STREAMFLOW ESTIMATION AND WATER USE PLANNING FOR SURFACE MINING IN NORTHERN ALASKA

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CHAPTER I

Introduction

The small-scale surface mining operation in northern regions is typically located in a remote, ungaged basin. With the recent increase in the economic potential of mining, the number of small operations is increasing. The year-to-year variability in precipitation in this region under study is of great concern to mine operators. Some years may yield only 5 to 6 inches of precipitation, while others may yield in excess of 15 inches. Such variability makes placer mining operations difficult to plan, since low water years may result in too little run-off for gravel washing and high water years may result in flooding of mining developments.

There are no official streamflow records in many watersheds in the northern region containing placer mining claims. However, information from other streams permits calculation of an estimated discharge for a watershed under consideration using average flows per square mile of drainage. It is to be emphasized that these are average or expected figures. In reality, one water year may be significantly wetter or drier than another. This high variability of annual precipitation is significant in relation to placer operations because of the large quantities of water required for this type of mining. Some operations can not operate in low water years. In the larger watersheds, operations may continue in low water years but at a slower pace because of longer water collecting times. The high variability of available run-off makes placer mining in many parts of Alaska difficult to plan on a long range basis because of the unpredictability of water supplies.

Therefore, characterization of summer streamflow regimes is important for the safety and design of operations, and for satisfying state and federal safety and environmental regulations. Currently, the streamflow methods used at these sites are generally patterned after those developed for temperate regions and are not necessarily sensitive to northern phenomena such as permafrost, icing and breakup.

With the imposition of more strict government regulations of mining activities, it is becoming increasing important to provide reasonable estimates of streamflow variability to determine how much water will remain after withdrawals for mining. The mining operator must be able to design his project so that he will not overestimate or underestimate the amount of water that will be available to him. This is important not only to make the mining operation economically viable, but also to protect it from flood damage.

Therefore, the small-scale mining operator in the North must be able to anticipate and accurately predict streamflow for any basin. This particular problem is not restricted to Alaska but is also encountered in Canada and even parts of northern contiguous states. Mining is generally not found in populated areas, so a limited data base is common. However, northern areas like Alaska encounter even more complications to the normal hydrological cycle since run-off characteristics are extremely different (Kane, 1979).

With the increased environmental concern especially evident in Alaska, this research report will help miners to accurately predict the impact on the water resources and the effect that the water resources will have on mining. By providing valid estimation procedures, one will be able to minimize the risk to a level that would be acceptable both to the operator as well as regulatory agencies.

Scope and Objectives

The project has examined summer streamflow characterization in northern basins, emphasizing techniques sensitive to northern phenomena in gaged basins. The methodology developed in an earlier work by the Water Research Institute (Carlson and Fox, 1980; Ashton and Carlson, 1983) has improved streamflow characterization for ungaged basins. In this project these techniques have been used for analyzing streamflow data and applied to interior basins ranging from arctic to subarctic to north temperate zones. The methodologies cited above were developed for situations where streamflow data and climatic data within a basin are very limited on nonexistent. Such techniques are, however, sufficiently flexible to utilize all available information within the sample and they do not require unobtainable data.

In this report, the available summer discharge records for all gaged basins within representative areas in Alaska have been analyzed with respect to their behavior. This has allowed for optimum design of estimation procedures for streamflow parameters, which generally rely on nonparametric statistical methods. Since, for any particular surface mining project, the range of data availability may go from occasional to continuous data, the estimation procedures should rely not only on patterns in the streamflow records but also change with the varied amount of information available. This has resulted in an estimation procedure that follows a flow-chart approach to allow the most efficient use of all available information.

The project has focused on developing standardized streamflow estimation procedures for different levels of surface mining activity versus differing availability of streamflow and climatic data. Decisions trees or flow diagrams based on data availability and size of mining operation have been developed, which will lead the operator to a particular procedure for estimating streamflow for that mining operation. These flow diagrams have been developed for each region simply because different stream regimes are anticipated within each region, and therefore different estimation procedures and parameter estimates may be necessary.

These flow diagrams will provide standardized procedures for estimating streamflows (given a certain amount of data and the size of the mining operation) but also includes estimates of the risk inherent in the procedure used to estimate each parameter. All flow charts are based on what the mining operators needs to know to successfully design and operate his long term mine plan. While the methodologies developed here will be useful in any surface mining situation, the scope of the work will be limited to a discussion of placer mining in the northern region of Alaska.

<u>Project Objectives</u>

The primary objectives of this project are to improve methods for determining summer streamflow and stream response to surface mining activities in remote northern regions. A secondary objective is to provide a handbook describing appropriate techniques for estimating streamflow regimes and their accuracy for use by small-scale surface mining operators in the North. These objectives are best met by several sub-objectives:

1. Improved streamflow characterization with emphasis on techniques sensitive to northern (arctic and subarctic) phenomena and ungaged basins.
2. Standardize procedures of streamflow characterization for design and operation of surface mining operations, and develop corresponding estimates of risk of uncertainity.

3. Develop a guide for use by mining operators to determine the water regime of a site for design and operation, and to meet state and federal requirements.

CHAPTER II

Placer Districts of Alaska and Characteristics of Placer Mines

The history of placer mining in Alaska and the locations of placer deposits have been documented in numerous publications of the U.S. Geological Survey, U.S. Bureau of Mines, and the Alaska Division of Mines and Geology. Most of this information has been summarized by Cobb (1973). His report contains nearly 500 references, and provides descriptions of the physiography, general geology, lode resources, and the history of placer mining for each mining region or district in Alaska. The most recent published compilation of active placer operations in the state is for the year 1975 (Carnes, 1976). A brief summary of the history and location of placer mines is also given in the U.S. Geological Survey professional paper 610 (Koschmann and Bergendahl, 1968). A recent survey has shown that, in 1982, 319 mechanized operators and 20 small recreational ventures produced an estimated 174,900 ounces of gold and over 20,000 ounces of by-product silver.

These figures represent an apparent increase in production of about 30 percent from 1981 to 1982. Available figures (Table 1 and Figure 1) on the total number of operations and the exact methods used by individual operations are considered incomplete because of the short-term nature of the operations.

Gold provinces of Alaska occupy the entire state, with the exception of the north slope. Figure 2 shows the mining regions and districts of Alaska (based on the 1954 USBM mining district classification). Although the exact methods used by individual operators are not well documented and will differ somewhat due to the variations in mine-site topography, water availability, overburden and placer types, and the basic mining techniques are well known and adequately summarized in the existing literature. (Koschmann and

Table 1. Gold production in Alaska by region, 1982
(Alaska Office of Mineral Development, 1983)

Region and district	Major operators	Production (troy ounces)
Northern	18	9,500
Chandalar		
Koyuk		
Noatak-Kiana		
Shungnak		
Western	34	34,550
Nome		
Kougarok		
Port Clarence		
Fairhaven		
Candle		
Ruby		
Solomon		
Koyuk		
Council		
Hughes		
Eastern Interior	201	88,500
Circle		
Livengood		
Fairbanks		
Forty-mile		
Manley-Eureka		
Rampart		
Richardson		
Bonnifield		
Kantishna		
Delta		
South-central	35	22,150
Cache Creek		
Nizina		
Chistochina		
Valdez Creek		
Kenai Peninsula		
Nelchina		
Southwestern	26	19,200
Innoko		
Tolstoi		
Iditarod		
Nixon Fork		
Nyac		
Crooked Creek		
Goodnews Bay		
Southeastern and Alaska Peninsula	5	1,000
Total	**319**	**174,900**

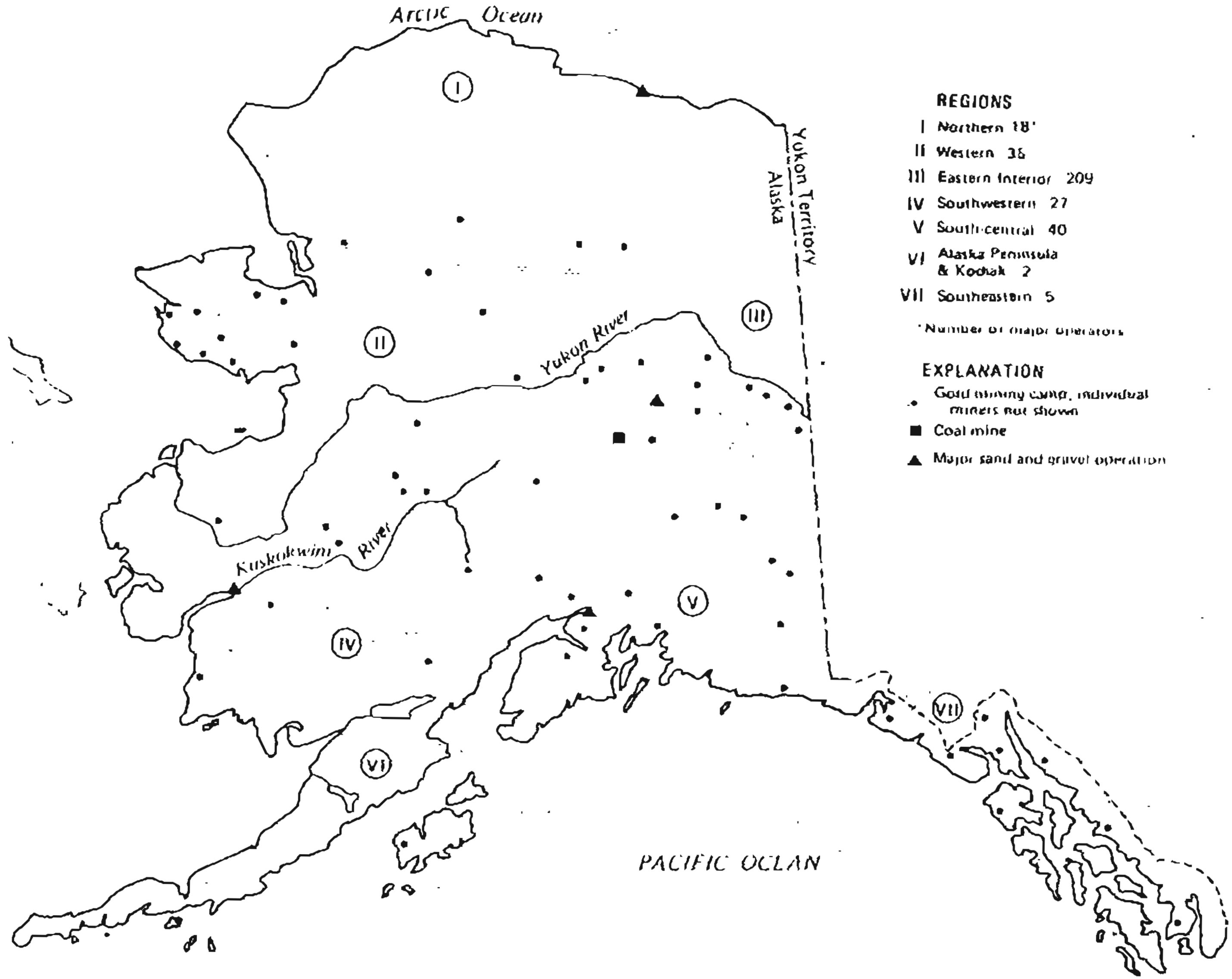


Figure 1. Producing mines and districts in Alaska, 1982

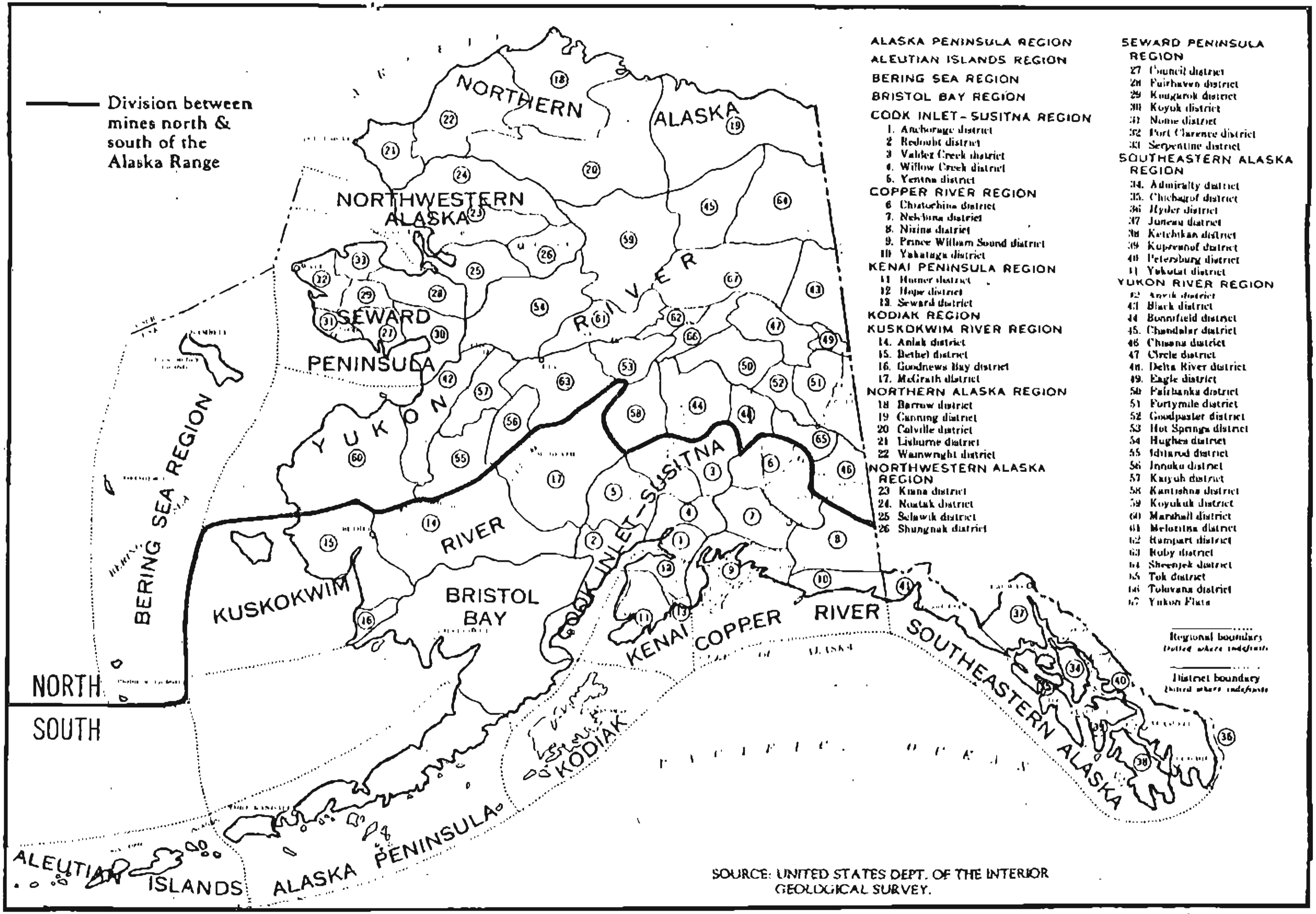


Figure 2. Mining regions and districts

Bergendahl, 1968; Wimmler, 1927; Thomas, 1959; Romanowitz, Bennet and Dane, 1970; Cobb, 1973) and will not be reviewed here.

Characteristics of Placer Mines

Placer mining operations are so variable that one could state that the only constant among operations is that each mine site has site specific conditions. The variability results from a number of factors, such as:

- topographical location,
- size of operation,
- amount and type of overburden,
- width of gold bearing strata,
- equipment and type of material processing,
- water availability and use,
- degree of water reuse or recycling,
- degree of waste water treatment,
- condition (clear vs. glacial) and size of source of water and receiving stream

Many of these factors are interrelated and some may dictate the type of mining operation. For example, size of operation may be a function of the availability of water and/or the type of the equipment used by the miner. Water availability may also influence the method of overburden removal, that is, hydraulic or mechanical, and the specific operating mode of the mine. For example, the removal of oversized material prior to sluicing results in less water being required to move the gold-bearing material through the sluice box. Full or partial reuse of sluice water is another method for reducing water use.

Primary equipment for moving material may consist of one type or a combination of many types, such as hydraulic giants, front-end loaders, backhoes, dozers, scrapers, draglines or dredges. Consequently, it will be difficult to find two identical placer gold mining operations in Alaska (R & M Consultants, Inc., 1982).

CHAPTER III

Water Use in Placer Mining and Water Resources Information

The estimation of water demand is the primary item in water supply planning. The purpose of water is in the gravity concentrating process used to separate the valuable constituents from the gangue material. However, associated operations such as hydraulic stripping, hydraulic elevating, hydraulic mining, stacking tailings, artificial thawing and dredge flotation requirements are also water dependent. Because each individual operation is different in regards to mining method, characteristics of the gravel, water availability and general topography, the duty of the hydraulic water is site specific. The duty of hydraulic water usually is stated in the United States as cubic yards of material mined for miner's inch day (MID). A miner's inch of water as generally accepted for a number of years is 1.5 cfm or 11.25 US gpm. A MID is the volume represented by a rate of flow of one miner's inch of water continuously for 24 hours.

Water duties obtained in mining gravel are reported extensively in the referenced publications. Generally in the larger mines the average was 3.0 to 4.0 cu. yd. per MID, though as high as 10 under favorable conditions; but at many mines, particularly small ones, it was less than 1.0 cu. yd. per MID. The water duty for stripping frozen muck, as is the case in many parts of Alaska, is extremely variable, depending on conditions as described above, and ranges from less than 1 to as much as 30 cu. yds. per MID. The averages achieved over a long-term of years at the large North American subarctic properties wore 15 to 19 cu. yd. per MID. Table 2 shows a recent survey (R & M Consultants, Inc., 1982) of few selected mining operations in Alaska and related water use. Figure 3 provides a nomogram for the determination of sluice box flow, given the box width, box flow depth and the box slope.

Table 3, from Peele (1940), illustrates the duty of water, under varying conditions, in Alaskan sluices.

The ideal mining situation is one in which the seasonal water supply is readily available in the creek on which the deposit is located. In these circumstances only minimum planning and preparation is necessary for impounding, transporting or recirculating the water. Unfortunately, this is usually the exception rather than the rule, and premining preparation usually includes ensuring an adequate seasonal supply. Premining planning in these situations should include determination of:

(a) The source and quantity of available water;

(b) the head obtainable on the field of operation;

(c) the nature of the ground, which determines the nature of the conduit employed;

(d) the cost of supply; and

(e) the form of mining for which, considering the nature of the deposit, the supply can best be utilized.

As discussed previously, the water supply has an important bearing on the method of working a property and on the plans to be employed. If ample water is available, under sufficient head, and the ground is suitable, the deposit may be broken down, carried into the sluices, washed and discharged into a tailing rice, solely by the application of water; and, with a cheap supply and adequate head, these are the conditions for hydraulic sluicing under the most favorable conditions. If a hydraulic giant is being used the effective range of the water-jet depends upon the water head. Optimal distances, however,

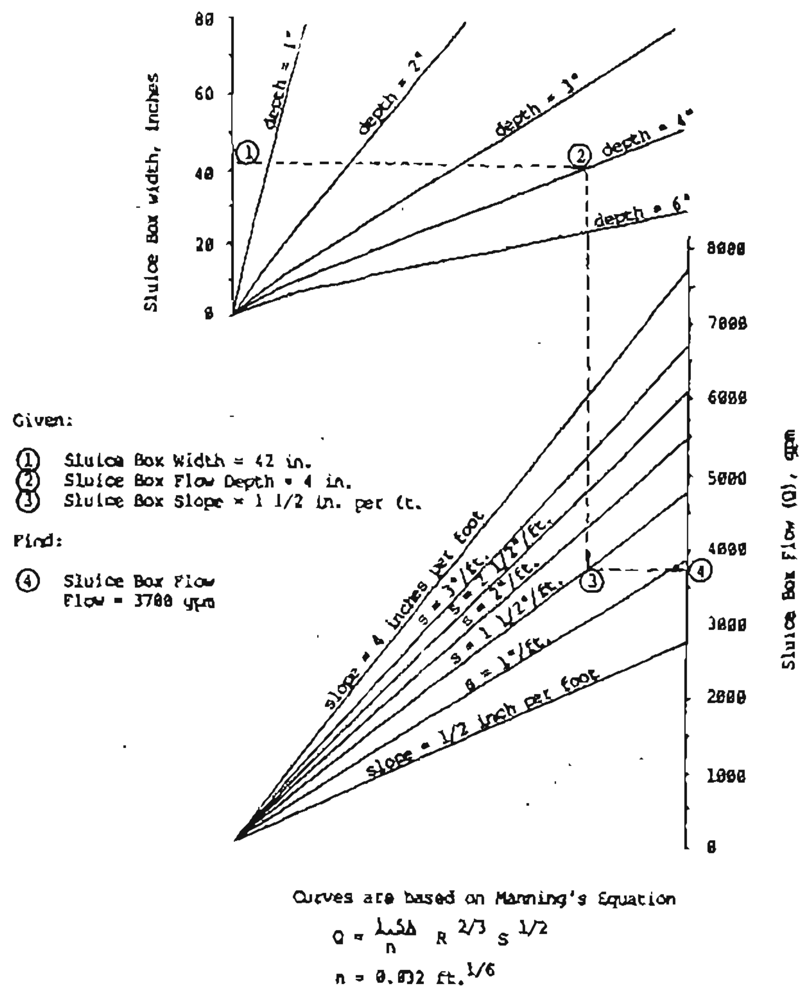


Figure 3. Determination of Sluice box flow (R & M Consultant, 1982)

Table 2
SUMMARY OF MINE OPERATION
(R & M Consultants, Inc., 1982)

Mine Site #	Creek	Type of Material Processing	Material Moved, Yd^3/Hour	Hours Worked	Water Required to Sluice One Cu. Yd.	Water Use Gal/min Sluicing
1	Fish	Elevated Sluice Box, Giant	75-100	8	Not Avail.	3680*
2	Fairbanks	Dragline Loading Sluice Box	25	9	1600	280
3	Gilmore	Cat Loading Sluice Box	90-100	8	1900-2100	3100
4	Eagle	Cat Loading Sluice Box	90-100	9	1800-2000	3000
5	Eagle	Cat Loading Sluice Box	80-100	2/10hr shifts	2900-3700	4900
6	Faith	Sluice Box with Grizzly	175	11	1300	3800
7	Mastodon	Vibrating Screen Conveyor	60	11	300	300
8	Miller	Cat Loading Sluice Box	150-200	8	1500-2000	1886
9	Mastodon	Vibrating Screens 80% Recycle	150	8	560(E)	1800
10	Mammoth	Loader Feeding Sluice Box	150-170	8	1200-1300	3300
11	Crooked	Cat Loading Sluice Box	150-200	8	1200-2000	4500
12	Deadwood	Loader Feeding Washing Bin	60-80	8	2900-3900	3900
13	Chena	Vibrating Screen to Conveyor to Trommel	150-200	8	390-520	1300
14	Flume	Cat Loading Sluice Box 80% Recycle	40-50	10	2600-3300	2200
15	Porcupine Sluice 2	Cat Loading Sluice Box	90-115	8.3	2700-3500	5400
16	Porcupine Sluice 1	Trommel	75-100	7.4	2000-2600	3300

Table 3: DUTY OF WATER IN ALASKAN SLUICES
(Peele, 1940)

Locality	Sluice Box Width In.	Sluice Box Depth In.	Sluice Box Grade In. per 12' box	Type of Riffle	Water Through Sluice, Miner's Inch	Duty	Nature of Gravel
Seward Pen:							
Big Hurrah Cr.	36	18	5	Rails	900	1.20	Unfrozen, med., much flat
Little Cr.	48	24	5-7	Angles and rails	---	1.37	Partly frozen, med.
Osborne Cr.	36	24	7	Blocks and rails	750	1.20	Partly frozen, heavy
Mt. McKinley Dist:							
Moore Cr.	24	20	6	Punched plate over matting and longit steel shod	300	1.60	Unfrozen, med., round
Fairbanks Dist:							
Pedro Cr.	36	30	11	Blocks	350	1.20	Partly frozen, heavy
Pedro Cr.	36	30	5	Rails	400	0.80	Partly frozen, med.
Yentna Dist:							
Peters Cr.	30	24	6	Rails	800	0.80	Unfrozen, med.; boulders
Kenai Dist:							
Crow Cr.	52	36	6	Rails	2600	0.50	Very coarse; many large boulders
Nizing Dist:							
Dan Cr.	48	44	5	Rails longit	---	0.32	Very coarse; many large boulders
Chititu Cr.	40	36	5-3/4	Rails longit	2200	0.42	Very coarse, many large boulders, also heavy

between the hydraulic monitor and the working face in relation to the mining conditions can be established mathematically. When the unit operates solely to disintegrate and dislodge the material the distance between it and the working face should not exceed one-quarter of the water head value in feet; if the purpose is dislodgement and concurrent transport of the material this distance should be equal to one-third of the water head (Popov, 1971). When the objective is only to transport the ground the distance may be equivalent to the range of the jet throw.

The velocity of water issuing from the giant's nozzle is determined by the equation:

$$v = \phi \sqrt{2gh}$$

Where v = rate of water outflow, m/sec

h = head at nozzle, in

g = acceleration due to gravity, m/sec^2

ϕ = Speed factor (depending upon the nozzle design, and ranges from 0.94 to 0.97).

With a definite nozzle diameter and velocity of outflowing water its consumption can be estimated from the equation:

$$\phi = u\pi \frac{d^2}{4} \cdot v \cdot \; m^3/sec$$

Where d = the diameter of nozzle outlet, mm

ϕ = water consumption, m^3/sec

v = rate of water outflow, m/sec

m = Coefficient of jet compression (u = 0.96 to 0.98)

The giants efficiency largely depends upon the properties of the ground, the height of the working face, the water head and the nozzle diameter. Table 4 lists approximate giant's efficiency standards and water consumption for various nozzle diameters.

Table 4. Hydraulic Giant's Efficiency Standards in Cubic Meters of Ground in Place During 1 Hour of Continuous Operation (Popov, 1971).

	Nozzle diameter, mm							
	50		63		75		100	
Category of ground	Water Consumption, litres /sec	Efficiency, m^3/ hr.	Water Consumption, litres /sec	Efficiency, m^3/ hr.	Water Consumption, litres /sec	Efficiency, m^3/ hr.	Water Consumption litres /sec	Efficiency, m^3/ hr.
I	49	14.6	78	23.4	112	33.8	196	59.0
II	49	8.8	78	14.0	112	20.5	196	35.4
III	49	4.9	78	7.8	112	11.2	196	19.6
IV	49	2.71	78	4.32	112	6.2	196	10.9
V	49	2.52	78	4.0	112	5.76	196	10.0

Category I includes peat with no roots, loose top soil, loose sandy-clayey ground;

category II—sandy pebbles or clay-cemented tough ground containing some pebble and coarse gravel (up to 30%);

category III—tough clays with boulders up to 50 cm in diameter amounting to 15% of the total, clay-bounded debris of bedrock, broken shales;

category IV—tough clays with boulders over 50 cm in diameter amounting to 3% of the total, unbroken marl and clay-cemented sandstones; weakly cemented conglomerates; frozen ground up to 30%;

category V--very tought clays with 50% of boulders over 50 cm in diameter, semibroken sandstones; frozen ground up to 50%.

Bench deposits are pre-eminently those to which the hydraulic method is applicable, because adequate grade for the disposal of tailing can generally be secured, and such benches are usually backed by mountains from which water under pressure can be had. Deposits in the beds of present streams, on the other hand, are less exploitable by hydraulicing. If water is cheap and plentiful, the hydraulic elevator might be used, as the wear and tear and the attention needed are small. On the other hand, the efficiency is low; and if the supply of water is not enough, it will be required to use the water in a more efficient way.

There are several alternatives in selecting a source and a means of transporting the water to the mining site. These include a diversion dam, a storage dam, a recirculating pond, a gravity ditch, a pumping plant or combination of these facilities. Selection of the method to be used is based on water requirements, water availability, life of the mine and the cost of the system.

In typically smaller scale operations where water is readily available, and only to be used for sluicing without storage, a simple diversion dam may be utilized. In this case, a pipe at the base of the dam may transport water to the head of the sluice or the dam gate may terminate at the sluice box, as shown in Figure 4.

However, present day highly mechanized operations require a good degree of maneuverability over shorter periods of time usually rely on small earth filled storage dams, or a conveniently located supply sources as shown in Figure 5. In these cases a pumping plant and pipeline or hose transportation system is utilized. When working bench grounds or other areas where water supply is low, it is often necessary to recirculate the water by constructing a storage dam below the sluicing operation, as shown in Figure 6. This may be detrimental to gold recovery if the recirculated water builds up a high solid content.

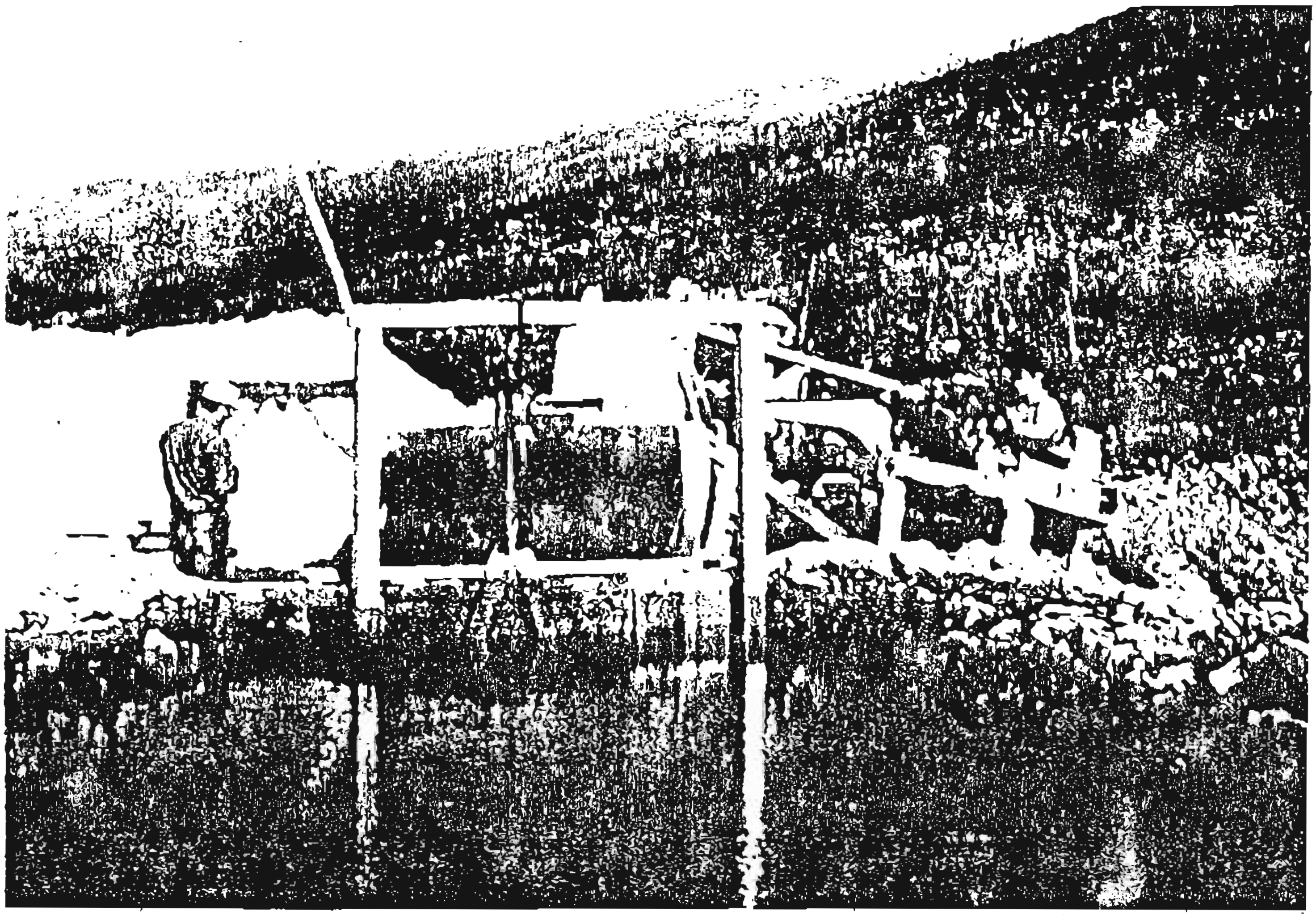


Figure 4. Diversion dam at Head of Sluice box

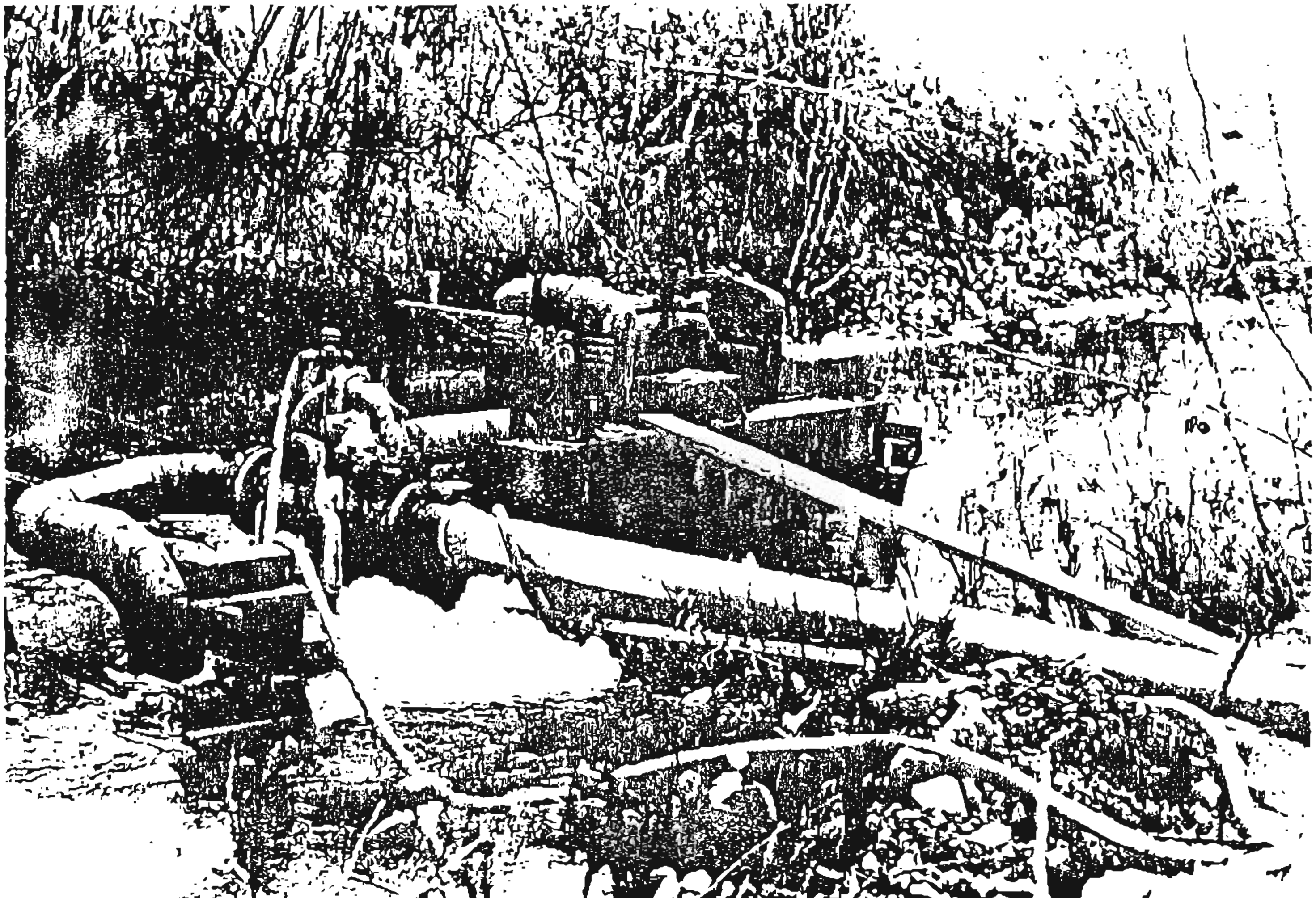


Figure 5. Storage pond and pump

Past operations requiring large volumes of water for hydraulic stripping, hydraulic mining, thawing and dredging relied heavily on gravity ditches to transport water long distances and pick up drainage from the surrounding watershed, as shown in Figure 7. In all cases, the best working results can be obtained only when the nature of water supply has been carefully studied and thoroughly understood.

BACKGROUND WATER RESOURCES INFORMATION

As part of every placer mining permit applications, the applicant must include background surface water information (Appendix A) that includes minimum, maximum, and average discharge conditions which identify crucial low flow and pick discharge rates of streams to identify seasonal variations.

One way of meeting these requirements without conducting long term stream gaging is through the use of regional frequency analysis. Lichty and Rightnour (1979) proposed a method to do this for mine areas where stream gages maintained by government agencies are dense enough to perform such analysis. The method is a modified USGS-Index Flood Method, and presents a map which indicates seasons of high and low flow. In the approach, critical low flow is constructed to be the 7-day 10-year low while peak flows are assumed to be peak daily averages. The method uses the following step-by-step procedure (Skelly and Loy, 1979):

1. Locate stream flow gaging stations in the mine area for watersheds of similar topographic, hydrologic and land cover conditions.
2. Determine the availability and reliability of the records for the gages and select the best suited for data collection.
3. Obtain daily flow records and perform high, low and average flow freqency analyses.

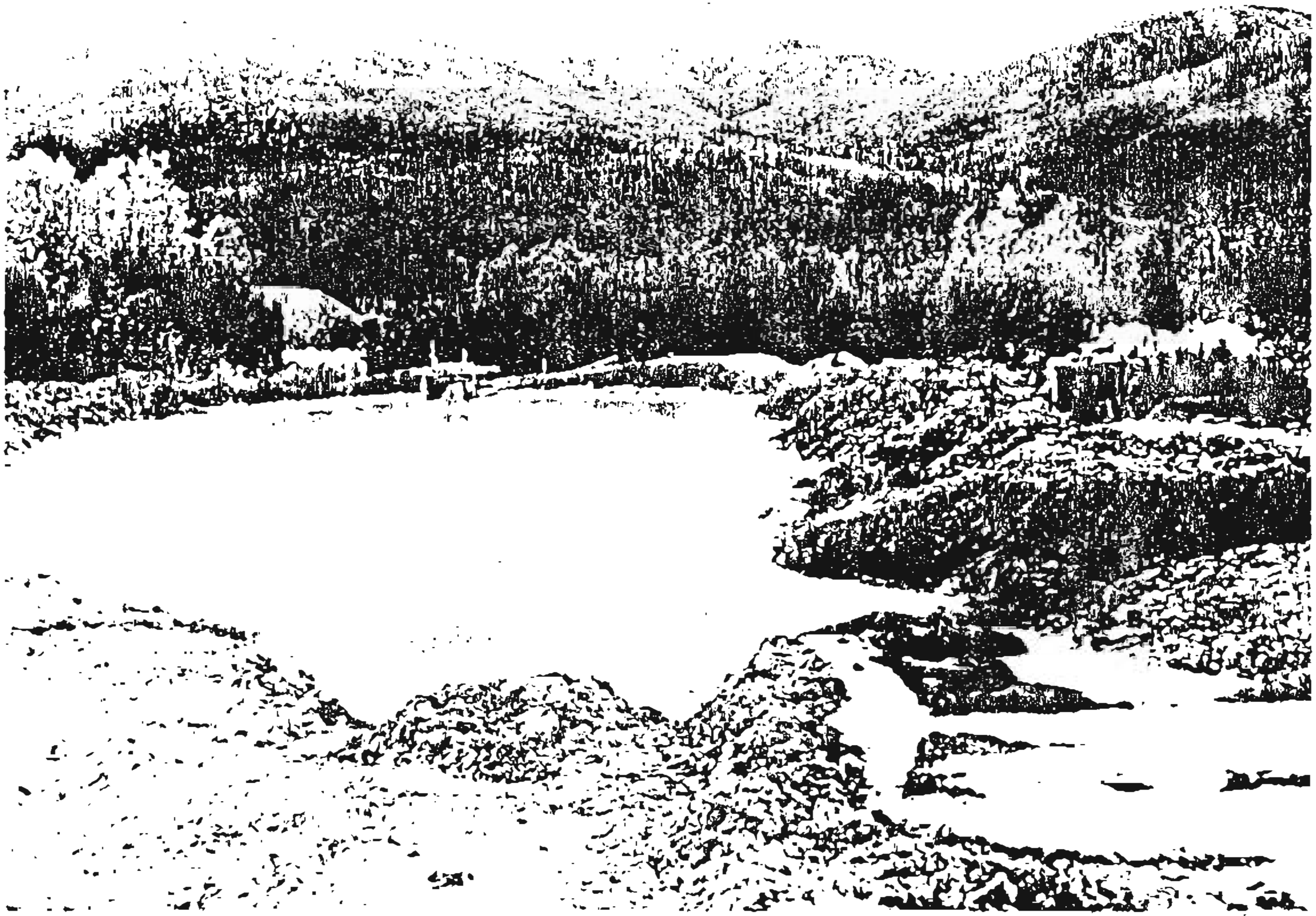


Figure 6. Recirculation Pond

Figure 7. Gravity ditch for hydraulic stripping

4. Perform a homogeneity test to determine pertinent data. Reject non-homogeneous gages.
5. Develop a high flow frequency curve using a modified index flood method.
6. Perform a mathematical regression to determine the index flood, low flow, and average flow as a function of drainage area.
7. Estimate seasonal variation of flow based on national correlation.

The procedure can be used quite successfully where gages are plentiful, but may not be applicable where gages are limited. In the regions where little or no existing data is available stream monitoring or other techniques for determining peak flows from ungaged watersheds need to be developed.

CHAPTER IV

Regional Frequency Analysis of Alaskan Streams

The velocity of flow and total discharge are extremely important for long range mine plan. Determination of baseline conditions for several variables (Table 5) involves analyses of existing flow information from the potential mining area with regard to daily maximum and minimum flows, yearly flows, as well as the rate of change of flow from maximum to minimum.

Prediction of adequate water availability would involve hydraulic-related calculations to estimate changes in daily maximum to minimum flow, as well as the time period over which these flow changes are anticipated to occur. Numerous mathematical models are available for accomplishing these predictions. The attached table (Table 6) from the Urban Institute (1976) provides a comparison of techniques used in the temperate region to estimate changes in stream flow.

An insufficient hydrologic data base exists for most large and small basins, owing to the lack of need in the past to collect data, and due to terrain accessability.

The runoff procedure is insufficiently understood and is complicated by specifically northern phenomena such as abrupt spring breakup, the general winter time stream-icing phenomena, and the lack of understanding of hydrologic relationship in a permafrost environment. These complications result in the general inability of techniques developed in the temperate regions to make good estimates of stream flow parameters and flood magnitudes (Carlson and Fox, 1974, 1976).

This section of the report is directed toward the examination and development of better methods for flood frequency design and stream flow

Table 5. Partial summary of items for consideration in evaluation of proposed mining and reclamation activities in a watershed

1. Location of existing or planned disturbances (mines, haulage-ways)

2. Sedimentation and erosion

   Loss of soil (rate and annual total)
   Effect on water character and treatment
   Effect of deposits on aquatic life, navigation, capacity
   Control techniques

      Sediment dams
      Runoff velocity reduction
      Diversion
      Revegetation

3. Water quality

   Chemical properties
   Physical properties
   Treatment methods

      Chemical
      Mechanical
      Alternate uses

4. Water supply

   Water quantity and sources
   Water flow characteristics
   Flood control installations and procedures
   Flood plain land and water use
   Water uses

5. Land use

   Present and projected future uses of land and reclaimed land.

Table 6. Comparison of techniques used to estimate change in stream flow* (Urban Institute, 1976)

	Types of Water Bodies	Watershed	Computing Requirements
Rational Method	Streams	Less than ~5 mi^2	Compilation of precipitation tables, manual computation
Flood Frequency Analysis	Streams, lakes estuaries	No limit	Access to a digital computer desirable to perform regression analyses and to fit flood data into the accepted distributional form
Hydrocomp Simulation Program (HSP)	Streams, lakes reservoirs	No limit	Designed for use on the IBM 360 or 370 computer

	Input	Cost	Output
Rational Method	Precipitation depth-frequency-duration tables, percent impervious ground cover in the watershed	Relatively low	Peak stream flow for storms of various degrees of severity
Flood Frequency Analysis	Stream flow records for gauged streams, watershed size and slope, average annual precipitation, and land use for numerous watersheds for several years	Low-medium (since additional time-consuming calculations are necessary)	Peak stream flow for storms of various degrees of severity
Hydrocomp Simulation Program (HSP)	Hourly precipitation and evaporation; extent, location and type of sewerage and ground cover	Approximately $10/ac for small watersheds, considerbly less for	Continuous stream flow hydrographs for as many points in the watershed and for as

Table 6. Comparison of techniques used to estimate change in stream flow* (continued)

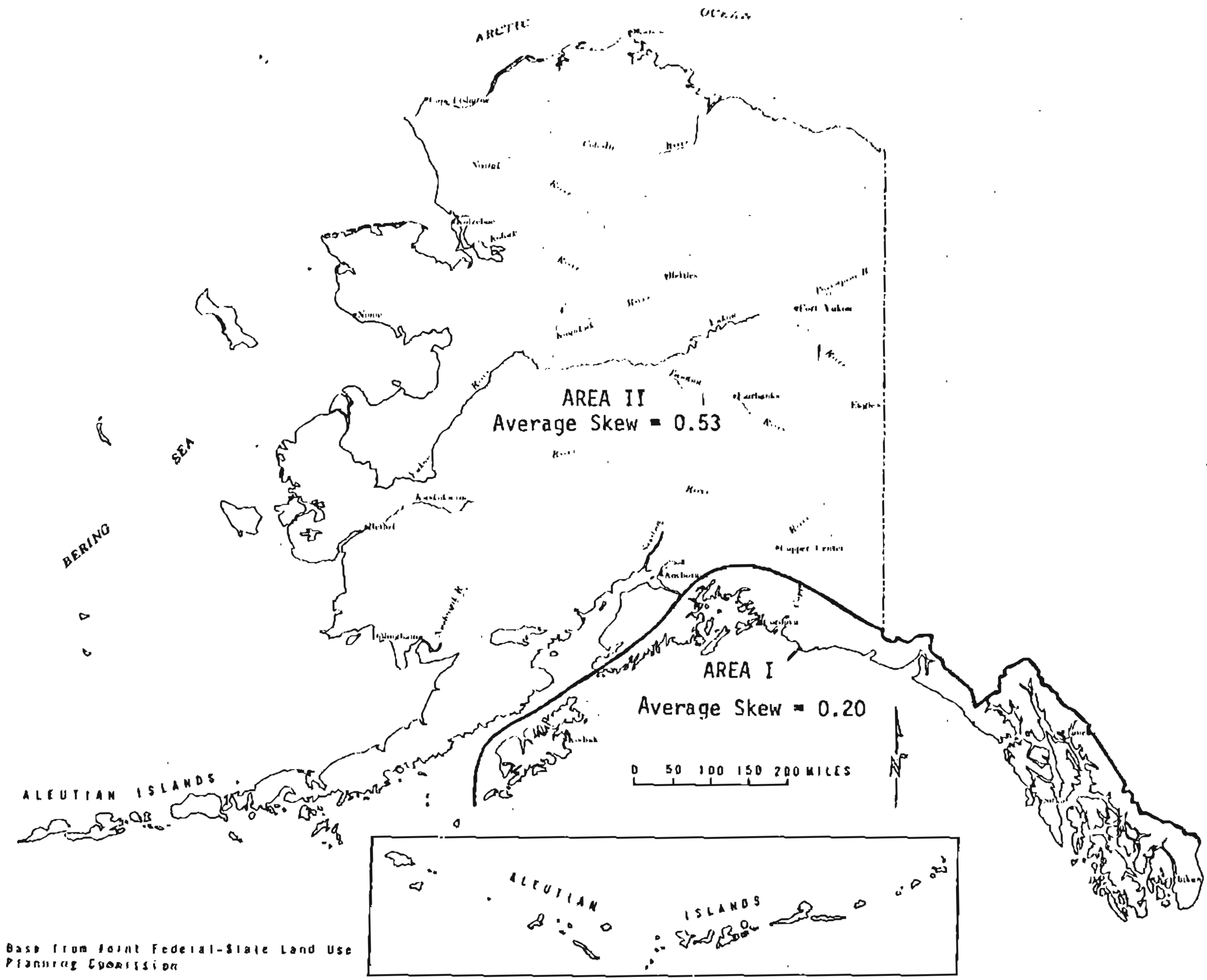
	Input	Cost	Output
	in watershed; channel configuration (for snowfall—daily and maximum and minimum temperatures, point, wind velocity, radiation and cloud cover desirable)	large ones	many years as desired

	Accuracy
Rational Method	Some reports of errors as great as 50% in reproducing past events
Flood Frequency Analysis	High for reproducing past events once it has been calibrated; unknown for future events
Hydrocomp Simulation Program (HSP)	High for reproducing past events and "good" for future events as rated by the developers, although no documentation is available

analyses in the northern sparse data region. The objective is to generate information useful for a design tool in estimation of high and low flow for specific durations and periods of the year. Using these methods, the design flow can be predicted during the critical period of the year.

Estimation of Stream Flow in Ungaged Basins

In a recent study Ashton and Carlson (1983) used streamflow data from continuously recording U.S. Geological Survey gaging stations in the hydrologically similar area (Area II), as shown in Figure 8, defined by Lamke (1979). Stations within this region were deleted from further consideration if the basin area was greater than 100 mi^2, 20% or more of the basin area was



Base from Joint Federal-State Land Use Planning Commission

Figure 8. Estimation of stream flow in ungaged basins

covered by glaciers, the streamflow was regulated, or there were less than five years of record as of November, 1981. Aleutian island stations, although within Lamke's region definition, were deleted from consideration. Outliers, discharge values which deviate from the general trend, and stations with periods of zero flow are treated as described in Kite (1977). Three periods of the year were selected for streamflow analysis: spring, April 1 to June 30; summer, July 1 to August 31; and fall, September 1 to November 30. For each period, the highest consecutive mean discharge with durations of one and three days and the lowest consecutive mean discharge with a duration of seven days were computed.

Single station data using multiple linear regression techniques was regionalized and then multiple linear regression equations were developed using basin and climatic characteristics to predict the 1- and 3-day duration, 2-year return period high flow and the 7-day duration, 5- and 10-year return period low flow. The regression equations that were developed for the 1- and 3-day duration high flow and 7-day duration low flow for the spring, summer and fall periods are given as:

$$Q = a\ A^b\ B^c\ C^d\ D^e \tag{4.1}$$

where

- Q = dependent variable, the discharge for a specific duration and return period;
- a = regression constant;
- b, c, d and e = regression coefficients for the independent variables;
- A, B, C and D = independent variables, basin and climatic characteristics.

Variables considered in the regression analyses were: drainage area; mean annual precipitation; percentage of drainage basin covered by forests, glaciers and lakes; main channel slope; stream length; mean basin elevation; mean minimum January temperature; 2-year, 24 hour precipitation intensity; and mean annual snowfall.

Estimation of High Flows

In their analysis (Ashton and Carlson, 1983) considered thirty-three gaging stations which met the criteria of basin size, percent of drainage area as glaciers, and length of record. For high flow the basin and climatic characteristics found significant are: drainage area, mean annual precipitation, mean minimum January temperature, and percent forest for spring; drainage area, mean annual precipitation, percent forest and channel slope for summer and drainage area and mean annual precipitation for fall. The 1- and 3-day duration, 2-year return period, high flow is predicted for ungaged basins using equation 2.

$$Q(m, n) = a A^{b} b^{c} (F + 1)^{e} \tag{4.2}$$

where

$Q(m, n)$ = dependent variable, the highest consecutive mean discharge for the mth period, where S is spring, Su is summer, and F is fall, and the nth duration where 1 is one day and 3 is three days, ft^3/s;

a = regression constant;

b, c, and e = regression coefficients for the independent variables (basin and climatic characteristics);

A = drainage area, mi^2;

= mean annual precipitation, inches;

F = Percentage of drainage basin covered by forest, expressed as a whole number.

The regression coefficients, with their associated average percent standard error, are given in Table 7. Table 8 presents the basin and climatic characteristics of the stations used in the analysis.

Estimation of Low Flows

Basin and climatic characteristics found significant for low flows are: drainage area and mean minimum January temperature during the spring and fall and drainage area and mean annual precipitation during the summer. The 7-day duration, 5- and 10-year return period low flow is predicted for ungaged basins using equation 3.

$$Q(m, n) = (a\, A^b\, P^c\, (T + 30)^d) - 1 \qquad (4.3)$$

where

$Q(m, n)$ = dependent variable, the lowest consecutive mean discharge for the mth period, where s is spring, su is summer, and f is fall, and the nth duration where 7 is seven days, ft^3/s;

a = regression constant;

b, c and d = regression coefficients for the independent variables (basin and climatic characteristics);

A = drainage area, mi^2;

P = mean annual precipitation, inches;

T = mean minimum January temperature, ^oF.

The regression coefficients, with their associated average percent standard error, are given in Table 7. Table 8 presents the basin and climatic characteristics of the stations used in this analysis.

The regionalization of single station data presented in that report provides a method to predict high and low flow for drainage basins smaller than 100 sq. miles in Alaska. Flow magnitudes can be predicted for the season of the year, flow duration, and the frequency of occurrence of interest. The regionalization provides the mine operator a means to predict design flows during the spring, summer and fall of the year. The mine operator can make a reasonable prediction of the design flow given the season of the year, whether high flow or low flow is of concern, and the duration of interest.

TABLE 7. Regression constants and coefficients for predicting high and low flows for selected durations and return periods.
(Ashton and Carlson, 1983)

Equation Number	Dependent Variable Qmn	Regression Constant a	Regression Coefficients b	c	d	e	f	Percent Average Standard Error
High flows with 2-year return period								
2a	Q(s,1)	2.712	0.812	0.831	-0.698	-0.396	—	22
2b	Q(s,3)	2.010	0.822	0.874	—	-0.393	—	24
2c	Q(su,1)	0.109	0.947	1.066	—	-0.405	0.323	16
2d	Q(su,3)	0.234	0.900	1.273	—	-0.359	—	20
2e	Q(f,1)	0.0744	0.773	1.331	—	—	—	21
2f	Q(f,3)	0.0632	0.783	1.336	—	—	—	20
Low flows with 5-year return period								
3a	Q(s,7)	0.0131	0.487	—	1.366	—	—	23
3b	Q(su,7)	0.0272	0.729	1.302	—	—	—	30
3c	Q(f,7)	0.00962	0.594	—	1.528	—	—	23
Low flows with 10-year return period								
3d	Q(s,7)	0.0147	0.452	—	1.331	—	—	23
3e	Q(su,7)	0.0252	0.716	1.292	—	—	—	32
3f	Q(f,7)	0.0106	0.575	—	1.478	—	—	23

Table 8. Basin and climatic characteristics of selected gaging stations (Ashton and Carlson, 1983)

Station No.	Station Name	Location Latitude (degrees)	Location Longitude (degrees)	Drainage Area (mi^2)	Main Channel Slope (ft/mi)	Stream Length (mi)	Mean Basin Elevation (ft)	Area of lakes and ponds (percent)	Area of forests (percent)
15207800									
15208100	Squirrel Creek at Tonsina	61.67	145.17	70.50	119	17.9	3,100	4	58
15244000	Ptarmigan Creek at Lawing	60.41	149.36	32.60	220	14.6	2,800	6	46
15246000	Grant Creek near Moose Pass	60.46	149.35	44.20	150	12.8	2,900	10	20
15254000	Crescent Creek near Cooper Landing	60.50	149.68	31.70	136	14.7	2,700	13	38
15260000	Cooper Creek near Cooper Landing	60.43	149.82	31.80	194	9.9	2,400	16	44
15260500	Stetson Creek near Cooper Landing	60.44	149.85	8.60	459	4.8	3,200	0	47
15261000	Cooper Creek at mouth near Cooper Landing	60.47	149.87	48.00	74.1	13.5	2,500	10	49
15264000	Russian River near Cooper Landing	60.45	149.98	16.80	116.0	23.5	2,100	4	51
15266500	Beaver Creek near Kenai	60.56	151.12	51.00	4.75	13.5	140	15	67
15272550	Glacier Creek at Girdwood	60.94	149.16	62.0	455	11.0	2,610	0	28
15273900	SF Campbell Creek at canyon mouth near Anchorage	61.15	149.72	25.2	255	9.2	2,760	1	8
15274000	SF Campbell Creek near Anchorage	61.17	149.77	30.4	246	11.5	2,530	1	26
15274300	NF Campbell Creek near Anchorage	61.17	149.76	13.4	389	10.6	2,670	2	30
15274600	Campbell Creek near Spenard	61.14	149.92	69.7	162	19.2	1,680	1	46
15275000	Chester Creek at Anchorage	61.20	149.84	20.0	226	11.4	800	1	61
15275100	Chester Creek at Arctic Blvd. at Anchorage	61.21	145.90	27.20	169	12.8	780	1	59
15277410	Peters Creek near Birchwood	61.42	149.49	87.8	133	21.0	3,150	0	23
15286000	Cottonwood Creek near Wasilla	61.57	149.41	28.50	44.0	11.4	500	6	85
15290000	Little Susitna River near Palmer	61.71	149.23	61.90	187	14.9	3,700	0	16
15297900	Eskimo Creek at King Salmon	58.69	156.67	16.10	18.2	7.3	140	5	14
15302800	Grant Lake Outlet near Aleknagik	59.80	158.55	34.30	82.66	9.0	876	12	52
15439800	Boulder Creek near Central	65.57	144.89	31.30	154.8	12.4	2,570	0	73
15476300	Berry Creek near Dot Lake	63.69	144.36	65.10	223	19.1	3,200	1	40
15515800	Seattle Creek near Cantwell	63.33	148.25	36.20	169	10.20	3,400	2	6
15534900	Poker Creek near Chatanika	65.16	147.48	23.1	130	9.75	1,710	0	91
15535000	Caribou Creek near Chatanika	65.15	147.55	9.19	229	3.5	1,640	0	97
15564877	Wiseman Creek at Wiseman	67.41	150.11	49.20	171	14.0	2,930	0	3
15565235	Ophir Creek near Takotna	63.15	156.52	6.19	79	6.4	1,070	0	86
15621000	Snake River near Nome	64.56	165.51	85.70	19.60	19.50	632	0	4
15668200	Crater Creek near Nome	64.93	164.87	21.90	145	9.2	1,620	1	3
15798700	Nunavak Creek near Barrow	71.26	156.78	2.79	13.0	2.5	40	22	0
15904900	Antigun River tributary near pump station 4	68.77	149.31	32.6	210	10.2	5,100	0	0

CHAPTER V

Flood Damage Probability Evaluations

It may be apparent that the maximum observed streamflow (the peak flow) observed on any stream over a period of one year varies from year to year in an apparently random fashion. This randomness has led to the use of probability and statistics in selecting capacity of flood water facilities.

Assessing benefits from flood probability evaluations and flood control projects and selecting the optimal solution is essentially a matter of managerial and engineering judgement. Water requirement and system structure can only be defined on the basis of a mining plan. However, the iterative and feedback nature of the process must be noted since mine production planning must consider the services required to support the operation. Therefore, it is recognized that mathematical models of the production system, economic model, stream flow estimation and probability evaluation can be regarded as useful tools that can help to evaluate specific issues, as the economic value of flood damage, that have great influence on the choice.

The return period of a T- year flood event is defined as an event of such magnitude that over a long period of time (much, much longer than T- years), the average time between the events having a magnitude equal to or greater than T-years event is T-years. Often the actual time between the occurrences of a T- year event is called the recurrence interval. Since the average time between occurrences of a T- year event is T- years, the probability of a T-year event in any given year is 1/T. Thus we have the relationship:

$$P_T = 1/T \tag{5.1}$$

Where T is the return period associated with an event Q_T and P_T is the probability of Q_T in any given year.

The damage done by a flood exceeding the protection level may depend on several factors:

— the development in the flooded area,

— existing structures,

— severity of the flood,

— flood protection system employed.

In selecting risk criteria two cases should be considered:

1. Any event exceeding the protection level is a catastrophic event, since it produces enormous damage to the property, that its occurrence can not be accepted.

2. Events exceeding the protection level are not catastrophic, since the damage produced is not so great to be surely unacceptable, and a certain risk level can be accepted.

Many government units have regulations governing the design period to be used. Often these return periods are based on the size of the structure and the consequence of the structural hydraulic capacity being exceeded. For example, Table 9 shows the design return period specified by Federal Regulation for surface mines of 1977.

Probability Evaluations

For the evaluation of specific probability, several assumptions must be made, that the peak flows from year-to-year are independent of each other. This means that the magnitude of a peak flow in any year is unaffected by the magnitude of a peak in any other year. It is also assumed that the statistical properties of the peak flows are not changing with time. This means that there is no changes going on within the watershed that results in changes in the peak flow characteristics of the watershed (Hann and Barfield, 1978).

Table 9. Design Return Periods for Certain Facilities Connected with Surface Mines

Item	Return Period
Water Quality Effluent Standards	10-year, 24-hour rain
Settling Ponds	
Volume of Runoff	10-year, 24-hour rain
Spillways (small ponds)	25-year rain
Spillways (large ponds)	100-year, 6-hour rain
Roads	
Out of Flood Plain	100-year
Water Control Structures	10-year

Under these assumptions, the occurrence of a T- year event is a random process meeting the requirements a particular stochastic process known as Bernoulli process. The probability of Q_T being equaled or exceeded in any year is p for all time and is unaffected by any prior history of occurrence of Q_T. If any event equaling or exceeding Q_T denotated as Q_T^*, than the Q_T^* is a Bernoulli random variable. The probability of K occurrence of Q_T^* in n years can be evaluated from the binomial distribution:

$$f(k, P_T, n) = \frac{n!}{(n-k)!\,K!} (P_T)^K (1-P_T)^{n-K} \qquad (5.2)$$

$$= 0.26$$

Where $f(K; P_T, n)$ is the probability of K occurrences of Q_T^* in n years if the probability of Q_T^* in any single year is P_T. For example, the probability of 2 occurrences of a 20 year event in 30 years is:

$$f(2; 0.05, 30) = \frac{30!}{(28)!\,2!}(0.05)^2(0.95)^{28}$$

$$= 0.26$$

In a large number of 30 year records, one would expect 26% of the records to contain exactly 2 peaks that equal to or excees Q_{20}. The other 74% of the 20-year records would contain 0, 1, 3, 4 . . . . or 30 peaks that equal or exceed Q_{20}. The probability of these later number of exceedances can be evaluated from equation 5.2 also. If this is done, the summation of the probabilities of 0, 1, 2, 3 . . . 30 peaks in 30 years equal to or greater than Q_{20} must equal 1.0 since all possibilities have been exhausted.

Equation 5.2 can be used to calculate the probability that a T- year event will be equaled or exceeded at least once in an n-year period by noting that 'at least once' means one or more. The probability of one or more exceedances is given by:

$$1 - f\ (0;\ P_T,\ n) = 1 - \frac{n!}{0!\ n!}\ P_T^{\,o}\ (1-P_T)^n$$

Since $P_T = 1/T$ and $0! = 1$, this relationship reduces to:

$$f(P_T,\ n) = 1-\ (1-1/T)^n \quad . \ . \ . \ . \ (5.3)$$

Where $f\ (P_T,\ n)$ is the probability that a T year event will be equaled or exceeded at least once in an n-year period. If n is set equal to T in equation (5.3), it can be shown that for large T, the probability, $f(P_T,\ T)$ approaches the constant 0.632. What this means is that if a structure having a design life of T- years is designed on the basis of a T- year event, the probability is approximately 0.63 that the design capacity will be exceeded at least once during the design life.

By specifying the acceptable probability of the designed capacity being exceeded during the design life of a structure, equation (5.3) can be used to calculate the required design return period. For example, if one wants to be 90 percent confident of not exceeding the design capacity of a structure in a

25-year period, the probability, $f(P_T, 25)$ would be $1 - 0.90 = 0.10$. Thus from equation 5.3:

$$0.10 = 1 - (1-1/T)^{25}$$

or $T = 238$ years.

To have a 90 percent confidence of not exceeding the design capacity in 25 year period the design capacity must be based on an event with a return period of 238 years. In this case the acceptable risk was only 10 percent, the degree of confidence was as high as 90 percent, the design life was 25 years and the required design return period was 238 years. Calculations like this can be carried out for various design lifes, design return period and acceptable risks. Figure 9 is based on such calculations and can be used to quickly determine the required design return period based on the design life and acceptable risk or probability of having the designed capacity exceeded (Hann and Barfield, 1978).

In this discussion it should be kept in mind that a high risk of having the design capacity exceeded may be acceptable since what is meant by exceeded is failure of the structure to handle the resulting flow in the manner the structure was designed to operate. Failure in this sense does not necessarily mean that the structure will be destroyed. For example, the failure of a road culvert to pass a peak flow may result in only minor flooding of a roadway or adjacent area and may be acceptable on a fairly frequent basis. On the other hand, failure of a settling pond may result in considerable damage to property and high risk of pollution downstream. Thus the selection of the acceptable risk and the design return period depend on the consequence of the design capacity being exceeded. Building the structure large enough to protect against extremely rare events is quite expensive while allowing the design capacity to be exceeded on a frequent basis may result in an accumulation of

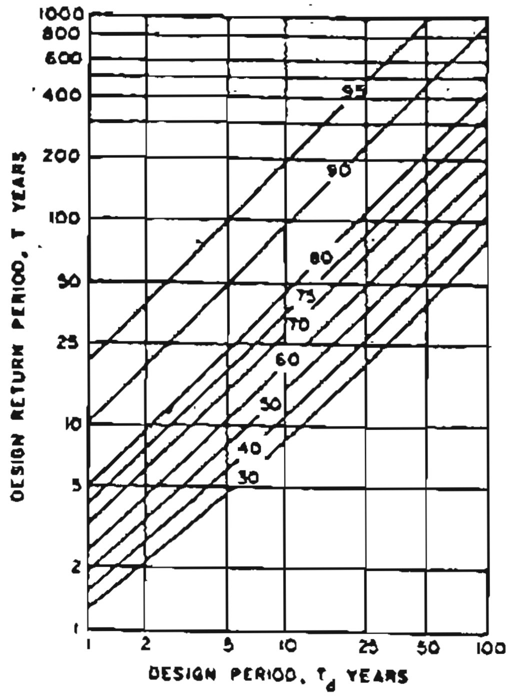


Figure 9. Design return period
(Hann and Barfield, 1978)

considerable economic loss. Thus the selection of the proper design return period is a problem in economic optimization and is beyond the scope of this project.

CHAPTER VI

Hydrologic Computations & Design Guide Lines for Flood Flow Buffers and Diversion Channels

Assigning a flood magnitude to a given return period requires knowledge of the flood flow characteristics of the basin of concern. The approach that is used to determine this relationship depends largely on the type, quantity and quality of hydrological data that is available and on the importance of the determination.

The possible situation that a placer mining operator might be faced with are as follows:

I. - A reasonably long record of stream flow is available at or near the point on the stream interest.

II. - A reasonably long record of streamflow is available on the stream of interest but at a point somewhat removed from the location of interest.

III. - A short streamflow record is available on the stream of interest.

IV. - No records are available on the stream of interest but records are available on the nearby streams.

V. - No streamflow records are available in the vicinity.

The methodologies that can be used for determination of flood frequency under various situations are shown as a flow diagram (Figure 10). The detailed methodology and hydrological computations for the determinations of flood frequency for the first three situations are adequately presented in any hydrology text. The determination of flood frequency estimations procedure in sparse data region (case IV & V) has been discussed in Chapter IV of this report, therefore, the scope of the work in this section will be limited to a discussion on hydrologic computations and use of the equations developed there

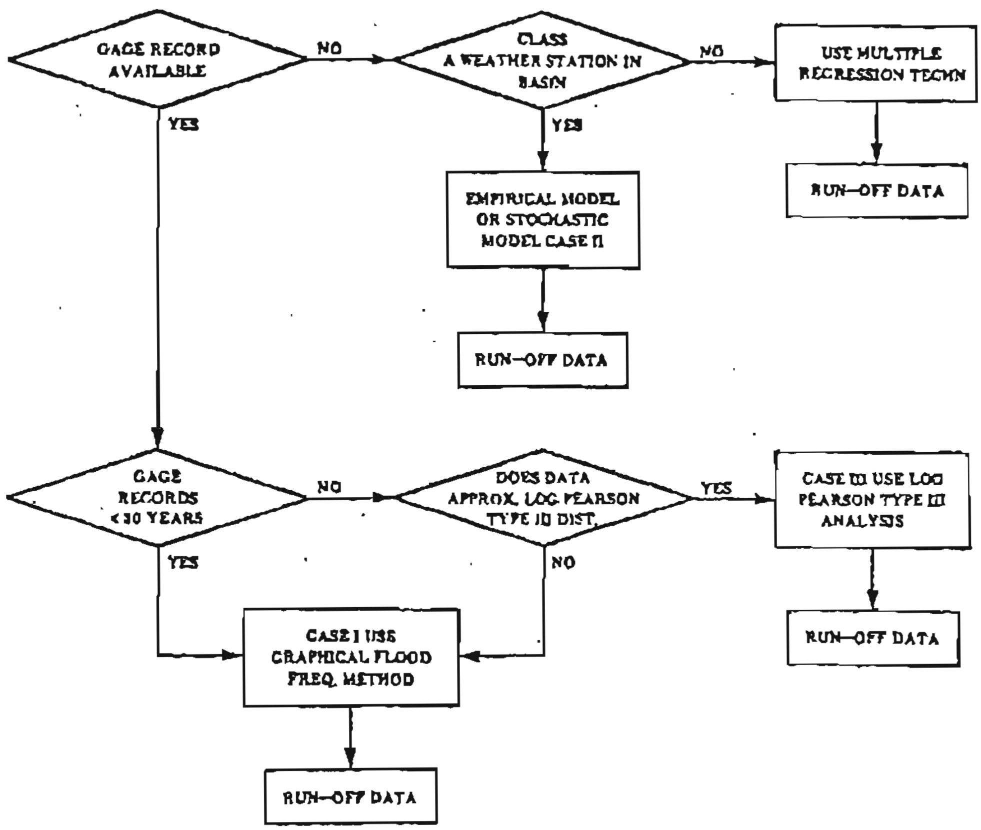


Figure 10. Stream flow analysis

for use in planning of mining operations. While the methodology discussed here will be useful in any surface mining situation, the scope of work will be limited to a discussion for the use of those equations in planning placer mining in the northern region of Alaska. A variety of hydrologic computation must be performed when designing a mining operation and sediment control facilities. For different situations the mining operator must use a specific rainfall event and determine runoff characteristics in one of the following form:

— Total runoff volume

— Runoff peak (High) flow

— Runoff low flow

— Plotting of hydrograph

Hydrologic computations of runoff for a given event are shown in the following examples:

Design Examples

The following examples are taken from a recent water research institute report (Ashton & Carlson, 1983) to illustrate the application of equation (4.2) and equation (4.3).

The streams used in these examples are hypothetical with the input data (drainage area, season of interest, mean annual precipitation, etc.) selected to illustrate selected applications of this report. For each mining site the mining operator must have, information regarding the size of the mine, whether high flow or low flow is of concern, the critical mining period, i.e., spring, summer or fall, and the tolerable delay, i.e., one or three days.

Example 1.

For creek A near Coldfoot on the Dalton Highway the 1-day, 2-year return period spring high flow and the 7-day, 5-year return period fall low flow have been determined to be important for mine planning.

From U.S. Geological Survey maps,

the drainage area is	$23.4 mi^2$
the percent drainage area as forest is	4%

From Figure 11

the mean annual precipitation is	19 inches

From Figure 12

the mean minimum January temperature is	$-18^{o}F$

For high flows: to compute the spring 1-day, 2-year return period flow use equation 4.2, the values for the coefficients were obtained from Table 7:

Equation 4.2

$$Q(S, 1) = 2.712 \; A^{0.812} \; P^{0.831} \; (F+1)^{-0.396}$$

$$Q(S, 1) = 2.712 \; (23.4)^{0.812} \; (19)^{0.831} \; (4+1)^{-0.396}$$

$$Q(S, 1) = Q\;(S,1) = 227 \; ft^3/S$$

For low flows: to compute the fall 7-day, 5-year return period flow use equation 4.3 with values of the coefficient from Table 7:

Equation 4.3

$$Q(f, 7) = (0.00962 \; A^{0.594} \; (T+30)^{1.528}) - 1$$

$$Q(f, 7) = (0.00962 \; (23.4)^{0.594} \; (-18+30)^{1.528}) - 1$$

$$Q(f, 7) = 1.8 \; ft^3/s$$

For this stream the design discharge are 227 ft^3/s for high flows and 1.8 ft^3/s for low flows.

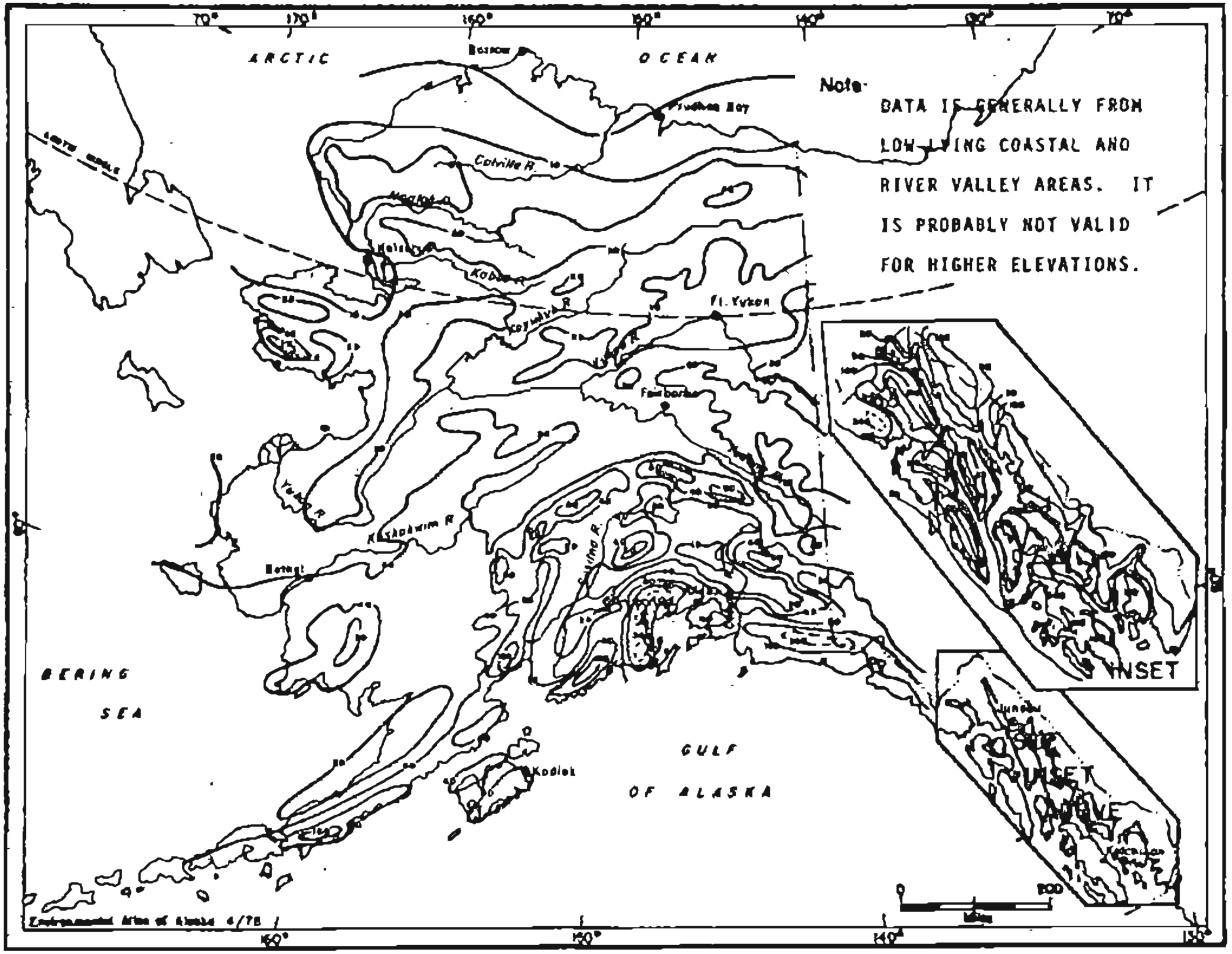


Figure 11. Mean Annual precipitation in Alaska

Source: Watson (1959)

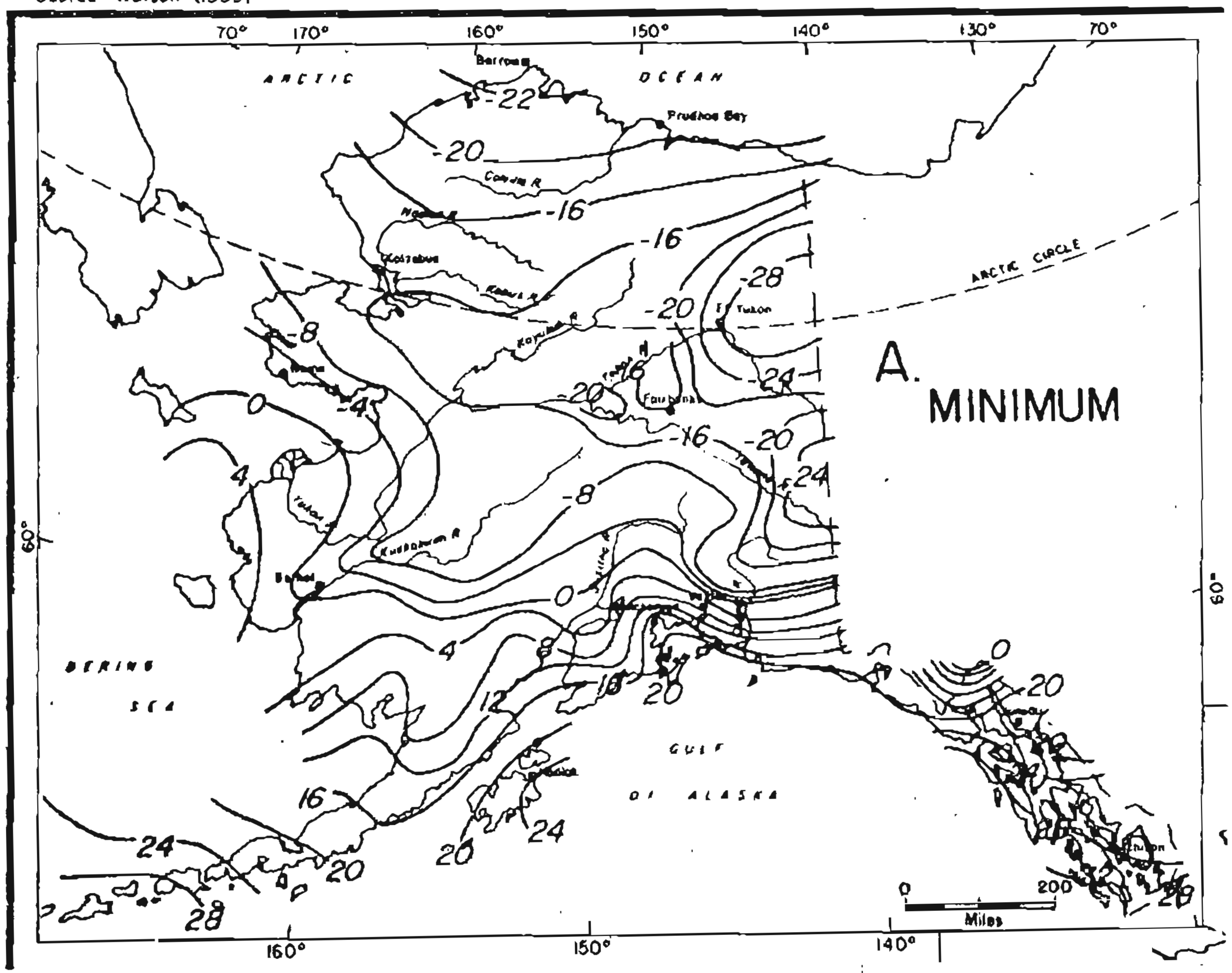


Figure 12. Mean Minimum January Temperature in Alaska

Example 2.

For creek B near Wasilla on the Parks Highway the 3-day, 2-year return period spring and summer high flows and the 7-day, 10-year return period summer and fall low flow have been determined to be important.

From U.S. Geological Survey maps,

the drainage area is 11.5 mi^2

The percent drainage area as forest is 67%

From Figure 11

the mean annual precipitation is 25 inches

From Figure 12

the mean minimum January temperature is $0^{o}F$

For high flows: to compute the spring 3-day, duration 2-year return period flow use equation 4.2 and values for the coefficient from Table 7:

Equation 4.2

$$Q(S, 3) = 2{,}010\ A^{0.822}\ P^{0.874}\ (F+1)^{-0.393}$$

$$Q(S, 3) = 2.010\ (11.5)^{0.822}\ (25)^{0.874}\ (67+1)^{-0.393}$$

$$Q(S, 3) = 48.0\ ft^3/s$$

To compute the summer 3-day, 2-year return period flow use equation 4.2 with values of the coefficient from Table 7:

Equation 4.2

$$Q(Su, 3) = 0.234\ A^{0.900}\ P^{1.273}\ (F+1)^{-0.359}$$

$$Q(Su, 3) = 0.234\ (11.5)^{0.900}\ (25)^{1.273}\ (67+1)^{-0.359}$$

$$Q(Su, 3) = 28\ ft^3/sec$$

For low flows: to compute the summer 7-day, 10-year return period flow use equation 4.3 and coefficient values from Table 7:

Equation 4.3

$$Q(Su, 7) = (0.0252\ A^{0.716}\ P^{1.292}) - 1$$

$$Q(Su, 7) = (0.0252\ (11.5)^{0.716}\ (25)^{1.292}) - 1$$

$$Q(Su, 7) = 8.3\ ft^3/s$$

To compute the fall 7-day, 10-year return period flow use equation 4.3 and the coefficient values from Table 7:

Equation 4.3

$$Q(f, 7) = (0.0106\ A^{0.575}\ (T+30)^{1.478}) - 1$$

$$Q(f, 7) = (0.0106\ (11.5)^{0.575}\ (0+30)^{1.478}) - 1$$

$$Q(f, 7) = 5.6\ ft^3/s$$

For streams with two critical mining periods select the highest high flow and the lowest low flow for the design discharge. For this stream the design discharges are 48 ft^3/s for high flows and 5.6 ft^3/sec for low flow.

Design Aids

The estimation of water demand is the primary item in water supply planning.

Reference may be made to the previous two examples for a procedure to determine high and low flow for a given water shed. The decision process involved at this stage is to determine if the water requirement for the design (planned) placer mined can be satisfied. Additionally, the mine planner must choose a mode of transportation of water, if the water supply is adequate. On the other hand, the water supply is inadequate during the critical mining period, the mine planner must decide on a storage dam and its size to assure maximum operating time in dry seasons. The decision process involved in planning is shown as a flow diagram in Figure 13.

Design of Stable Channels

Design events are stipulated in the regulations for each type of channel that may occur in mining activities. Table 11 summarizes these requirements.

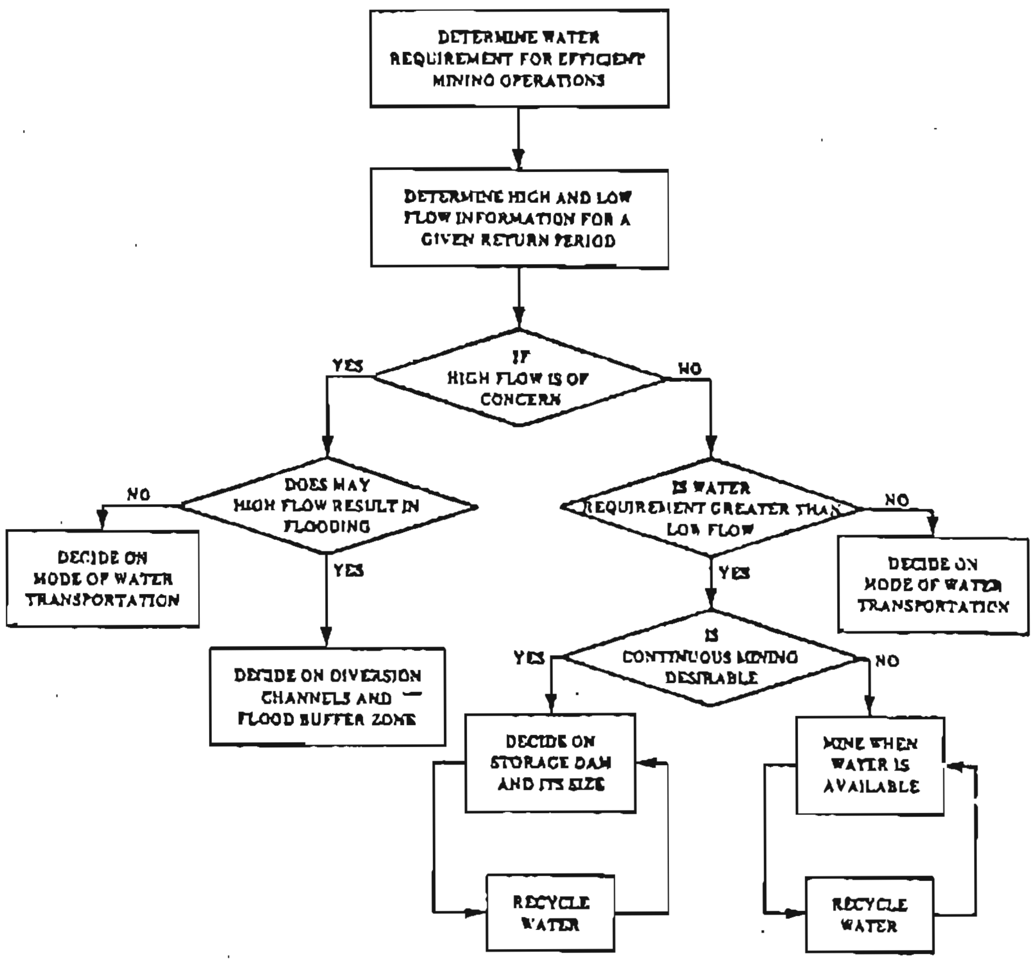


Figure 13. Flow diagram for the estimation of water demand

TABLE 11
SUMMARY OF DESIGN STORM CRITERIA
FOR CHANNEL DESIGN

Channel Situation	Design Storm
A. Runoff/Shallow Groundwater Diversions and Collection	
1. Temporary	2-year 24-hour
2. Permanent	10-year 24-hour
B. Stream Channel Including Banks and Floodplain	
1. Temporary	10-year 24-hour
2. Permanent	100-year 24-hour

The channels should be designed to hold the peak flow for the given event. This estimation of flow can be obtained by the method described in the previous steps.

The critical factors in diversion channel design are:

1. The amount of water to be conveyed.
2. Character of ground
3. Maximum velocity that will not permit erosion.
4. Maximum safe slope of the banks within the water way.
5. Seepage losses.

The amount of water to be conveyed in a channel is determined from the available supply and the amount required for the operation. Maximum safe velocities are a matter of experiment, and experimental determination should be made for all important installations. The maximum velocity should be used where possible to permit the smallest channel and the lowest cost. The following table (Table 12) will act as a general guide:

Table 12: Maximum Mean Velocities for Diversion Channel

Material	Velocity in Ft. per sec.
Very light loose sand	2.0 to 2.5
Average sandy soil	2.0 2.5
Average loam or alluvial soil	2.75 3.0
Stiff clay or ordinary gravel	4.0 5.0
Coarse gravel or cobbles	5.0 6.0
Conglomerate, cemented gravel, soft rock	6.0 8.0
Hard rock	10.0 15.0

In cases where it is more desirable to maintain the maximum head it is necessary to design the channel for less than the maximum velocity. Nevertheless, the velocity should not fall much below 2 f.p.s.. Lower velocities permit silting.

The maximum safe bank slope within the waterway should be used to minimize the amount of excavation. The following table (Table 13) from the Handbook of Applied Hydraulics is recommended for a guide for unlined channels:

Table 13: SUGGESTED BANK SLOPES FOR UNLINED CHANNELS

For cuts in firm rock	1/4:1
For cuts in fissured or partly disintegrated rock, tough hard pan . .	1/2:1
For cuts in cemented gravel, stiff clay soils, ordinary hardpan . . .	3/4:1
For cuts in firm, gravelly, clay soil, or for side-hill cross section in average loam	1:1
For cuts or fills in average or gravelly loam	1 1/2:1
For cuts or fills in loose sandy loam	2:1
For cuts or fills in very sandy soil	3:1

When designing permanent stream diversions in lieu of leaving the stream buffer zone, it may be practical to design and construct the channel in two stages: a main channel to carry the 10-year event, and a floodway that adds sufficient capacity to carry the required 100-year event. In this case, illustrated in Figure 14, the main channel and floodway must be treated separately when determining stable conditions.

A final caution when determining the design storm for permanent stream diversions is the requirement that the constructed channel must have, at a minimum, the capacity of the original channel immediately up and downstream of the diversion. Therefore, these capacities must be checked before the design storms are computed in case they exceed the design conditions. If the two stage option is selected, the capacity of the main channel and then the total channel with floodplain should be checked.

The capacity of a channel may be determined by use of the Manning Equation:

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

Where: Q is the capacity in cfs.

n is the Manning's roughness coefficient.

A is the area of the channel section in square feet.

R is the hydraulic radius defined as area (A) divided by the wetted perimeter in feet.

S is the channel slope in feet/foot.

The Manning's roughness coefficient varies in natural channels usually from 0.030 to 0.060 with 0.030 being relatively smooth channels with little or no growth and 0.060 being rough rocky channels with vegetation.

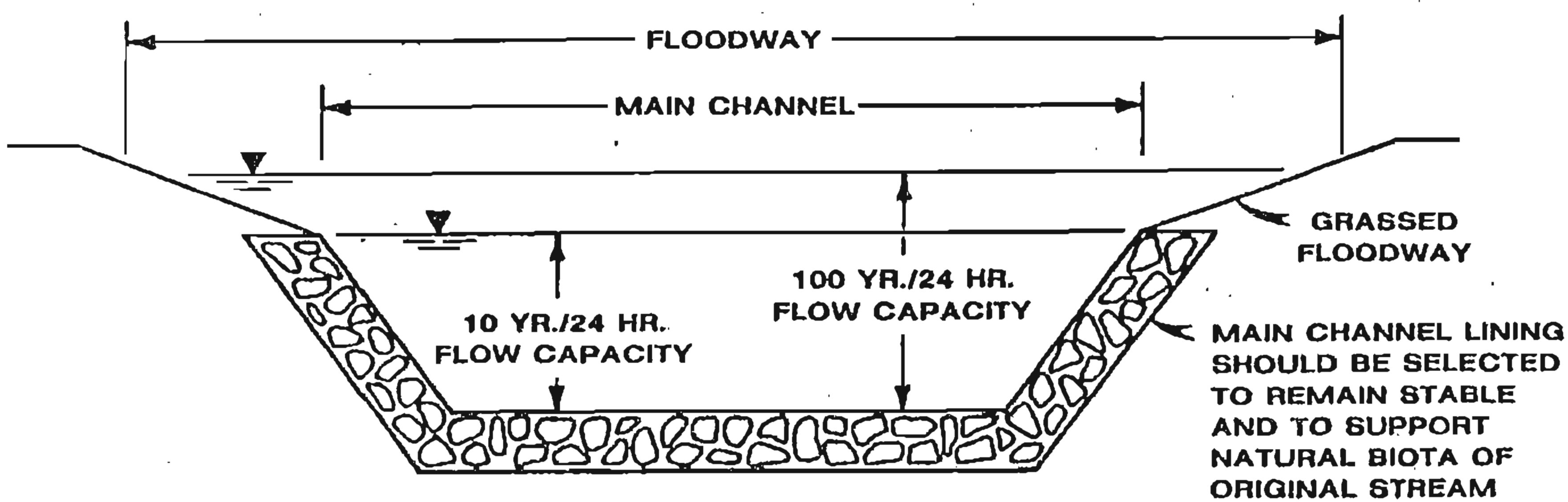


Figure 14. Permanent Stream diversion cross section
(Skelly and Loy, 1979)

Manning's equation is best used for velocities between 1 and 6 feet per second, but is fairly reliable up to 10 feet. For hydraulic raddii greater than 10 feet, velocities greater than 10 ft. per sec., or slopes flatter than 1 in 10,000 should be used with caution. For R or v greater than 20. it is unreliable. Results from this formula must not be expected to be consistently closer than 5%. An uncertainty of x% in selecting a value of n will result in an uncertainty of 2x in computed slope and x in computed velocity.

Lined canals are rarely used in placer mining but when employed it is rarely safe to increase the bank slope on that account.

Seepage losses must be taken into consideration but are rarely predictable with any degree of accuracy. Seepage in new channel is generally higher than old ones. The closer the channel is to the water table, the smaller the seepage loss. Seepage loss in frozen ground is negliable, but the ground may not remain permanently frozen (once the ditch is put into use). Seepage losses can, however, be controlled by various methods. The following table (Table 14) will serve as a guide to possible seepage losses — the first figure given is for old ditches — the second figure for new.

Design procedures for design of stable channels is designated by the following step by step approach (Hilchey, 1947).

Step 1

Determine the location of and drainage area to the channel and compute peak flow rate for design storm as outlined above.

Step 2

Determine the slope of the channel and select a channel shape. For small drainage areas, V-shaped ditches are often used while trapezoidal channels are usually used for larger areas and flows. The side slopes of your channel should never be steeper than 2:1, primarily for maintenance reasons. When

Table 14: CONVEYANCE LOSSES IN CUBIC FEET PER SQUARE FOOT OR WETTED PERIMETER FOR CANALS NOT AFFECTED BY THE RISE OF GROUND WATER

Material	Cubic feet per square ft. in 24 hours
Impervious clay loam	0.25 - 0.35
Med. Clay loam underlain with hardpan not more than 2 or 3 ft below bed	0.35 - 0.50
Ordinary clay loam, silt soil, lava ash loam	0.50 - 0.75
Sandy or gravelly clay loam, cemented gravel, sand and clay	0.75 - 1.00
Sandy loam	1.00 - 1.50
Loose sandy soils	1.50 - 1.75
Gravelly sandy soils	2.00 - 2.50
Porous gravelly soils	2.50 - 3.00
Very gravelly soils	3.00 - 6.00

rock riprap is used as a lining, the angle of repose of the lining should be considered when side slopes are selected. Generally, 2.5:1 minimum will be sufficient unless the rock is rounded and is less than 6-inches in mean diameter; in this case, 3:1 side slopes should be used.

Step 3

Determine the maximum permissible depth of flow to maintain a stable channel.

Step 4

Use the maximum permissible depth of flow, the channel geometry, the channel slope, the Manning's Equation (see Table 15 for "n") to compute the maximum design flow in the channel maintaining stable conditions.

Step 5

If the computed maximum design flow does not equal or exceed the peak design storm, the channel lining and/or channel shape can be altered to increase the available flow. Channel shape can increase design flow by either

Table 15.
MANNING'S ROUGHNESS COEFFICIENT (n)
FOR VARIOUS CHANNEL LININGS

Lining	n
Bare Soil	0.023
Jute Mesh	0.023
Vegetation	
Retardance A	0.160
Retardance B	0.080
Retardance C	0.050
Retardance D	0.040
Retardance E	0.030
Rock Riprap	$0.0395\ D_{50}^{1/6}$*

NOTE: The values of n listed above are good average value for computation. The SCS Engineering Field Manual presents charts that show a relationship between n and R if additional values are desired.

*D_{50} is the mean rock size in feet.

widening the bottom or flattening the side slopes. When a channel is being constructed, the slope of the channel may also be somewhat variable, flattening the slope will increase the maximum design flow. Remember that a minimum freeboard of 0.3 feet must be maintained above the design water surface.

Storage Dams and Storage Estimates

Storage dams must be used where the minimum daily run-off does not equal or exceed the minimum daily water requirements. The size of a storage dam depends on:

- the amount that minimum run-off falls below minimum requirements.
- the amount and distribution of run-off peaks, and
- economic factors.

Estimation procedures for the storage of water is designated by the following step by step approach:

Step 1: After all available data such as run-off low flow, high flow, etc. have been assembled, plot the hydrographs.

Step 2: Decide as to how much water is to be used. If it is necessary to operate a full season regardless of the run-off, the amount of water should be somewhat less than the minimum recorded average. If it is considered better to operate with more water with the probability of being forced to close down early in dry season, water could be used at something approaching the average seasonal run-off.

Step 3: If the alternative to operate a full season regardless of the run-off is choosen, decide an average rate of water use, (say n cfs) which is less than the low run-off flow.

Step 4: Draw the n c.f.s. line on the low run-off flow hydrograph and measure the shaded area below the curve. This area represents the amount of storage which must be developed. Knowing that an area of one square inch on the hydrograph is equivalent to 397 acre-feet, the actual quantity is readily computed. Go to Step 7.

Step 5: If the alternative to operate with more water is choosen, all surplus from the spring run-off must be stored against the following shortage to assure maximum operating time in the dry season. Decide an average rate of use (say Y c.f.s) which is more than the low run-off data.

Step 6: Draw the Y c.f.s line on the hydrograph showing the greatest spring run-off. The required storage in this case is the excess volume of run-off during the spring high flow. Computations are the same as previously.

Step 7: The required storage for a given dry period should be checked against the excess run-off in the period immediately before, to make sure that there is no accumulated storage. If there is an accumulated storage, the storage area should be increased accordingly.

FLOOD-FLOW BUFFER DESIGN

Flood-flow buffers should be designed to prevent the diversion of an active channel through the material site. The design life is usually some finite period ranging from 5 years to possibly 50 years or more for some sites.

The recommended design procedure is to consider the lateral activity of the particular stream based on its channel configuration and historical migration pattern. The stream size, soil composition of the buffer material, vegetative cover, permafrost banks, and channel aufeis are also important considerations affecting the stability of the buffer. The hydrology of the stream must be considered to evaluate the frequency that the buffer will be flooded. Each of these are discussed in more detail elsewhere (Woodward Clyde Consultants, 1980). However, some pertinent information are provided here for ready reference.

Buffer height and buffer width are interrelated to a certain degree. If the buffer is high enough to keep all but the largest of floods out of the material site, only bank erosion needs to be considered in buffer design. This may be the situation for many material sites located on terraces. If the buffer is low and is flooded frequently by larger flows, erosion of the surface of the buffer, headward erosion of the upstream face of the material site, and scour within the site must be considered in the buffer design. The height of natural buffers is fixed at the level provided by nature. Design options include increasing buffer width to account for low height, building up the buffer height by adding a dike on the stream side, or building a completely separate buffer structure. These options are discussed in more detail in a subsequent paragraph.

To evaluate the frequency of flooding, hydrologic and hydraulic analyses must be carried out. The details of these analyses are too complex to explain here, some methodologies have been discussed already in Chapter IV. However, appropriate references are given to allow the user to study the subject further.

- A hydraulic analysis is required to evaluate what discharge will initiate overtopping of the buffer. Cross sections of the stream, extending up to the level of the buffer on both banks, are necessary for this analysis. It is preferable to have five or more cross sections through the reach of river adjacent to the buffer. The Manning equation or, perferably, a backwater program, should be used to calculate the discharge corresponding to the stage that would overtop the buffer. Discussions of these analyses are provided in most open-channel hydraulics textbooks (Chow, 1959), and in other references (Bovee and Milhous, 1978; U.S. Army Corps of Engineers, 1976).
- A flood frequency analysis provides an estimate of the recurrence interval or probability of exceedance of the discharge which just overtops the buffer. Detailed discussion of flood frequency analyses are included in most hydrology textbooks, U.S. Water Resources Council (1977), and Lamke (1979). Lamke (1979) provides equations for determining flood discharges for rivers in Alaska for the following recurrence intervals (Table 16) and corresponding exceedance probabilities:

Table 16:

Recurrence interval (years)	Exceedance probability (%)
1.25	80
2	50
5	20
10	10
25	4
50	2
100	1

With the discharge and its frequency of occurrence known, the probability of that flood occurring over the design life of the buffer is needed. Table 17 below provides the probability of occurrence of a flood of a specified recurrence interval during a specified buffer design life.

Table 17. Probability of Occurrence[a] (%) of a Specified Flood During a Specified Design Life

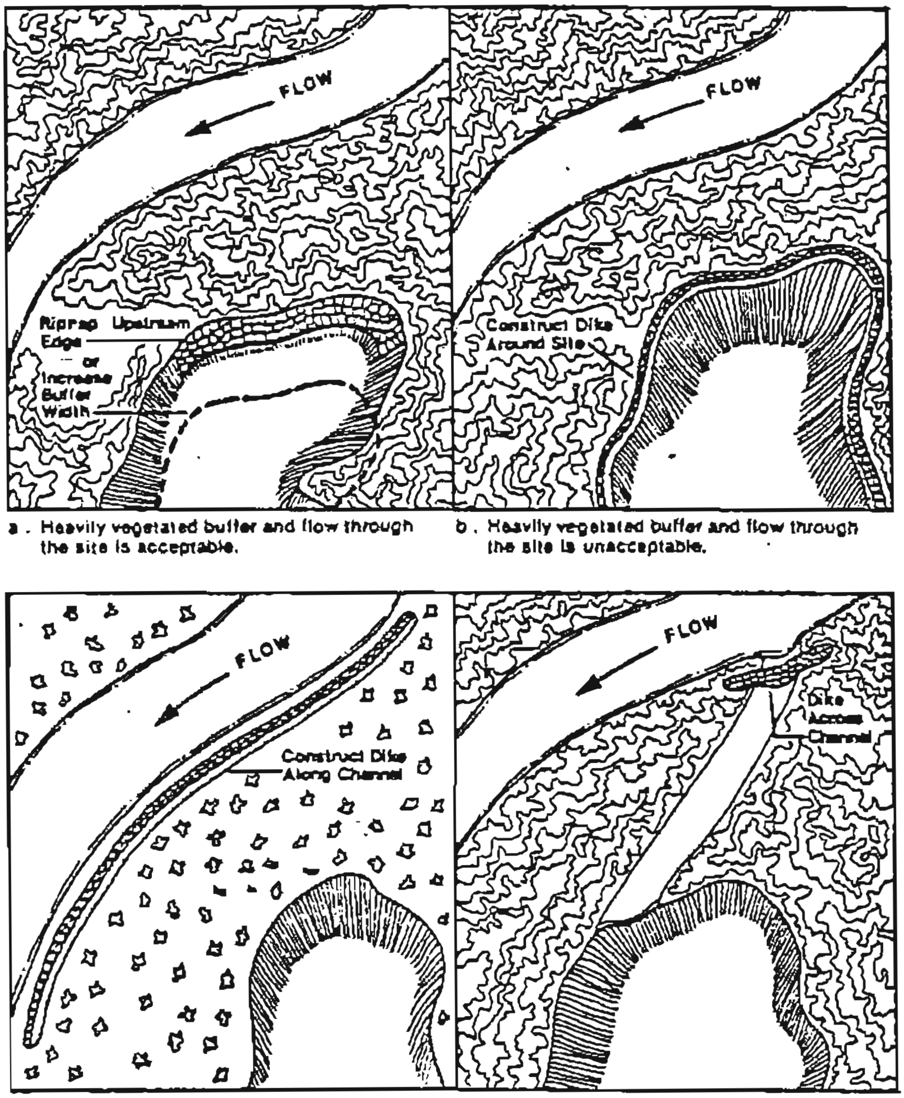
Flood		Buffer design life (years)							
Recurrence interval (years)	Exceedance probability (%)	2	5	8	10	20	25	50	100
1.25	80	96	99+	99+	99+	99+	99+	99+	99+
2	50	75	97	99+	99+	99+	99+	99+	99+
5	20	36	67	83	89	99	99+	99+	99+
10	10	19	41	57	65	88	93	99	99+
25	4	8	18	28	34	56	64	87	98
50	2	4	10	15	18	33	40	64	87
100	1	2	5	8	10	18	22	39	63

[a]Probability of Occurrence = 1 − (1 − Exceedance Probability)$^{\text{Design Life}}$

With the known probability of flow through the site during the design life of the buffer, the user can evaluate the consequences. If the probability is low, the width of the buffer can be designed based on lateral migration alone. If the probability is high, one of several design options are recommended.

- o If the buffer is heavily vegetated, and if flow through the material site is acceptable, riprap the upstream edge of the material site to prevent headward erosion; or, increase the width of the buffer to allow for erosion loss (Figure 15(a)).
- o If the buffer is heavily vegetated, and flow through the site is unacceptable, construct a dike surrounding the material site designed for a flood with an acceptabily low probability of occurrence (Figure 15(b)).
- o If the buffer is lightly vegetated, build a dike along the river side of the buffer designed for a flood with an acceptably low probability of occurrence (Figure 15(c)).
- o If the buffer contains a high-water or abandoned channel, build a dike along the river side of the buffer to keep flow out of the channel; the dike should be designed for a flood with an acceptably low probability of occurrence (Figure 15(d)).

As an example of buffer height design, consider the material site location shown in Figure 16. The buffer width has been estimated by historical erosion techniques. Cross sections are surveyed as shown (two additional cross sections were collected further downstream). A backwater analysis was run to find that discharges of 103 m^3/s and 89 m^3/s overflowed the buffer at Cross Sections 3 and 7, respectively. A flood frequency analysis indicated that these discharges had recurrence intervals of 35 and 25 years. The design life of the buffer is 25 years. Thus, from Table 17, at Cross Section 7 there is a 64 percent chance of getting flow into the downstream end of the material site within the 25-year life. This chance is acceptable to the user because the flow would primarily be backwater and would have relatively low erosion potential. At Cross Section 3 the upstream buffer has a 50 to 60 percent chance of overtopping the buffer. The user finds this to be unacceptable, but since there is a relatively small chance of substantial flow entering the pit from the upstream side, he recommends riprapping the upstream bank of the pit.



a. Heavily vegetated buffer and flow through the site is acceptable.

b. Heavily vegetated buffer and flow through the site is unacceptable.

c. Lightly vegetated buffer and flow through site is acceptable.

d. Highwater or abandoned channel through heavily vegetated buffer and flow through site is acceptable.

Figure 15. Schematic of recommended options if the probability of flow through the site is high (Woodward Clyde Consultant, 1980)

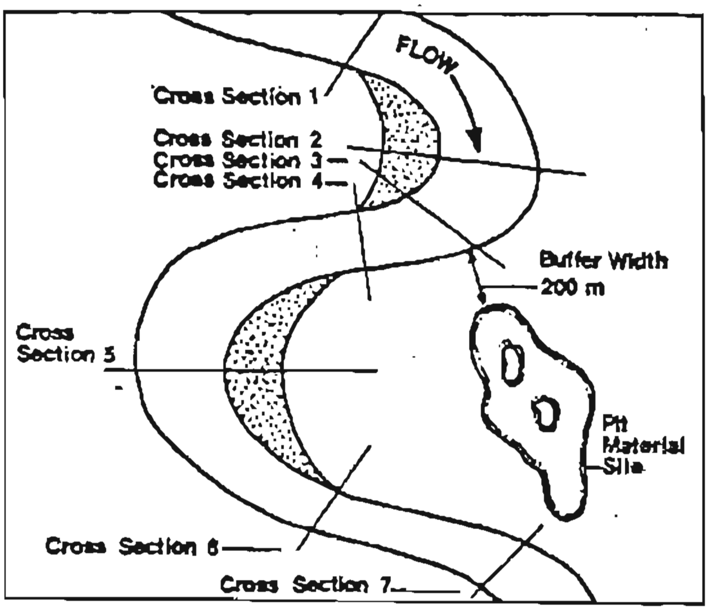


Figure 16. Schematic of an example of buffer height design
(Woodward Clyde Consultant, 1980)

APPENDIX

ANNUAL PLACER MINING APPLICATION
LAND USE AND WATER USE PERMITS AND MINING LICENSE

APPLICANT AND SITE INFORMATION

DO NOT MARK IN THIS SPACE

Do you plan to do the following work on the claim(s)?

☐ Explore ☐ Mine ☐ Transport Equipment

Are the mining claims:

☐ Federal ☐ State

Check box(es) and list number(s) if you have had any of the following permits for these claims:

☐ DNR Land Use Permit No.:

☐ Fish & Game - Habitat Protection Permit No.:

☐ DNR - Water Use Permit ADL No.:

☐ DEC - Wastewater Discharge Permit No.:

☐ Revenue - Alaska Mining License No.:

☐ EPA-NPDES Wastewater Discharge Permit No.:

Check the mining district in which the claims are located:

☐ Fairbanks
☐ Seward Peninsula
☐ Circle
☐ Kuskokwim
☐ Iditarod
☐ Innoko
☐ Forty Mile
☐ Hot Springs
☐ Koyukuk-Chandalar
☐ Other (Specify):

Check the box to indicate who controls land on which the claim(s) are located or which is crossed for access:

☐ U.S. National Park Service
☐ U.S. Military
☐ U.S. Forest Service
☐ U.S. Bureau of Land Management
☐ U.S. Fish & Wildlife Service
☐ State of Alaska
☐ City (Specify):
☐ Borough (Specify):
☐ Native Corporation (Specify):
☐ Other (Specify):

Claim Owner's Full Legal Name		Street Address or P.O. Box		
City	State	Zip Code	Home Telephone	Office Telephone
Name of Lease Holder (if rights to claim are leased)		Street Address or P.O. Box		
City	State	Zip Code	Home Telephone	Office Telephone
Name of Operating Company or Authorized Representative in Field		Street Address or P.O. Box (Specify if Summer Address is different from Winter)		
City	State	Zip Code	Home Telephone	Office Telephone
Employer I.D./Social Security Number	Month Start Up This Year	Month Shut Down This Year	Number of People Working	
On what creek(s) are your claim(s) located?		Claim(s) Location: Section(s), Township(s), Range(s), Meridian		

DESCRIPTION OF OPERATION

List type, size, purpose and number of pieces of equipment to be used on the claim.

Which equipment listed above is used for the removal of overburden?

Which equipment listed above will be used in the stream?

Beginning and ending dates for transportation of equipment across country TO a claim:

Beginning and ending dates for transportation of equipment across country FROM a claim:

If using a hydraulic giant, list nozzle size, number of nozzles, feet of head and total amount of water CFS or GPD:

List type and amount of explosives to be used:

If explosives will be used in or near streams or other bodies of water, indicate when, where and why they will be used.

Type of overburden: (O.K. to check more than one)

☐ Rock ☐ Sand ☐ Silt ☐ Clay ☐ Organic Material
☐ Other (Specify):

Depth of Overburden

Amount of Overburden to be removed:

Number of acres:

Average depth in feet:

PLACER MINING METHOD

Check method of mining and processing: (O.K. to check more than one.)

☐ Suction Dredge Intake Size ______ (in.) Pump Capacity ______ (GPM)

☐ Sluice Pump Intake Size ______ (in.) Pump Capacity ______ (GPM)

☐ Bucket Line Dredge Size of Buckets ______ (cu. ft.)

☐ Washing Plant Type ______ Vol. Material Processed/hr. ______ (cu. yd.)

☐ Sluice Box Length ____ (ft.) Width ____ (in.)

Depth of Water in Box ____ (in.) Slope ____ (in./ft.)

☐ Chemical Treatment ☐ Mercury ☐ Cyanide ☐ Other

Describe process on a separate sheet.

Daily Volume of Material Processed: ______ (cu. yds.)

WATER SUPPLY

Type of Dam (Exclude settling ponds):

☐ Earthfill ☐ Timbercrib ☐ Concrete ☐ Other: ______

☐ Temporary ☐ On-stream

☐ Permanent ☐ Off-stream

Size of Dam

Length Width at Crest Width at Base Height

Storage Capacity: (Indicate Length and Width of Area and Depth of Water)

Spillway Capacity: (CFS)

FUEL

Fuel Stored: (List type of fuel)

Amount of fuel stored:	Distance from stored fuel to nearest body of water:
Method of transporting fuel:	Amount of fuel transported per trip:

RECLAMATION

Indicate method of reclaiming area of operation.
(O.K. to check more than one)

☐ Level tailings piles ☐ Reestablish stream channels

☐ Regrade contours ☐ Remove toxic materials

☐ Respread topsoil ☐ Other: (Specify)

☐ Revegetate or reseed

WATER USAGE

What % of a natural stream is diverted for any reason: %	What % of diverted water is used for mining: %	Of water used for mining what % is recycled: %

Amt. of water required: Qty: ☐ GPM ☐ GPD ☐ CFS	Usage: Hours/Day	Usage: Days/Week

Date commenced operations: Month Year	Years needed to mine out claim:
If water is not used for mining, is it routed around the treatment pond? ☐ YES ☐ NO	When wastewater is returned to a stream, is it treated? ☐ YES ☐ NO

Condition of stream above claim, prior to discharge of wastewater:

☐ Clear ☐ Murky ☐ Muddy

If "muddy", is it: ☐ Natural ☐ Manmade

☐ Other: (Describe)

Method of taking water:

☐ Diversion Ditch ☐ Headgate ☐ Pump: Intake size ______ (in.)

WASTE WATER TREATMENT

Capacity of Settling Pond(s): Indicate length and width of area and depth of water for each pond.

1. 2.

3. 4.

ACCESS

If access roads will be built to the claim, indicate:

Length ______ (ft.) Depth of Material ______ (ft.)

Width ______ (ft.) Kind of Material Used ______

DRILLING

Estimated maximum depth: (ft.)	Number of holes drilled:
Diameter of holes drilled: (in.)	Type of drill used:

Signature of Applicant	Relationship to Claim(s): ☐ Owner ☐ Leasee ☐ Operator	Date

CLAIM LISTING

Please list all claim names within this claim group. Each claim will be specifically named on your water permit or certificate. You may use the number of each claim on your sketch and USGS map for convenience.

No.	CLAIM NAME	ADL/BLM No. if known	No.	CLAIM NAME	ADL/BLM No. if known
1.			9.		
2.			10.		
3.			11.		
4.			12.		
5.			13.		
			14.		
			15.		
8.			16.		

Form 00-001 (Rev. 12/81)

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