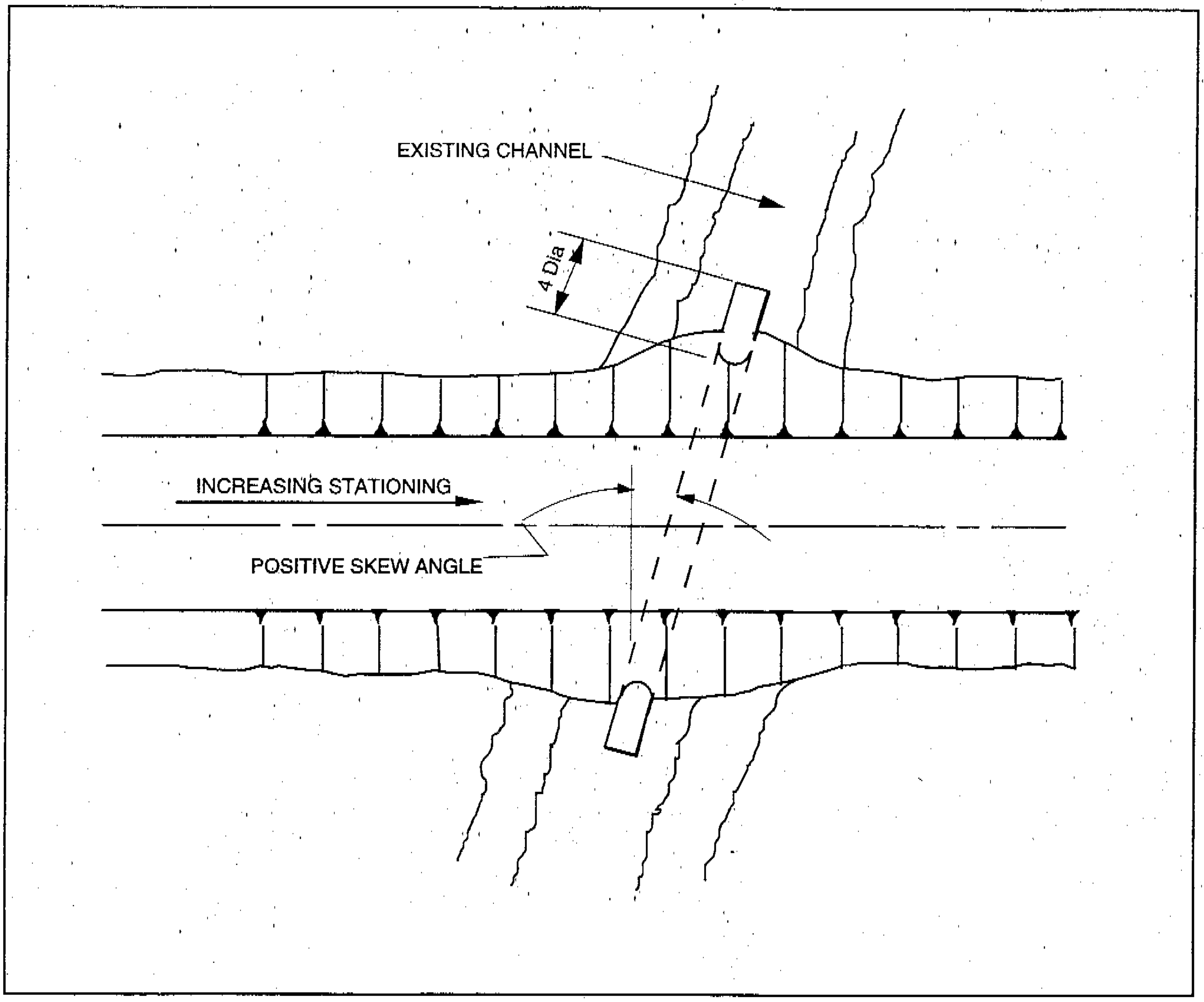

Stream Crossing Design Procedure for Fish Streams on the North Slope Coastal Plain

by
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BP EXPLORATION

Alaska Department
of Environmental Conservation

STREAM CROSSING DESIGN PROCEDURE FOR FISH STREAMS ON THE NORTH SLOPE COASTAL PLAIN OF ALASKA

June 1994

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1. PURPOSE

Improper design and installation of cross-drainage structures in streams can result in two major problems. First, barriers to the free movement of fish may be created, and second, increased maintenance may be required. This is especially true in the oil fields on Alaska's North Slope, where ice and snow accumulations and discharges during spring breakup can cause significant problems for such structures.

This manual was developed to help North Slope operators avoid these problems by providing a consistent set of design and installation standards for culverts, bridges, and pipeline crossings of fish streams. It is hoped that the manual will be used by all North Slope operators. These standards should reduce the impact of such structures to wetlands and waters and should reduce permitting and project review times for both industry and state agencies. Figure 1-1 provides a summary decision diagram to guide the stream crossing design process.

This project involved the development of a cross-drainage-structure design manual with standardized specifications and procedures for fish stream crossings on the North Slope. The project was identified as a priority in the Alaska Department of Environmental Conservation's (ADEC's) 1990 Nonpoint Source Pollution Control Strategy developed under Section 319 of the Clean Water Act. The need for the manual of standardized procedures and design standards for fish stream crossings to reduce nonpoint source pollution and improve fish passage was identified by the Nonpoint Source Oil and Gas Working Group consisting of industry, borough, and agency representatives. The manual was funded through a Section 319 nonpoint source grant provided by ADEC and the U.S. Environmental Protection Agency and through a matching amount of funding provided by BP Exploration (Alaska) Inc.

This project is a companion task to a project completed by the Alaska Department of Fish and Game (ADF&G) under the same work plan. The ADF&G project involved surveying ten stream crossings and verifying the success of fish-habitat remedial actions undertaken by industry at five high-priority stream crossings. The database and conclusions generated from the ADF&G effort were intended to, and have, supported improvements in this manual of design criteria and specifications for North Slope fish stream crossings as identified by ADF&G in 1989.

This manual is divided into the following major sections:

1. Purpose
2. Biological Background
3. Route Planning and Design Criteria
4. Assembly and Review of Existing Data
5. Calculations of Design Discharge
6. Evaluation of Channelized Flow
7. Design and Installation of Culverts
8. Design and Installation Considerations for Bridges
9. References

The following federal and state laws and regulations are among those that apply to construction of stream crossings on the North Slope:

Federal

- United States Code, Title 16 - Conservation (Fish and Wildlife Coordination Act of 1934, as Amended)
- Code of Federal Regulations, Title 33 - Navigation and Navigable Waters
- Executive Order 11988 - Floodplain Management Guidelines
- Code of Federal Regulations, Title 18 - Conservation of Power and Water Resources
- Code of Federal Regulations, Title 40 - Protection of the Environment, Part 125, Discharge into Water (National Pollutant Discharge Elimination System)
- Code of Federal Regulations, Title 43 - Public Lands, Interior
- Public Law 92-500, Sections 840 and 870 - Clean Water Act

State of Alaska

- Alaska Statutes, Title 16.05 - Fish and Game Code
- Alaska Statutes, Title 46.03 - Environmental Conservation
- Alaska Statutes, Title 46.15 - Water Use Act
- Alaska Administrative Code, Title 5, Fish and Game
- Alaska Administrative Code, Title 5, Fish and Game, Chapter 95
- Alaska Administrative Code, Title 18 - Environmental Conservation, Chapter 70, Water Quality Standards, and Chapter 72, Waste Water Disposal

NOTE:
Section and figure references are to this report

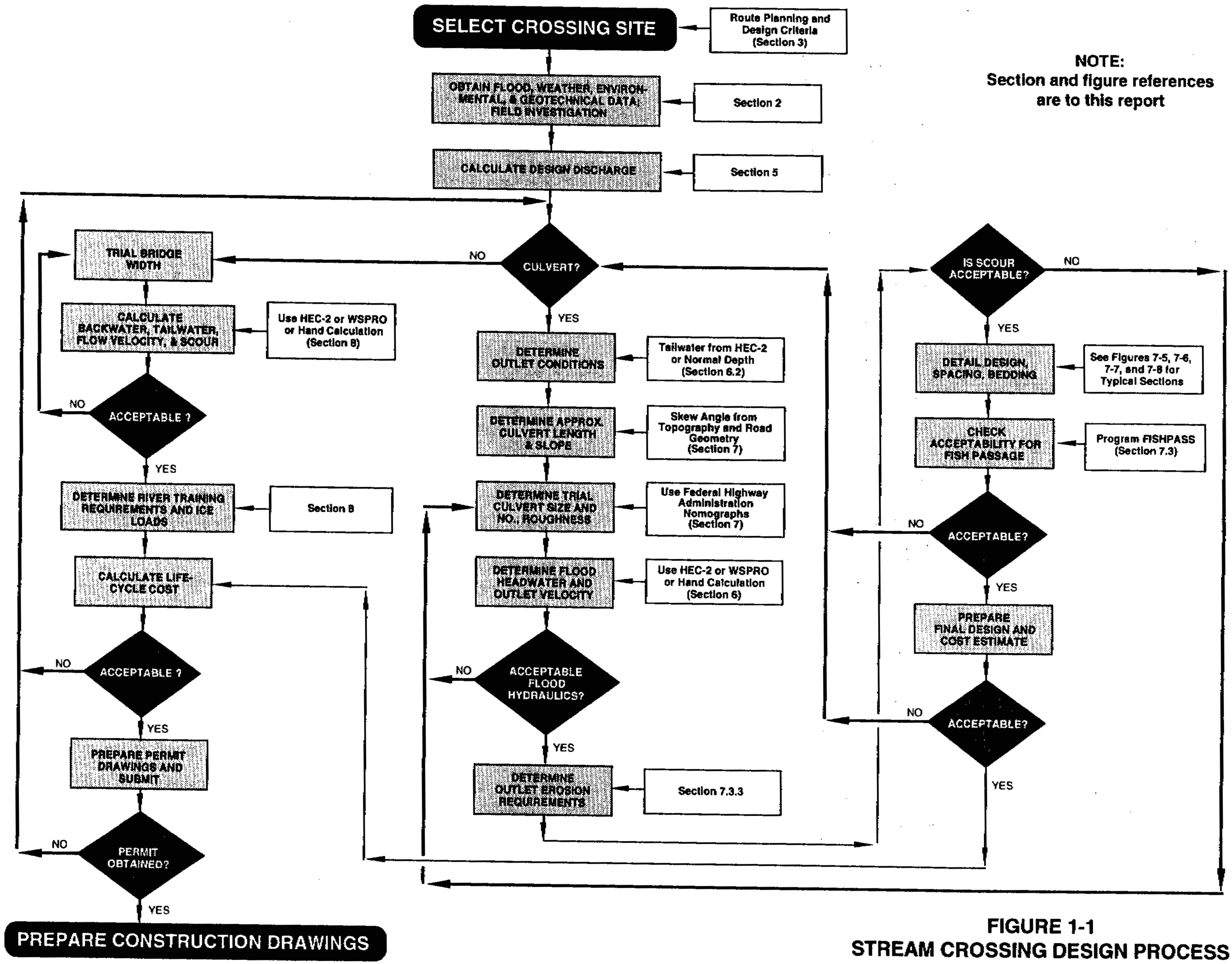


FIGURE 1-1
STREAM CROSSING DESIGN PROCESS

2. BIOLOGICAL BACKGROUND

Key biological factors for stream crossing design are the fish species present, migration timing, distance to upstream spawning areas, and swimming capabilities and behavior of the fish when stressed to the limit by a culvert. The Fairbanks office of the Habitat and Restoration Division of the Alaska Department of Fish and Game (ADF&G) collects relevant data. Because data are not currently available for all North Slope streams, it is necessary to inform ADF&G of proposed routes sufficiently far in advance of project design so that ADF&G can acquire adequate information. At least one year should be allowed where possible so that ADF&G can survey the streams during both winter and summer. ADF&G may select a design species and time of migration for each stream crossed; this information provides the basis for planning and design of culverts. ADF&G will also identify fish spawning and overwintering areas which must be avoided.

2.1 FISH SWIMMING ANATOMY

Fish swim using two separate muscle systems. The red muscle system, which is similar to that of humans and is used for longer-term activities, functions in an aerobic state. These muscles are used for continuous, slow swimming requiring low power output. As in human muscles, the power output of red muscles in fish decays after long periods of use. White muscles, on the other hand, provide elevated power for very short durations. Severe white muscle usage leaves the fish in a state of white muscle exhaustion requiring long periods of rest to recuperate (Behlke 1987).

In passing a typical culvert, a fish may use white muscles to enter the difficult high-velocity water at the culvert outlet and red muscles to swim through the slower water of the culvert barrel. If a second region of high-velocity water is encountered, the fish may not be able to negotiate it because the energy stored in the white muscle system is depleted.

Behlke has observed that although fish may be capable of expending swimming energy at certain maximum rates, they may not choose to do so when confronted with specific obstacles. Behlke speculates that fish swimming in culvert barrels of unknown extent attempt to minimize power output by consistently moving ahead slowly in the culvert, though they may be physically capable of moving ahead faster (Behlke 1987, Behlke et al. 1989). His theory is supported by field observations of fish passage through culverts taking longer than expected.

2.2 HYDRAULICS

Fish swimming through a culvert must expend enough power to overcome the hydraulic resistance offered by the flowing water while making headway through the culvert. Resistance, as recognized in engineering terms, consists of three components: profile drag force, gradient force, and the force of acceleration of the fish's mass (refer to Figure 2.1). The following discussion is a summary of the more detailed analysis contained in Behlke et al. (1991).

The most significant of these is *profile drag*, F_D, which is always opposed to the direction of the fish's motion. This force consists of both skin friction and pressure forms. It may be expressed as:

$$F_D = C_D \rho S V_{fw}^2 / 2 \qquad \text{Eq. 2.1 (Behlke et al. 1991)}$$

where,

C_D is the drag coefficient.

ρ is the mass density of water in which the fish swims.

S is the surface area of the fish.

V_{fw} is the swimming velocity of the fish with respect to the water.

For North Slope conditions, Equation 2.1 can be reduced to show that:

$$F_D \propto L^{1.8} V_{fw}^{1.8} \qquad \text{Eq. 2.2}$$

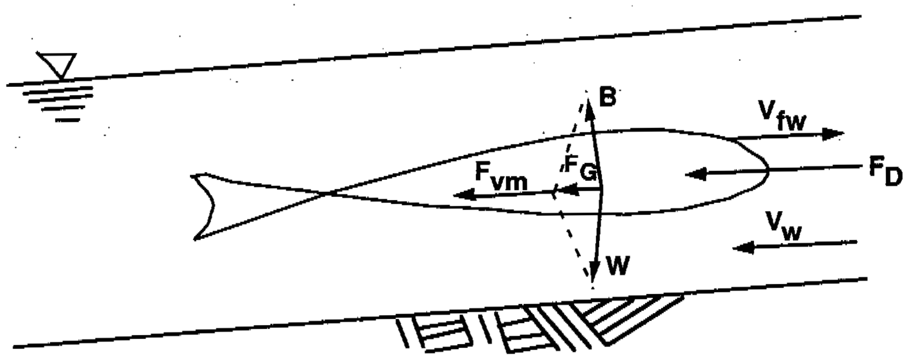
where L is the total length of the fish.

Thus, the drag force experienced by a fish is nearly proportional to the square of the fish's swimming velocity.

The second force opposing fish motion is the *gradient force*, F_G, which is the vector resultant of the fish's weight and its buoyant force with respect to the surrounding pressure gradient. The pressure gradient is normal to the hydraulic grade line discussed in Section 7.1. Behlke has shown that the gradient force, F_G, is :

$$F_G = W(\sin\phi + \cos\phi\,(\tan(\theta - \phi))) \qquad \text{Eq. 2.3}$$

where,



B	=	Buoyant force
W	=	Weight of fish
V_w	=	Velocity of water
F_G	=	Gradient force
F_{vm}	=	Virtual mass force
V_{fw}	=	Swimming velocity of fish with respect to water
F_D	=	Profile drag

Reference: Behlke et al. (1991)

FIGURE 2-1
FORCES ACTING ON A SWIMMING FISH

W is the fish's weight.

θ is the slope of the water velocity vector, or for open channels the slope of the hydraulic grade line, S_f.

ϕ is the angle with respect to horizontal along which the fish swims.

For small angles (less than 6°), sin ϕ and tangent ϕ are close to the slope, S_E, and the equation reduces to:

$$F_G = W S_E \qquad \text{Eq. 2.4}$$

where,

S_E is the slope of the energy grade line (see Section 7.1), which usually is approximated by the slope of the water surface S_w.

Where the slope of the flow channel (i.e., the slope of the culvert) is small, the gradient force is not significant. However, a change in elevation at the inlet or outlet of a culvert causes a fish to expend considerable energy in passing such an obstacle.

The remaining force acting on a swimming fish is the force due to acceleration of the fish's mass, or the *virtual mass force*, F_{vm}. This force is generated in accordance with Newton's Second Law. When a fish accelerates with respect to the surrounding water, this force is opposite to the direction of the fish's relative acceleration. The virtual mass force is expressed as:

$$F_{vm} = 1.2\ (W/g)\ a_{fw}$$

where

a_{fw} is the relative acceleration of the fish with respect to the surrounding water.

The constant 1.2 accounts for the added mass of that portion of water near the boundary that accelerates with the fish (Webb 1975).

The virtual mass force becomes significant where an obstacle exists that would require the fish to accelerate in order to overcome it. This could be at the inlet or outlet of a culvert, or a weir in a flow channel.

In designing a culvert for the passage of fish, one must consider the forces experienced by an upstream-migrating fish. The hydraulic conditions experienced where the fish actually swims must not overpower the fish's capabilities.

3. ROUTE PLANNING AND DESIGN CRITERIA

Early in the planning process, it is important to consult with the Alaska Department of Fish and Game (ADF&G) and the Alaska Department of Environmental Conservation (ADEC), as appropriate, so that they can gather fisheries and water quality information on the streams for which crossings are planned. At least one year should be allowed where possible. Failure to consult with ADF&G may result in expensive modifications to fish stream crossings after construction.

Many fish passage and water quality problems associated with road crossings of streams can be avoided by careful route planning based on the following guidelines:

- Avoid anadromous fish spawning habitat.
- Avoid fish overwintering habitat.
- Locate crossings in areas of minimal stream slopes and widths.
- Align road to provide a crossing at right angles to stream flow.
- Avoid crossing in areas where icings occur.
- Consider all life-cycle costs in deciding between culverted and bridged crossings.
- Consider future access needs so that new roads are minimized.
- Minimize the number and size of stream crossings.
- For larger fish streams, consider both bridges and culverts.
- Consider maintenance and abandonment costs.
- Investigate proposed routes early on the ground with appropriate personnel to identify environmental factors (acceptability of crossing sites; the species, timing of fish populations which must be accommodated; etc.).
- Locate road alignments on drainage divides where possible.
- Locate stream crossings where the stream is well incised and not prone to diversion.
- Consider the effects of blockage and diversion of streams by snow drifts formed at elevated crossing structures.
- Analyze the height of embankment necessary to prevent overtopping if drainage structures become blocked.

Hydraulic and environmental factors should be considered when a new route is first proposed because they may have an important bearing on costs and environmental feasibility. Once the route is selected, stream crossing alternatives can be evaluated in more detail.

Once a route has been determined, the design criteria must be identified before the detailed design of the crossing can proceed. One must systematically consider each criterion. It is also necessary to re-evaluate previous decisions against newly developed information as the design develops. If necessary, steps may be repeated. Engineers will approach this process in a variety of ways and may modify or skip steps as appropriate.

The design procedures presented in this manual are based on standard procedures that have been tailored to North Slope conditions. Alternative designs are provided. The final choice among these alternatives is the most economic design which meets the following criteria:

- ***Vehicle Requirements.*** Includes wheel loads, lane widths, speed, sight distances, etc.
- ***Access and Flood Passage.*** Provides access to project facilities for vehicles and personnel during floods of up to a 50-year return period, unless, after operational needs are analyzed, access is necessary only at some lesser return period, or if temporary inaccessibility from time to time is acceptable.
- ***Effectiveness.*** Provides standard design and specifications which allow for effective maintenance of drainage and erosion control features during the project life and which minimize failure and replacement.
- ***Environmental.*** Minimizes disturbances to wetlands and critical fish and wildlife habitat and ensures free passage of fish.
- ***Erosion.*** Minimizes erosion and thermal degradation of permafrost.
- ***Icings.*** Minimizes creation of icing problems and accommodates expected icing in design.
- ***Snow Drifting.*** Minimizes snow drifting that can block culverts.
- ***Disruption.*** Prevents unnecessary alterations to surface water hydraulics or configurations.
- ***Navigation.*** Permits passage of small craft on navigable streams.
- ***Construction.*** Includes temporary drainage structures to provide access during construction; these are to be removed when construction is done.
- ***Compliance.*** Complies with all applicable codes and regulations.

4. ASSEMBLY AND REVIEW OF EXISTING DATA

Once the general location or range of locations for a route with stream crossings has been selected, all available information pertinent to the study should be assembled, reviewed, and evaluated (Table 4-1). The results of this evaluation will identify the need for gathering further information.

A multidisciplinary team experienced in route location, hydraulics, and fisheries biology should investigate the route during the early studies of conceptual design. ADF&G should be informed of routes considered so that they can gather necessary data in a timely manner. Field investigations, which collect necessary data not available from office studies, usually include surveying river cross-sections at and near proposed crossing locations. Surveys should extend about ten channel diameters (widths) both upstream and downstream of a proposed crossing to accurately define the stream's hydraulic characteristics. All existing high-water marks from past floods should be located and surveyed. Field investigations may include soil borings to determine foundation conditions.

TABLE 4-1
ASSEMBLY AND REVIEW OF EXISTING DATA

DATA	EXPLANATION
Maps, Charts, & Aerial Photos	Valuable information on channel stability can often be obtained by comparing old maps or photographs to recent ones.
Existing Crossings	The type, dimensions, and past performance of nearby stream crossings, particularly on the same stream, will yield the most valuable design information for new crossings. Maintenance and environmental personnel should be interviewed and problems carefully noted.
Channel Geometry	Representative hydraulic geometry of streams to be crossed should be determined from existing mapping and air photos. Pertinent factors include cross-sectional dimensions of the channel and floodplain, overbank height, meander radius, planform, channel slope, location of hydraulic control points such as rapids or constrictions, and particularly the location of points from which backwater may divert from the channel or endanger property. The investigation should also consider channel stability.
Water Level & Discharge Data	Stream flow and flood data from the methods provided in Section 5 should be supplemented with site-specific investigations.
Environmental	Species of fish present and regulatory requirements for passage may be obtained from the Habitat and Restoration Division of the Alaska Department of Fish and Game. Regulatory requirements may include specifications to facilitate fish passage and limitations on times or methods of construction. Environmental requirements are a mandatory part of crossing design.
Ice & Debris	Local evidence of ice thickness and dimensions of moving sheets, as well as dates of breakup and freezeup, are necessary to determine ice loads on piers or to plan maintenance requirements for culverts or bridges.
Snow & Snow Drifting	The local tendency for snow drifting must be evaluated to minimize the need for pre-breakup maintenance.
Coastal Processes	For stream crossings near the coast, data are needed on past lunar and estimated future storm-tide levels, as well as wave heights and runup. Storm surge estimates are provided by Wise et al. (1981).
Geotechnical	Excavation and pile driving records and soil test results from existing nearby structures should be collected for a description of bed and bank materials and evidence of past scour levels.
Climatological	Climatological data are necessary to evaluate design parameters in regions where poor records of historic stream-flow events exist. The data should include precipitation duration and intensity estimates, which are useful for flood and runoff estimation. Wind data may be necessary for wave height estimation, while snowfall and temperature data are used for estimating snowmelt flood. This information is normally available from the Arctic Environmental Information and Data Center, part of the University of Alaska Anchorage, as well as many other sources.
Land Use	Present and postulated land use are important because development of a watershed increases the rate of runoff and the supply of sediment to streams. Crossing design should provide for anticipated future changes in the watershed and for the impact of backwater on land use.
Flooding Limits	Determine the height and susceptibility to flood damage of property that may be damaged by backwater, either water or ice, which a stream crossing may induce.

5. CALCULATION OF DESIGN DISCHARGE

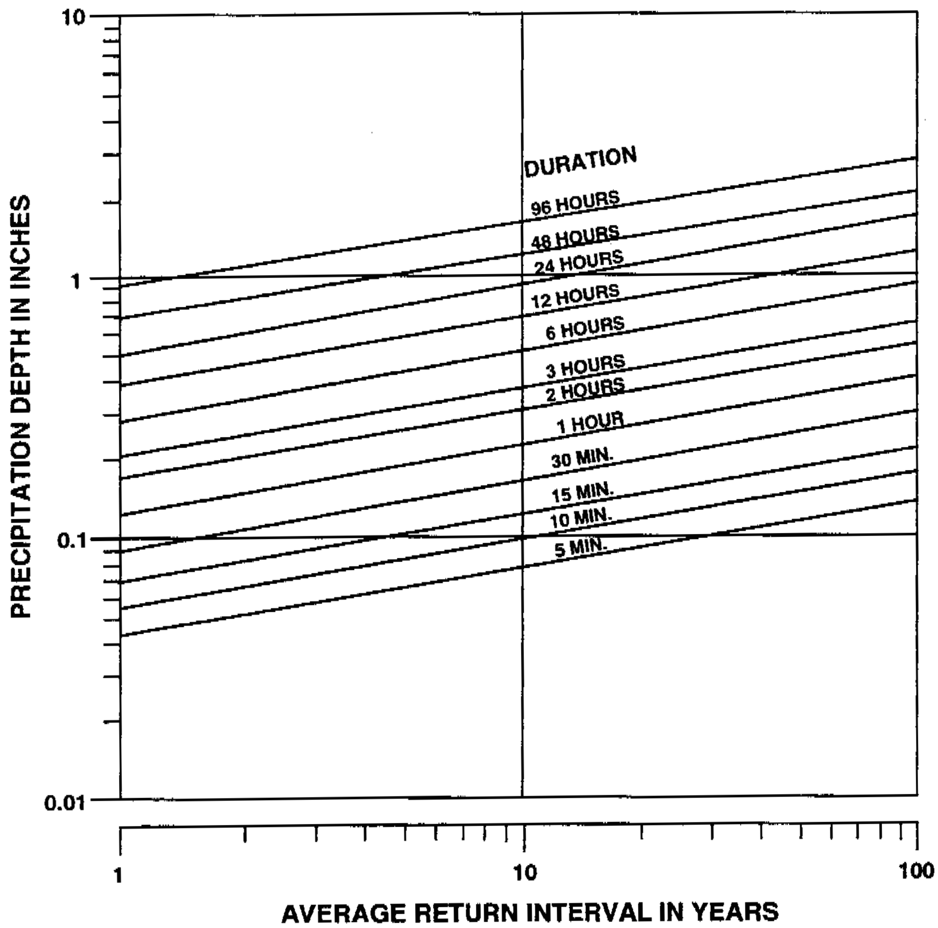
The first step in designing a crossing is to determine the amount of water that must pass through the crossing. This section provides an overview of the climate and hydrology of the Arctic Coastal Plain of northern Alaska and gives methods for determining design discharges.

5.1 GENERAL HYDROLOGY

The Arctic Coastal Plain is a region of low relief dominated by shallow drained-lake basins, wind-oriented lakes, and ice-wedge polygons. The surface, which tends to be covered with ice-rich sands and silts, has been altered by both wind and water erosion and elevated by the formation of ground ice. During rainfall or snowmelt events, the first runoff is as sheet flow. Because of the frozen ground, infiltration is practically nonexistent. The first minor streams, called first-order stream channels, begin with the melting of near-surface ground ice, normally along the boundaries of ice-wedge polygons. These first drainages do not erode soil material to form their channels; rather, they are formed solely by the subsidence of soils due to the melting of ground ice. As the streams combine and grow larger, they develop the capacity to erode their bed and banks and to transport sand and gravel. Arctic Coastal Plain streams of all sizes do not display the regular stream forms of temperate zone streams. Lateral erosion is controlled by frozen ground and, during early breakup, by snowdrifts. Since the most important factor controlling erosion is the frozen state of the soils, the most effective action to prevent erosion is to protect that state.

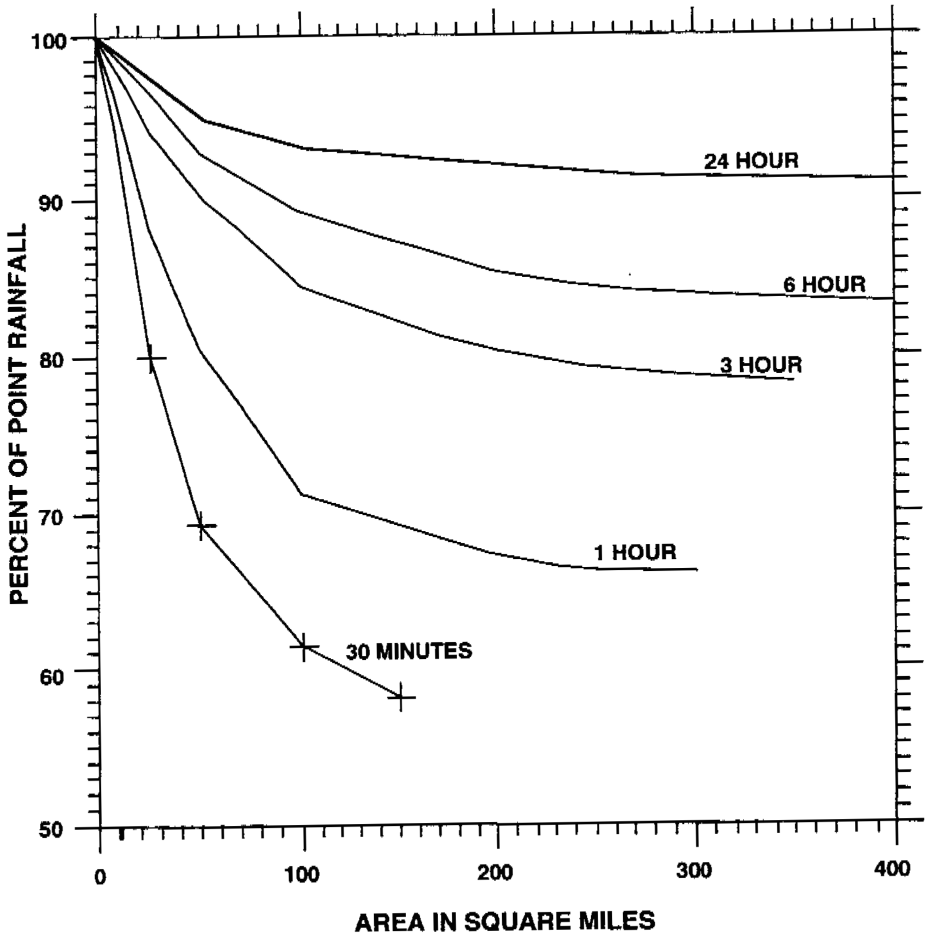
5.2 PRECIPITATION INTENSITY, FREQUENCY, AND DURATION

Precipitation rates on the Arctic Coastal Plain are low compared with most areas of Alaska. The most severe storms result from the movement of large-scale frontal systems from the southwest. Thunderstorms and associated intense precipitation bursts are known but rare (Ohtake 1979). Figure 5-1 provides probability estimates of point precipitation depths to be expected for selected time durations on the coastal North Slope. Figure 5-2 provides a method of adjusting this point precipitation to averages over larger areas, while Table 5-1 gives an estimate for each month of the average depth of precipitation at Prudhoe Bay.



References: U.S. Weather Bureau (1963)
Miller (1965)

FIGURE 5-1
DEPTH, DURATION, AND FREQUENCY OF PRECIPITATION



Reference: U.S. Weather Bureau (1963)

FIGURE 5-2
ADJUSTMENT OF POINT RAINFALL FOR INCREASED AREA

TABLE 5-1
ESTIMATED AVERAGE MONTHLY PRECIPITATION
AT PRUDHOE BAY

MONTH	PRECIPITATION (inches)
January	0.5
February	0.5
March	0.4
April	0.5
May	0.4
June	0.8
July	2.0
August	2.3
September	1.3
October	1.2
November	0.7
December	0.4
ANNUAL	11.0

NOTE: Precipitation estimates above are based on the limited data available from the U.S. Soil Conservation Service (SCS) Wyoming-type snow gauge located at Prudhoe Bay near the intersection of the Spine Road and Putuligayuk River Road. The catch of precipitation by this gauge is roughly double the catch recorded by conventional rain gauges, due to the inability of the conventional gauges to measure snowfall in windy environments. The data presented in this table were derived by correlating the average monthly precipitation measured at the SCS gauge with the long-term monthly distribution of annual precipitation measured at Barrow.

5.3 FLOODS

5.3.1 Snowmelt Floods

The annual spring breakup, which occurs around the beginning of June, provides the largest floods on North Slope coastal streams. Snowmelt flood peaks can be modeled analytically by considering the physics of snowmelt (Carlson et al. 1974). However, modeling North Slope breakup floods has not been encouraging as a predictive tool because of the sparsity of sufficiently detailed snowpack and meteorological data and because of the uncertainties caused by damming of meltwater by snowdrifts and ice. Design predictions can best be obtained through statistical analysis of past floods, as described in Section 5.4.

5.3.2 Rainfall Floods

Within the limited available experience, large summer-rainfall floods have not occurred on the smaller coastal plain streams of the North Slope; however, such floods do occur on the larger streams. The lack of rainfall floods on smaller streams is the result of the capacity of the seasonally thawed tundra and thaw lakes to store and retard runoff.

Flood peaks from rainfall can be modeled using several runoff models provided by the HYDRO computer program, part of the larger HYDRAIN program. Alternatively, HEC-1 is a similar U.S. Army Corps of Engineers program. (See references for source of programs.) The rainfall rates which are required input for these programs are provided in Figures 5-1 and 5-2.

5.3.3 Ice and Snow Blockage

During the long winter, thick sheets of ice form on larger streams and lakes; and smaller streams, which are normally dry, become blocked with snowdrifts. These winter ice and snow blockages play three important roles during breakup:

- They collect and release runoff from snowmelt.
- They decrease the channel area available to convey water, thus increasing the water level for a given discharge. More area is flooded and greater freeboard must be provided for riparian structures. The increased stage is also responsible for the third role:
- They may cause flow to be diverted between adjacent stream channels.

Construction of roads, pads, or above-ground pipelines across stream channels increases the tendency for ice or snow blockage at the crossing. Because stream blockages are often caused by snowdrifts at these crossings, such drifts, including those at both culvert inlets and outlets, should be cleared before breakup. The cost of preventing and removing snowdrifts from culverts and bridges should be recognized in plan formulation. In some cases, life-cycle costs may be reduced by using large multiplate culverts or bridges rather than several small culverts.

5.4 DESIGN FLOODS

5.4.1 Peak Discharge

Maximum annual discharges of small streams draining the coastal plain historically have resulted solely from snowmelt. Larger streams originating in the Brooks Range have both snowmelt and rainfall floods. The purpose of this section is to provide guidance for estimating the peak discharge for the small streams draining less than 200 square miles of the coastal plain.

The only continuously gauged stream draining only the coastal plain is the Putuligayuk River. The U.S. Geological Survey and ARCO Alaska, Inc. have recorded annual peak discharge there since 1970. A continuous record of daily average flow and minor peaks was obtained from 1970 to 1979; however, since 1979, only annual peaks have been acquired. Table 5-2 provides statistics of annual peak flow from that station derived by the method prescribed by the U.S. Water Resources Council (1981). By this analysis, data from one year, 1961, were disregarded as being too low. Because the record is not sufficiently long to reliably define the statistical skew, a skew of zero is assigned.

To facilitate crossing designs in the Kuparuk field, ARCO monitored a number of minor streams during breakup from 1978 to 1984. This program provided estimates of flood frequency for a few small streams with drainage areas ranging between 29 and 96 square miles. Drainages in the Kuparuk area typically have fewer lakes and ponds than most coastal plain streams. ARCO's 1984 Kuparuk Breakup Study provides a flood history for these streams (Drage et al. 1984).

A statistical relationship for these streams developed from ARCO data is shown on Figure 5-3 as a function of drainage area. Figure 5-4 provides an adjustment (developed by Jones and Fahl [1993]) accounting for the percentage of the drainage area in lakes and ponds. Used together, the two figures provide a conservative statistical estimate of annual peak discharge to be expected for coastal plain streams. This relationship may be used if site-specific data are not available; however, because of the wide spread in the 95 percent confidence shown in Table 5-2, site-specific data should be gathered if possible. This may be done by analyzing high-water marks from past floods, determining channel capacity, or gauging for a short time and correlating with data from the longer-record stations.

TABLE 5-2
FLOOD STATISTICS FOR THE
PUTULIGAYUK RIVER AT SPINE ROAD NEAR DEADHORSE

USGS Station	15896700
Drainage Area	176 square miles
Mean Logarithm, X	3.5019
Standard Deviation, σ	0.1823
Computed Skew, G	-0.2699
Adopted Skew, G	0.0000
Systematic Events, n	22

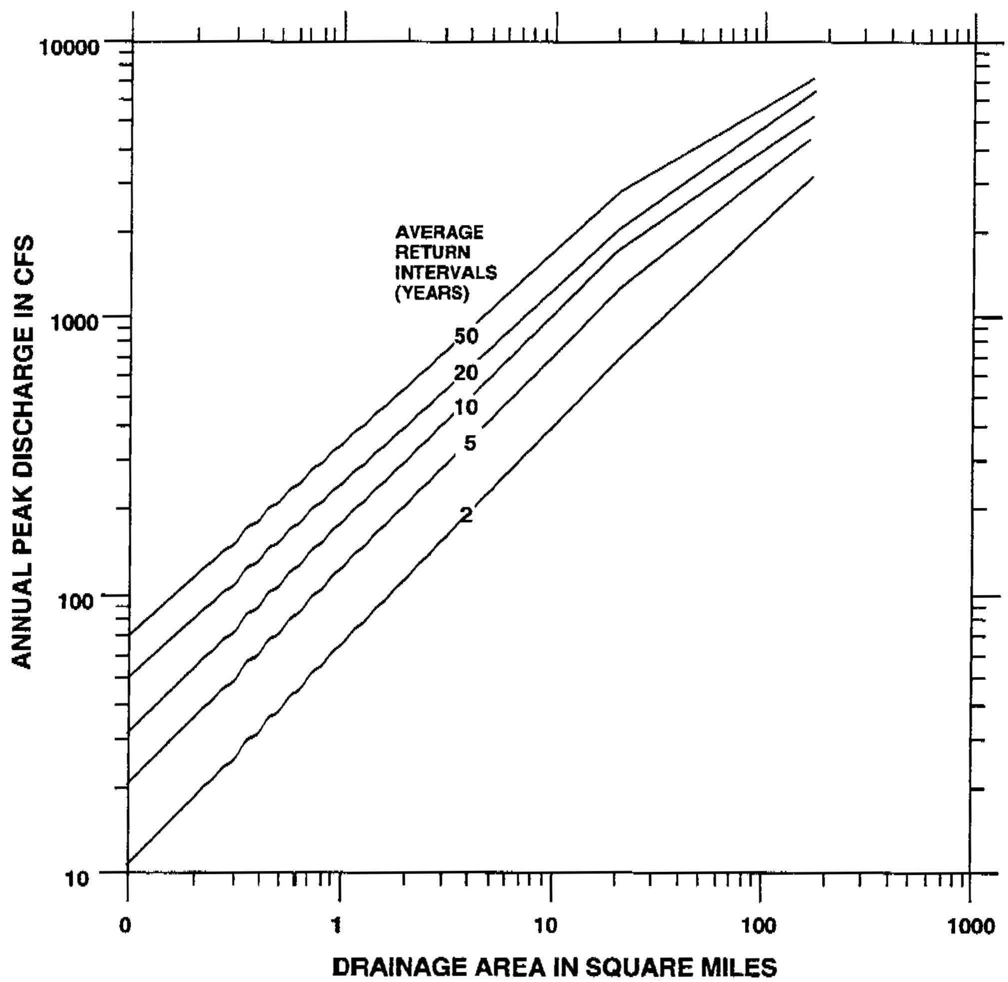
EXCEEDANCE PROBABILITY	RETURN INTERVAL (years)	COMPUTED PROBABILITY (cfs)	EXPECTED PROBABILITY (cfs)	90% CONFIDENCE LIMIT 0.5 (cfs)	0.95 (cfs)
0.002	500	10600	12700	16600	8050
0.005	200	9360	10700	14000	7250
0.010	100	8430	9360	12200	6640
0.020	50	7520	8130	10500	6040
0.050	20	6330	6650	8460	5220
0.100	10	5440	5600	6970	4570
0.200	5	4520	4590	5550	3870
0.500	2	3180	3180	3700	2730
0.800	1.25	2230	2200	2610	1820
0.900	1.1111	1850	1800	2210	1450
0.950	1.0526	1590	1520	1930	1190
0.990	1.0101	1200	1080	1520	824

Exceedance Probability: The probability that a random event, such as a flood, will exceed a given magnitude in any one year.

Return Interval: The average number of years between occurrences of a flood of a given magnitude.

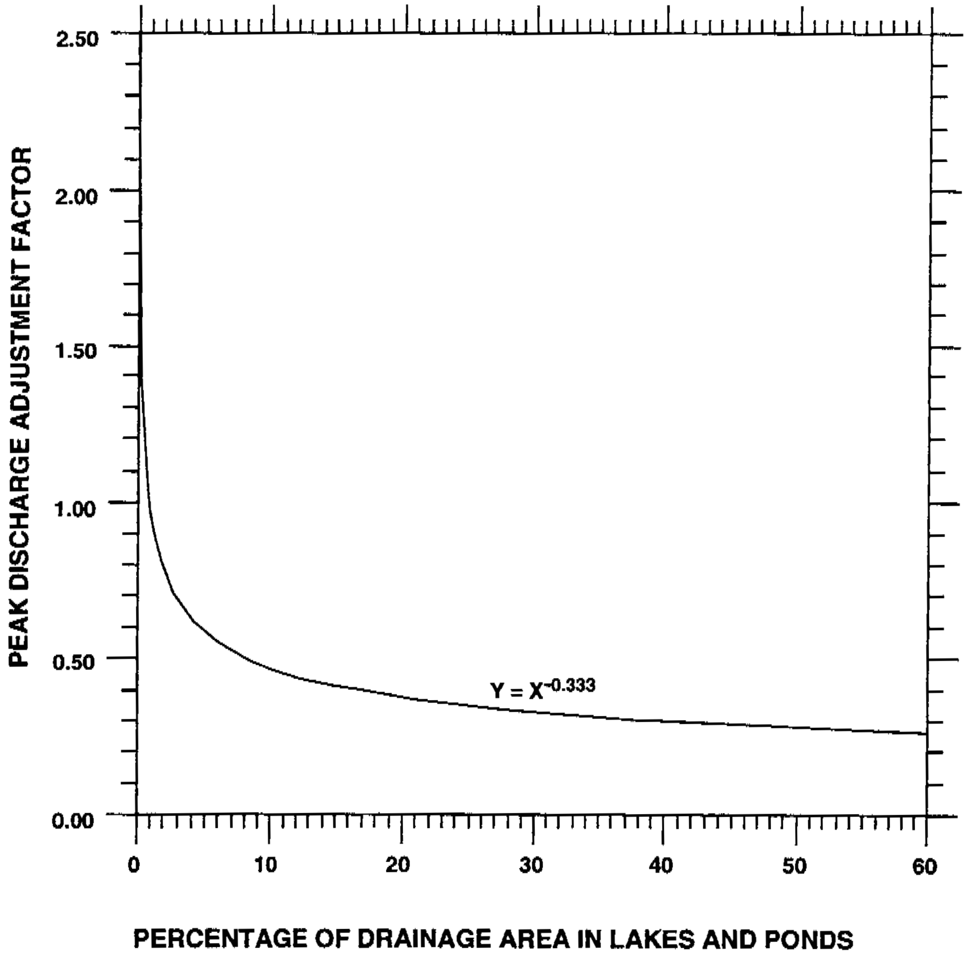
Computed Probability: The computed discharge of the flood of the listed Exceedance Probability derived from the available flood history.

Expected Probability: The average of the true probabilities of all flood magnitude estimates for a specified flood frequency that might be made from successive samples of a given size. This is a function of the length of record available.



NOTE: Criteria are derived from ARCO Kuparuk observations on several small streams and from available U.S. Geological Survey data. The relationship is based on streams with minimal lakes or ponds. Results should be adjusted using Figure 5-4 to account for storage of flood water in lakes and ponds.

FIGURE 5-3
AVERAGE ANNUAL FLOOD PEAK FREQUENCY FOR NORTH SLOPE STREAMS HAVING LESS THAN 2% OF THE DRAINAGE AREA IN LAKES OR PONDS



Reference: Jones and Fahl (1993)

FIGURE 5-4
ADJUSTMENT FACTOR FOR FLOOD PEAKS FROM FIGURE 5-3

5.4.2 Reduction of Peak by Storage

The numerous lakes and ponds of the coastal plain serve to reduce flood peaks by storing water on the rising limb of the flood hydrograph and releasing it on the falling limb. The impact of this storage on flood peaks has been estimated by considering the known variation of peak discharge as a function of the percent of drainage area in lakes and ponds. For this purpose, lakes and ponds are those areas shown in blue on USGS 1:63,360 scale (1 inch = 1 mile) maps. This relationship is shown on Figure 5-4. Water stored in these shallow lakes is released throughout the summer. The average August flow is about 0.042 cfs per square mile, while the average maximum day for August is 0.142 cfs per square mile. This value may be slightly higher for drainages with a high percentage of lakes.

5.4.3 Flood Timing

All floods used in the above analysis have occurred during breakup. The earliest date of crest on the Putuligayuk River is June 2 and the latest is June 17, while the average date of the annual crest is June 9, with a standard deviation of 3.57 days. Later breakups tend to be associated with greater snowpacks and larger floods. Larger streams originating in the Brooks Range, such as the Sagavanirktok and Kuparuk, tend to break up earlier than the Putuligayuk; and small coastal-plain streams crest a few days later.

5.4.4 Flood Stage

Because of channel blockage by snow and ice, the maximum flood stages usually occur before the time of maximum discharge during breakup. In natural channels, the maximum stage is usually limited by bank height because of the large amount of overbank storage and water conveyance provided by the flat terrain of the coastal plain. Higher stages are experienced where roads and pads cross the floodplain and restrict overbank flow. Backwater from flow over bottomfast sea ice in the estuary affects stages near the mouths of coastal plain streams until late June. Estuaries and the lower portions of coastal streams are impacted by storm surge tides during the ice-free period, when winds are from the west or northwest. Estimates of storm surge height are provided by Wise et al. (1981) and other proprietary sources. During the brief ice-free summer, stages above the tide level are governed by channel hydraulics, which may be determined using the programs WSPRO or HEC-2.

5.4.5 Fish Passage Design Flood

The Alaska Department of Fish and Game (ADF&G) specifies the time and nature of fish passage for each stream proposed for crossing. For many streams, fish must be able to pass upstream through culverts during the flow that is exceeded only for 48 hours during the mean annual flood. This discharge can best be estimated by applying to the mean annual peak flood (derived by the method in Section 5) the ratio of the 24-hour delay flood to the peak flood that has been measured at similar basins. The 24-hour delay flow may range from about 95% to 40% of the peak flow but will probably average about 70% of peak flow. Very few hydrographs have been measured for North Slope streams; however a few exist from ARCO's planning of Kuparuk facilities (Drage et al. 1984). It is possible to synthesize hydrographs using methods provided by the U.S. Army Corps of Engineers in the HEC-1 program or many other similar programs.

For other streams, fish may not be present during breakup, and the fish passage design flood specified by ADF&G may be that existing during normal summer flow. This flood is a very small (a fraction of the spring breakup flood), and its magnitude may be estimated by comparison with known hydrographs as described above.

Measured hydrographs are almost nonexistent for the North Slope. In order to facilitate an economical design, hydrographs should be measured whenever possible for streams to be crossed.

5.5 RISK OF FLOOD EXCEEDING DESIGN FLOOD

Risk is a statistical concept defined as the probability of an event occurring within a specified number of events. For crossing design, the risk is the chance that a flood exceeding the design flood will occur within a given time period, usually the project's expected life. Risk is estimated by the binomial distribution. The probability that a flood, F, will occur at least once in n successive years is:

$$R = 1-\{P(F)\}^n \qquad \text{Eq. 5.1}$$

where

R is the probability that an event F will occur at least once in n years and

P is the probability of the event F occurring in any one year. P is described in Section 5.4.

Thus, if a project has a design life span of 25 years, a 2,440-year return period will result in a 10% chance of failure during the project life. Another example follows:

What is the probability that the flood that occurs on the average once in every 10 years ("10-year flood") will occur in the next 10 years?

$$R = 1 - (1 - 1/10)^{10}$$

$$= 1 - 0.34868$$

$$= \quad 0.65$$

Thus, there is a 65% chance that a 10-year flood will be exceeded during the next 10 years. (This does not imply that 65% is an acceptable risk. What constitutes an acceptable risk must be determined on a project-specific basis.)

6. EVALUATION OF CHANNELIZED FLOW

A natural or man-made channel is defined as a waterway having a bed and banks confining moving water. Water flows in channels in accordance with the general laws of physics: gravity provides energy and friction provides resistance to flow. Since the concepts relating to open channel flow include some of the most complex in hydraulics, the intent of this section is to provide design procedures for the simpler cases normally encountered.

6.1 CRITICAL FLOW

In any channel cross-section, a given discharge may flow at an infinite product of depth and average velocities. The particular depth and velocity at which it will flow are functions of the geometric properties of the section, the stream slope, and the channel roughness. Each combination of depth and velocity represents a particular level of energy in the flow. One combination of depth and velocity represents the minimum possible energy level for the given discharge; all other combinations have higher energy levels. The depth at this minimum energy level is called *critical depth*, and the associated velocity is called the *critical velocity*. The channel slope which produces this depth and velocity is called the *critical slope*.

The concept of critical depth, which occurs often in nature, provides a useful starting place for hydraulic computations. For example, flow over a spillway or road passes through critical depth. The critical flow depth for wide, rectangular channels or weirs may be expressed as:

$$d_c = 0.315\, q^2/3 \qquad \text{Eq. 6.1}$$

where

d_c is the critical depth in feet and

q is the unit discharge in cfs per foot of width.

6.2 NORMAL DEPTH

Normal depth occurs at uniform flow, which exists when the water velocity does not change from section to section. This occurs when the gravitational forces causing flow are equally opposed by the frictional forces resisting flow. The average water velocity in a section at uniform flow can be calculated by Manning's formula:

$$V = (1.489\, R^{2/3}\, S^{1/2})/n \qquad \text{Eq. 6.2}$$

where

V is the average water velocity in the channel cross-section in feet per second.

R is the hydraulic radius in feet = A/P. (A is the wetted area of the cross section in square feet. P is the wetted perimeter which is the length of the cross section boundary in feet in contact with the section.)

S is the slope of the channel in feet per foot.

n is Manning's roughness coefficient (Table 6-1).

The above analytical relationship may be used to develop normal flow characteristics for channels of any shape. These characteristics are applicable to most flow conditions, provided the water surface is not significantly affected by either backwater or drawdown or varying rapidly with time. If any of these conditions is encountered, the backwater flow may be determined using the computer programs WATSPRO or HEC-2. The Federal Highway Administration (1961) provides design charts solving Manning's formula for a wide range of conditions. Backwater is a flow condition usually occurring upstream of a channel restriction such as a culvert. In uniform channels with subcritical flow, the curve is concave upwards, and velocities decrease downstream. The term is used in a generic sense to describe all computed water surface profiles.

TABLE 6-1
MANNING'S ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

CHANNEL TYPE	CONDITION	n
Natural Channels	Tundra Beaded Drainages	
	Low Stage	0.050
	Flood Stage	0.050
	Gravel-Bed Streams, Steep Banks	
	Low Stage	0.040
	Flood Stage	0.030
	Overbank	0.020
Artificial Channels	Frozen Silt	0.020
	Gravelly Sand	0.025
	Riprap (1-foot diameter)	0.035
	Sand Bags	0.030
	Plastic Filter Cloth	0.015
	Articulated Concrete Mats (Armorflex)	
	Open Blocks	
	30-pound	0.031
	50-pound	0.032
	70-pound	0.034
	Flow Over Ice	
	Smooth Ice	0.010
	Blocky Rough Ice	0.030
	Floating Ice Cover	
	Smooth Ice w/o Drifting Blocks	
	Beginning of Winter	0.012
	Middle of Winter	0.010
	Rough Ice with Drifting Blocks	0.025
	Culverts	See Table 7-3

Note: Many cross-section boundaries are composed of more than one material. In this case use a perimeter-length-weighted average n value.

Reference: ARCO (1984)

7. DESIGN AND INSTALLATION OF CULVERTS

Culverts are relatively short pipes placed under a road or other embankment to convey water from one side to the other. Culverts must convey design flood flows without unacceptable damage to the road or adjacent property; and for streams supporting fish, culverts must not impede the free passage of fish (see Section 2). This section provides a discussion of culvert hydraulics and design procedures for both cases.

7.1 BASIC CULVERT CHARACTERISTICS

Figure 7-1 is a definition sketch for culvert flow, while Table 7-1 defines symbols and units for culverts. Section 1 represents the approach channel. The total energy available to force water through the culvert consists of the static head, h_1, acting at the upstream Section 1, plus the velocity head, $V_1^2/2g$, in feet, of the approach flow. Section 2 represents the loss of energy due to contraction at the culvert entrance. The loss of energy between Sections 1 and 2 is due to friction and flow contraction in the approach channel. This loss is normally computed by the methods of channelized flow provided in Section 6. The energy lost at the culvert entrance is due to the sudden constriction and consequent expansion of the flow. This loss is calculated as a coefficient of discharge, C, times the velocity head in the barrel. Entrance loss coefficients are derived experimentally and are provided in Table 7-2.

An additional energy loss occurs as the result of the friction of the water flowing in the culvert barrel between Sections 2 and 3. This may be calculated using Manning's roughness coefficient (n values of Table 7-3) or the computer programs WATSPRO or HEC-2. All of the energy remaining is in the velocity head at the outlet and is dissipated in the downstream channel. It is this energy lost at the outlet that provides the most serious impediment to upstream fish passage. Because both tailwater elevation and pipe geometry vary, six possible flow conditions exist. These conditions are shown in Figure 7-2 and described in the following paragraphs, while methods for analysis are provided in Norman et al. (1985).

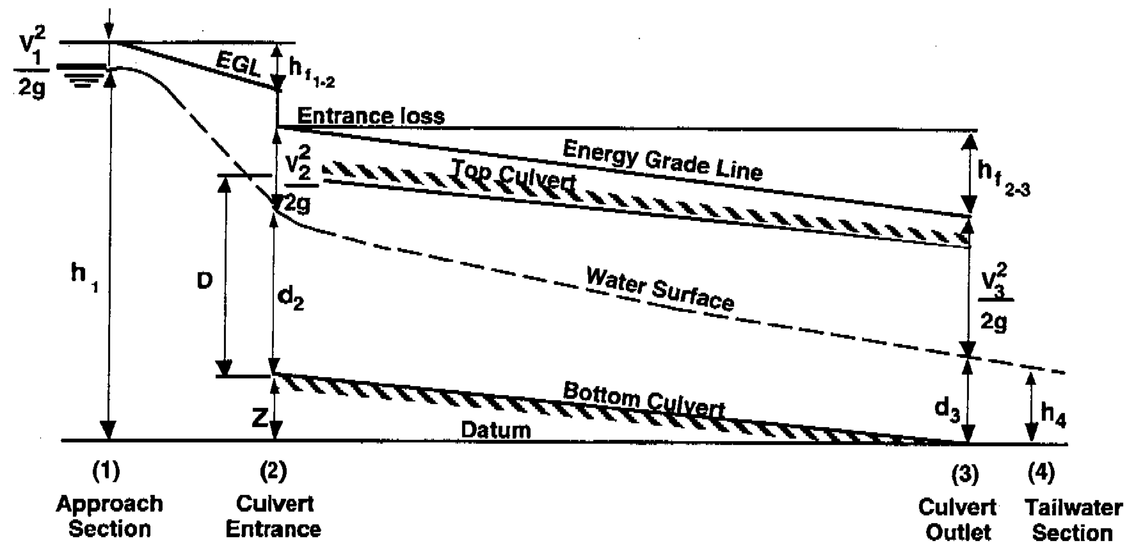


FIGURE 7-1
DEFINITION SKETCH OF CULVERT FLOW

TABLE 7-1
SYMBOLS AND UNITS FOR CULVERTS

Symbol	Definition	Unit
A	Area	ft^2
A_0	Area of culvert barrel	ft^2
A_c	Area of section of flow at critical depth	ft^2
b	Width of contracted flow section for box culvert	ft
C	Coefficient of discharge; also, coefficient for computing various culvert properties; subscripts refer to specific items, as *a* for area, *k* for conveyance, *m* for mean depth, *p* for wetted perimeter, *q* for discharge, *r* for hydraulic radius, and *t* for top width	
D	Maximum inside vertical dimension of culvert barrel, or the inside diameter of a circular section (For corrugated pipes, D is measured as the minimum inside diameter)	ft
D_m	Maximum inside diameter of pipe culvert at entrance	ft
d	Depth of flow measured from the lowest point in the cross section for culverts	ft
d_c	Maximum depth in critical-flow section	ft
d_m	Mean depth	ft
EGL	Energy grade line	ft
F	Froude number	
g	Gravitational constant (acceleration)	ft/sec^2
H_0	Specific energy	ft
h	Static or piezometric head above an arbitrary datum	ft
h_c	$d_c + z$ for type 1 culvert flow	ft
h_e	Head loss due to entrance contraction	ft
h_f	Head loss due to friction	ft
h_v	Velocity head at a section	ft
K	Conveyance of a section	ft^3/sec
K_c	Conveyance of critical depth section	ft^3/sec
K_0	Conveyance of full culvert barrel	ft^3/sec
k	Adjustment factor; subscripts refer to specific items, as *a* for skewed abutments with dikes, *L* for length, *r* and *R* for radius, *w* for length of wingwalls, and θ for wingwall angle.	
L	Length of culvert barrel, bridge abutment, or broad-crested weir in direction of flow	ft
L_p	Distance a culvert barrel projects beyond a headwall or embankment	ft
L_w	Distance from approach section to entrance of culvert, upstream side of contraction, or crest of weir	ft
m	Channel-contraction ratio	
n	Manning roughness coefficient	$ft^{1/6}$
n_c	Composite value of roughness coefficient	$ft^{1/6}$

TABLE 7-1 (Cont'd)
SYMBOLS AND UNITS FOR CULVERTS

Symbol	Definition	Unit
P	Wetted perimeter of cross section of flow	ft
P_p	Wetted perimeter of the paved invert of a culvert	ft
Q	Total discharge	ft^3/sec
R	Hydraulic radius	ft
R_0	Hydraulic radius of a culvert barrel	ft
r	Radius of entrance rounding	ft
S	Friction slope	
S_c	Bed slope of culvert for which the normal depth and the critical depth are equal	
S_0	Bed slope of culvert barrel	
T	Width of a section at the water surface	ft
V	Mean velocity of flow in a section	ft/sec
V_0	Full culvert velocity	ft/sec
w	Measure of the length of a wing-wall or chamfer	ft
x	Length of part-full flow	ft
z	Elevation of a point above a datum	ft
1,2	Subscripts which denote the location of cross sections or section properties in downstream order	
α	Velocity-head coefficient	
θ	Acute angle between a wingwall and plane of contraction or headwall; and the bevel angle	
$<$	Less than	
$\leq$	Equal to or less than	
$>$	Greater than	
$\geq$	Equal to or greater than	

Reference: Bodhaine (1968)

TABLE 7-2
CULVERT ENTRANCE LOSS COEFFICIENTS

ENTRANCE TYPE	$(h_1 - Z) / D$						
TYPE 1, 2, & 3 FLOW (partly full pipe, low head)	1.60	1.40	1.20	1.00	0.80	0.60	0.40
Steel Line Pipe - Any Diameter							
Flush in a vertical headwall	0.80	0.83	0.86	0.88	0.91	0.93	0.93
Mitered to a 1 on 2 slope	0.70	0.73	0.82	0.87	0.90	0.90	0.88
Projecting one diameter	0.72	0.74	0.77	0.79	0.82	0.83	0.84
24 Inch CMP*							
Flush in a vertical headwall	0.86	0.87	0.92	0.93	0.93	0.93	0.93
Mitered to a 1 on 2 slope	0.70	0.73	0.82	0.87	0.90	0.90	0.88
Projecting one diameter	0.77	0.78	0.83	0.84	0.88	0.90	0.91
60 Inch CMP*							
Flush in a vertical headwall	0.82	0.85	0.88	0.91	0.93	0.93	0.93
Mitered to a 1 on 2 slope	0.70	0.73	0.82	0.87	0.90	0.90	0.88
Projecting one diameter	0.74	0.77	0.79	0.82	0.84	0.86	0.86
TYPE 4 AND 6 FLOW (full pipe, high head)							
Steel Line Pipe - Any Diameter							
Flush in a vertical wall	0.84	0.84	0.84	0.84	***	***	***
Mitered to a 1 on 2 slope	0.74	0.74	0.74	0.74	***	***	***
Projecting one diameter	0.76	0.76	0.76	0.76	***	***	***
24 Inch CMP							
Flush in a vertical headwall	0.87	0.87	0.87	0.87	***	***	***
Mitered to a 1 on 2 slope	0.74	0.74	0.74	0.74	***	***	***
Projecting one diameter	0.78	0.78	0.78	0.78	***	***	***

TABLE 7-2 (Cont'd)
CULVERT ENTRANCE LOSS COEFFICIENTS

ENTRANCE TYPE	$(h_1 - Z) / D$						
TYPE 4 & 6 FLOW (full pipe, high head) (cont'd)							
48 Inch CMP**							
Flush in a vertical wall	0.85	0.85	0.85	0.85	***	***	***
Mitered to a 1 on 2 slope	0.74	0.74	0.74	0.74	***	***	***
Projecting one diameter	0.76	0.76	0.76	0.76	***	***	***
TYPE 5 FLOW (partly full pipe, high headwater)							
Steel Line Pipe - Any Diameter							
Flush in a vertical head wall	0.47	0.44	***	***	***	***	***
Mitered to a 1 on 2 slope	0.44	0.43	***	***	***	***	***
Projecting one diameter	0.42	0.40	***	***	***	***	***
24 Inch CMP**							
Flush in a vertical head wall	0.49	0.45	***	***	***	***	***
Mitered to a 1 on 2 slope	0.45	0.41	***	***	***	***	***
Projecting one diameter	0.44	0.41	***	***	***	***	***

* Coefficients for pipes 72" and larger are the same as for steel line pipe.
** Coefficients for larger pipes are the same as for steel line pipe.
*** Cannot exist

NOTES:

1. For classification of culvert flow, see Figure 7-2
2. Coefficients for pipe diameters not provided may be obtained by interpolation.

Reference: ARCO (1984)

TABLE 7-3
MANNING'S ROUGHNESS COEFFICIENTS FOR CULVERTS

CLASSIFICATION	TYPE	DIAMETER*	n VALUE
Welded Steel		All	0.012
Annular Corrugations	2-2/3" pitch 1/2" rise	All	0.024
	3" pitch 1" rise	30" - 144"	0.027
	6" pitch 1" rise	30" - 60"	0.025
		66" - 96"	0.024
		108" - 144"	0.023
	6" pitch 2" rise	60" - 72"	0.034
		84" - 96"	0.033
		108" - 132"	0.032
		144" - 156"	0.031
		168" - 180"	0.030
		192" - 216"	0.029
		228" - 240"	0.028
		252" - 276"	0.027
Helical Corrugations	2-2/3" pitch 1/2" rise	24"	0.016
		36"	0.019
		48"	0.020
	3" pitch 1" rise	36"	0.021
		48"	0.023
		54"	0.023
		60"	0.024
		66"	0.025
		72"	0.026

*For pipe arch culverts, use rise as diameter

Reference: ARCO (1984)

7.2 FLOW TYPES AND CHARACTERISTICS

As shown in Figure 7-2, six flow types have been defined for culverts. Type 3, or tranquil flow, is best suited for fish passage and is thus the most desirable flow type for culvert design on the North Slope. A culvert may operate as one type of flow during flood peaks and the desirable Type 3 flow during normal flow. Type 3 flow will generally not be possible within 48 hours of the design flood discharge.

7.2.1 Type 1: Critical Depth at the Inlet and Barrel

Type 1 flow occurs frequently on small, steep streams where the allowable headwater depth is limited by topography and where tailwater depths are low. The capacity of the culvert is limited solely by conditions at the inlet. As shown in Figure 7-2, Type 1 flow passes through the critical depth at the inlet, the barrel slope is greater than critical, and the headwater-to-depth ratio is less than 1.5. The culvert barrel flows partially full. Type 1 flow will not usually allow acceptable water velocities for fish passage. This type can be altered to Type 3 (which can pass fish) by increasing the tailwater elevation artificially.

7.2.2 Type 2: Critical Depth at Outlet

Type 2 flow passes through critical depth at the outlet. Because the headwater-to-depth ratio is less than 1.5, the flow depth at the inlet is above critical, the barrel slope is low enough so that the flow does not accelerate past critical in the barrel, and the outlet is high (usually perched) so that the flow passes through critical. The high-velocity water associated with this type of exit is difficult for fish to navigate and tends to create excessive scour. This flow can be altered to Type 3 by lowering the pipe or artificially raising the tailwater.

7.2.3 Type 3: Tranquil Flow

Type 3 flow exists when the flow depth exceeds critical depth throughout the length of the culvert. This requires headwater-to-depth ratios less than 1.5, and the inlet functions as a weir, with flow above critical depth. This flow type is most suitable for fish passage and is the most desirable. Because of the wide range of possible tailwater conditions, Type 3 flow cannot be reliably solved by the nomographs in Norman et al. (1985); however, the flow can be determined by backwater calculations using WSPRO or HEC-2. The adequacy of Type 3 flow for fish passage can be determined using Behlke's program FISHPASS. The FISHPASS program uses hydraulic formulas to caculated profile drag, non-Archimedean buoyant forces, and virtual mass forces in order to quantify the hydraulic forces within a culvert that weak-swimming fish can sustain without exhaustion. These forces are compared to those allowable, and unsuitable culvert designs are rejected.

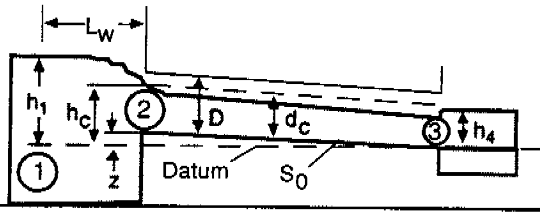
TYPE	EXAMPLE	DISCHARGE EQUATION

1
CRITICAL DEPTH AT INLET

$\frac{h_1 - z}{D} < 1.5$

$h_4 / h_c < 1.0$

$S_0 > S_c$



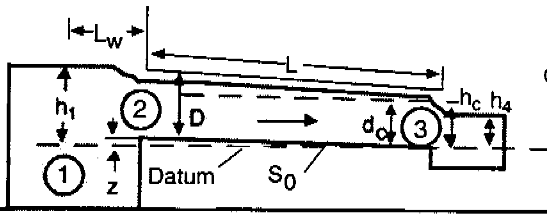
$$Q = CA_c \sqrt{2g\left(h_1 - z + \alpha_1 \frac{V_1^2}{2g} - d_c - h_{f_{1,2}}\right)}$$

2
CRITICAL DEPTH AT OUTLET

$\frac{h_1 - z}{D} < 1.5$

$h_4 / h_c < 1.0$

$S_0 < S_c$



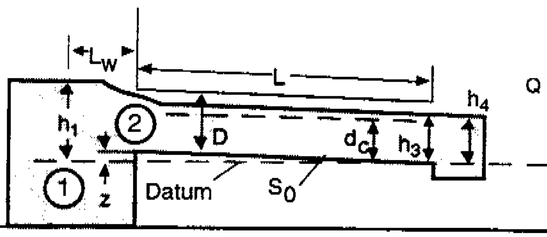
$$Q = CA_c \sqrt{2g\left(h_1 + \alpha_1 \frac{V_1^2}{2g} - d_c - h_{f_{1,2}} - h_{f_{2,3}}\right)}$$

3
TRANQUIL FLOW THROUGHOUT

$\frac{h_1 - z}{D} < 1.5$

$h_4 / D \leq 1.0$

$h_4 / h_c > 1.0$

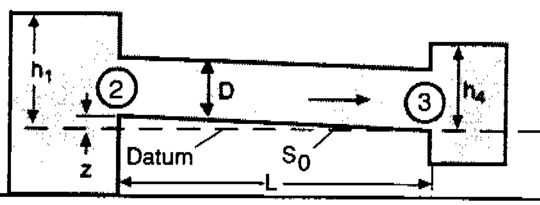


$$Q = CA_3 \sqrt{2g\left(h_1 + \alpha_1 \frac{V_1^2}{2g} - h_3 - h_{f_{1,2}} - h_{f_{2,3}}\right)}$$

4
SUBMERGED OUTLET

$\frac{h_1 - z}{D} > 1.0$

$h_4 / D > 1.0$

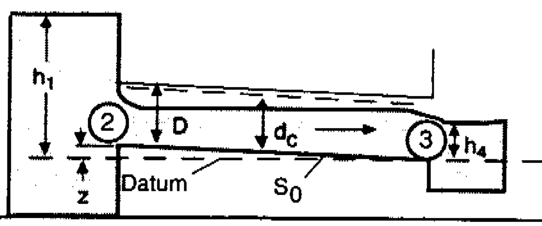


$$Q = CA_o \sqrt{\frac{2g\,(h_1 - h_4)}{1 + \frac{29\,(C^2 n^2 L)}{R_o^{4/3}}}}$$

5
RAPID FLOW AT INLET

$\frac{h_1 - z}{D} \geq 1.5$

$h_4 / D \leq 1.0$

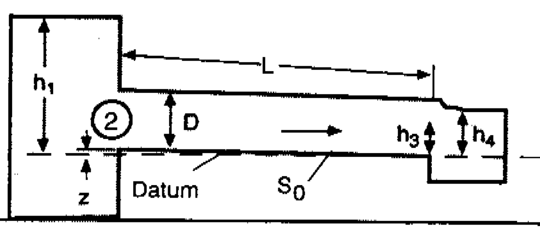


$$Q = CA_o \sqrt{2g\,(h_1 - z)}$$

6
FULL FLOW FREE OUTFALL

$\frac{h_1 - z}{D} \geq 1.5$

$h_4 / D \leq 1.0$



$$Q = CA_o \sqrt{2g\,(h_1 - h_3 - h_{f_{2,3}})}$$

Reference: Bodhaine (1964)

FIGURE 7-2
CLASSIFICATION OF CULVERT FLOW TYPES

7.2.4 Type 4: Submerged Outlet

In Type 4 flow, both the inlet and outlet are submerged. This type is hydraulically very efficient; however, it is rarely found on the North Slope because of the flat topography. Discharge for Type 4 flow exists when both the inlet and outlet are submerged and the culvert flows full. For this type of flow, the discharge produced by a given head may be easily obtained using the outlet control nomographs in Norman et al. (1985).

7.2.5 Type 5: Rapid Flow at Inlet

Type 5 flow resembles flow under a sluice gate, where capacity is controlled completely by conditions at the inlet. The headwater-to-diameter ratio normally exceeds 1.5, which ensures a full pipe at the entrance. The entrance contracts the flow, and the culvert barrel is neither flat enough nor long enough to cause the friction losses that would make the pipe flow full and become Type 6. The tailwater depth must be below the top of the pipe at the exit so that the flow does not become Type 4. Type 5 flow occurs only with short, steep culverts, which are not likely to be found on the North Slope. This is an undesirable flow type because a high headwater-to-diameter ratio will cause severe erosion and difficult fish passage at the outlet and inlet. Discharge for a given head may be determined with the outlet control nomographs provided in Norman et al. (1985).

7.2.6 Type 6: Full Flow with Free Outfall

Type 6 flow occurs when the culvert is full and under pressure, with the tailwater below the top of the culvert. This is an impossible condition for fish passage because of the strong vortex generated at both the inlet and outlet. This type of flow evolves from Type 5 flow when the contracted entrance is able to expand and contact the top of the pipe. Once the jet expands to the top of the pipe, the supply of air is cut off and the pipe flows full for its entire length. Within a certain range, Type 5 and 6 flows may alternate. The inlet control nomographs in Norman et al. (1985) may be used to provide a conservative solution.

7.3 DESIGN

7.3.1 Flood Passage Design

Culverts are normally designed to pass the 50-year design flood, with a headwater-to-depth ratio no greater than 1.5 to limit vortex erosion. Alternatively, a specific design study evaluating costs of larger culverts against the damage caused by excessive headwater may be accomplished. Norman et al. (1985) contains procedures for this type of study, which the Federal Highway Administration calls a risk analysis.

The first step in culvert design is to determine the design discharge using the methods described in Section 5. The second step is to determine the tailwater elevation, which is independent of culvert geometry. The tailwater elevation may be determined using the open channel methods described in Section 6. The third step normally is to select a trial culvert diameter. The nomographs in Norman et al. (1985) may then be used to determine the resulting headwater elevation for all except Type 3 flow. This condition, which occurs most frequently, must be solved analytically using models such as WSPRO or HEC-2. Alternatively, the program HYDRAIN also contains two programs, CDS-Culverts and HY-8, which may be used to provide solutions for flood design.

7.3.2 Fish Passage Design

Fish passing upstrêam must be capable of swimming into the culvert, upstream through the barrel and out through the culvert inlet. The Alaska Department of Fish and Game (ADF&G) normally requires that at least one culvert in each battery of culverts be capable of passing fish. More that one culvert able to pass fish may be required for very wide batteries with many culverts.

Fish passage design should be viewed from the perspective of the fish's power and energy expenditures in the various parts of the culvert: outlet, barrel, and inlet (Behlke 1990). The engineer must rely upon open-channel flow hydraulics to predict the fish's power and energy expenditures during the passage. Data requirements are the same as for flood design except the design discharge is the discharge at the time specified by ADF&G.

To pass fish successfully, culverts must operate with Type 3 flow. Culvert hydraulics may be analyzed using the HEC-2 or WSPRO program using the coefficients in Tables 7-2 and 7-3. The ability of the culvert to pass fish may be estimated using the FISHPASS program, which considers profile drag, non-Archimedean buoyant forces, and virtual mass forces, as well as the power and energy expenditures of the fish.

As a guideline based on experience, a culvert will be adequate for fish passage if it is properly designed to pass a 50-year flood with acceptable headwater, it is installed at a slope of less than 0.5%, and its invert is at least 20% of the diameter below the stream bed.

Analysis of many culverts shows that most of a fish's energy is expended in the white muscle mode in passing through the culvert outlet. Thus, it is necessary that the tailwater be high enough so that the outlet flow is subcritical. In some instances, it may be necessary to construct weirs in the downstream channel in order to increase tailwater height. Care must be exercised in the design of tailwater control structures so that the fish passage problem is not merely shifted downstream. See Federal Highway Administration (1983) for guidance in the design of downstream energy dissipation structures.

7.3.3 Outlet Erosion

Scour holes will develop at the exit of any culvert. These holes may eventually lower the channel to the point that a once-acceptable installation will no longer pass fish. Outlet erosion occurs as the result of dissipation of energy concentrated by the culvert. Two types of solutions exist. The first consists of estimating the size of the scour hole that will naturally develop and accepting that change in the tailwater depth. The second consists of providing a structural solution, such as a scour mat downstream of the culvert to resist erosion.

Scour Holes

Scour can impact the stability of embankment slopes and can create a pool which attracts fish so that they remain at freezeup and do not survive. Impacts of scour can be judged once the dimensions are estimated. Estimating erosion at culvert outlets is difficult because of many complex factors, including the discharge, the culvert size, soil type, duration of flow, and tailwater depth. This section presents a method used by the Federal Highway Administration to estimate scour at culvert outlets. The solution is in the form of a dimensionless scour-hole geometry which is:

$$\alpha_e\left(\frac{Q}{g^{1/2}D^{5/2}}\right)^{\beta}\left(\frac{t}{t_0}\right)^{\theta}$$

where D is the diameter of the culvert, in feet. For non-circular or partially full culverts, the diameter, D, should be replaced by an equivalent depth, y_e, defined as:

$$y_e = (A/2)^{1/2}$$

where

A is the cross-sectional flow area at the culvert outlet.

Q is the discharge in cfs.

g is the acceleration of gravity = 32.2 feet per second squared.

t is the time of flow Q in minutes.

t_0 is the base time in minutes used in the experiment = 316 minutes unless specified otherwise.

$\alpha_e = 0.63^{2.5\beta-1}$ for h_s, w_s, and L_s, or

$\alpha_e = 0.63^{2.5\beta-3}$ for Vs.

τ = critical tractive shear stress on the bed material in pounds/ft^2

The values of the experimental coefficients α_e, ß, and θ are provided in Table 7-4.

The point of maximum scour is at about 40 percent of the scour hole length. Side slopes are at the material's angle of repose. Typical scour hole depths are about twice the culvert diameter. Scour can be reduced by increasing the culvert's diameter.

Erosion Control Mats

If scour is not acceptable, a structural limit on scour may be provided. Structures traditionally consist of pre-formed and lined basins, rigid concrete stilling basins, or horizontal riprap aprons. Basins are not desirable on the North Slope because the culvert outlet must be accessible for snow removal before breakup. Because riprap is normally not available locally, a standardized mat design based on a Corps of Engineers design is provided (Figure 7-3). This mat is made of concrete block; however, riprap or articulated concrete-filled mattresses of equivalent weight may be substituted. The mat must be large enough to allow for energy dissipation on the mat. Corry et al. (1983) provide an extensive discussion of erosion control structures. The State of Alaska may require that erosion control mats be buried below the invert of the culvert. The need for erosion control can be minimized by using larger culverts and by lowering the culvert invert.

7.3.4 Flotation Control

Corrugated metal pipe culverts frequently fail because flotation causes bending at the inlet. Forces causing flotation arise from two causes under typical North Slope conditions. The first and least common cause is an open-water condition arising from differences in hydrostatic pressure acting on the inside and outside of the culvert. Figure 7-4 is a definition sketch for a culvert with a common projecting entrance. The flotation force, F, acting on the culvert is the difference between the buoyant force acting upward on the outside of the culvert and the weight of the contained water acting downward on the invert of the culvert. The external flotation force acts from d_1, and the internal downward force results from d_2. The resultant force acts upward at a point midway between the inlet and the intersection of the culvert's top with the embankment and is equal to the mass of the water in the culvert between d_1 and d_2 acting over the length L.

The second and most significant case under North Slope conditions occurs during breakup when the culvert entrance becomes completely blocked with ice. In that case, d_2

TABLE 7-4
EXPERIMENTAL COEFFICIENTS FOR CULVERT OUTLET SCOUR

MATERIAL	NOMINAL GRAIN SIZE d_{50} (mm)	SCOUR EQUATION	DEPTH h_s				WIDTH W_s				LENGTH L_s				VOLUME V_s			
			α	β	θ	α_e	α	β	θ	α_e	α	β	θ	α_e	α	β	θ	α_e
Uniform Sand	0.20	V-1 or V-2	2.72	0.375	0.10	2.79	11.73	0.92	0.15	6.44	16.82	0.71	0.125	11.75	203.36	2.0	0.375	80.71
Uniform Sand	2.0	V-1 or V-2	1.86	0.45	0.09	1.76	8.44	0.57	0.06	6.94	18.28	0.51	0.17	16.10	101.48	1.41	0.34	79.62
Graded Sand	2.0	V-1 or V-2	1.22	0.85	0.07	0.75	7.25	0.76	0.06	4.78	12.77	0.41	0.04	12.62	36.17	2.09	0.19	12.94
Uniform Gravel	8.0	V-1 or V-2	1.78	0.45	0.04	1.68	9.13	0.62	0.08	7.08	14.36	0.95	0.12	7.61	65.91	1.86	0.19	12.15
Graded Gravel	8.0	V-1 or V-2	1.49	0.50	0.03	1.33	8.76	0.89	0.10	4.97	13.09	0.62	0.07	10.15	42.31	2.28	0.17	32.82
<u>Cohesive Sandy Clay</u>																		
60% Sand PI 15	0.15	V-1 or V-2	1.86	0.57	0.10	1.53	8.63	0.35	0.07	9.14	15.30	0.43	0.09	14.78	79.73	1.42	0.23	61.84
Clay PI 5-16	Various	V-3 or V-4	0.86	0.18	0.10	1.37	3.55	0.17	0.07	5.63	2.82	0.33	0.09	4.48	0.62	0.93	0.23	2.48

V-1. FOR CIRCULAR CULVERTS (Cohesionless material or the 0.15-mm cohesive sandy clay)

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{or } \frac{V_s}{D^3}\right] = \alpha\left(\frac{Q}{\sqrt{g}\; D^{5/2}}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

V-2. FOR OTHER CULVERT SHAPES (Same material as above)

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \text{or } \frac{V_s}{y_e^3}\right] = \alpha_e\left(\frac{Q}{\sqrt{g}\; y_e^{5/2}}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

V-3. FOR CIRCULAR CULVERTS (Cohesive sandy clay with PI = 5-16)

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, \text{or } \frac{V_s}{D^3}\right] = \alpha\left(\frac{\rho V^2}{\tau_c}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

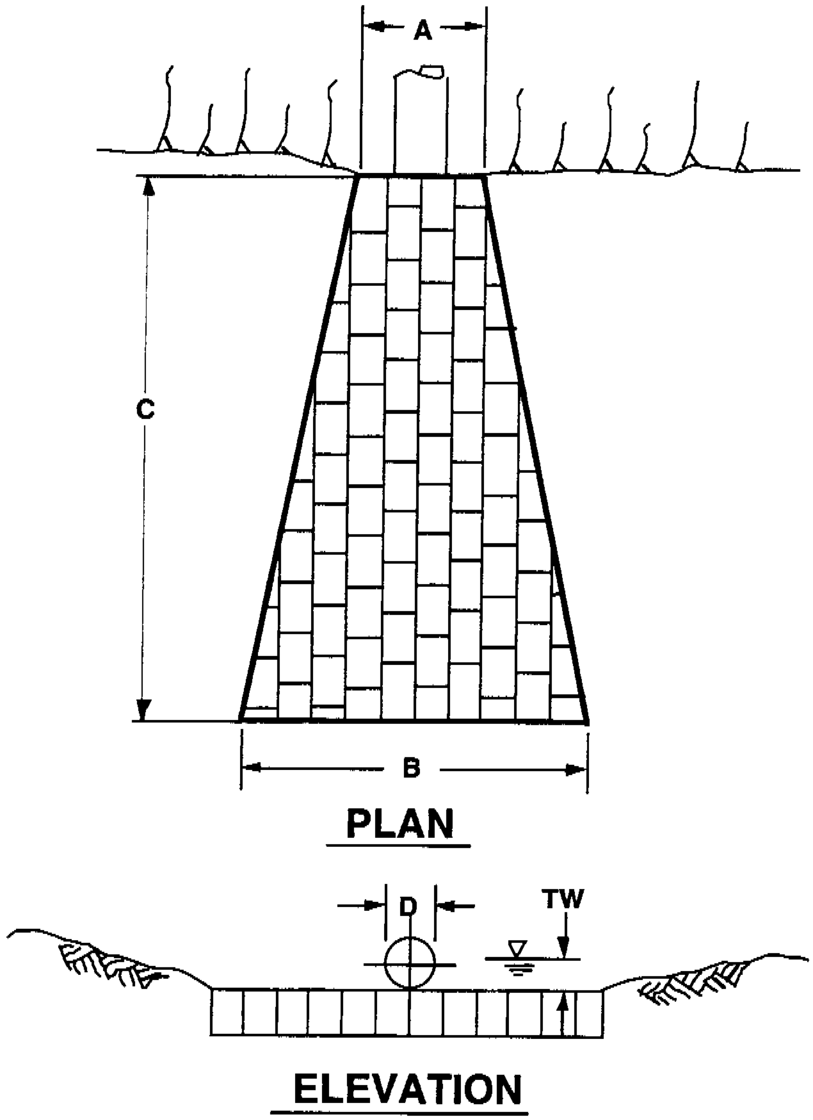
where t_o = 316 min.

V-4. FOR OTHER CULVERT SHAPES (Cohesive sandy clay with PI = 5-16)

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, \text{or } \frac{V_s}{y_e^3}\right] = \alpha_e\left(\frac{\rho V^2}{\tau_c}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

Reference: Corry et al. (1983, Table V-1)



$A = 3D$

$C = 1.7\ D\ (Q/D^{2.5}) + 8$

$B = A + 0.4C$ if tailwater is above pipe centerline

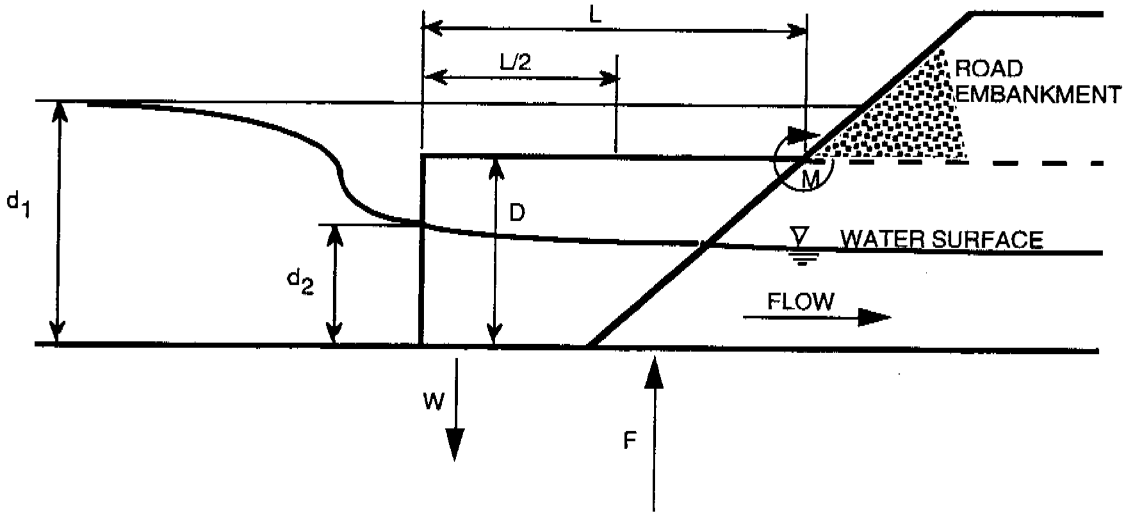
$B = A + C$ if tailwater is below pipe centerline

$W = 79\ [(0.02\ D^2/TW)\ (Q/D^{2.5})^{1.33}]^3$

W = Weight of concrete block in pounds per square foot assuming unit weight of concrete is 150 lb / cu ft.

Reference: Bohan (1970)

FIGURE 7-3
CONCRETE SCOUR MAT AND DIMENSIONS



F = the resultant hydrostatic force acting upward on a culvert entrance, in pounds

D = the culvert diameter in feet

L = the exposed culvert length in feet

d_1 = the headwater depth above the bottom of the culvert inlet in feet

d_2 = the depth of water at the culvert barrel inlet, in feet

M = bending moment

FIGURE 7-4
CULVERT FLOTATION CONTROL

becomes zero and the flotation force is d_1 acting over the length L. If the headwater is ponded above the pipe, the force on a round pipe is:

$$F = 49 \, L \, D^2$$

The force on an arch pipe or arch is approximately:

$$F = 62.4 \, A \, L$$

where A is the end area of the pipe.
There are several solutions to the problem. These include the following:

- Provide a rigid pipe capable of resisting the upward forces without bending. Steel line pipes used as culverts have not been known to fail.
- Provide a concrete weight at the pipe end equal to F/2 to counteract the upward bending moment generated by F.
- Provide an entrance mitered to the slope so that the resultant uplift force, F, acts close to the point at which the top of the culvert is restrained. This reduces the bending moment about that bound by reducing L.
- Provide a vertical headwall to eliminate L.
- Prevent scour around the inlet which tends to increase L.

7.3.5 Geotechnical

This section provides construction aspects related to soil and climatic conditions, soil properties for calculation of culvert stress, and recommended typical sections. The *soil reaction modulus* is the unit pressure developed as the side of the pipe deflects a unit distance outward into the side fill. This modulus is dependent on the soil type, void ratio, and the soil's thermal state and ice content. Recommended soil modulus design values for typical soils encountered on the North Slope are provided in Table 7-5.

In order that culverts not lose soil support from excessive thaw settlement, it is desirable to maintain the permafrost below the pipe. Table 7-6 provides estimates of thaw depth, thaw strain, and thaw settlement for four design cases.

7.4 INSTALLATION GUIDELINES

Table 7-7 contains a summary of culvert installation guidelines. Figures 7-5 and 7-6 provide a typical section for locations where a culvert is placed on thaw-unstable soils and gravel is used for insulation. Figures 7-7 and 7-8 provide details applicable to multiple culverts placed on thaw-unstable soil.

TABLE 7-5
TYPICAL SOIL MODULUS VALUES
FOR TYPICAL CONFINED GRANULAR EMBANKMENT SOILS

EMBANKMENT TYPE	PSI
Winter placement (frozen)	600
Nominal Summer placement (thawed)	1000
Minimum 90% Standard Proctor	2000

Reference: ARCO (1984)

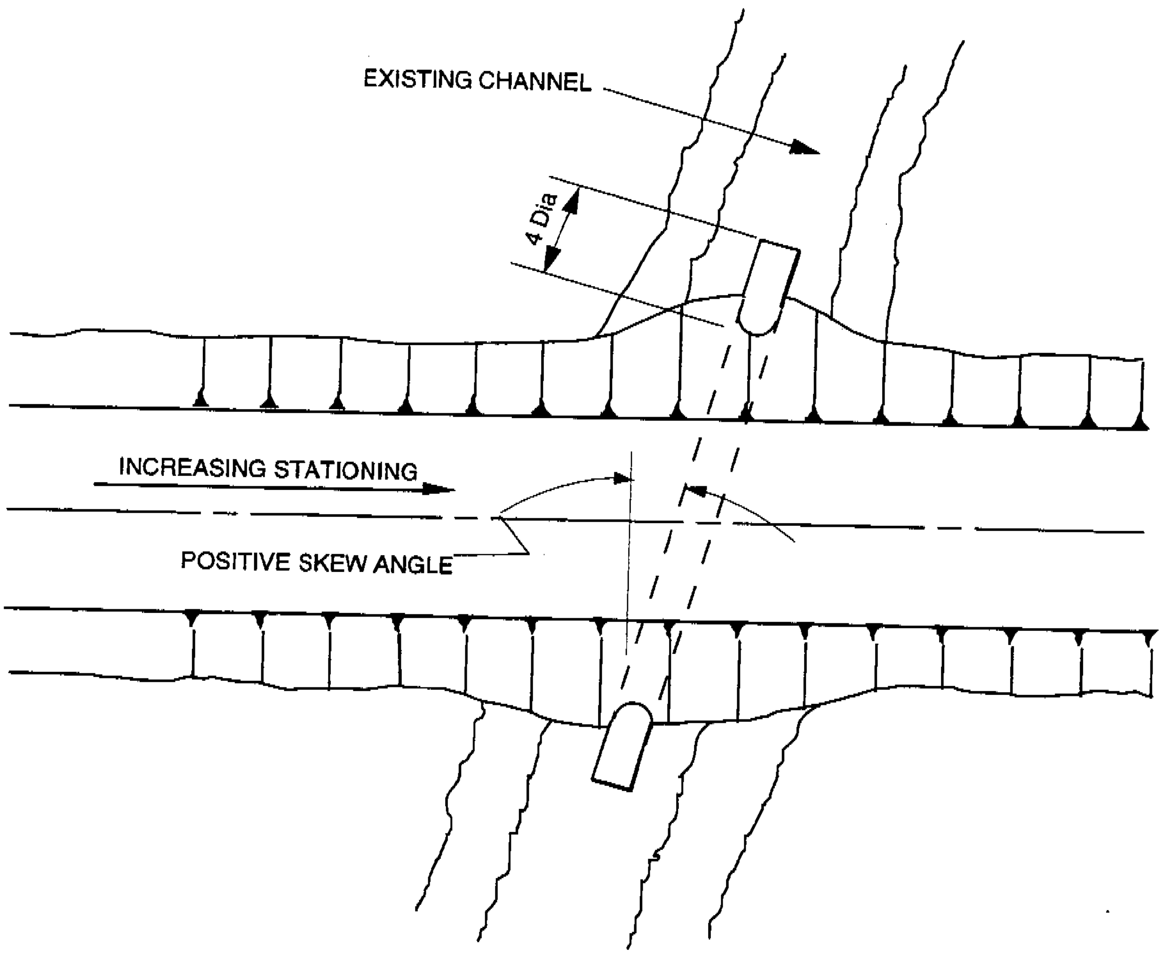
TABLE 7-6
ESTIMATED THAW SETTLEMENT BENEATH CULVERTS

DESIGN CASE	DEPTH OF THAW (feet)	THAW STRAIN (percent)	THAW SETTLEMENT (feet)
3" insulation over silt	0.2	12	0.02
Summer placed gravel	2.0	<2	<0.04
1' Peat over silt	0.9	40	0.36
Silt	1.5	12	0.18

Reference: ARCO (1984)

TABLE 7-7
CULVERT INSTALLATION GUIDELINES

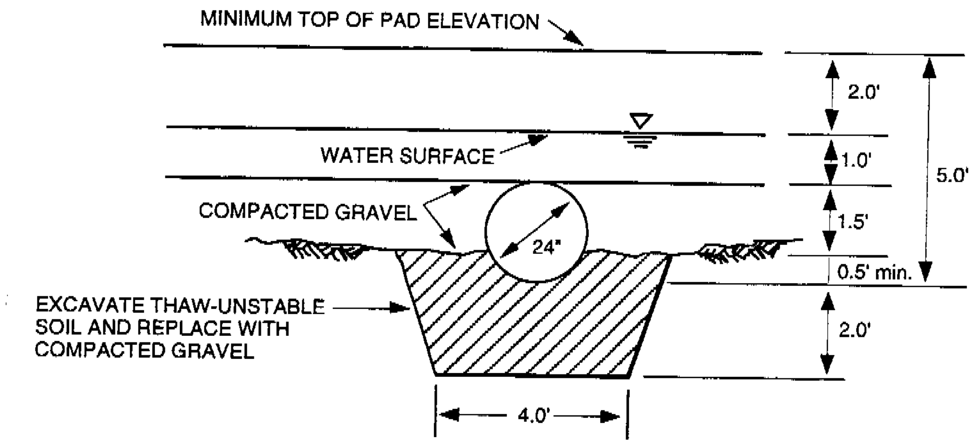
DATA	EXPLANATION
Dimensions	The minimum culvert size is 24-inch diameter, except for fish-bearing waterbodies, where a minimum diameter of 48 inches normally should be used.
Material	Culverts with diameters 36 inches or smaller shall be fabricated from lengths of steel pipe manufactured in accordance wtih ASTM A-211. Pipe shall be fabricated from carbon steel meeting the requirements of ASTM A-570 Grade D or approved equal. Culverts larger than 36 inches in diameter may be fabricated from galvanized corrugated metal pipe.
Excavation	Thaw-unstable native soils shall be excavated to a minimum depth of two feet below the culvert and a minimum width of two culvert diameters.
Bedding	Bedding for steel pipe culverts shall consist of thawed pit-run gravel which contains no stones larger than 4 inches and no more than 12 percent passing the number 200 sieve of the fraction of material passing the 3-inch sieve. The material shall be free of ice, muck, organic material, and frozen clumps. Bedding shall be placed in the excavated trench in uniform 6-inch loose measurement layers and compacted to a minimum density of not less than 95 percent of the maximum density as determined by AASHTO T180 Method D. Bedding shall be placed and compacted to the plane of the culvert invert. At that time the culvert shall be placed to the line and grade determined by the design engineer. Bedding shall then be placed and compacted to a height not less than 25 percent of the culvert's diameter above the invert and to a width equal to the ditch width shown in Figure 7-6. Each lift shall be placed on both sides of the culvert and compacted prior to placing subsequent lifts on either side.
Laying	The lower segment of the pipe shall be in contact with the bedding through its full length. Alignment and grades shall be as determined by the design engineer. Pipe sections may be field joined by a single-V-groove weld, AISC joint designation B-P2 or approved equal.
Backfilling	Culvert shall be inspected for damage, settlement, and proper alignment before backfill is placed. Backfill shall be placed so that the slope of the embankment exposed to the stream shall not be steeper than two (horizontally) to one (vertically).



Reference: ARCO (1984)

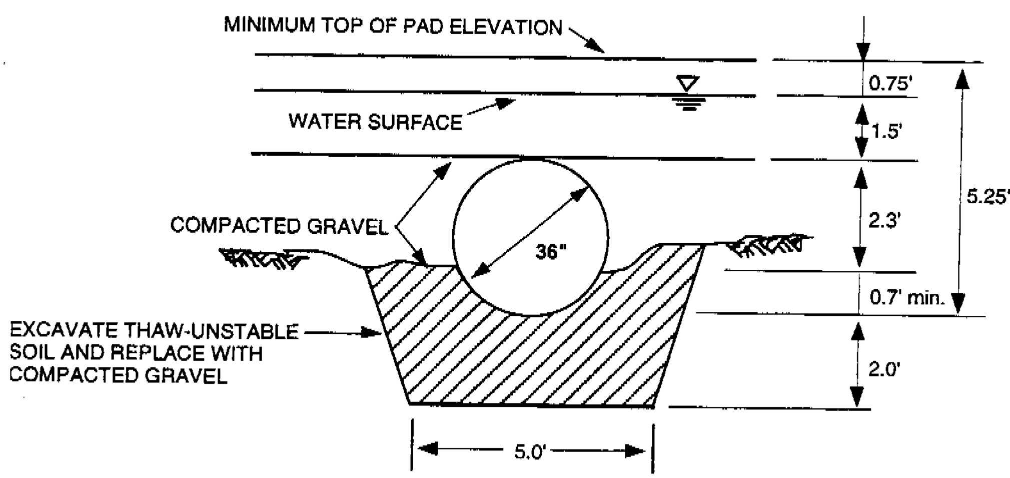
FIGURE 7-5
CULVERT INSTALLATION DETAIL: PLAN VIEW

NOT TO SCALE



TYPICAL SECTION - 1

24" IRON PIPE CULVERT, THAW-UNSTABLE SOIL



TYPICAL SECTION - 2

36" IRON PIPE CULVERT, THAW-UNSTABLE SOIL

Reference: ARCO (1984)

FIGURE 7-6
CULVERT INSTALLATION DETAIL: CROSS SECTIONS

8. DESIGN AND INSTALLATION CONSIDERATIONS FOR BRIDGES

A bridge is a structure, including supports, erected over a depression or an obstruction such as a body of water, a road, or a railway having a track or a passageway for carrying traffic and having a length of opening greater than 20 feet (Simons and Sentürk 1992). Bridge design requires consideration of all of the factors described above to determine length, freeboard and training requirements.

8.1 PIER AND ABUTMENT TYPES AND SPANS

Bridge piers increase backwater and scour at bridges. Adding piers reduces individual span length and therefore cost up to a limit; there is an optimum number of spans to minimize project costs. Perpendicular abutments induce deeper scour but provide greater flow area than do sloping, spill-through abutments. Therefore, choice of span length between piers requires a trial and error economic analysis to determine the most cost-effective arrangement. Scour estimation procedures are provided by Neil (1972), Simons and Sentürk (1992), and Bradley (1970), as well as many other sources, and are not repeated here.

8.2 WATERWAY OPENING

Waterway opening requirements determine span length and affect bridge costs more than any other factor. Required openings are determined by trial and error.

A trial length is assumed, then backwater and scour are computed for that trial length for the design flood conditions. The length is adjusted and the computations are repeated.

Graphic procedures for backwater computation and scour estimation are provided by Bradley (1970). Alternatively, estimates may be made using the Corps of Engineers Backwater Computation Program "HEC 2," or by the Federal Highway Administration's Computer Program "WSPRO."

Trial span widths, W, may be determined using one of several regime relationships such as Lacy's regime width equation which is:

$$W = C\,Q^{1/2}$$

where

W is the Lacey Regime width, which is the width of a channel conveying the discharge Q that has attained more or less a stable width with respect to erosion and deposition. This width should not constrict the channel at the design discharge.

C ranges between 1.8 and 2.7 with 1.8 for very stable channels and 2.7 for sandy shifting channels. Use 1.8 for small streams with frozen banks. Large streams tending to braid require 2.7 or special study.

Q is the design discharge.

Repeated economic evaluations of the span and backwater usually result in a final span much shorter than the original Lacy trial span. Figure 3.1 from Neill (1972) provides the hydraulic design process.

8.3 FREEBOARD

Freeboard is the distance from the design water surface to the lowest part of the span. Freeboard must be high enough to pass any floating ice or debris (about five feet) and for navigable streams freeboard must allow passage of water craft as required by Coast Guard regulations.

8.4 SCOUR

Scour affects pile and abutment embedment length for bridges.

Three types of scour exist. The first is long-term degradation, which is the continuing progressive lowering of a stream's bed due to changes in the stream's discharge or sediment load. Degradation is independent of the bridge and according to Simons and Sentürk (1992) is evaluated by an extensive study of the stream's nature. The second type of scour is general scour, or contraction scour. Such scour results from the acceleration of flow through a constriction and is best estimated at a bridge by Laursen's equation (Simons and Sentürk [1992]). General scour lowers the entire width of the stream bed. Local scour results at a pier or abutment as the result of the vortex generated. Local scour may also result around ice jams. Simons and Sentürk (1992) and by Neill (1972) provide methods for estimating local scour, which is additive to general scour.

The computer program SCOUR in the larger program HY-TB automates the computation of scour.

8.5 ICE LOADS

Ice loads on piers are a function of ice thickness, mass, velocity, and temperature as well as the size and shape of the pier. Ice loads on bridge piers may be estimated by a standard formula. Procedures for estimating ice loads are provided by Maattanen (1983) and by Neill (1972).

8.6 GEOTECHNICAL

Geotechnical interests in bridge design are primarily concerned with requirements for pile, piers and abutments.

8.7 STRUCTURAL

A preliminary structural design and cost estimate is necessary for each trial span length to enable a final span choice.

8.8 ENVIRONMENTAL

Bridges are not considered as restrictions to fish passage. However, construction or backwater may impact water quality and other environmental aspects. Timing of in-stream construction to avoid seasonal fish use is necessary.

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COMPUTER PROGRAMS REFERENCED

HYDRAIN
SCOUR
WSPRO
WATSPRO
HEC-1
HEC-2
HY-TB

Transportation Research Center
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FISHPASS

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