should also determine Manning's n, the slope between Section 1 and 3, and the substrate classification for permissible velocities or tractive force calculations.

c. Determine a rating curve for the reach between Sections 1 and 3 using the average of the areas and hydraulic radii for several depths including the design flood depth.

d. Calculate the conveyances K for Section 1 and the constricted channel at Section 3 and the ratio L/b (embankment and constriction breadths).

e. Calculate m (channel constraction) from conveyances K (Eq. 2) with m and L/b go to figure for constriction type (Figures 3.4.14 to 3.4.22) to determine C' the coefficient of discharge (standard value).

f. Determine if the constriction type or location require any modification to C' i.e. C' x k , k_F etc. = C. See Table 3.4.5 for figure location for modification factors.

g. With n and m given enter Figure 3.4.24 to determine h_1*/ h. (If the constriction under investigation is not Type I a C' and k_F are determined for the m, L/b and the Froude number in Figure 3.4.15 a and b. The C' x k_F = C_{basic} is used with C determined in f. In Figure 3.4.25 to obtain k_a. k_a times h_1*/ h will give the corrected h_1*/ h).

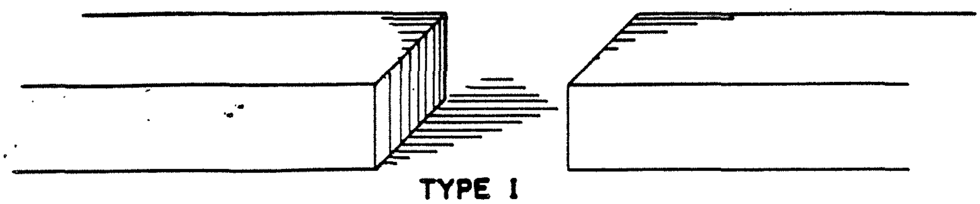
h. With the value h_1*/ h estimate h (Trial starts 0.9 x $(V_1 \frac{A_1}{A_3})^2/2g$ and calculate h_1*

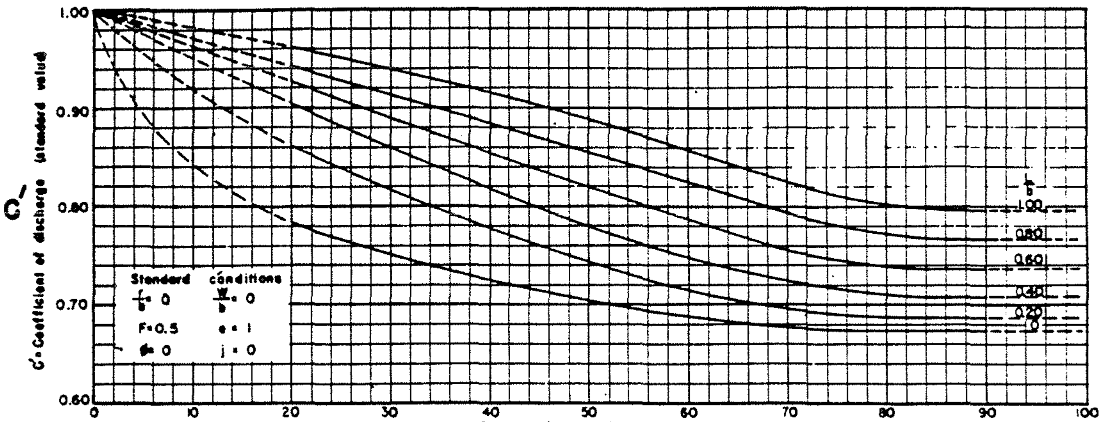
i. Calculate trial A_3 using depth of flow calculated in c less (h - h_1*) which is y_3.

j. Using Eq. 5 and 4 and correcting C value in Eq. 4 for Froude number k_F calculate h, if value agrees with that assumed in h, the solution is reached. If not repeat h, i and j with a new estimate of h.

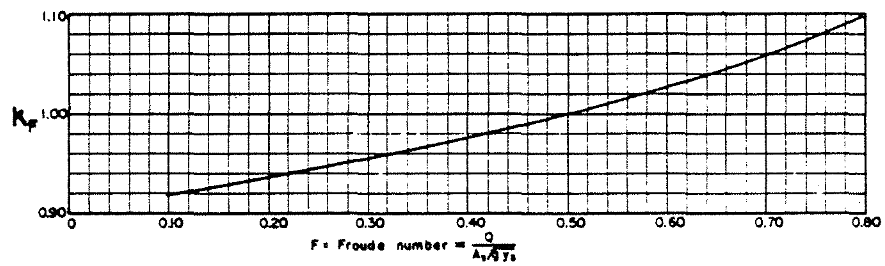
k. When solution is reached check V_3 velocity per tractive force or permissible velocity criteria, if satisfactory or close to satisfactory a analysis at the 10 year flood may not be necessary.

l. For fish watercourses where the two year flood water prism is constricted, perform analysis a thru j to determine that velocity V_3 does not exceed requirements stipulated in Table 3.2.1.

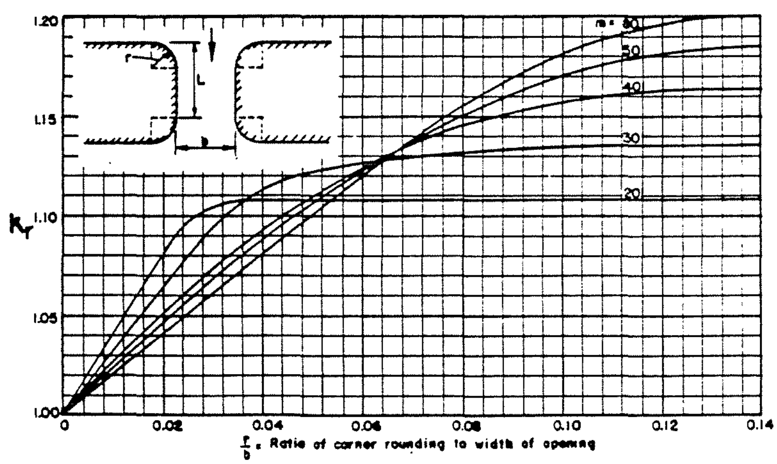




A. Base curve for coefficient of discharge



B. Variation of discharge coefficient with Froude number



C. Variation of discharge coefficient with entrance rounding

Figure 3.4.15.--Type 1 opening, vertical embankment, vertical abutment.

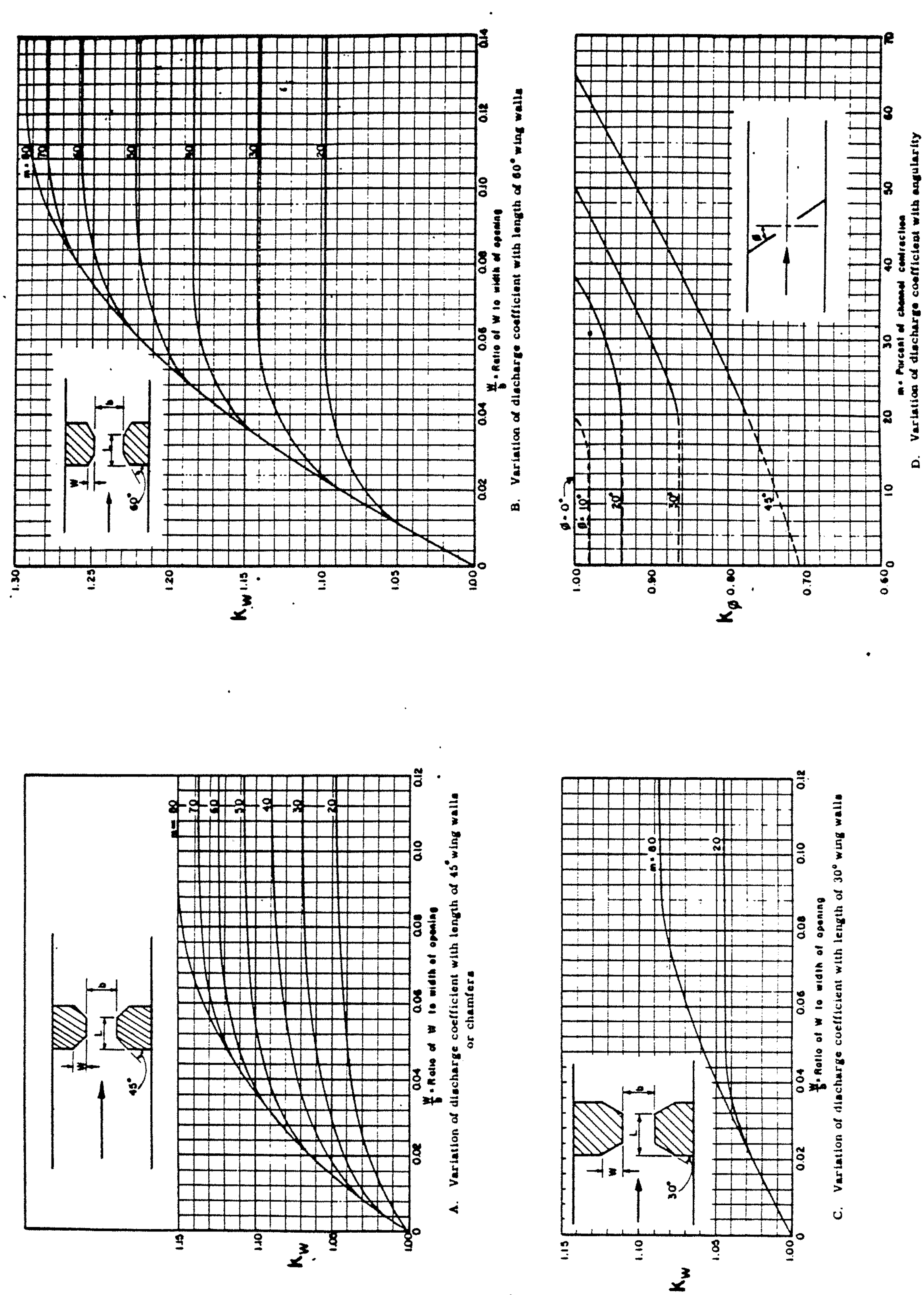
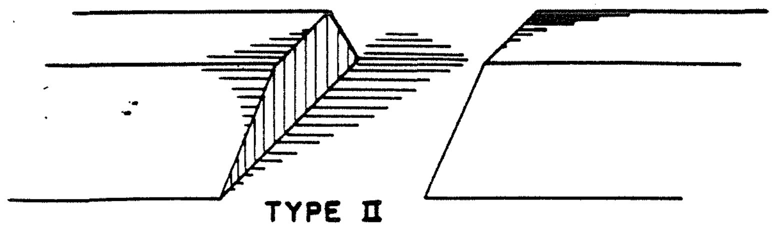
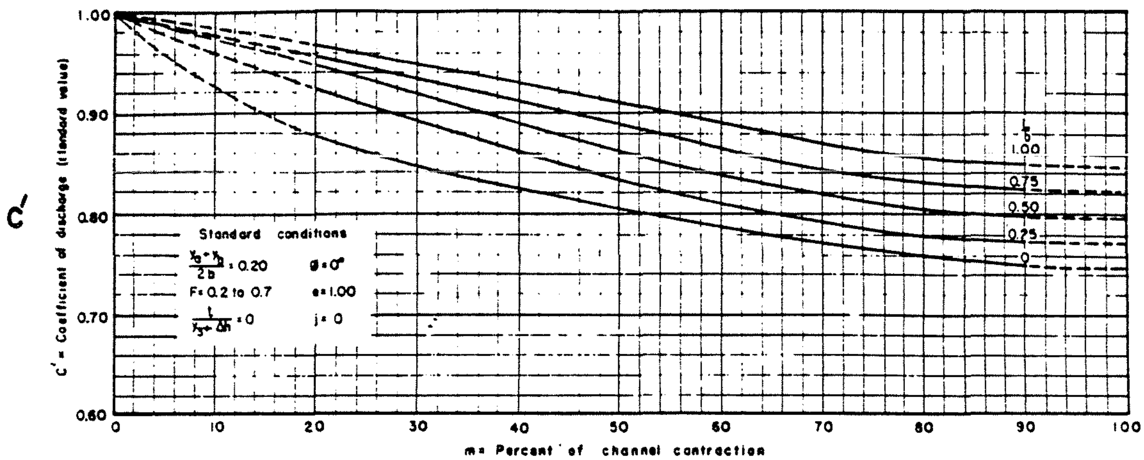
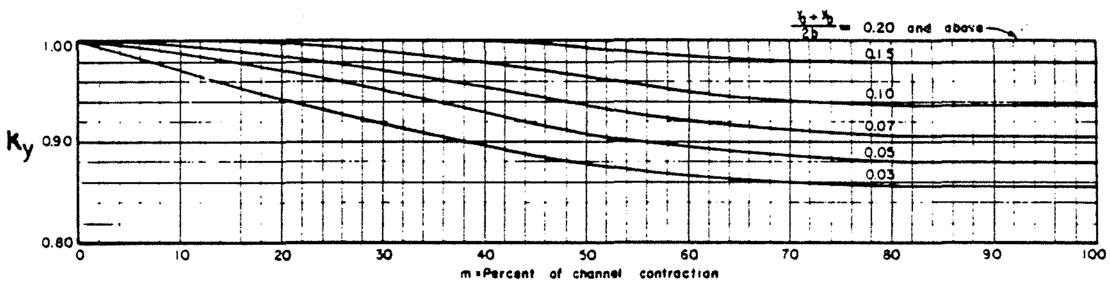


Figure 3.4.16.--k_w and k_ϕ curves for vertical embankment and abutment of Type I opening see figure 3.4.23.

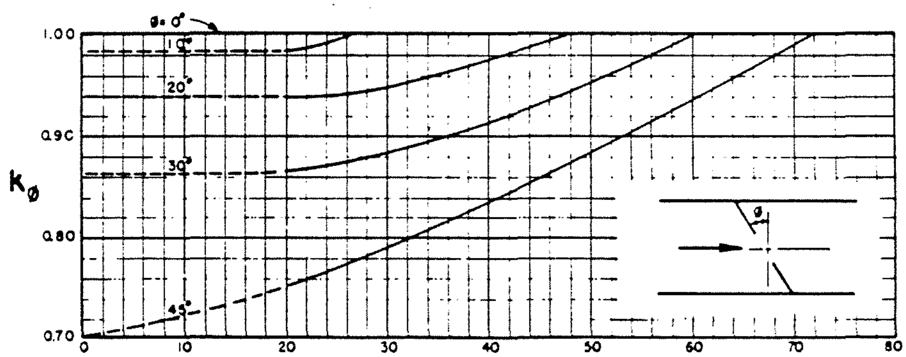




A. Base curve for coefficient of discharge

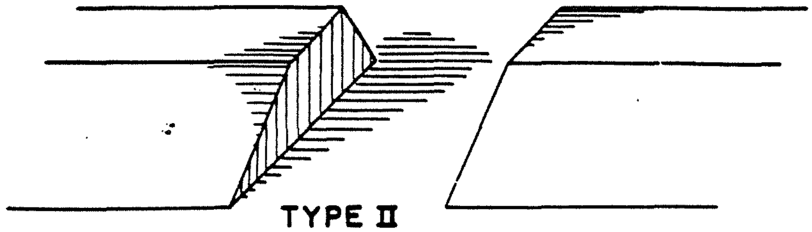


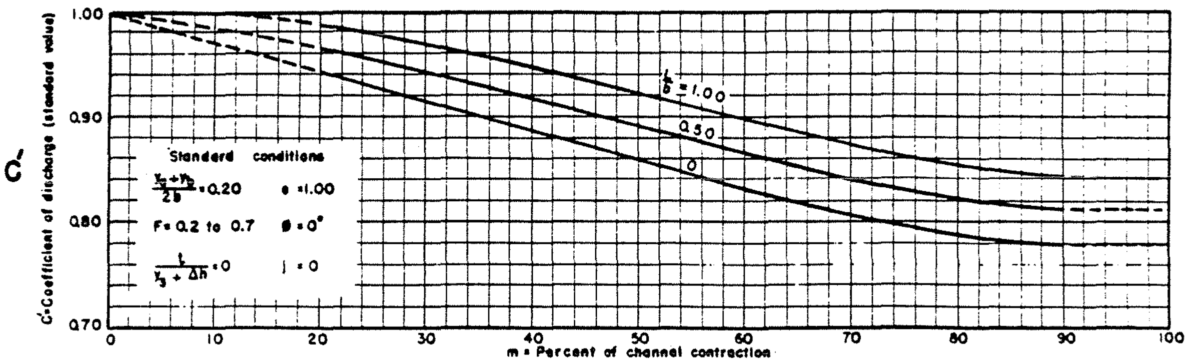
B. Variation of discharge coefficient with $\frac{y_a+y_b}{2b}$ ratio



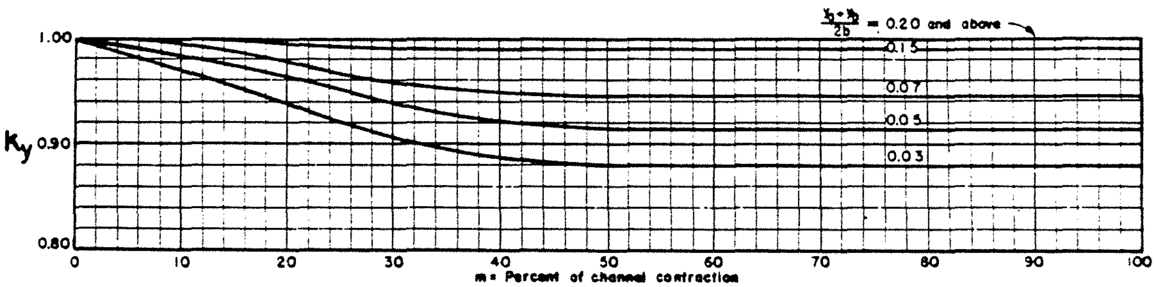
C. Variation of discharge coefficient with angularity

Figure 3.4.17.--Type II opening, embankment slope 1 to 1, vertical abutment. See figure 3.4.23.

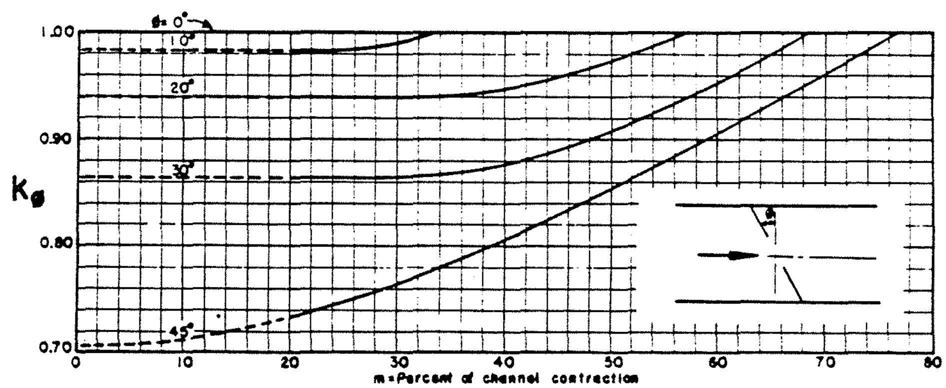




A. Base curve for coefficient of discharge



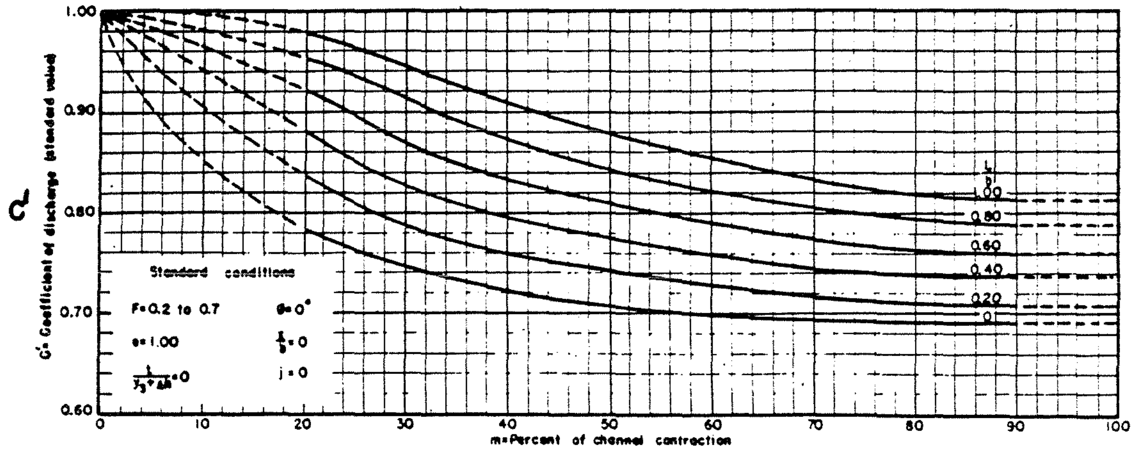
B. Variation of discharge coefficient with $\frac{y_a + y_b}{2b}$ ratio



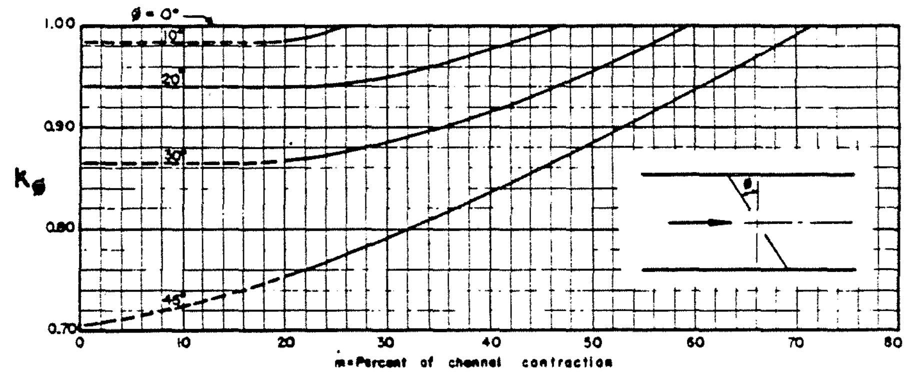
C. Variation of discharge coefficient with angularity

Figure 3.4.18.--Type II opening, embankment and abutment slope 2 to 1, vertical abutment. See figure 3.4.23.

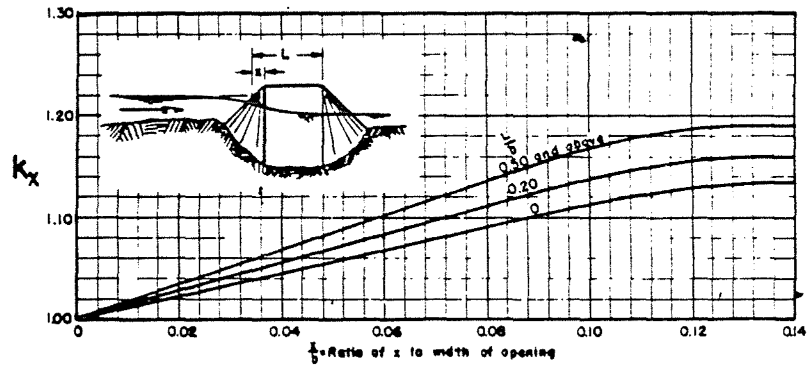
TYPE III



A. Base curve for coefficient of discharge

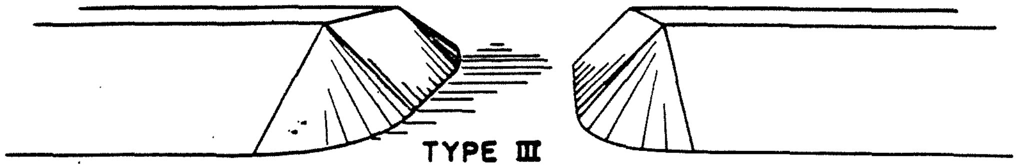


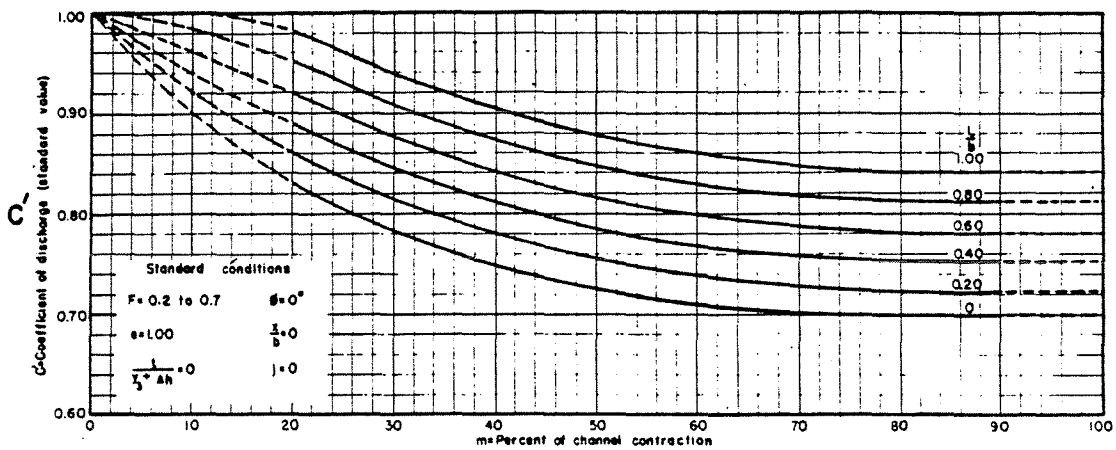
B. Variation of discharge coefficient with angularity



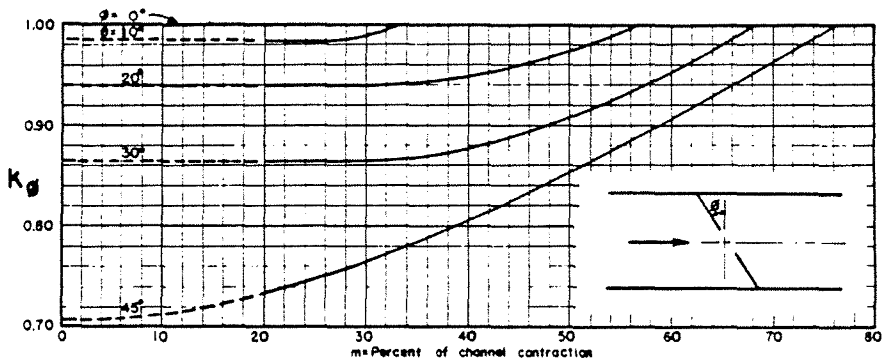
C. Variation of discharge coefficient with $\frac{x}{b}$ ratio

Figure 3.4.19.--Type III opening, embankment and abutment slope 1 to 1. See figure 3.4.23.

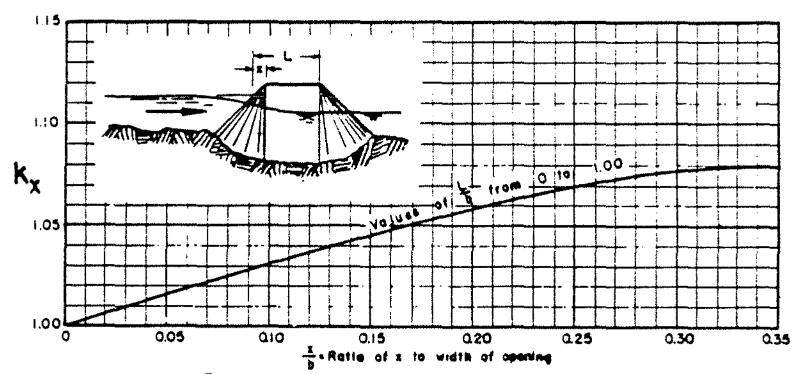




A. Base curve for coefficient of discharge

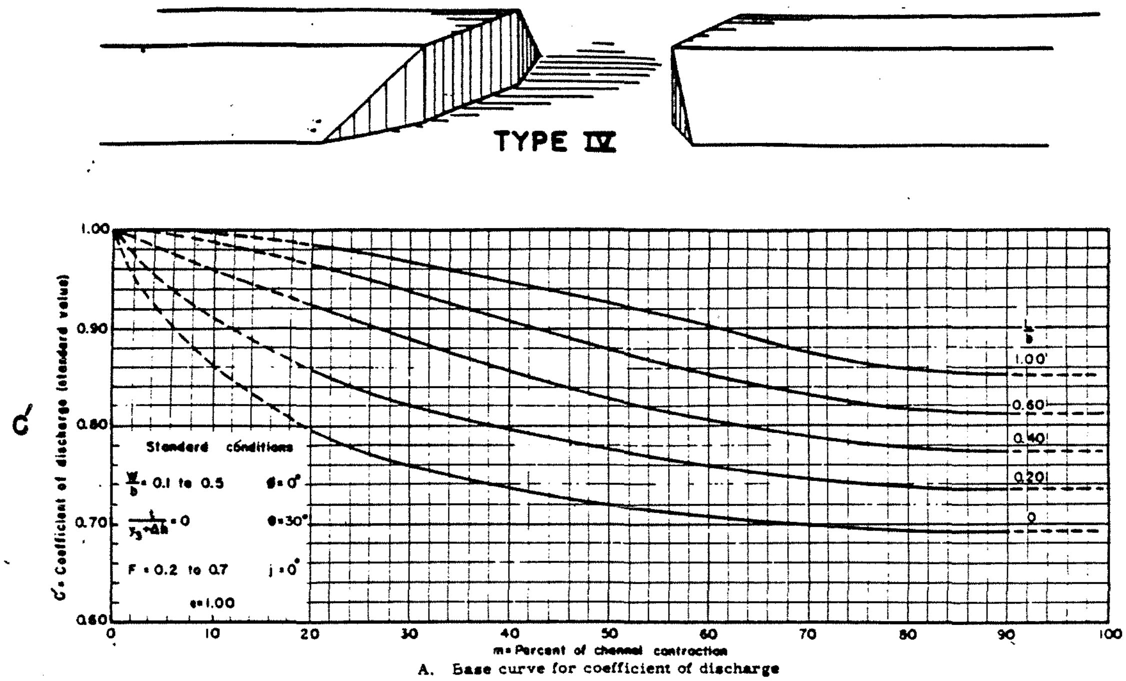


B. Variation of discharge coefficient with angularity

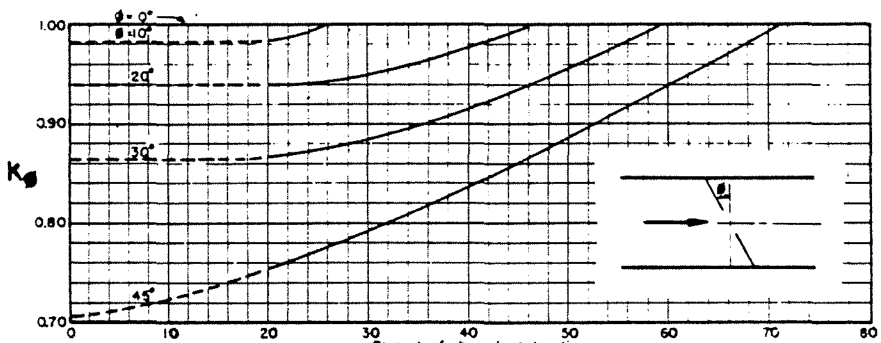


C. Variation of discharge coefficient with $\frac{x}{b}$ ratio

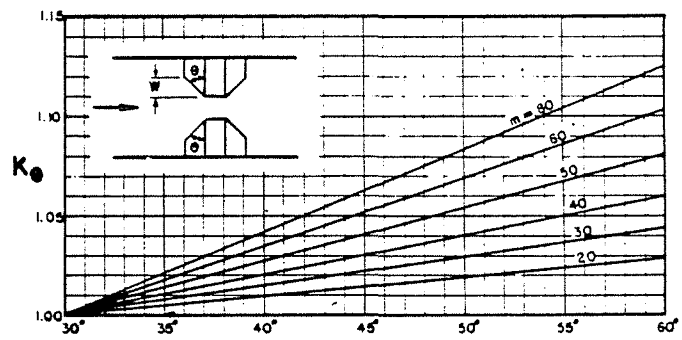
Figure 3.4.20.--Type III opening, embankment and abutment slope 2 to 1. See figure 3.4.23.



A. Base curve for coefficient of discharge

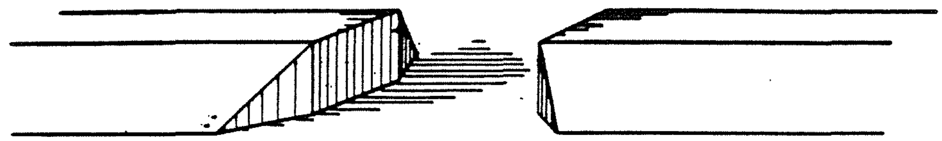


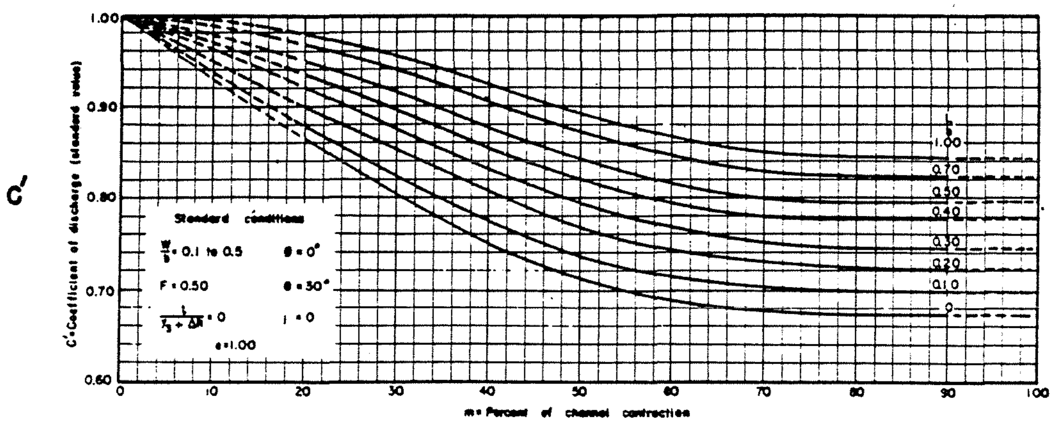
B. Variation of discharge coefficient with angularity



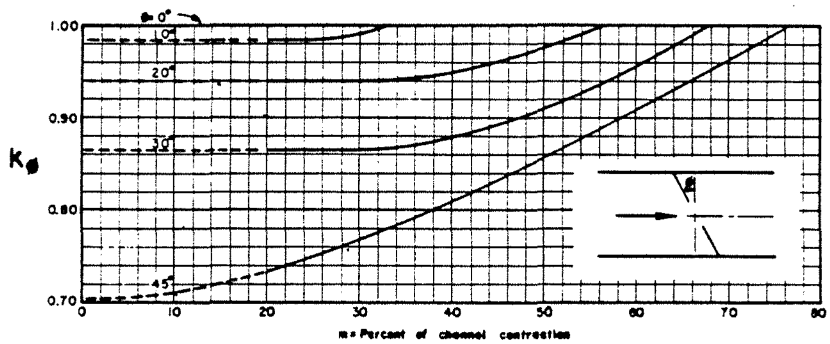
C. Variation of discharge coefficient with wing-wall angle

Figure 3.4.21.--Type IV opening, embankment slope 1 to 1, vertical abutment with wing walls. See figure 3.4.23.

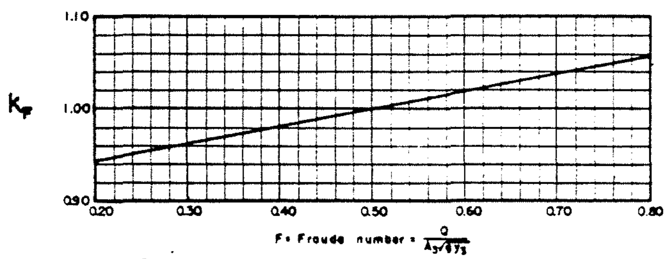




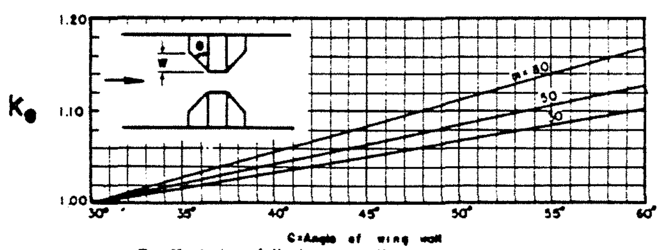
A. Base curve for coefficient of discharge



B. Variation of discharge coefficient with angularity

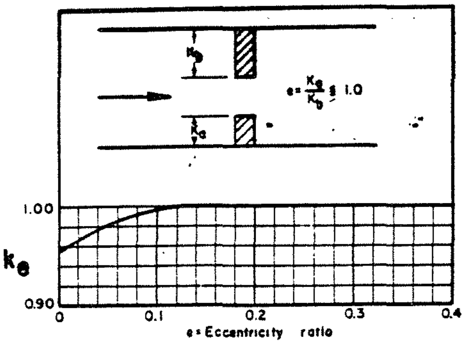


C. Variation of discharge coefficient with Froude number

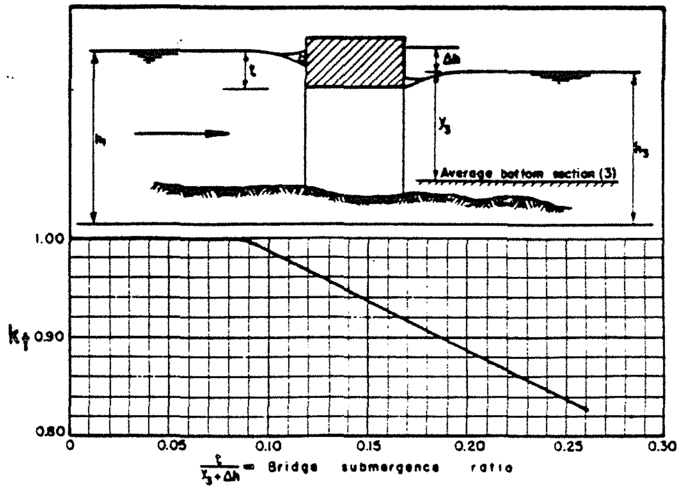


D. Variation of discharge coefficient with wing-wall angle

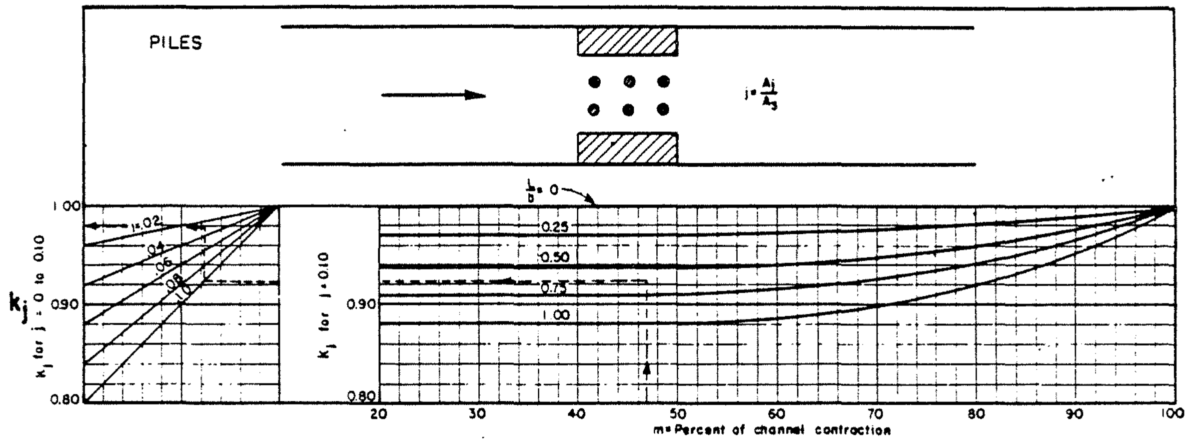
Figure 3.4.22.--Type IV opening, embankment slope 2 to 1, vertical abutment with wing walls. See figure 3.4.23.



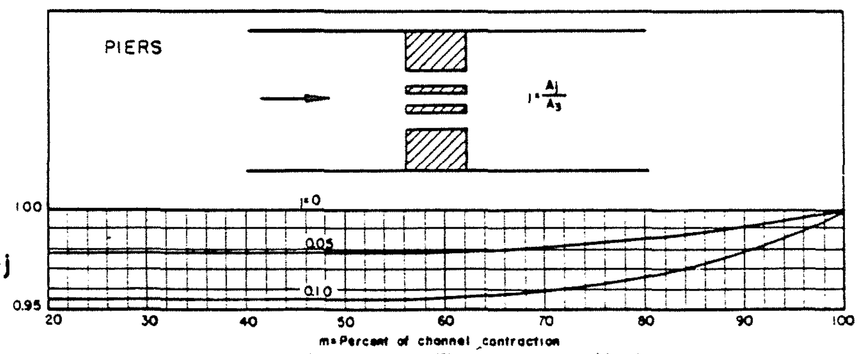
A. Variation of discharge coefficient with eccentricity



B. Variation of discharge coefficient with degree of submergence of bridge



C. Variation of discharge coefficient with area of bridge piles



D. Variation of discharge coefficient with area of bridge piers

Figure 3.4.23.--Types I-IV openings, k_e, k_t, and k_j curves.

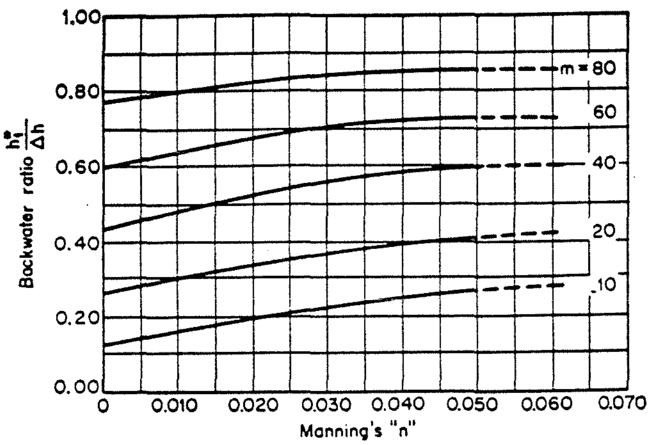


Figure 3.4.24. The effect of channel roughness on the backwater ratio for basic-type constrictions.

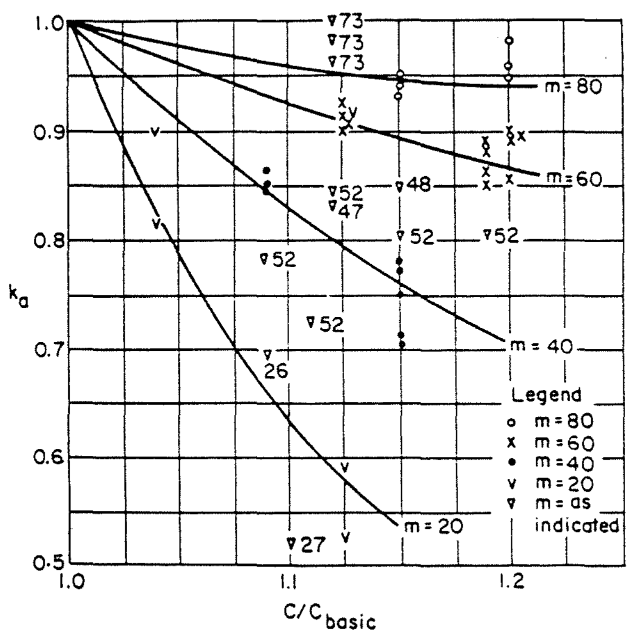


Figure 3.4.25. The effect of constriction on the backwater ratio.

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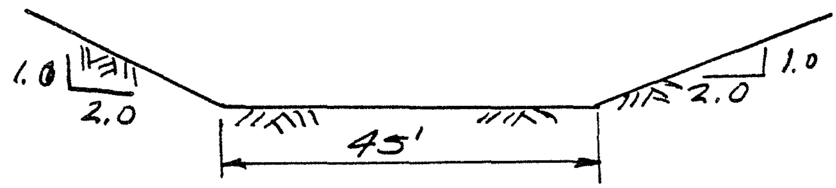
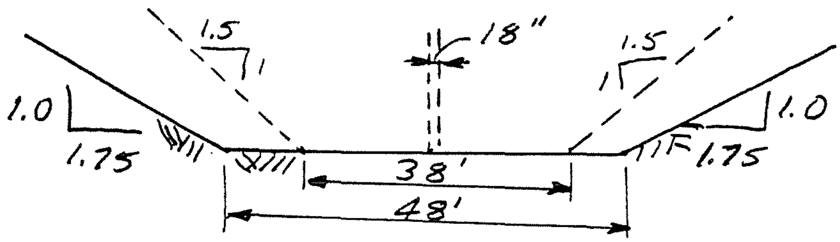
Site investigation data

Section 1





Section 3

$n = 0.035 \quad S = 0.0012$

Channel composed of gravels & cobbles less than 6" in diameter

Watercourse classified as Type Group II by F Scale

Design Q 100yr flood

$Q_{100} = 3,620$ cfs $\quad Q_{10} = 1200 \quad Q_2 = 250$

(Step C)

Average section between 1 and 3 the channel values are base 46.5 side slopes 1.875:1 (SS)

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

$$= \frac{1.486}{.035}\left[(46.5+1.875y)\,y\right]\left[\frac{(46.5+1.875y)y}{46.5+\sqrt{1.875^2+1^2}\,(2y)}\right]^{2/3}(.0012)^{1/2}$$

solving for depth (y)

	Y	Velocity ft/sec
Q = 3620 cfs	10 ft.	5.54
Q = 1200	5.38 ft.	3.94
Q = 250	2.15 ft.	2.30

Susitna Hydroelectric Project

Culvert Design Data Sheet

Location: Township T32N Range R6E

Section 30 Meridian 62°50'20"N

Project Feature: (project access road, material site access road, etc.)

Station: 1200' Upstream of River Y Junction

Type Water Course A B C -

User Fish Group (B & C only) I II III IV

Drainage Area: 6050 acres

Q_2: 250 cfs Q_{design}: 3620 cfs

Frequency of Q_{design}: 50 years

Watercourse Area for Q_2: 108.6 ft^2 Gradient 0.0012 ft/ft

Watercourse depth of flow for Q_2: 2.15 ft

Classify channel substate: Insignificant amount of fines, gravels to cobbles less than 6 inches

Channel configuration: Braided Meandering Straight

Other (describe) Bridge

Culvert Type: —— Other Bridge with center pier

Size: —— Width at channel 38' SS=1½:1

Slope: Same as river ft/ft Length: 50' ft.

V, Q_2: ______ ft/sec V, Q_{design}: 7.34 ft/sec

HW/D, Q_2: —— % HW/D, Q_{design}: —— %

Attested to by:

F. Scale
Fisheries Biologist

S. Rule
Design Engineer

Design is based on Type III constriction with SS = 1.5:1 with 38' between toe of slopes bridge will have a central concrete pier support 18" thick see dash outline on Section 3 outline page 1.

(step d)

Conveyance Section 1 (K_1)

$$K_1 = \frac{1.486}{n} A_1 r_1^{2/3} = \frac{1.486}{0.035}\left[(45+2(10))10\right]\left[\frac{(45+2(10))10}{45+10(4.47)}\right]^{2/3}$$

$= 103{,}206$

Conveyance Section 3 (K_3

n correction for piers $n_{concrete}$ 0.015

$$\bar{n} = \frac{\left[P_c n_c^{1.5} + P_w n_w^{1.5}\right]^{2/3}}{(P_c + P_w)^{2/3}} = \frac{\left[10(.015)^{1.5} + (17.25+3.61(10)).035^{1.5}\right]^{2/3}}{(10+17.25+3.61(10))^{2/3}}$$

$= 0.032$

$$K_3 = 2\,(\text{openings})\ \frac{1.486}{0.032}\left[\left(17.25+\frac{10}{2}(1.5)\right)10\right]\left[\frac{\left(17.25+\frac{10}{2}(1.05)\right)10}{10+17.25+1.8(10)}\right]^{2/3}$$

$= 71{,}278$

$L = 38.75'$ (see Figure 3.4.19C) $x = 3.75$

$b = 38'$

$L/b \approx 1.0$

(step e)

$$m = \left(1 - \frac{K_3}{K_1}\right)100\% = \left[1 - \frac{71278}{103206}\right]100\%$$

$= 30.9\%$

C' by interpolation for SS 1.5:1 from Figures 3.4.19 and 3.4.20

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SS = 1:1   C' = 0.925
Use 0.923 for SS = 1.5:1
SS = 2:1   C' ≈ 0.921

(Step f) Review modification factors to C'
Table 3.4.5

k_F N.A. k_ϕ N.A. k_θ N.A.

k_e N.A. k_j yes Fig 3.4.23 k_r N.A.

k_t N.A. k_x yes Figs. 3.4.19c, 3.4.20c k_y N.A.

$$J = \frac{A_J}{A_g} = \frac{10'(1.5')}{(38' + 10' \times 1.5)10'}$$

$= 0.03$

k_j from Fig. 3.4.23 for $m = 30.9$; $J = 0.03$

$k_j = .989$

k_x interpolated from Figs. 3.4.19c and 3.4.20c

for $L/b = 1$ & $\frac{X}{b} = \frac{3.75}{38} \simeq 0.10$

	SS = 1:1	1.5:1	SS = 2:1
k_x	1.17		1.03
k_x		1.10	

$C = C' k_j k_x = .923(.989)(1.10)$

$= 1.004 > 1.0$ Use 1.0

$C = 1.0$

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	COMPUTED SR CHECKED CC	DATE 8/25/84
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(step g)

Since constriction is not Type I determine C' and k_F from Fig 3.4.15 A & B

For $m = 30.9$ & $L/b = 1.0$

A $C' = .938$

$$\text{Froude No.} = \frac{Q}{A_3\sqrt{g Y_3}} = \frac{3620}{((38+1.5(10))10)\sqrt{32.2(10)}}$$

$= 0.38$

B $k_F = 0.975$

$C_{basic} = C' k_F = 0.938(0.975)$

$= 0.915$

$$C/C_{basic} = \frac{1.0}{0.915}$$

$= 1.09$ with Figure 3.4.25

$k_2 = 0.77$

From Fig. 3.4.24 with $n = 0.032$ & $m = 30.9$

$h_1^*/\Delta h = 0.49$

$$h_1^*/\Delta h^{\text{corrected}} = \frac{h_1^*}{\Delta h}(k_2) = 0.49(0.77)$$

$= 0.377$

(step h)

$$\text{Trial } \Delta h = .9\frac{\left(V_1\frac{A_1}{A_3}\right)^2}{2g}$$

$$= .9\frac{\left(5.54\left(\frac{654}{530}\right)\right)^2}{2(32.2}$$

$= 0.653$

$$h_1^* = 0.377 \Delta h$$
$$= 0.377(.653)$$
$$= 0.246$$

(Step i)

$$y_3 = y_3 - \Delta h + h_1^* = 10 - 0.653 + 0.246$$

$$y_3 = 9.593$$

(Step j)

$$h_f = b\left(\frac{Q}{\sqrt{K_1 K_3}}\right)^2 + L\left(\frac{Q}{K_3}\right)^2$$

$$= 38\left(\frac{3620}{\sqrt{103206(71278)}}\right)^2 + 38\left(\frac{3620}{71278}\right)^2$$

$$= 0.068 + 0.098$$

$$h_f = 0.166$$

$$\Delta h = \frac{V_3^2}{2gC} - \frac{V_1^2}{2g} + h_f$$

$$= \frac{\left(\frac{3620}{(36.5+9.593(1.5))9.593}\right)^2}{64.4(1.0)} - \frac{5.54^2}{64.4} + 0.166$$

$$\Delta h = 0.543 \neq 0.653$$

Repeat steps h, i & j for new trial Δh

<u>Trial $\Delta h = .525$</u>

(Step h) $h_1^* = .377(.525) = 0.198$

(Step i) $y_3 = 10 - 0.525 + 0.198 = 9.673$

$$\Delta h = \frac{\left(\frac{3620}{(36.5+9.673(1.5))9.673}\right)^2}{64.4} - \frac{5.54^2}{64.4} + 0.166$$

<u>$\Delta h = 0.525$</u> OK

Refering to Figure 3.4.13

The backwater effect $h_1^* = 0.377(0.525') = 0.198'$

the velocity (average) at Section 3

$$V_3 = \frac{3620}{(36.5' + 9.673'(1.5))\,9.673'}$$

$= 7.34$ ft/sec

Repeat Calculations for Q_2

REFERENCES

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

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

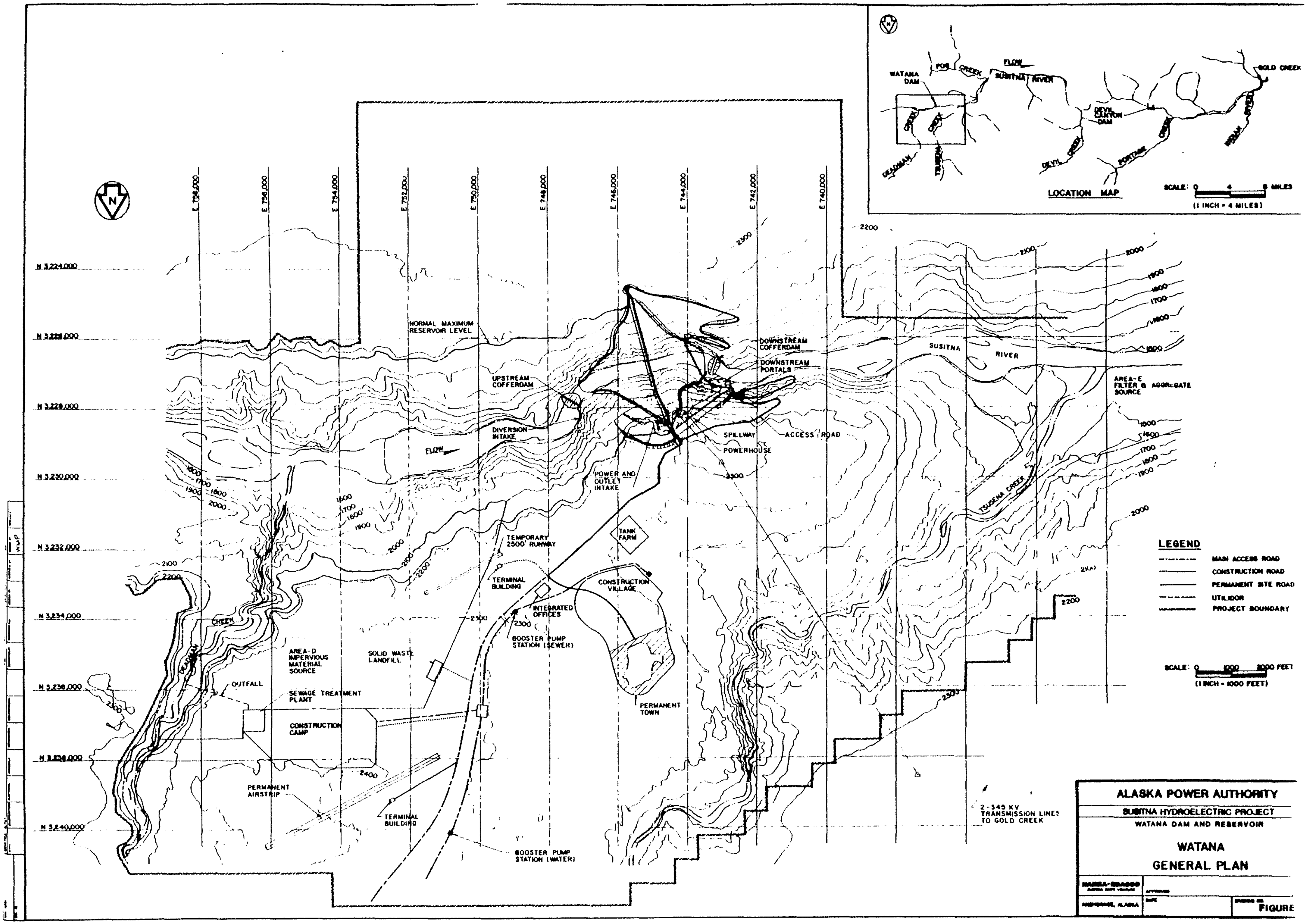
5. "Open-Channel Hydraulics", by Ven Te Chow, McGraw-Hill Book Company, New York City.

6. U.S. Geological Survey, Circular No. 284, 1953 Computation of Peak Discharge at Contractions C.E. Kindwater, R.W. Carter, H.J. Tracy.

7. Backwater Effects of Open Channel Constrictions, Transactions, ASCE, Vol. 120, pp. 993-1006, 1955.

APPENDIX A

PROJECT DRAWINGS



ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

WATANA DAM AND RESERVOIR

WATANA
GENERAL PLAN

FIGURE

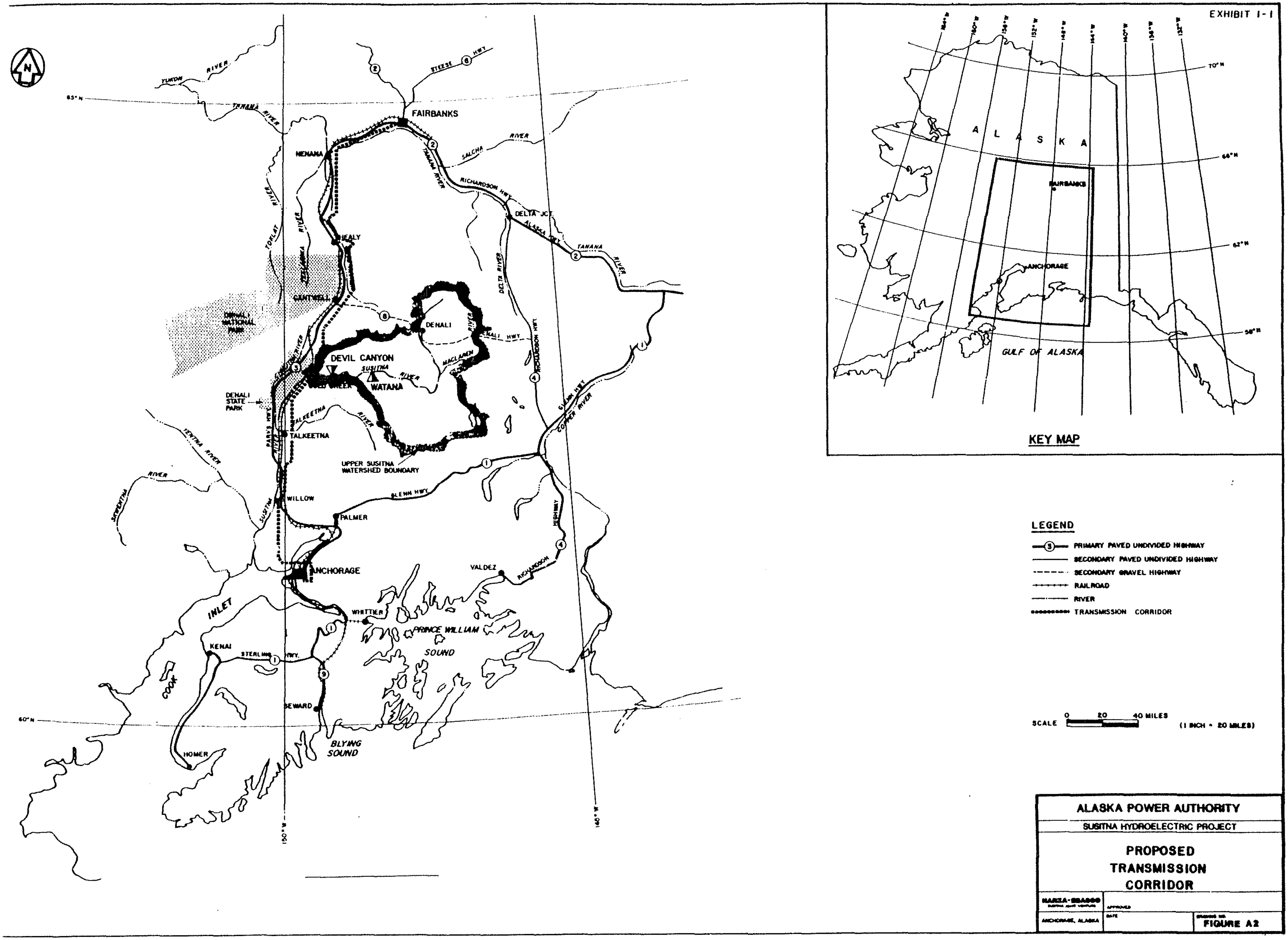
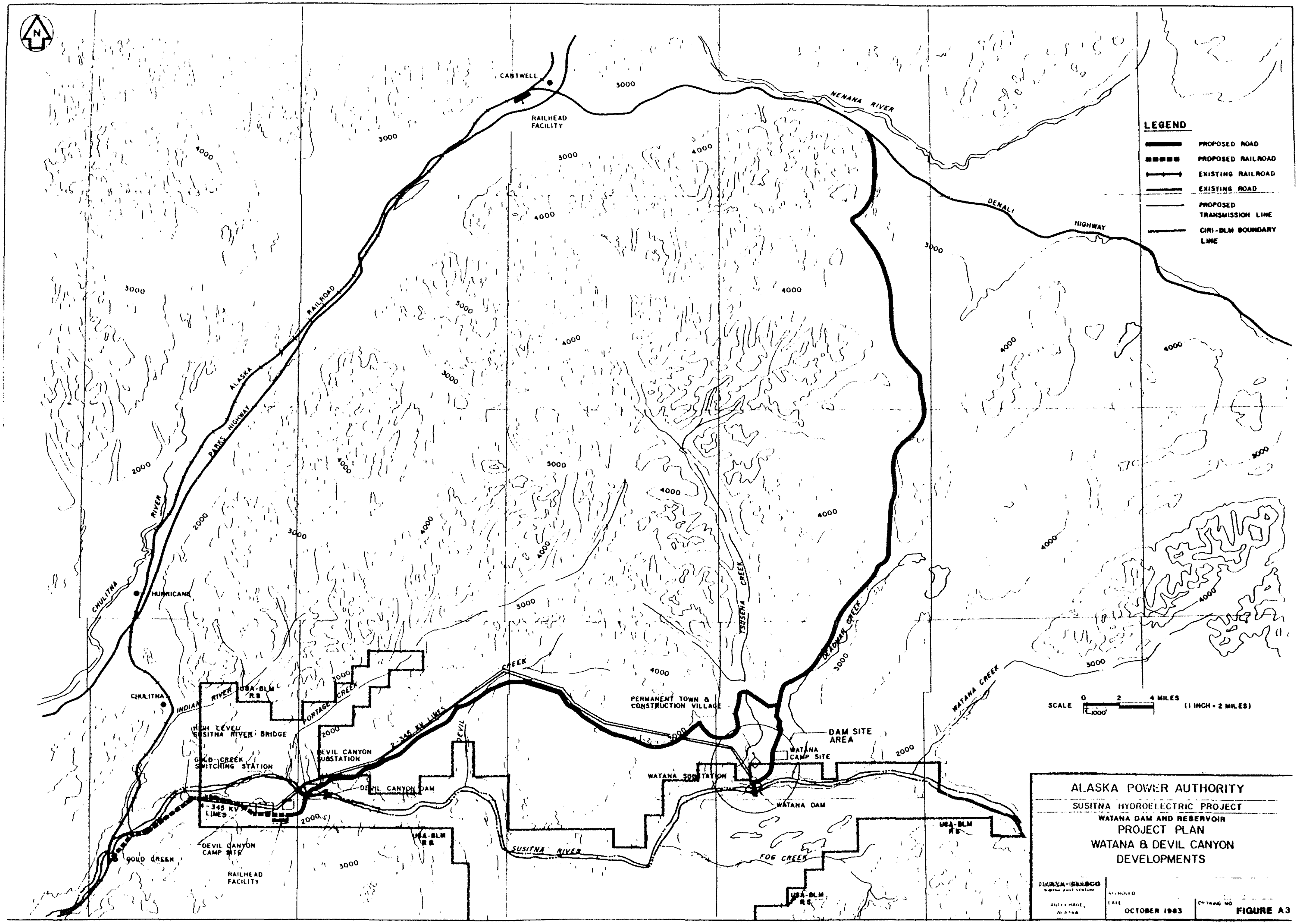
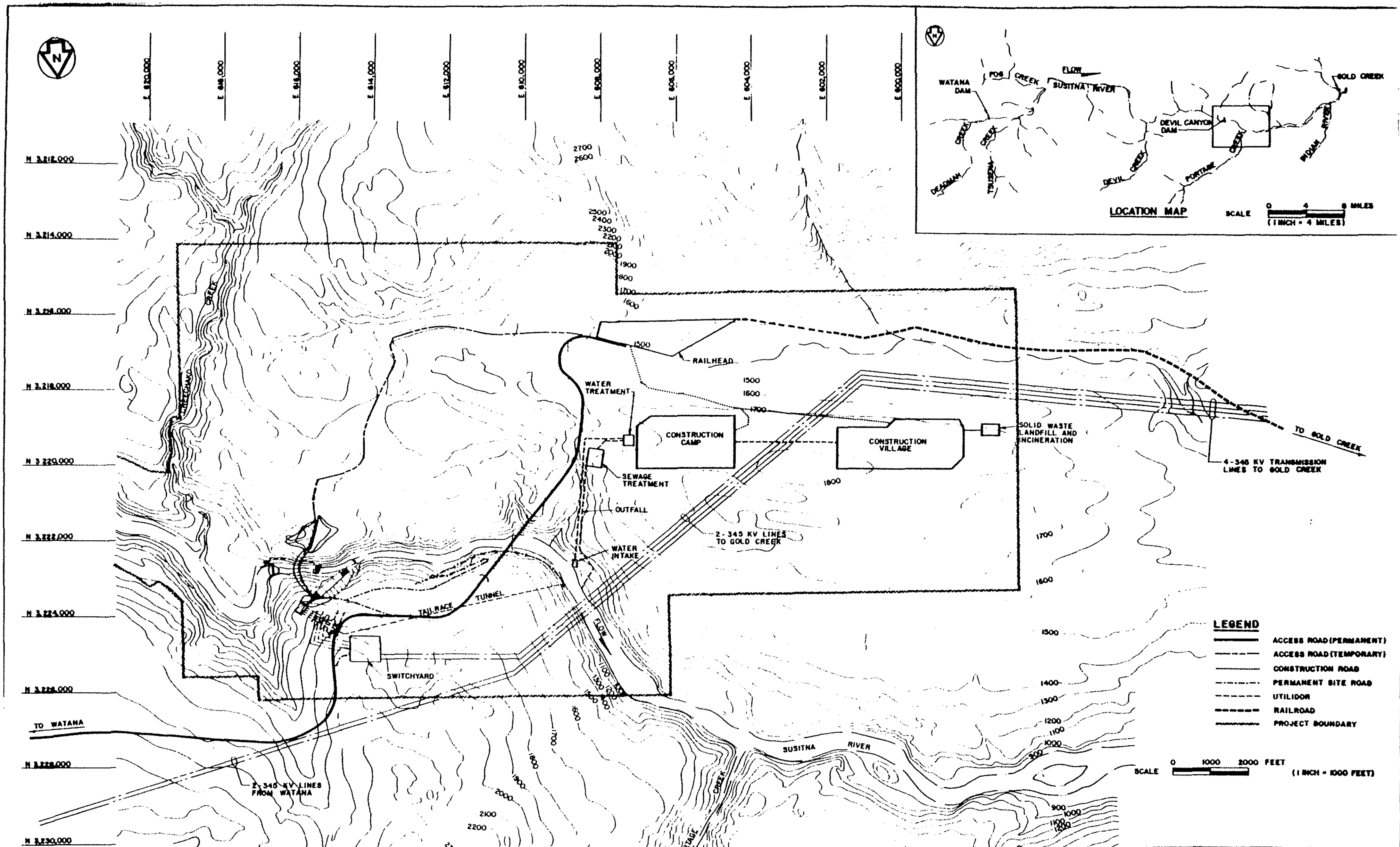


EXHIBIT I-I
FAIRBANKS
NENANA
HEALY
CANTWELL
DENALI NATIONAL PARK
DENALI
DEVIL CANYON
WATANA
DENALI STATE PARK
TALKEETNA
UPPER SUSITNA WATERSHED BOUNDARY
WILLOW
PALMER
ANCHORAGE
VALDEZ
WHITTIER
PRINCE WILLIAM SOUND
KENAI
STERLING HWY.
SEWARD
HOMER
COOK INLET
BLYING SOUND
DELTA JCT.
YUKON RIVER
TANANA RIVER
SALCHA RIVER
RICHARDSON HWY
ALASKA HWY
DELTA RIVER
DENALI HWY
MACLAREN RIVER
SUSITNA RIVER
TALKEETNA RIVER
YENTNA RIVER
SKWENTNA RIVER
GLENN HWY
COPPER RIVER
STEESE HWY
PARK'S HWY
150° W
146° W
60° N
ALASKA
FAIRBANKS
ANCHORAGE
GULF OF ALASKA
KEY MAP
LEGEND
PRIMARY PAVED UNDIVIDED HIGHWAY
SECONDARY PAVED UNDIVIDED HIGHWAY
SECONDARY GRAVEL HIGHWAY
RAILROAD
RIVER
TRANSMISSION CORRIDOR
SCALE 0 20 40 MILES (1 INCH = 20 MILES)
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
PROPOSED TRANSMISSION CORRIDOR
HARZA-EBASCO
ANCHORAGE, ALASKA
FIGURE A2


LEGEND
PROPOSED ROAD
PROPOSED RAILROAD
EXISTING RAILROAD
EXISTING ROAD
PROPOSED TRANSMISSION LINE
CIRI-BLM BOUNDARY LINE
CANTWELL
RAILHEAD FACILITY
NENANA RIVER
DENALI HIGHWAY
ALASKA RAILROAD
PARKS HIGHWAY
CHULITNA RIVER
HURRICANE
CHULITNA
INDIAN RIVER
PORTAGE CREEK
HIGH LEVEL SUSITNA RIVER BRIDGE
GOLD CREEK SWITCHING STATION
DEVIL CANYON SUBSTATION
DEVIL CANYON DAM
DEVIL CANYON CAMP SITE
GOLD CREEK
RAILHEAD FACILITY
SUSITNA RIVER
FOG CREEK
TSUSENA CREEK
DEADMAN CREEK
WATANA CREEK
PERMANENT TOWN & CONSTRUCTION VILLAGE
DAM SITE AREA
WATANA CAMP SITE
WATANA SUBSTATION
WATANA DAM
USA-BLM R.S.
SCALE 0 2 4 MILES (1 INCH = 2 MILES)
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
WATANA DAM AND RESERVOIR
PROJECT PLAN
WATANA & DEVIL CANYON DEVELOPMENTS
OCTOBER 1983
FIGURE A3



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SUSITNA HYDROELECTRIC PROJECT

WATANA DAM AND RESERVOIR

DEVIL CANYON

GENERAL PLAN

HARZA-EBASCO

ANCHORAGE, ALASKA

FIGURE A4

APPENDIX B

RAINFALL FREQUENCY DATA FOR ALASKA

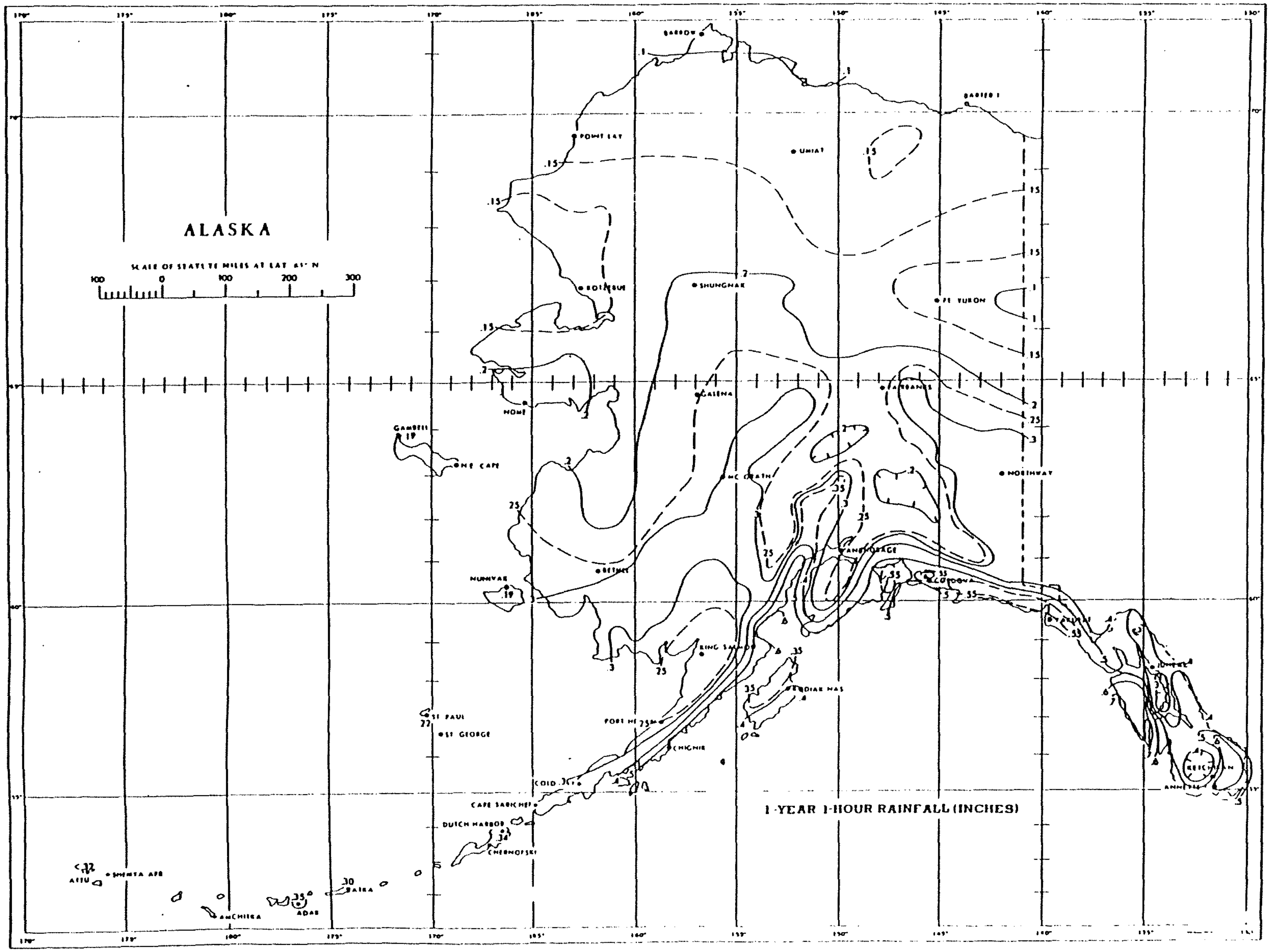

FIGURE B1 —1 yr. 1 hr. rainfall (in.).

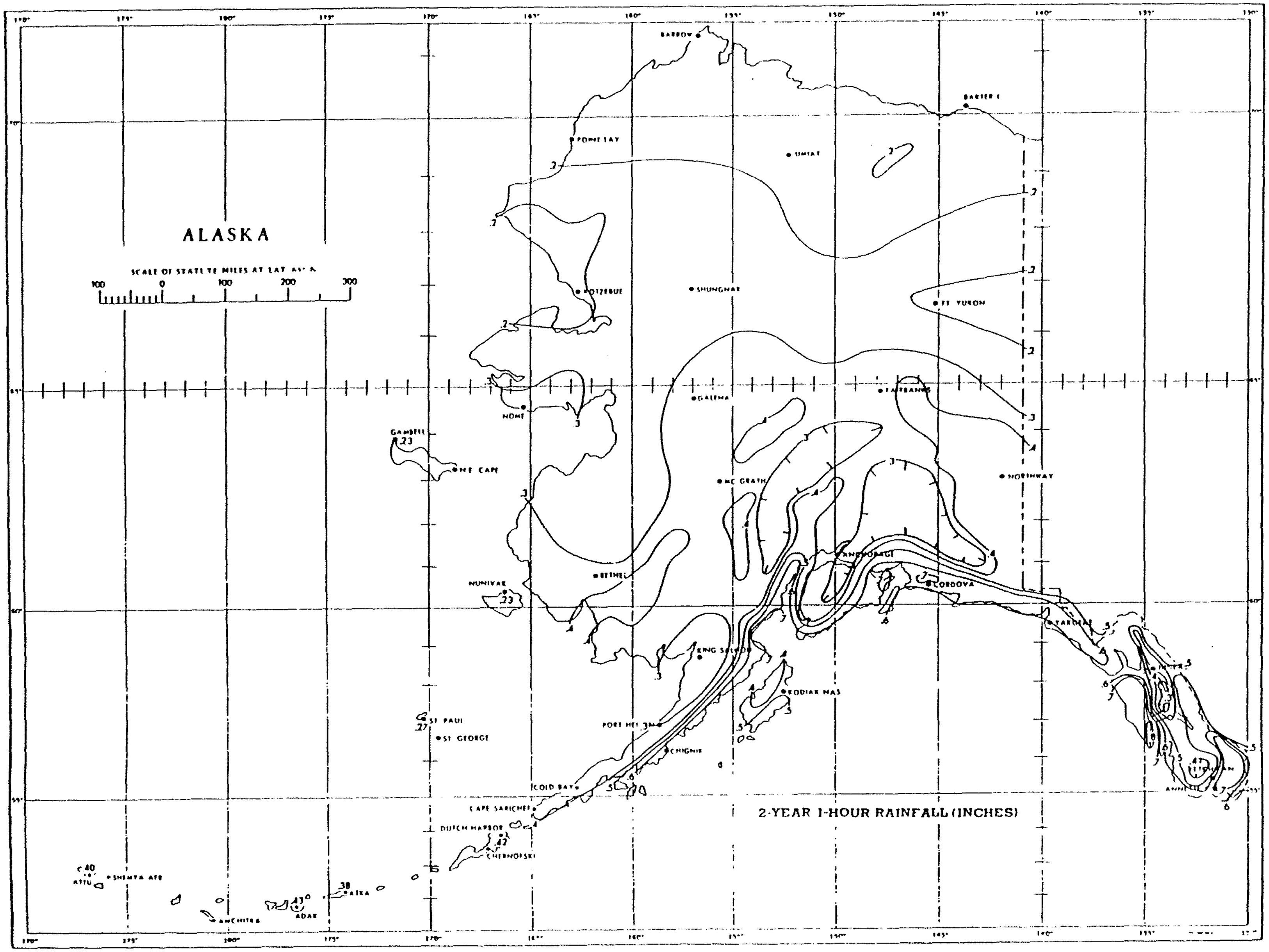

FIGURE B2 --2 yr. 1-hr. rainfall (in.).

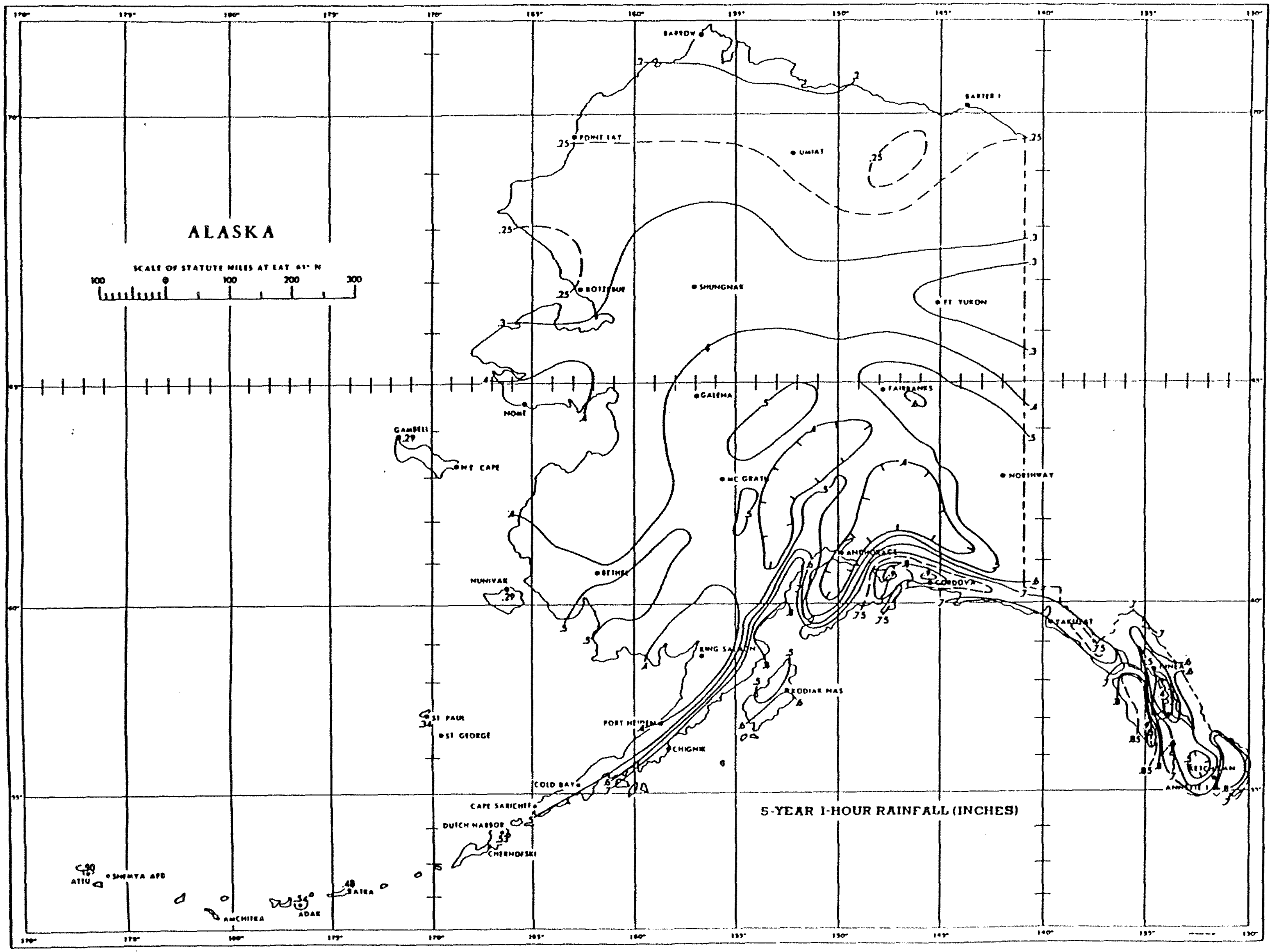

FIGURE B3 —5-yr. 1-hr. rainfall (in.).

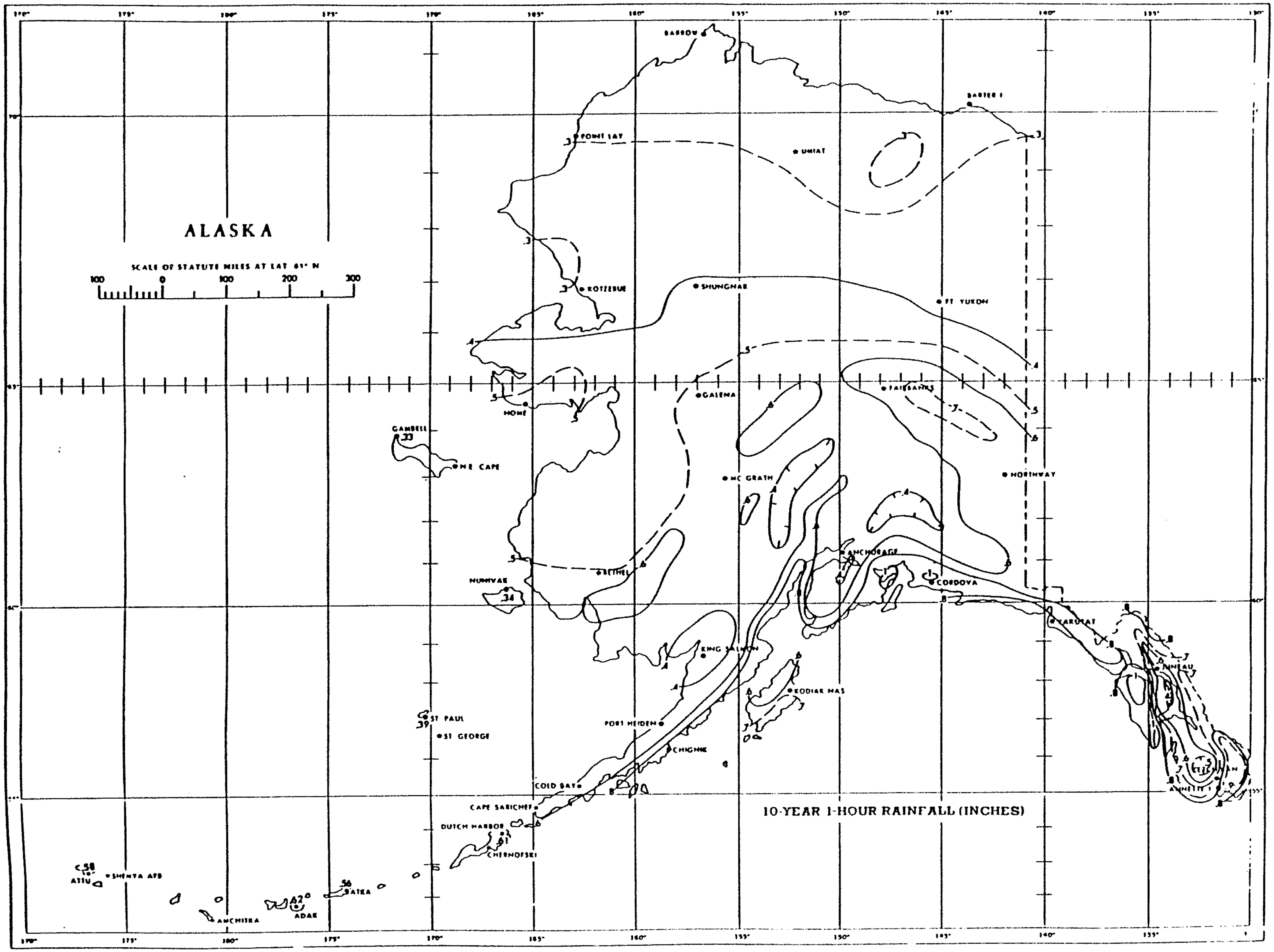

FIGURE B4 —10-yr. 1-hr. rainfall (in.).

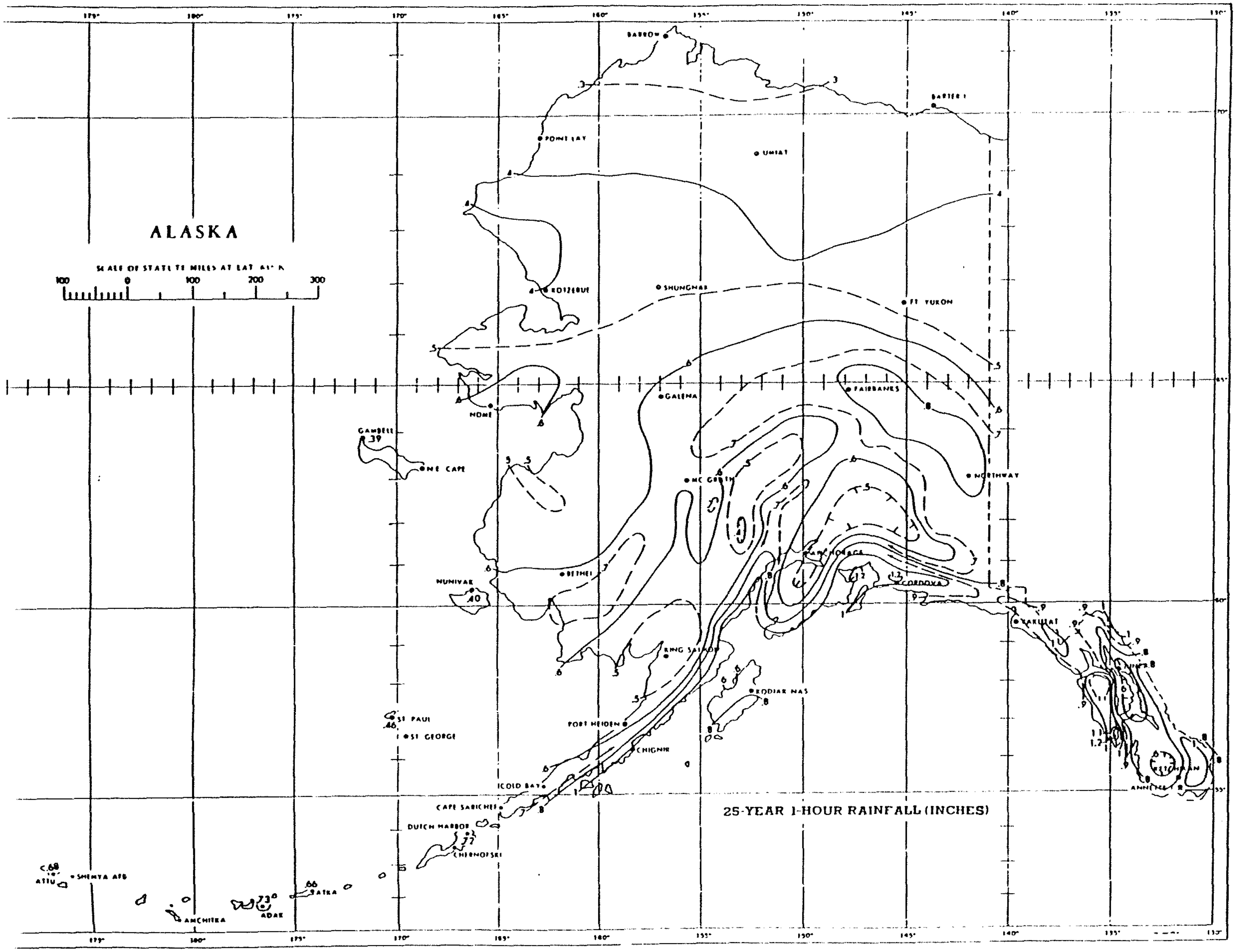

FIGURE B5 —25-yr. 1-hr. rainfall (in.).

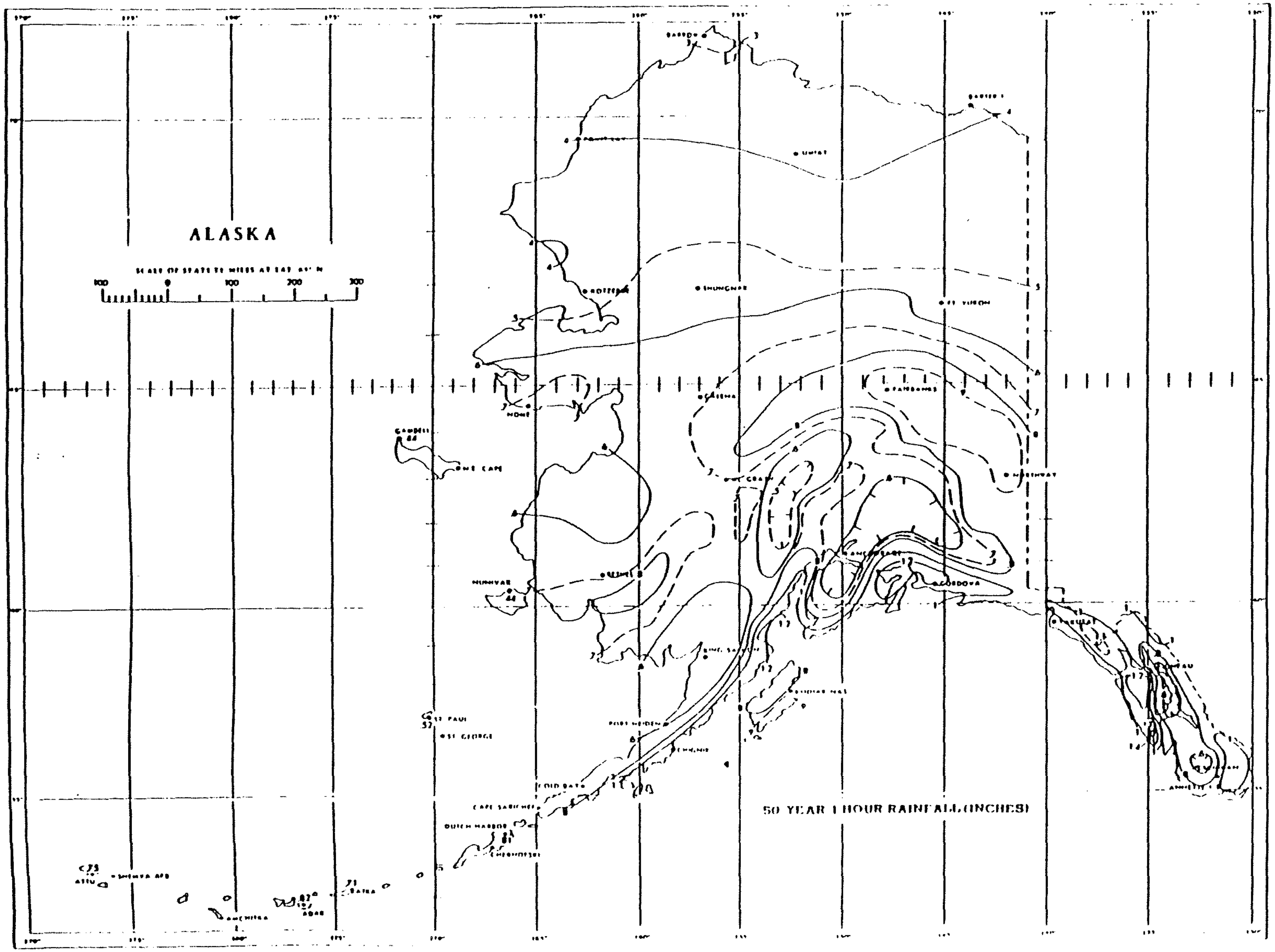

FIGURE B6 —50 yr 1 hr rainfall (in.)

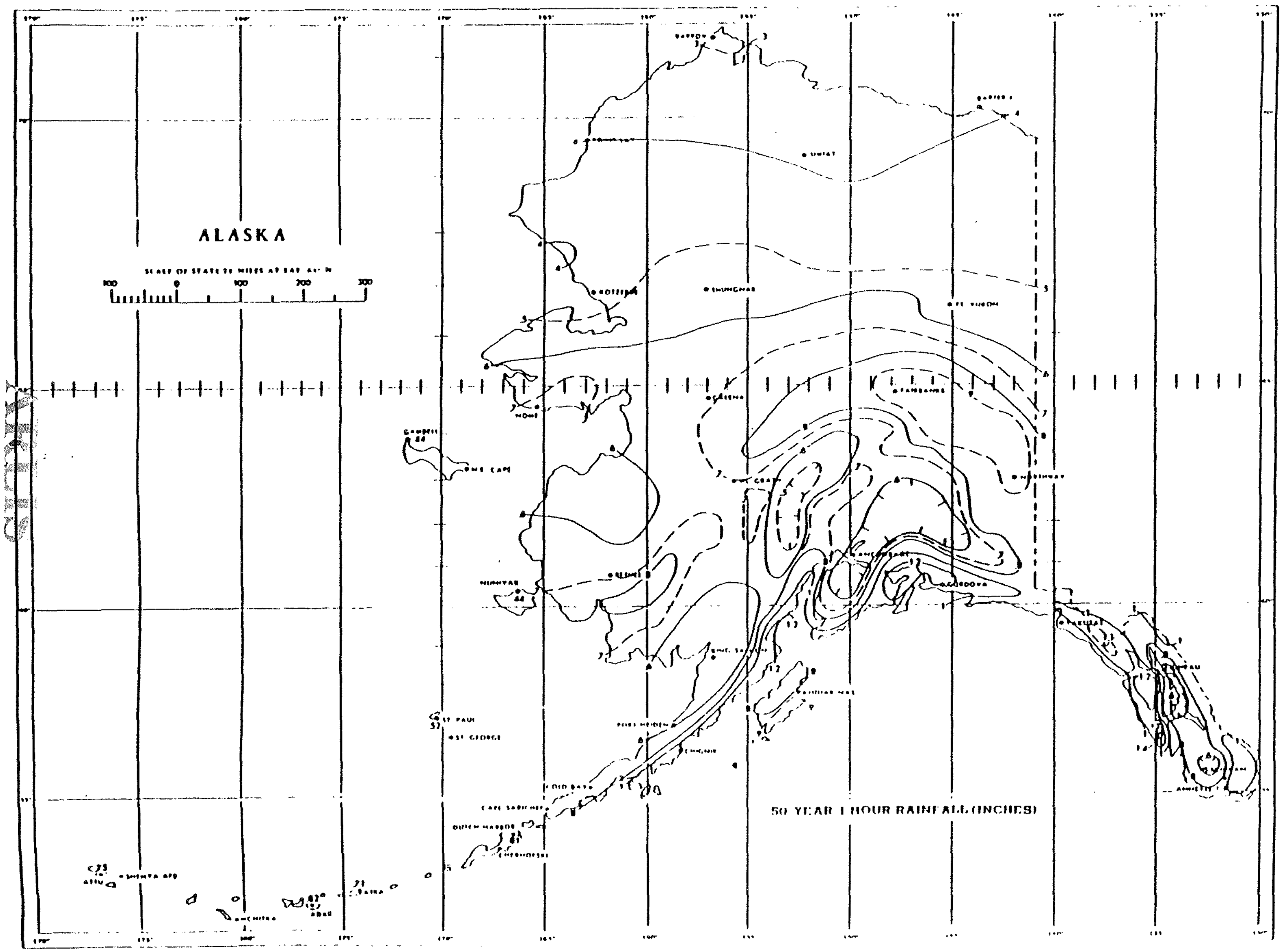

FIGURE B6 —50 yr 1 hr rainfall (in)