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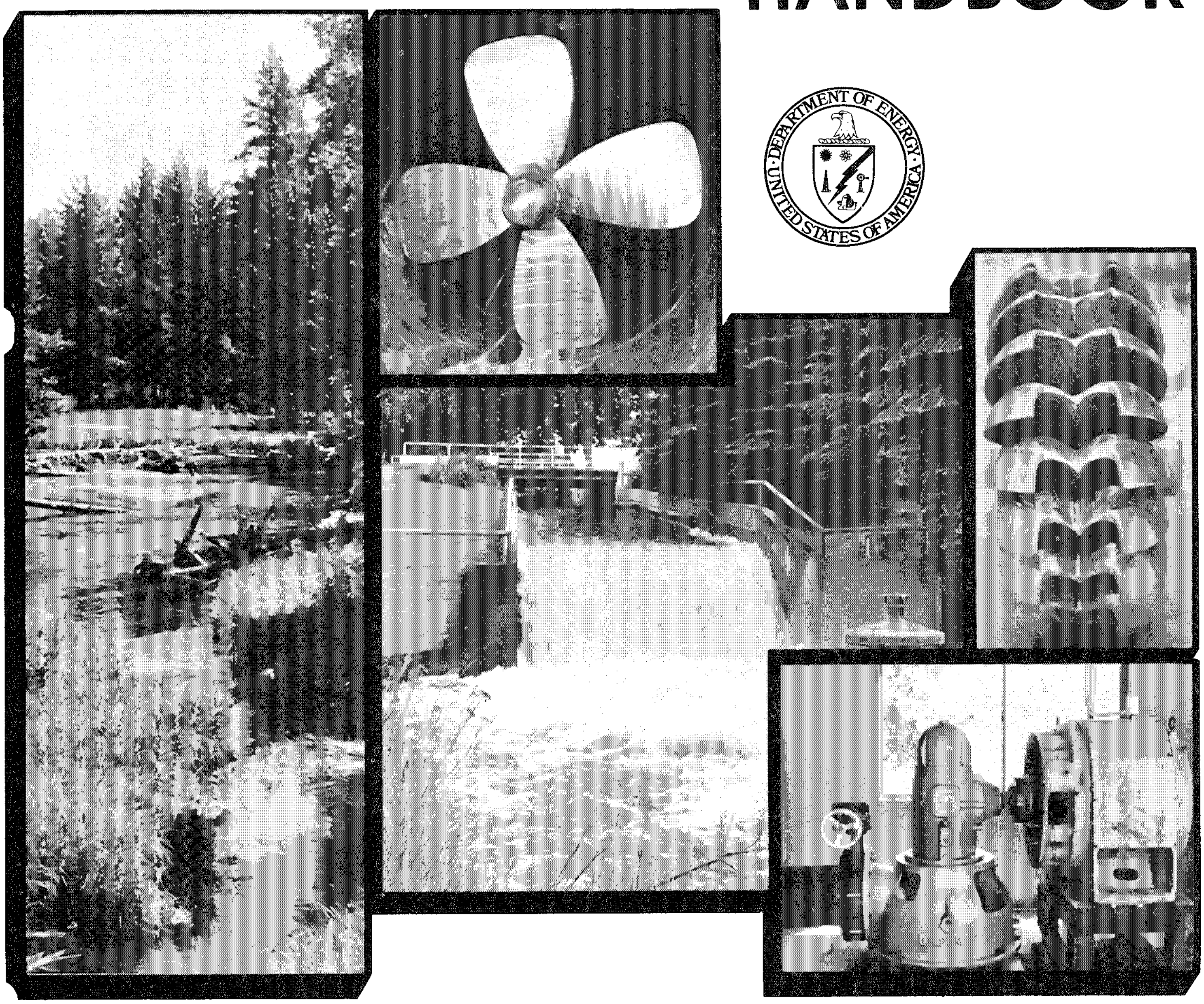
# MICROHYDROPOWER HANDBOOK



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# MICROHYDROPOWER HANDBOOK
Volume 1

J. D. McKinney, EG&G Idaho
Prof. C. C. Warnick, University of Idaho
B. Bradley, Bradley Engineering
J. Dodds, EG&G Idaho
T. B. McLaughlin, EG&G Idaho
C. L. Miller, EG&G Idaho
G. L. Sommers, EG&G Idaho
B. N. Rinehart, EG&G Idaho

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EG&G Idaho, Inc.
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# 1. INTRODUCTION

For centuries, energy from falling water has been converted by man to perform useful work. At the turn of the century, this country was dotted with thousands of picturesque water wheels being turned by the weight or velocity of falling water. The turning water wheels converted the energy of the falling water into mechanical energy, or shaft horsepower. Usually, the water wheel turned a shaft that was connected to some work process such as a gristmill. Today's modern turbines, although they look much different than the old water wheels, represent refinements of similar technology--a more efficient way of converting the energy of falling water to mechanical energy, resulting in faster shaft rotation (see Figure 1-1). If the shaft from a turbine is connected to an electric generator, the two pieces of equipment become known as a hydroturbine-generator unit or a hydroelectric-generator. In general, it can be said that the modern turbine rotates much faster than a water wheel. The faster speed is an advantage in hydroturbine-generator units.

The size of hydroturbine-generator units can vary from a very small turbine connected to a car alternator to a large unit like those in Grand Coulee Dam on the Columbia River. Microhydropower plants are the smallest of the turbine-generator units, producing 100 kilowatts (kW) or less of power [134.1 horsepower (hp) or less]. This handbook only considers units in this size range that convert the mechanical shaft energy of a turbine into electric energy from a generator.

There are many types and makes of turbines. This handbook discusses those turbines that would most likely be connected to small generators producing less than 100 kW of electric power. Water wheels are not addressed in this handbook.

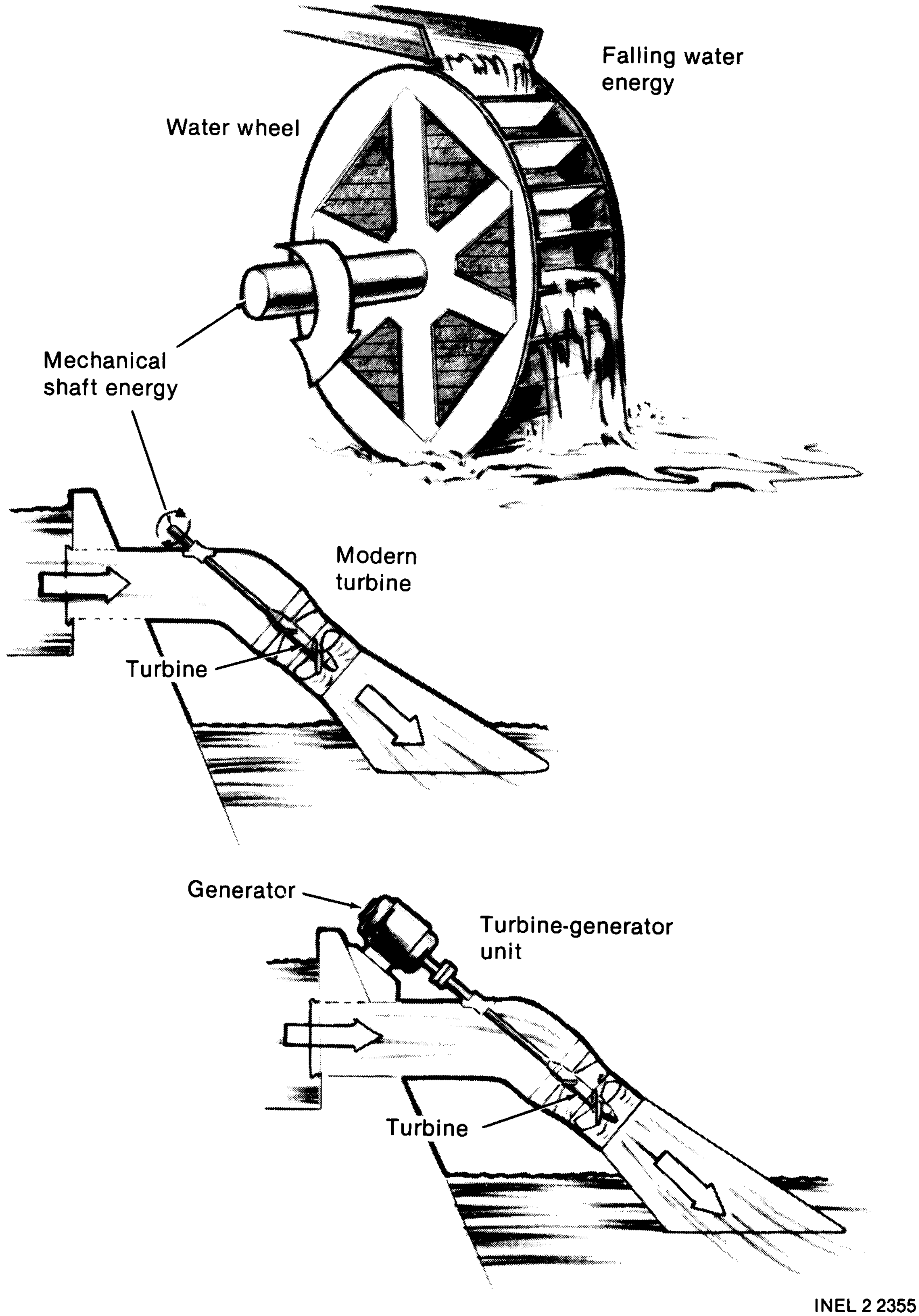



Figure 1-1. Comparison of water wheel and water-driven turbine.

## 1.1 Purpose

The purpose of this handbook is to provide a mechanically proficient lay person with sufficient information to evaluate microhydropower site potential, lay out a site, select and install equipment, and finally, operate and maintain the completed system. The actual construction details of the site are not included; rather, pointers are given as to what help he should expect from a construction contractor, and general guidelines on construction details are provided. In addition, information about obtaining financing and permits is provided. To help offset the cost, the person performing the work, referred to as the "developer," is encouraged to do as much of the work as possible. However, developers with major areas of uncertainty should consider professional assistance.

The handbook has been written with the aim of keeping the format simple and straightforward. The reader is encouraged not be intimidated by what may be unfamiliar or appear too technical. The handbook assumes that the reader has little working knowledge of hydropower or the engineering concepts behind the use of hydropower. The reader is encouraged to take the time to read and understand each section of the handbook, especially the mathematics, tables, charts, and graphs. A thorough understanding of the information presented in the handbook will greatly enhance the chances for a successful development--one that produces the energy expected and saves the developer money and time in the long run. Keep reading and studying the contents of this handbook; don't give up!!! The mathematical procedures presented in this handbook are limited to multiplication, division, and square roots. More sophisticated procedures may yield greater accuracy, but for the purposes intended, the procedures presented should be sufficient.

Figures 1-2 and 1-3 show simple tools and aids you can use in making and recording calculations, making sketches, and doing similar work that will be needed during the site development process. A pocket calculator can be of great benefit in performing many of the calculations. You will also need a ruler or scale, a triangle, and graph paper or quadrille paper. Graph paper with 10 divisions to the inch and a scale with the same

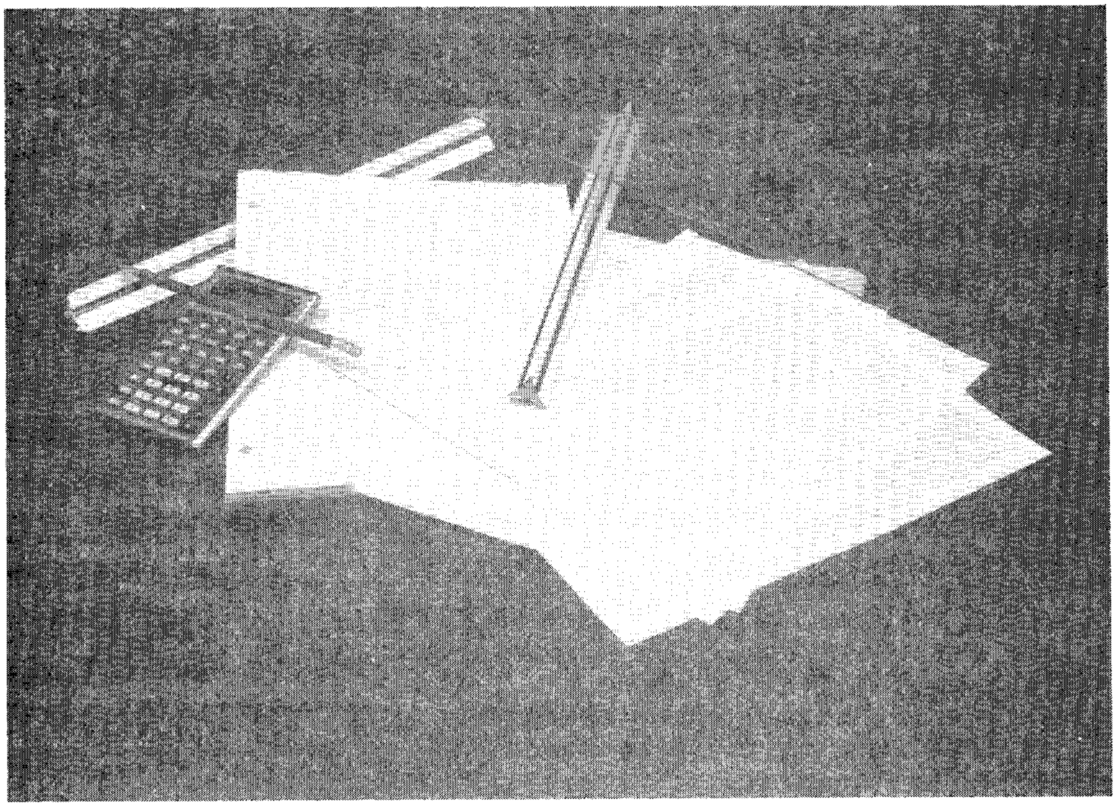


Figure 1-2. Simple tools and aids.

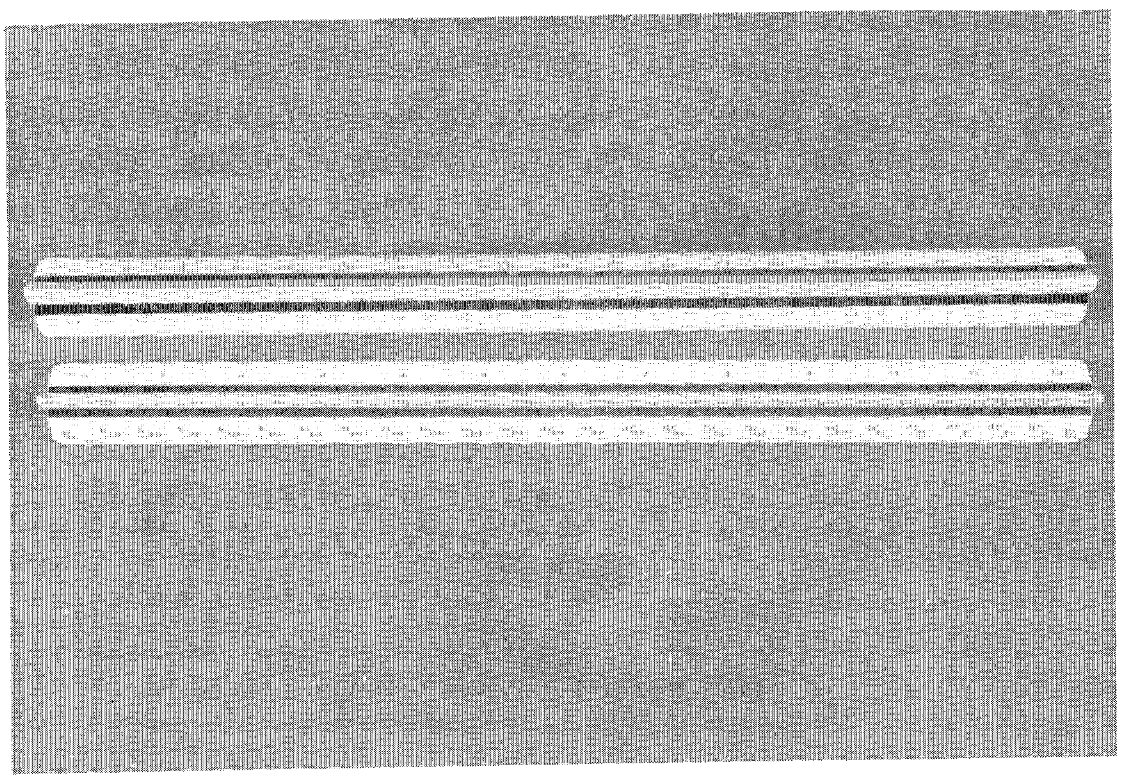


Figure 1-3. Types of scale.

divisions can be very helpful. All of these items are easily obtainable at a stationery or office supply store.

## 1.2 Cost of Development

At the time of this writing, 1982, the typical development cost may range from $1,000 to $4,000 per installed kW. A typical single-family home will require a peak power demand of 5 kW. (If electricity is used for heating, the demand may reach 12 to 20 kW.) This means that a developer who wants to install a unit for personal use may have to invest from $5,000 to $20,000.

The installation cost can vary greatly, depending on the work required to prepare the site and on the physical dimensions of the turbine-generator unit to be used. The handbook demonstrates how to estimate installation cost and how to balance design tradeoffs, cost, and projected energy production.

## 1.3 Category of Developer

The majority of people interested in microhydropower are motivated by a desire to be energy independent. The remainder of those interested desire to produce as much energy as possible as a source of revenue and will generally install larger units than those in the first category. This handbook addresses both categories of interest.

You are encouraged to determine which of the two categories applies to you. It will make a difference in how you design your microhydropower system. If you are unsure which category you should be in, you should evaluate the site as a Category 2 developer and make a final decision later as to which category you build to.

Category 1. The primary motivation is to supply electricity where a utility source is not available, or to develop a separate source of electrical energy and thus become energy independent. The developer in this category is more interested in generating only what energy is needed and in having that energy available for as much of the year as possible. The developer is not interested in recovering the maximum energy available from the stream. As a result, the system will be designed for the minimum stream flow of the year. The Category 1 developer will generally have a smaller investment than the Category 2 developer.

Category 2. The primary motivation is to produce the maximum energy available from the stream for the dollar invested. The developer may or may not plan to use the energy generated. The operating capacity will generally be greater than 50 kW. The design flow will usually be 70 to 75% of the maximum annual stream flow.

## 1.4 Organization of the Handbook

The handbook is intended to aid the individual who has a small site that does not justify the expense of professional engineering services. The handbook is divided into eight sections, including the Introduction. You are encouraged to peruse the handbook and obtain a general knowledge of its content before starting actual development. Sections 2 through 8 represent the major steps that any site development must go through. With the exception of Sections 7 and 8, which cover financial and institutional requirements, the sections are presented in the order that must be followed in development. The actual development steps given in Sections 2 through 6 should be completed in the sequence presented to the maximum extent possible. Sections 7 and 8 should be read and followed from the beginning of development through actual operation. Subsection 1.5 below describes the actual steps in more detail.

Several appendices are provided. They contain technical data and conversion tables, descriptions of the development of two example sites, a discussion of applicable federal laws, lists of federal and state agencies, a list of equipment manufacturers, and a glossary of terms.

## 1.5 Event Sequence

To help in using this handbook, an event sequence--a logical sequence of major steps to be followed in developing a microhydropower plant--is presented below, with a brief narrative describing each event or step. Each event is also cross-referenced to the appropriate section in the handbook. A developer who follows this sequence should encounter minimal delays in the development process and should gain the most from this handbook.

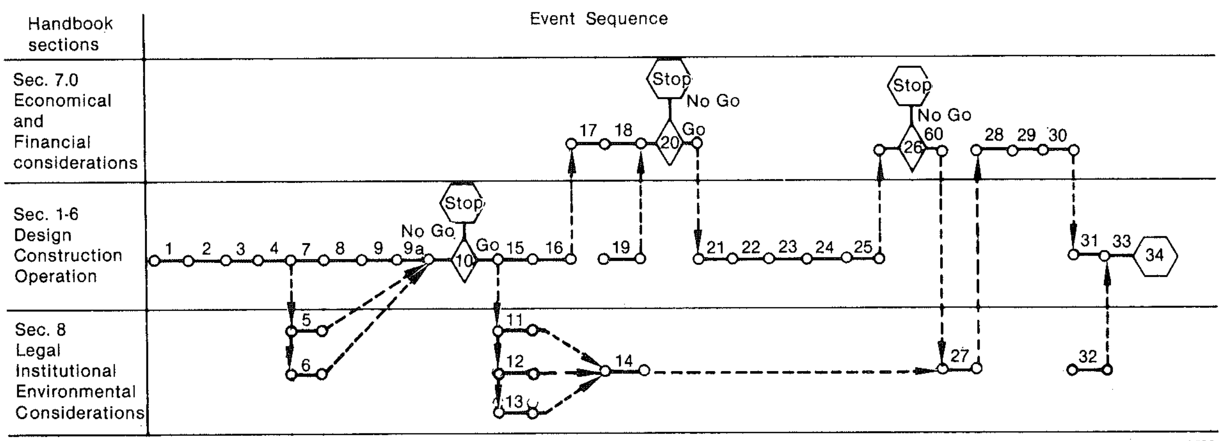
In reading through the event descriptions for the first time, you may encounter terms that are confusing and statements that, without more background, have little meaning to you. Don't worry about the details of the events at this time. The sequence is set up for continual reference as you work through the handbook. As you proceed, the terms and statements should become clear. Keep this subsection marked for quick reference, and as you finish each step refer back to be sure you pursue the next most important step.

Not all events listed below must be done one at a time. There are some steps that can be performed simultaneously. Figure 1-4 is a graphical representation of the events. The figure will be explained in more detail after the narrative description.

### 1.5.1 List of Events

1. Lightly Review the Handbook. Skim through the handbook. Become familiar with its organization and with the major subjects covered in each section. As you are reviewing, study the figures and diagrams in particular. Refer to the glossary (Appendix G) for meanings to unfamiliar terms. Don't worry if you don't understand some of the sections. They should become clear as you work with them.

2. Read Sections 1 and 2 and Subsection 8.1. The most important items in Section 1 are the event sequence and the determination




All categories

1. Lightly review the Handbook
2. Read Sections 1 and 2 and Subsection 8.1
3. Determine power requirements (Subsection 3.1)
4. Make site inspection (Subsection 3.2)
5. Make initial contact with state and local agencies (Subsection 8.2)
6. Make initial contact for Federal land-use Permit (Subsection 8.3.3 and Appendix C)
7. Determine available flow (Subsection 3.3)

Category 1 with an existing dam, canal drop, or industrial waste discharge site

8. Measure head and distance (Subsection 3.4)
9. Determine design capacity (Subsection 3.5)

Category 1 with a run-of-the-stream site

8. Determine design head (Subsection 3.5)
9. Measure head and distance (Subsection 3.4)

Category 2 with an existing dam, canal drop or industrial waste discharge site

8. Measure head and distance (Subsection 3.4)
9. Determine plant capacity (Subsection 3.6)

9.a Determine annual energy production (Subsection 3.7)

Category 2 with a run-of-the stream site

8. Determine plant capacity (Subsection 3.6)
9. Measure head and distance (Subsection 3.4)

9.a Determine annual energy production (Subsection 3.7)

All catagories

10. Go-No Go
11. Determine federal requirements (Subsection 8.3)
12. Obtain state and local permits (Subsection 8.2)
13. Obtain federal-land use Permit (Subsection 8.3.3)
14. File for FERC license Subsection 8.3)
15. Read section on turbines (Subsection 4.1)
16. Contact manufactures and suppliers (Subsection 4.2)
17. Determine market potential (Subsection 8.4 and Section 7.0)
18. Determine financing options (Subsection 7.0)
19. Make preliminary cost estimate (Subsection 4.3.1)
20. Go-No Go (Subsection 4.3.1)
21. Select the design criteria (Subsection 4.3.2)
22. Design the system (Subsections 4.4 through 4.8)
23. Assemble the design package (Subsection 5.1)
24. Negotiate an equipment package (Subsection 5.3)
25. Make a project cost estimate (Subsection 5.2)
26. Go-No Go (Subsection 5.2)
27. Obtain FERC license (Subsection 8.3)
28. Finalize the marketing contract (Subsection 8.4)
29. Develop financial package (Subsection 5.0)
30. Obtain financing (Subsection 7.0)
31. Finalize design (Subsection 5.4)
32. Obtain local building permit (Subsection 8.2)
33. Construct the system (Subsection 5.0)
34. Operate the system (Subsection 6.0)

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Figure 1-4. Event sequence.

of your development category. Section 2 begins the actual work of the handbook by defining such terms as head, flow, and kilowatt. The power equation is presented in Subsection 2.5. This equation is the basis for all hydropower development. Subsection 2.6 describes the types of microhydropower source. The type of source you have (Run-of-the-Stream, Existing Dam, Canal Drop, or Industrial Waste Discharges) establishes how you design your system. Subsection 2.7 identifies the characteristics of the two example sites used in the handbook. Subsection 8.1 describes some of the environmental considerations you may have to address.

3. Determine Your Power Requirements (Subsection 3.1). In Subsection 3.1 you will determine how much power you should have to meet your needs. This number is important as a base for comparison with the power-generating potential of the your source. Category 1 developers will use the required power as their first design point and base the rest of their system design on it.

4. Make Site Inspection (Subsection 3.2). Subsection 3.2 leads you through a detailed inspection of your site. You are given a list of things to consider in your inspection. After the inspection, you should establish a preliminary layout for your site.

5. Make Initial Contact with State and Local Agencies (Subsection 8.2). At this time, it is important to establish the initial contact with the state agencies. Subsection 8.2, State and Local Requirements for Development, tells how to proceed and identifies what should be discussed in these contacts.

6. Make Initial Contact for Federal Land-Use Permits (Subsection 8.3.3 and Appendix C). If any part of your site will involve the use of federal lands, you should contact the appropriate agency at this time to determine the land-use requirements. Subsection 8.3.3 and Appendix C tell what steps to take.

7. Determine Available Flow (Subsection 3.3). Subsection 3.3 presents various methods for measuring stream flow. If you are a Category 2 developer, you will be given additional instructions on how to develop a flow duration curve. The curve is important in selecting the right equipment and determining how much energy can be generated in a year.

Note: How you proceed through the next two events depends on which category of developer you are and what type source you are developing.

Category 1 with an Existing Dam, Canal Drop, or Industrial Waste Discharge Site

8. Measure Head and Distance (Subsection 3.4). Subsection 3.4 describes various survey methods useful in measuring head and discusses the measurement of intake to powerhouse distances.

9. Determine Design Capacity (Subsection 3.5). Subsection 3.5 has several parts to choose from. You should follow Subsection 3.5.2 to calculate the design power capacity of the site. This number can then be compared with your power requirements, which were determined in Step 3. If the calculated power is less than the requirements, the site will not produce all the power you need. If the calculated power is more than your requirements, then you may be able to sell the excess power to a utility, or you can reduce the design flow to meet only your needs.

Category 1 with a Run-of-the-Stream Site

8. Determine Design Head (Subsection 3.5). Run-of-the-Stream developers have the option of determining how much head they wish to develop. Section 3.5.1 shows you how to determine the amount of head required to meet your needs.

9. Measure Head and Distance (Subsection 3.4). Subsection 3.4 describes various survey methods.

Category 2 with an Existing Dam, Canal Drop, or Industrial Waste Discharge Site

8. Measure Head and Distance (Subsection 3.4). Subsection 3.4 describes various survey methods useful in measuring head and discusses the measurement of intake to powerhouse distances..

9. Determine Plant Capacity (Subsection 3.6). Subsection 3.6 explains plant capacity and gives a rule-of-thumb method for analyzing plant capacity.

9a. Determine Annual Energy Production (Subsection 3.7). Subsection 3.7 discusses annual energy production and explains plant factor.

Category 2 with a Run-of-the-Stream Site

8. Determine Plant Capacity (Subsection 3.6). Run-of-the-stream developers will evaluate plant capacity for a range of heads.

9. Measure Head and Distance (Subsection 3.4). A survey should be conducted to determine if the head ranges are reasonable.

9a. Determine Annual Energy Production (Subsection 3.7). Subsection 3.7 discusses annual energy production and explains plant factor.

Note: All categories of developers proceed from here.

10. Go/No-Go. This is a logical place to address the first Go/No-Go decision, which is a decision whether or not to proceed with the development. This event does not refer to any particular section in the handbook. Instead, it is a simple comparison of the power required (determined in Step 3) and the power that can be produced (calculated in Step 8 or 9). If the power required is less than or equal to the calculated power, your decision can be "go" and

you should proceed to the next event. If the power required is considerably more than the calculated power, then you might consider a "no-go" decision and find some other means of generating power.

11. Determine Federal Requirements (Subsection 8.3). Read Subsection 8.3 and determine which exemption category you think your site fits into. It is a good idea to write or call the FERC to verify the procedure you think appropriate.

12. Obtain State and Local Permits (Subsection 8.2). In accordance with your initial contacts and Section 8.2, obtain all the state and local permits needed to accompany your FERC license request. (The term "License" in this handbook also implies exemption).

13. Obtain Federal Land-Use Permits (Subsection 8.3.3). If you are going to use federal lands in any way, you must have a permit from the agency with jurisdiction over the land before the FERC will grant a license. In Step 6 you determined what requirements would be imposed on you. To obtain a permit, you must be prepared to show how you will comply with the requirements.

14. File for FERC License (Subsection 8.3). In accordance with Subsection 8.3, file with the FERC for the appropriate exemption.

15. Read Section on Turbines (Subsection 4.1). Subsection 4.1 describes the various types of turbine available to microhydropower developers. Read the section and determine which type or types best fit your site.

16. Contact Manufacturers and Suppliers (Subsection 4.2). Subsection 4.2 presents a form to fill out and send to the turbine manufacturers and suppliers. The manufacturer or supplier completes the form or supplies the requested information in some other way, and then returns the information to you. With this information in hand, you will now be able to determine the preliminary site economics and establish the design criteria.

17. Determine Market Potential (Subsection 8.4). If you are a Category 2 developer or a Category 1 developer who might sell excess power, read Section 8.4 "Marketing" and then contact local utilities to determine their interests.

18. Determine Financing Options (Section 7.0). Section 7.0 discusses several financing options. Review the section to determine which options might be available to you.

19. Make Preliminary Cost Estimate (Subsection 4.3.1). After the forms are returned from the manufacturers and suppliers, you can make a preliminary cost estimate for the project. This first rough-cut estimate of the project cost should be considered preliminary, but it should indicate the financial magnitude of the project.

20. Go/No-Go (Subsection 4.3.1). The first Go/No-Go decision (Step 10) was based on the power potential of the site. This decision is based on the economic potential of the site. If you consider the development worth the investment, proceed. If not, drop it.

21. Select the Design Criteria (Subsection 4.3.2). If your decision is "go," it is time to select the best turbine-generator and establish the design criteria that will be used for the design work in the remainder of Section 4.0.

22. Design the System (Subsections 4.4 through 4.9). Follow the procedure in Subsections 4.4 through 4.9 to design the system.

23. Assemble the Design Package (Subsection 5.1). In Subsection 5.1, you will assemble the designs of Section 4.0 into a design package and check to make sure that the system will fit together (that is, verify dimensions, lengths, flows, velocity, etc.). Correct any deficiencies to make sure that you are aware of all costs which can be identified.

24. Negotiate An Equipment Package (Subsection 5.1.2). Contact the manufacturer(s) and supplier(s) identified in Step 21 and negotiate or receive bids for the equipment package.

25. Make a Project Cost Estimate (Subsection 5.1.4). From the information in Section 4.0 and Subsection 5.1.4, make a detailed project cost estimate for the complete project.

26. Go/No-Go (Subsection 5.1.5). Like the previous Go/No-Go decision, this decision is based on the economics of the system.

27. Obtain FERC License (Subsection 8.3). Most lending institutions will require an FERC license before they will loan money for a hydropower project. Federal law requires a license before construction begins. In Step 14, you filed for a license. You will have to have a license before proceeding much further.

28. Finalize the Marketing Contract (Subsection 8.4). If you plan to sell power, it is time to negotiate a firm price for the power and obtain a legal contract.

29. Develop Financial Package (Section 7.0 and Appendix A-5). Develop a financial package to present to the lending institutions.

30. Obtain Financing (Section 7.0). Obtain the financial resource required to construct the project.

31. Finalize Design (Subsection 5.1). If the purchased equipment requires changes in any of the design criteria, correct the design to account for the changes. If it does not, use the design from Step 23 for construction of the project.

32. Obtain Local Building Permits (Subsection 8.2). Obtain county and/or city permits before starting construction.

33. Construct the System (Section 5.2). Procure equipment, construct the system, and install all components.

34. Operate the System (Section 6.0). Section 6.0 describes some of the things that should be considered during the first startup of the system. This section also tells you how to bring the system on line.

Each of the 34 events are shown in Figure 1-4 as an activity line. The event is referenced to the activity line by placing the event number above the line. The figure shows which events can be performed simultaneously. The figure shows event sequence only; it does not represent a time frame for doing the work.

## 1.6 Event Schedule

Table 1-1 shows the typical range of time that might be required to complete each event shown in the logical sequence of events (Figure 1-4).

TABLE 1-1. TYPICAL TIME RANGE FOR MICROHYDROPOWER DEVELOPMENT EVENTS

| Events | Typical Time Range[a] (months) | |
|---|---|---|
| All Categories | Low | High |
| 1. Lightly review the handbook | -- | -- |
| 2. Read Sections 1 and 2 and Subsection 8.1 | -- | -- |
| 3. Determine your power requirements | 1/4 | 1 |
| 4. Make site inspection | 1/4 | 1/2 |
| 5. Make initial contact with state and local agencies | 1 | 2 |
| 6. Make initial contact for Federal land-use permits | 1/2 | 2 |

TABLE 1-1. (continued)

| Events | Typical Time Range[a] (months) | |
|---|---|---|
| All Categories | Low | High |
| 7. Determine available flow | 1/4 | 12 |
| Category 1 with an Existing Dam, Canal Drop, or Industrial Waste Discharge Site | | |
| 8. Measure head and distance | 1/4 | 1 |
| 9. Determine design capacity | 1/4 | 1/2 |
| Category 1 with a Run-of-the-Stream Site | | |
| 8. Determine design head | 1/4 | 1/2 |
| 9. Measure head and distance | 1/4 | 1 |
| Category 2 with an Existing Dam, Canal Drop, or Industrial Waste Discharge Site | | |
| 8. Measure head and distance | 1/4 | 1 |
| 9. Determine plant capacity | 1/4 | 1/2 |
| 9a. Determine annual energy production | 1/4 | 1/2 |
| Category 2 with a Run-of-the-Stream Site | | |
| 8. Determine plant capacity | 1/4 | 1/2 |
| 9. Measure head and distance | 1/4 | 1 |
| 9a. Determine annual energy production | 1/4 | 1/2 |
| 10. Go/No-Go | -- | -- |
| 11. Determine federal requirements | 1/4 | 1/2 |
| 12. Obtain state and local permits | 2 | 6 |
| 13. Obtain Federal land-use permits | 2 | 6 |
| 14. File for FERC license | 1/2 | 1 |
| 15. Read section on turbines | -- | -- |

TABLE 1-1. (continued)

| Events | Typical Time Range[a] (months) | |
|---|---|---|
| All Categories | Low | High |
| 16. Contact manufacturer and suppliers | 1 | 3 |
| 17. Determine market potential | 1 | 3 |
| 18. Determine financing options | 1 | 3 |
| 19. Make preliminary cost estimate | 1/4 | 1/2 |
| 20. Go/No-Go | -- | -- |
| 21. Select the design criteria | 1/4 | 1/2 |
| 22. Design the system | 3 | 6 |
| 23. Assemble the design package | 1 | 2 |
| 24. Negotiate an equipment package | 1 | 3 |
| 25. Make a project cost estimate | 1 | 2 |
| 26. Go/No-Go | -- | -- |
| 27. Obtain FERC license | 1 | 6 |
| 28. Finalize the marketing contract | 1 | 3 |
| 29. Develop financial package | 1 | 3 |
| 30. Obtain financing | 3 | 6 |
| 31. Finalize design | 1/4 | 2 |
| 32. Obtain local building permits | 1/4 | 1/4 |
| 33. Construct the system | 3 | 12 |
| 34. Operate the system | -- | -- |

a. The reader is cautioned not to add up the columns of months to obtain total elapsed time, since many of the events are simultaneous.

## 2. WHAT IS HYDROPOWER?

Hydropower is the power derived from the natural movement or flow of masses of water. Most commonly, this power is harnessed by taking advantage of the fall of water from one level to another, thus exploiting the effect of gravity. The energy of the falling water is converted to mechanical energy by the use of a turbine. Microhydropower turbines come in many shapes and sizes--from waterwheels, to pumps used as turbines (where water is forced through the pump in the opposite direction), to squirrel cage turbines, called crossflow turbines. Once the turbine is used to convert water energy to mechanical energy, the mechanical energy in turn can be used to perform work or converted to some other form of energy, such as electrical energy (called hydroelectric energy). The energy-producing potential at any given hydropower site depends on the energy of the water, which in turn depends on the distance the water falls, called the head, and on the amount of water flowing.

The actual amount of mechanical or hydroelectric energy produced at such a site also depends on the efficiency at which the turbine or turbine-generator unit can convert the water energy to the other forms of energy. Sites with modern microhydroelectric units will have efficiencies ranging from 40 to 75%. In other words, 40 to 75% of the energy-producing potential is actually converted into useful energy.

This section of the handbook discusses the recent history of waterpower, definitions of head and flow, the definition of kilowatt, the power equation (a more detailed development of the power equation is given in Appendix A-1), types of microhydropower source, and two example sites. (The example sites are discussed in the remaining sections of the handbook and are presented as complete examples in Appendix B.) A reader who is familiar with these subjects may want to skim over this section.

## 2.1 History and Typical Microhydropower Systems

The use of hydropower as a source of mechanical energy dates back more than 2,000 years to the earliest waterwheels. Such wheels in one form or another were the primary source of power for many centuries. French engineers started making improvements in waterwheels in the mid 18th century and continued to lead the field until the mid 19th century. A French military engineer, Claude Burdin (1790-1873), first used the term "water turbine" from the Latin turbo: that which spins. (Although water wheels fit this definition, they are not now classed as turbines by most of those working in the hydropower field.) The first commercially successful new breed of hydraulic turbine was a radial-outflow type. The water entered at the center of the turbine and flowed outward through the turbine runners (blades). The turbine was developed by a student of Burdin, Benoit Fournegron (1802-1867). In 1836, a patent was awarded to Samuel B. Howd of Geneva, New York for a radial "inflow" turbine. The idea was perfected by James B. Francis and Uriah A. Boyden at Lowell, Massachusetts in 1847. In its developed form, the radial inflow hydraulic turbine, now known as the Francis turbine, gave excellent efficiencies and was highly regarded.[a]

Another class of turbine used the concept of a vertical wheel driven by a jet of water applied at one point in its circumference. The approach led ultimately to the Pelton wheel, which uses a jet or jets of water impinging on an array of specially shaped buckets closely spaced around the rim of a wheel. The Pelton wheel was developed at the end of the 19th century by a group of California engineers, among them Lester A. Pelton (1829-1908).[b]

Waterwheels and modern turbines are often differentiated by stating that modern turbines are smaller, run at higher speeds, will work submerged,

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a. The Design and Performance Analysis of Radial-Inflow Turbines, Volume 1, Northern Research and Engineering Corporation, Cambridge, Massachusetts.

b. "The Origins of the Water Turbine," Norman Smith, Scientific American, January 1980, Vol. 242.

can use a wide range of heads of water, and, finally, are more powerful or more efficient.[a] Waterwheels, on the other hand, produce shaft mechanical power with slow rotational speed and high torque. The rotation speed might range from 6 to 20 revolutions per minute (rpm). Where water wheels were used in industry, power was transmitted by pulleys and belts to perform work such as milling and grinding or operating saws, lathes, drill presses, and pumps. These operations needed the higher torque and only modest rpm.

It is worth noting that water wheels offer high torque and thus are capable of driving heavy, slow-turning mechanical equipment. If that is the type of power you need, you should look at the possibility of using a waterwheel. They will operate even with large variations in the water flow rate, and they require minimal maintenance and repair. In addition, trash racks and screens are usually not required, since most waterwheels can operate with dirt, stones, and leaves entrained in the water.

Electric generators, however, require rotation speeds ranging from 720 to 3,600 rpm. Generators operating at higher speeds are smaller and cost less than those operating at lower speeds. For this reason, the modern turbine is favored for the generation of electricity.

The generation of electric power from flowing water has been a source of energy in the United States for a century. The first electricity from hydropower was produced in 1882 by a 12.5-kilowatt (kW) plant in Appleton, Wisconsin. Since then, the number of hydroelectric power generating facilities in the U.S. has grown to more than 1,300, and total capacity now surpasses 76,000 megawatts (MW).

Early hydroelectric power plants were small, and the power they produced went to nearby users. But by the early 1900s, design and engineering advances had opened the way for larger facilities and greater transmission distances. Improvements in dam construction equipment and techniques made much larger dams possible, while the use of alternating current (a-c)

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a. "The Origins of the Water Turbine," Norman Smith, Scientific America, January 1980, Vol. 242.

generators, transformers, and the development of suspension-type insulators led to long-distance, high-voltage power transmission systems.

By the 1920s, emphasis had shifted to the development of large hydroelectric power projects, and as time went by, smaller developments--those under 25 MW--were more and more ignored. During the 1950s and 1960s, a combination of economic factors--the need to replace worn out turbine-generator equipment and the availability of inexpensive fossil fuel--made it appear that a number of smaller hydropower facilities built early in the century had outlived their usefulness, and many of these were shut down and disposed of by their owners. Recently, however, the rapidly rising costs of fossil fuels and the high cost of meeting environmental standards for new thermal power plants have prompted a new look at hydropower's role in the national energy picture. And because almost all of the economically feasible and environmentally acceptable sites for large hydropower projects have already been developed, this new look at hydropower is focusing on smaller installations.

## 2.2 Head

Hydropower has been defined as energy available from falling water--water that is changing elevation. If the change in elevation is measured, the measured distance is called "head." Head is usually measured in feet. For example, if a stream is impounded by a small dam and the upstream pool behind the dam is 20 feet higher than the stream below the dam, the head of water at the dam is 20 feet (see Figure 2-1). Likewise, if a road crosses a stream where the elevation of the water is 6,020 feet above sea level, and at some distance downstream the road crosses the stream again where the elevation of the water is 6,000 feet, the available head between the two crossings is 20 feet.

Thus, head is vertical change in elevation measured in feet. Feet of head is a convenient way of expressing the theoretical energy available for any given amount of water. The mathematical symbol used for head is "h." Subsection 3.4 of the handbook discusses how to measure head.

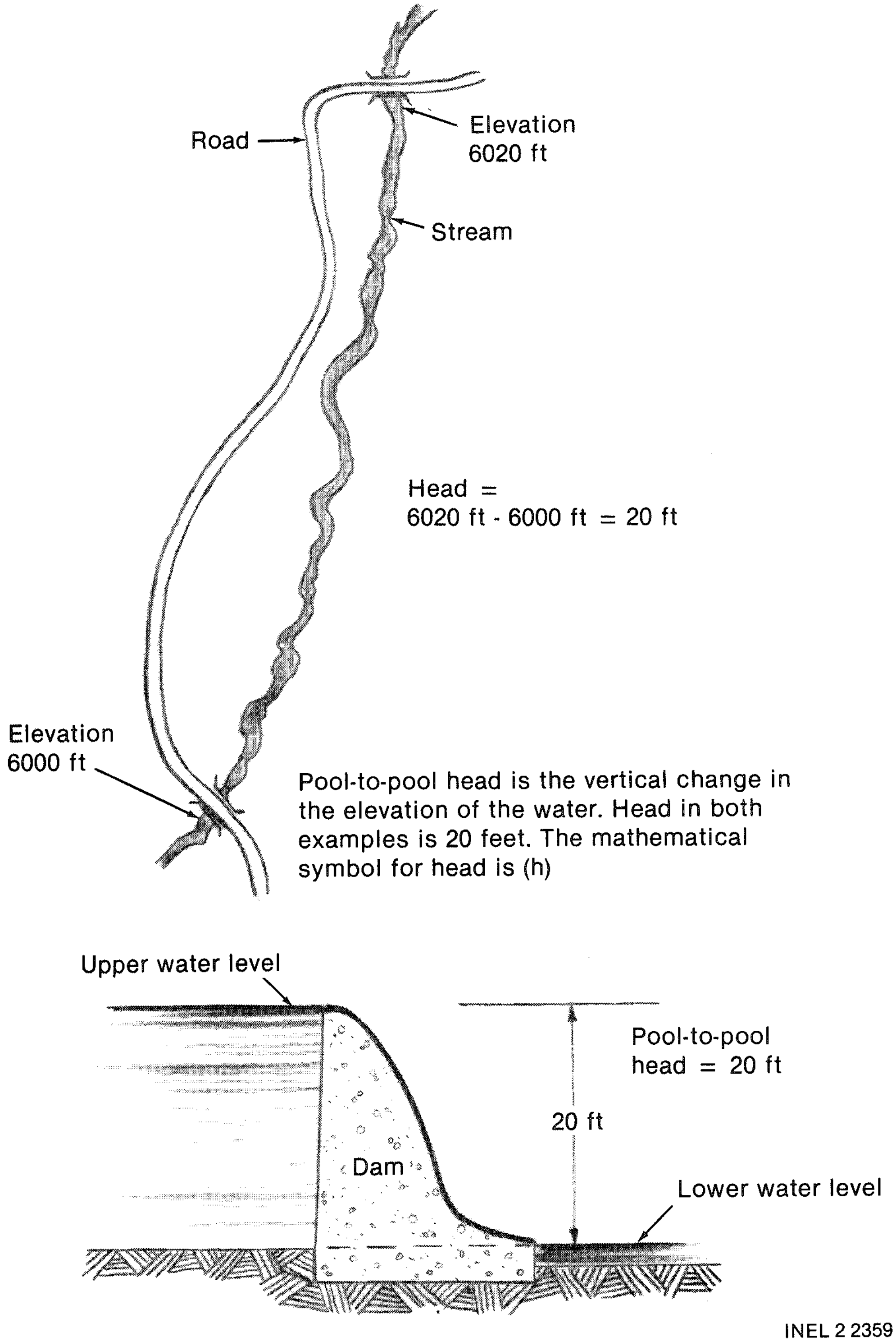


Figure 2-1. Head illustrated.

NOTE: You should be aware that two terms are used for head, and that you must know the difference between these terms when dealing with turbine manufacturers so that you will convey the proper information for turbine selection. The head given above, 20 feet, is termed the pool-to-pool head (sometimes referred to as the gross head). This is the total hydraulic head available to perform work. The turbine manufacturer sizes his turbine for net effective head (net head). Net head is the pool-to-pool head less hydraulic losses from friction, entrance losses, trashrack losses, etc. Calculation of these losses is discussed in Subsection 4.5. It is important that you make it clear to the manufacturer or engineer which head you are referring to in your discussions or correspondence.

## 2.3 Flow

To compute theoretical power from a hydropower site, the head and the volume of water flowing in the stream must be known. The gallon is a standard unit for volume. The cubic foot is another unit of volume that may not be as familiar. The cubic foot is the standard unit of volume in hydropower. One cubic foot of water contains 7.481 gallons (see Figure 2-2).

1 cubic foot ($ft^3$) of water = 7.481 gallons (gal)

Flow is the volume of water passing a point in a given amount of time. For example, if a pipe has water running into a 1 $ft^3$ container and it takes 1 minute to fill the container, the "flow" of the water out of the pipe is 1 cubic foot per minute (see Figure 2-3). The time period for measured flow can either be a minute or a second. In microhydropower, you may encounter both units, depending on the literature you read. It is important to remember that since a minute is 60 times longer than a second, flow per minute is 60 times larger than the same flow per second.

In this handbook, flow is expressed in cubic feet per second. The mathematical symbol for flow is "Q".

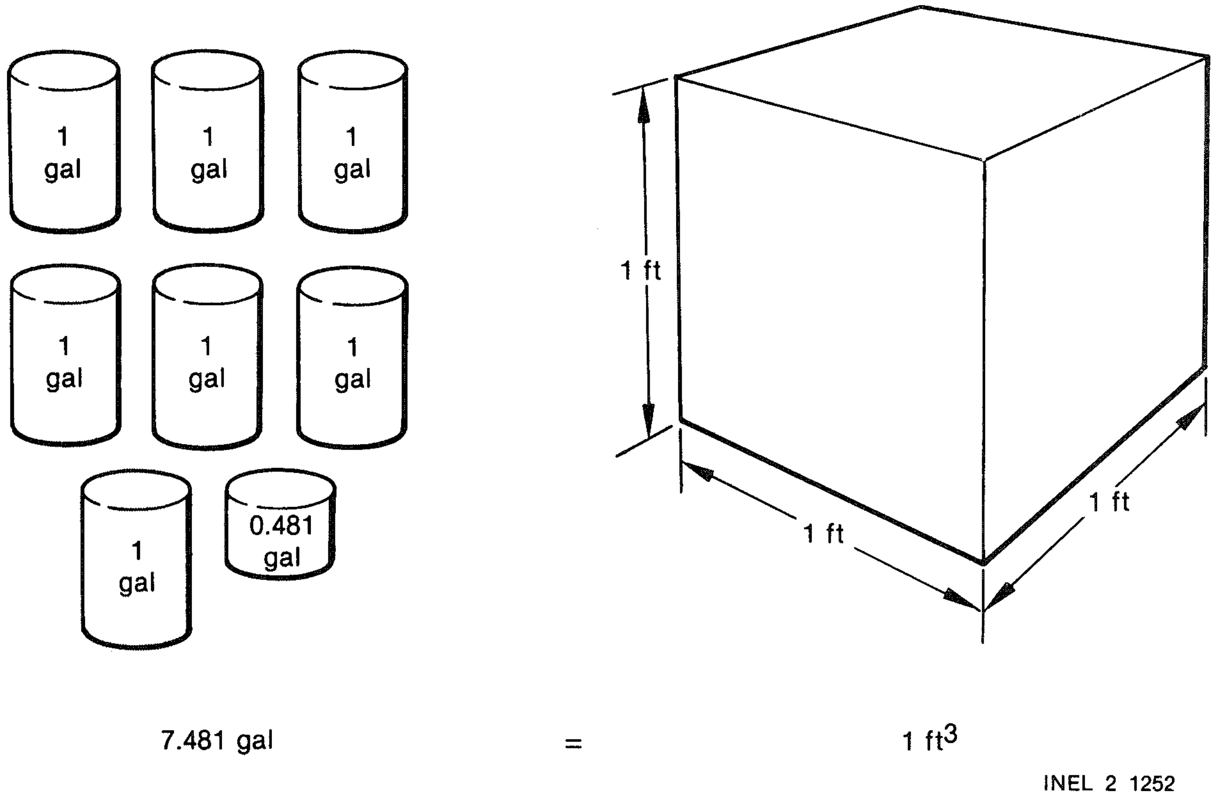


Figure 2-2. Cubic foot illustrated.

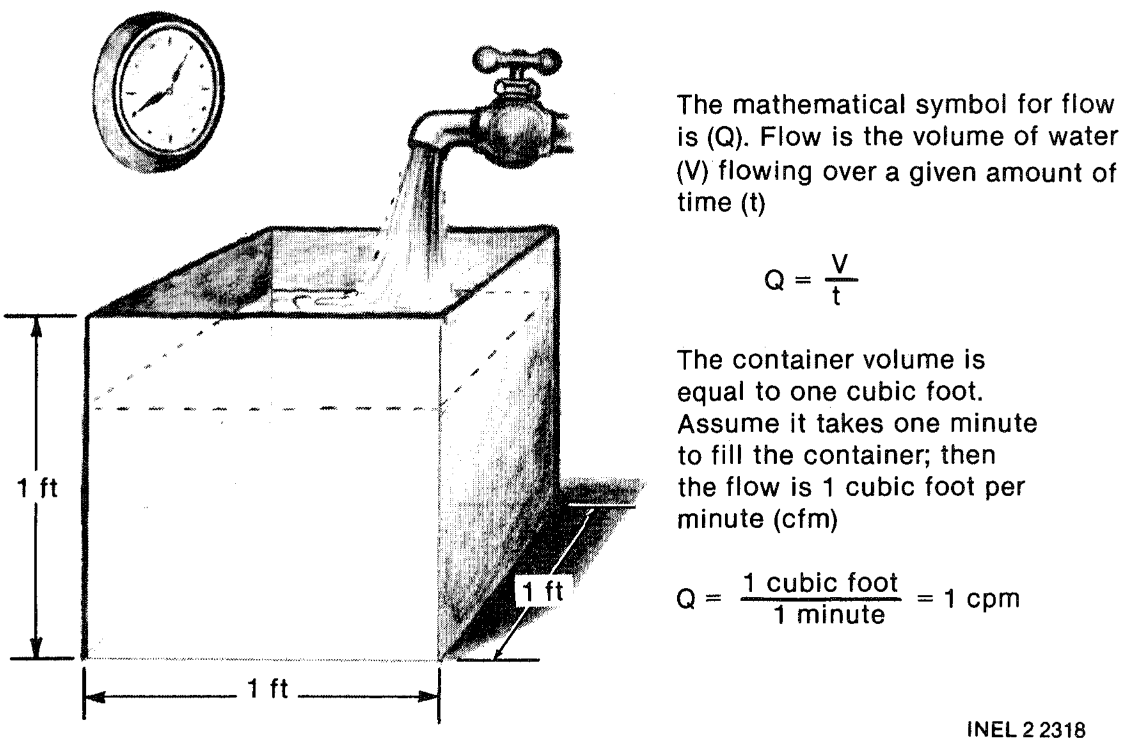


Figure 2-3. Cubic foot per minute illustrated.

$$Q = \frac{V}{t} \quad (2\text{-}1)$$

where

$Q$ = flow in cubic feet per second (cfs)

$V$ = volume of water in cubic feet ($ft^3$)

$t$ = time of measurement in seconds (sec).

Assuming that the volume of water is 1 cubic foot and the time of measurement is 1 second, the flow, Q, would equal 1 cubic foot per second, expressed as 1 cfs. Subsection 3.3 of the handbook discusses how to measure flow in cfs.

## 2.4 Kilowatt

The basic unit of electrical power used is the kilowatt, abbreviated as kW. A kilowatt is equal to 1,000 watts (W). For example, ten 100-watt light bulbs burning at the same time would require one kilowatt. If the lights were to burn for one hour, the amount of energy used would be one kilowatt-hour, abbreviated as kWh (see Figure 2-4). The kilowatt-hour is the standard measurement of energy from which most domestic electric bills are computed.

The term "microhydropower" is applied to any hydroelectric plant that generates 100 kW or less. One thousand (1,000) 100-W light bulbs burning at one time would require 100 kW:

1,000 light bulbs x 100 W = 100,000 W

100,000 W ÷ 1,000 = 100 kW.

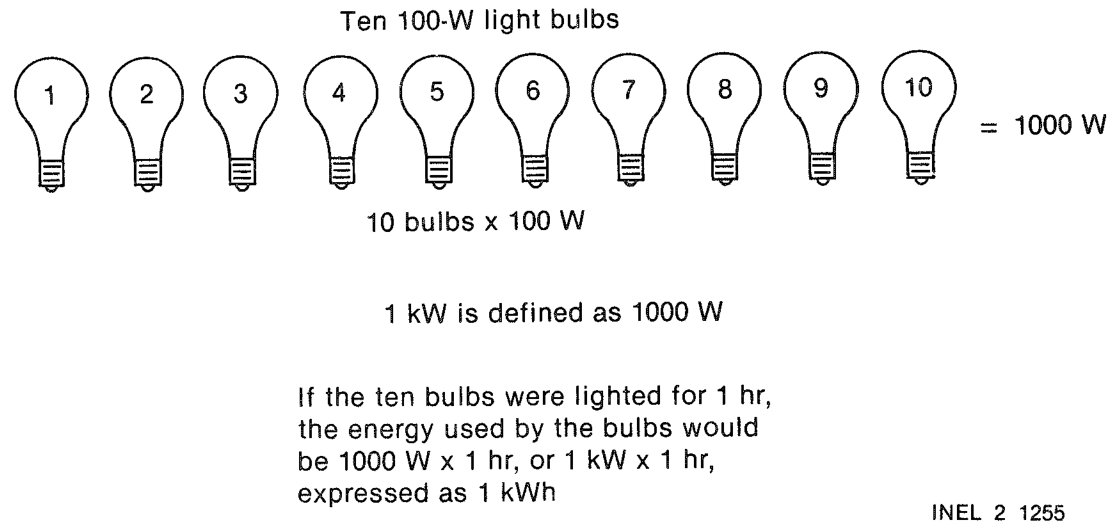


Figure 2-4. Kilowatt illustrated.

If the lights were to remain on for one hour, the amount of energy used would equal 100 kWh:

100 kW x 1 hr = 100 kWh.

If the energy costs 50 mills (5¢) for each kWh, then 1,000 lights burning for one hour would cost $5.00:

1,000 lights x 100 W x 1 hr = 100 kW x 1 hr = 100 kWh

100 kWh x $0.05 per kWh = $5.00.

Each additional hour the lights remained on would cost another $5.00. The average household (not including electric heat) consumes about 1,000 kWh per month.[a] Assuming the same cost of 5¢ per kWh, the monthly electric bill would be approximately $50.00.

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a. The Publication, Electrical World Directory of Electric Utilities, 1979-1980, 88th Edition, McGraw-Hill Publishing Co., which gives average residential consumption for most utilities in the United States, shows that this average varies widely depending on the utility and location. The value chosen for presentation here (1,000 kWh/month) is approximately midrange.

In actual practice, a typical home has a peak demand of about 5 kW. This means that at some time during a typical month there will be a period during which the household will be consuming power at a rate of 5 kW. A large group of homes taken together would have an average peak demand of about 2.5 kW per home, and an average demand of 1.4 kW. The average peak demand per home is reduced for a group of homes because not all appliances are in use at the same time, and the more homes, the more the peak is spread out. This would indicate that a stand-alone 100-kW plant could actually supply the energy needs of 35 to 40 homes, assuming that the annual production is 50% of the theoretical maximum from the 100-kW plant. (The reason for the 50% assumption is explained later in the handbook.) If a 100-kW hydropower plant is used in place of diesel power units, the plant would displace diesel fuel at the rate of 22 gallons per hour (gph), or about 100,000 gallons per year (gpy).

If you develop a microhydropower plant that produces more energy than you consume, you may be able to sell the excess power to the local utility. For example, if an average of 25 kW is available for utility buyback 40% of the time during a year and the utility agrees to pay you 50 mills per kWh, you would receive $4,380 per year in revenue from the utility:

24 hr/day x 365 days/yr x 0.40 (time available is 40%) = 3,504 hr/yr

3,504 hr/yr x 25 kW = 87,600 kWh/yr

87,600 kWh/yr x $0.05 = $4,380 annual revenue.

The cost of installing a microhydropower plant typically ranges from $1,000 to $4,000 per kW of installed capacity. A 30-kW plant might cost anywhere from $30,000 to $120,000. It is the intent of this handbook to help keep installation costs to a minimum.

## 2.5 Power Equation

If you plan to develop a microhydropower site, you must become familiar with the basic power equation:

$$P = \frac{Q \times h \times e}{11.81} \qquad (2\text{-}2)$$

where

| | | |
|---|---|---|
| P | = | power in kW |
| Q | = | flow in cfs |
| h | = | head in feet (pool-to-pool or net effective head, depending on the efficiency factor selected) |
| e | = | efficiency (to be explained) |
| 11.81 | = | conversion constant for power in kW divided by the density of water. |

A more detailed development of the power equation is provided in Appendix A-1. Equation (2-2) is the standard equation that is used throughout the remainder of the handbook to calculate power in kW.

Any power-producing system produces less power than is theoretically available. The efficiency factor, e, of any given system is the actual power produced divided by the theoretical power available, expressed as a percentage:

$$e = \frac{P}{P_{th}} \times 100 \qquad (2\text{-}3)$$

where

$e$ = efficiency of the system in percent

$P$ = actual power produced

$P_{th}$ = theoretical power available

100 = conversion to percent.

For a microhydropower system, the efficiency may range from 40% to 75%. The efficiency depends on site conditions, the equipment used, and the actual installation method.

The mechanical shaft efficiency of the turbine and associated equipment depends on the following:

- Flow variation effects on the turbine
- Head variation effects on the turbine
- Flow restriction or disturbances at the intake structure
- Friction losses in the penstock, valves, and other associated equipment (penstock is pipe that conveys water to the turbine)
- Turbine losses due to friction and design inefficiencies
- Configuration of draft tube (pipe that carries water away from turbine).

If the efficiency is to include the generator output, the following equipment-related losses also reduce the overall efficiency value:

- Speed increaser losses due to friction and design inefficiency
- Generator losses due to friction and design inefficiency.

The actual efficiency of a particular installation cannot be determined until the site is operational and the head and flow are known for any given power output. Section 4 discusses in more detail how to estimate friction losses and other variables that affect the overall efficiency. For the remainder of the discussion in this section and Section 3, the assumed value for efficiency is 60%. For individual developments, 60% can be used for first-cut calculations of power output, but this value should be refined (up or down) as more is learned about the particular site. The 60%-efficiency figure can be used with the pool-to-pool head since it contains the efficiency losses for head that one would expect to find at a typical site. The 60% efficiency can be used with the pool-to-pool head as it contains the efficiency losses for head which one would expect to find at a typical site.

If the efficiency is assumed to be 60%, then from Equation (2-2)

$$P = \frac{0.60}{11.81} \times Q \times h$$

$$P = 0.051 \times Q \times h \quad .$$

The equation solves for the power produced (P), which is dependent on the value of the two variables, flow (Q) and head (h). If we assume that the flow is 10 cfs and the head is 10 feet, the equation solves for power as follows:

$$P = 0.051 \times 10 \times 10$$

$$= 0.051 \times 100$$

$$= 5.1 \text{ kW} \quad .$$

Now, notice what happens if the flow doubles to 20 cfs:

$$P = 0.051 \times Q \times h$$

$$= 0.051 \times 20 \times 10$$

$$= 10.2 \text{ kW} .$$

In other words, doubling the flow doubled the power. Next, see what happens if the flow returns to 10 cfs and the head is doubled to 20 feet:

$$P = 0.051 \times Q \times h$$

$$= 0.051 \times 10 \times 20$$

$$= 10.2 \text{ kW} .$$

The same power is produced by doubling either the head or the flow. The point is that head and flow have an equal effect on the power equation. Figure 2-5 shows this relationship. Another difference to note in the figure is that the higher head option uses smaller equipment.

## 2.6 Microhydropower Sources

A microhydropower system can be developed from either a natural source or a manmade structure. Natural sources include a stream without a dam (Figure 2-6), a waterfall (Figure 2-7), a spring branch, or even a natural lake. Manmade sources include any structure used to increase head or provide a source of water other than a natural source. Examples of manmade sources are dams (Figure 2-8), canal drops (Figure 2-9), and industrial or domestic wastewater discharge.

### 2.6.1 Natural Sources (Run-of-the-Stream)

A microhydropower system developed on a natural stream is referred to as "run-of-the-stream." Figures 2-10 and 2-11 show two such systems with

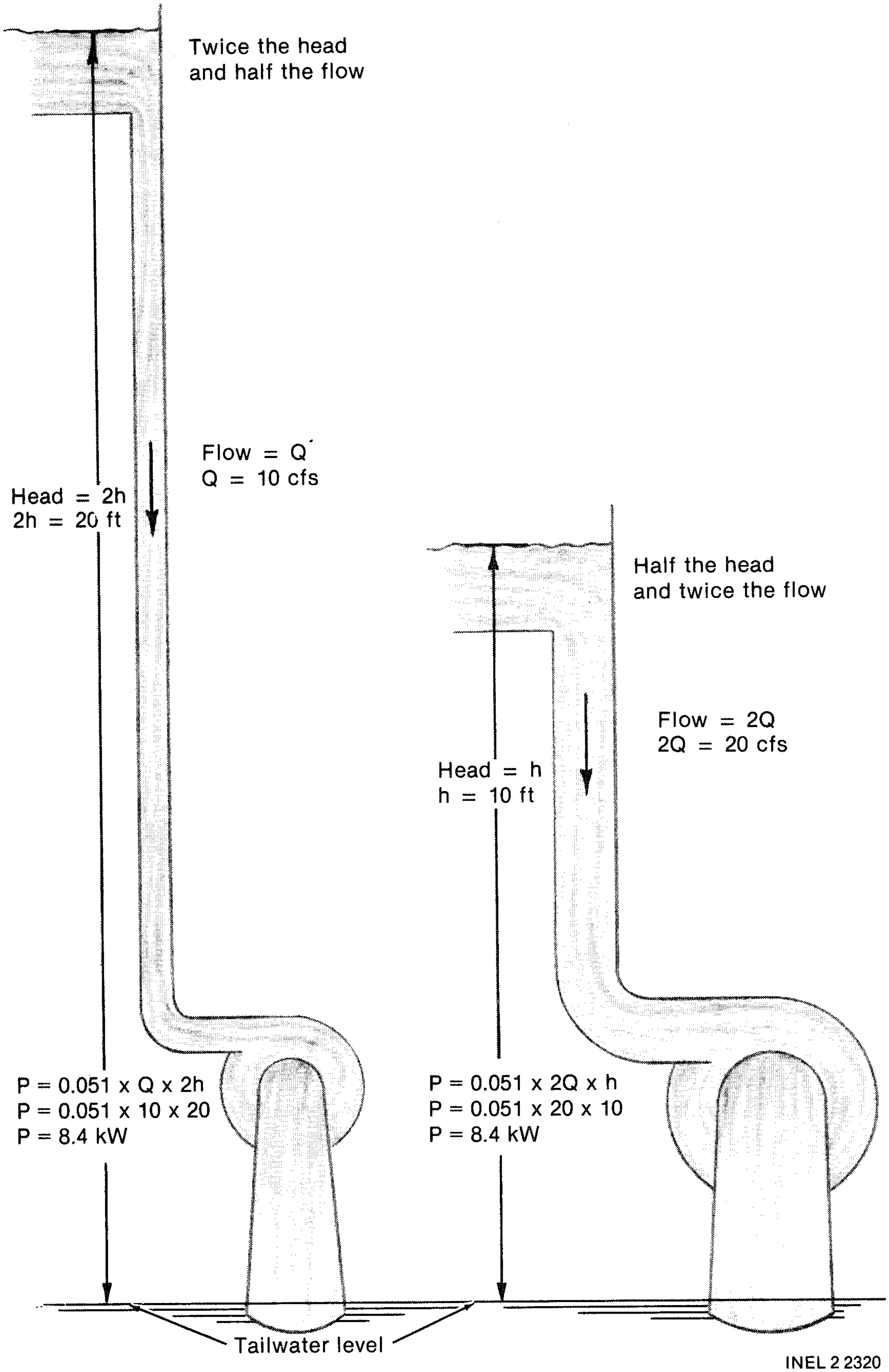


Figure 2-5. Effect of doubling either head or flow.

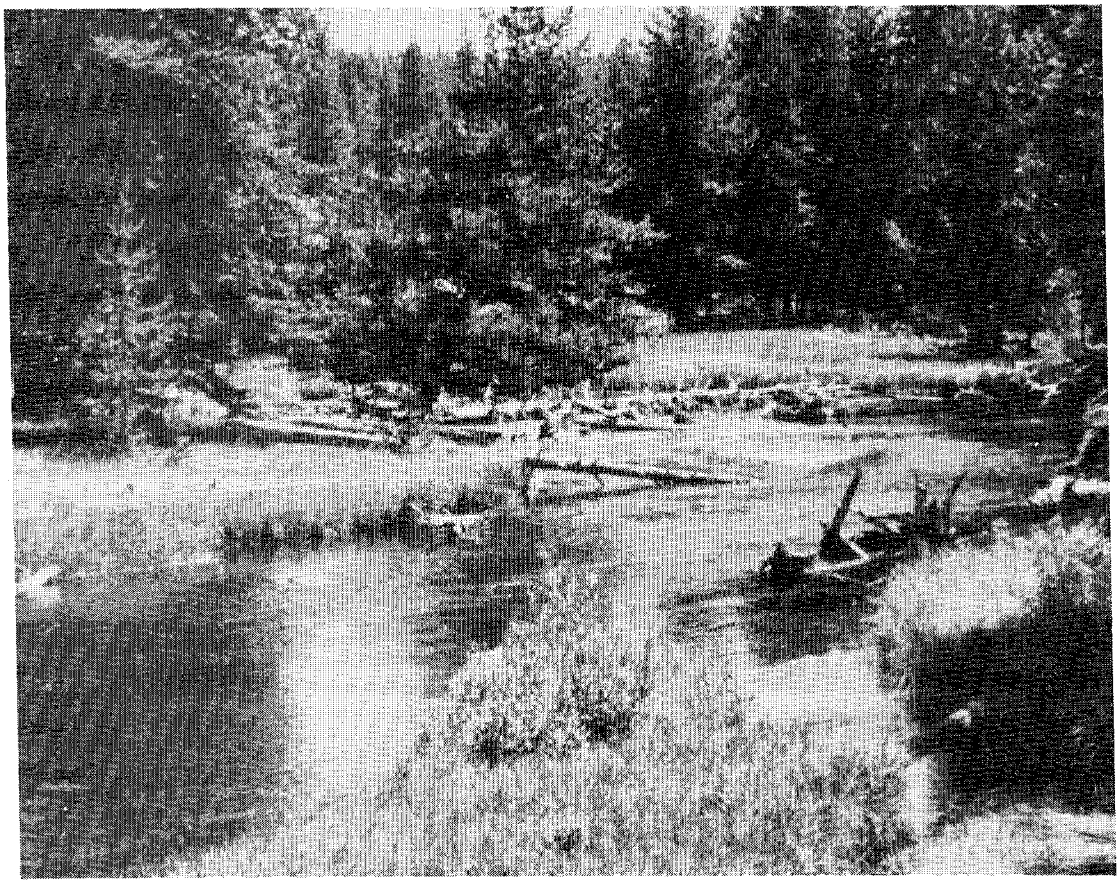

Figure 2-6. Stream without a dam.

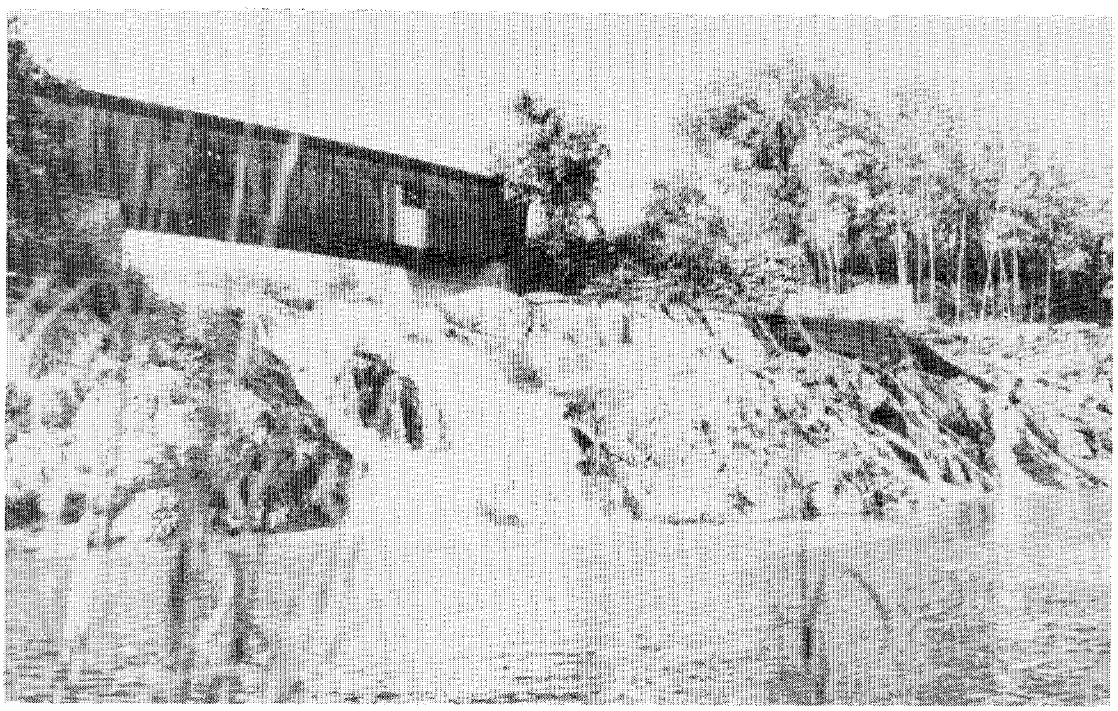

Figure 2-7. Waterfall.

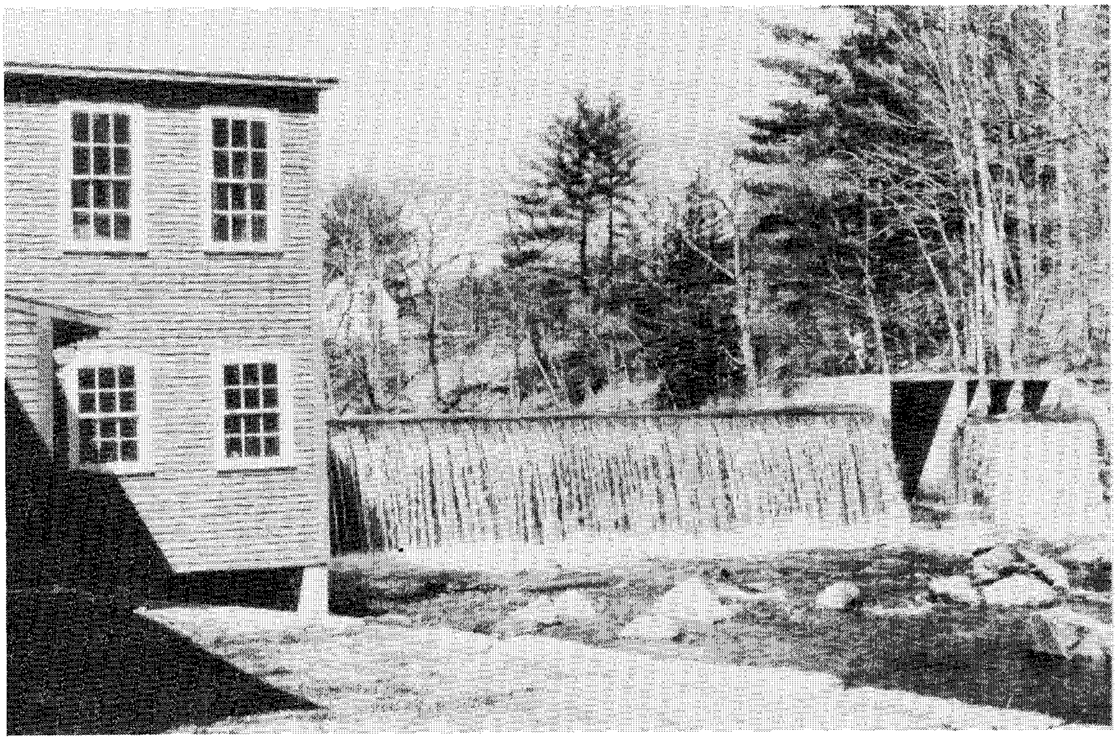

Figure 2-8. Dam.

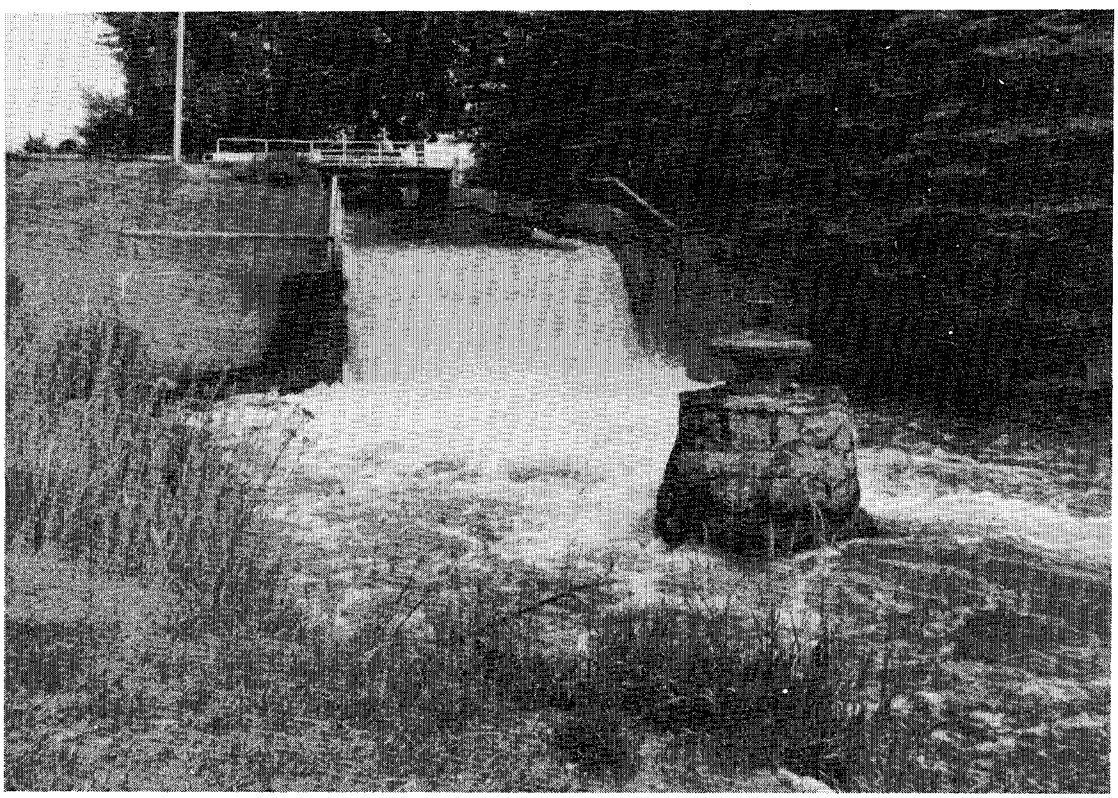

Figure 2-9. Canal drop.

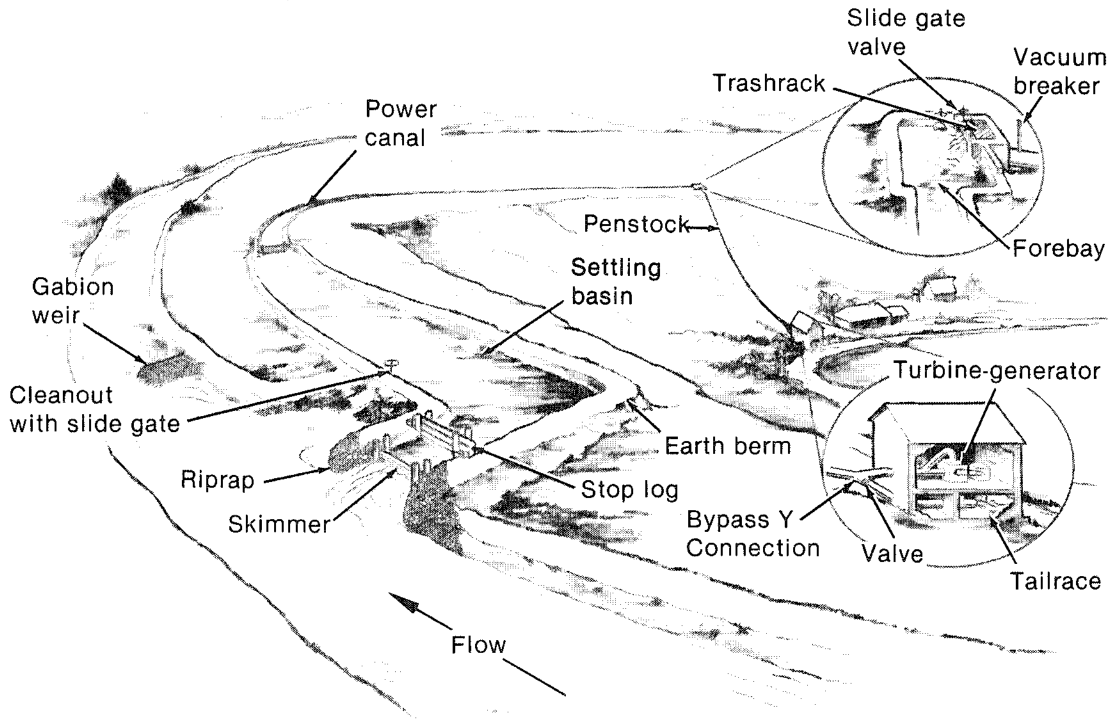


Figure 2-10. Run-of-the-stream development.

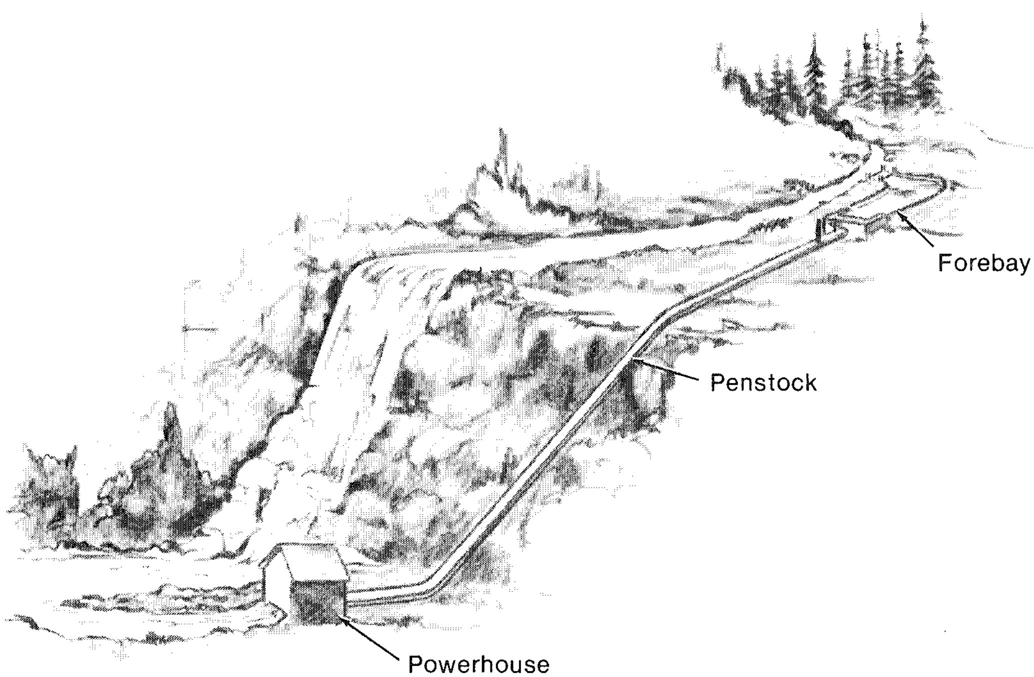


Figure 2-11. Run-of-the-stream development.

appropriate nomenclature. In a run-of-the-stream development, an intake structure diverts water from a stream to a penstock. The structure consists of:

- Stream Diversion Works--The diversion works divert water from the stream into the intake system.

- Settling Basin--The settling basin is located near the diversion works and is used to settle out suspended material before the water enters the power canal.

- Power Canal--The power canal carries water from the diversion works and settling basin to the forebay. A canal is useful where the water can be carried at approximately the same elevation to a point from which the penstock can be made as steep, straight, and short as possible.

- Forebay--The forebay is a settling basin designed to settle out suspended material before the water enters the penstock. Some type of forebay is required in all run-of-the-stream developments.

- Penstock Intake Structure--The penstock intake structure provides the transition from the forebay to the penstock. It also provides the framework for the trashracks and intake gates.

The penstock carries the water from the forebay to the turbine. Ideally, the penstock should be as steep, straight, and short as possible. The powerhouse contains the turbine-generator, controls, and associated equipment, and the tailrace returns the water to the stream.

The design head can be adjusted depending on the available flow and power requirements (Subsection 3.4.1). Therefore, the location of the intake structure is a function of how much head is needed. At natural sources that capitalize on the change in elevation of a waterfall, the head is set by the elevation of the waterfall, and the design procedures should be the same as for manmade sources where the head is established by the characteristics of the site.

Natural sources may have aesthetic value, which should be considered. For example, if all the water flow from a waterfall or a stream is to be used for power production, the waterfall or a portion of the stream will be dried up. If only a portion of the flow is used, aesthetic and other environmental effects are minimized.

Natural sources are subject to annual stream variation. For Category 2 developers, power generating potential will vary with the flow.

### 2.6.2 Manmade Sources

Existing manmade sources can generally be modified to install a microhydropower system without much of an environmental impact. The construction of a dam for the sole purpose of developing microhydropower systems is generally economically prohibitive. However, if a dam is being built for other purposes, a microhydropower system may be a logical and economical addition to the project. Small dams typically have a relatively small change in elevation (head), 35 feet or less. With a small head, the flow has to be larger to produce a given amount of power, and larger flow means bigger turbines--and thus more expense than for installations operating with a larger head to produce the same amount of power. Figure 2-12 shows a possible installation at an old mill site. Figure 2-13 shows a siphon penstock that could be used on an existing dam at which there is no way to draw the water out of the reservoir.

In certain parts of the country, manmade structures such as canal drops provide excellent opportunities for hydropower production. Flow can be seasonal, but it is generally constant during the months of operation. For canals where the flow is seasonal, care should be taken to ensure that enough energy can be produced annually to justify the expense. Figure 2-14 shows such an installation.

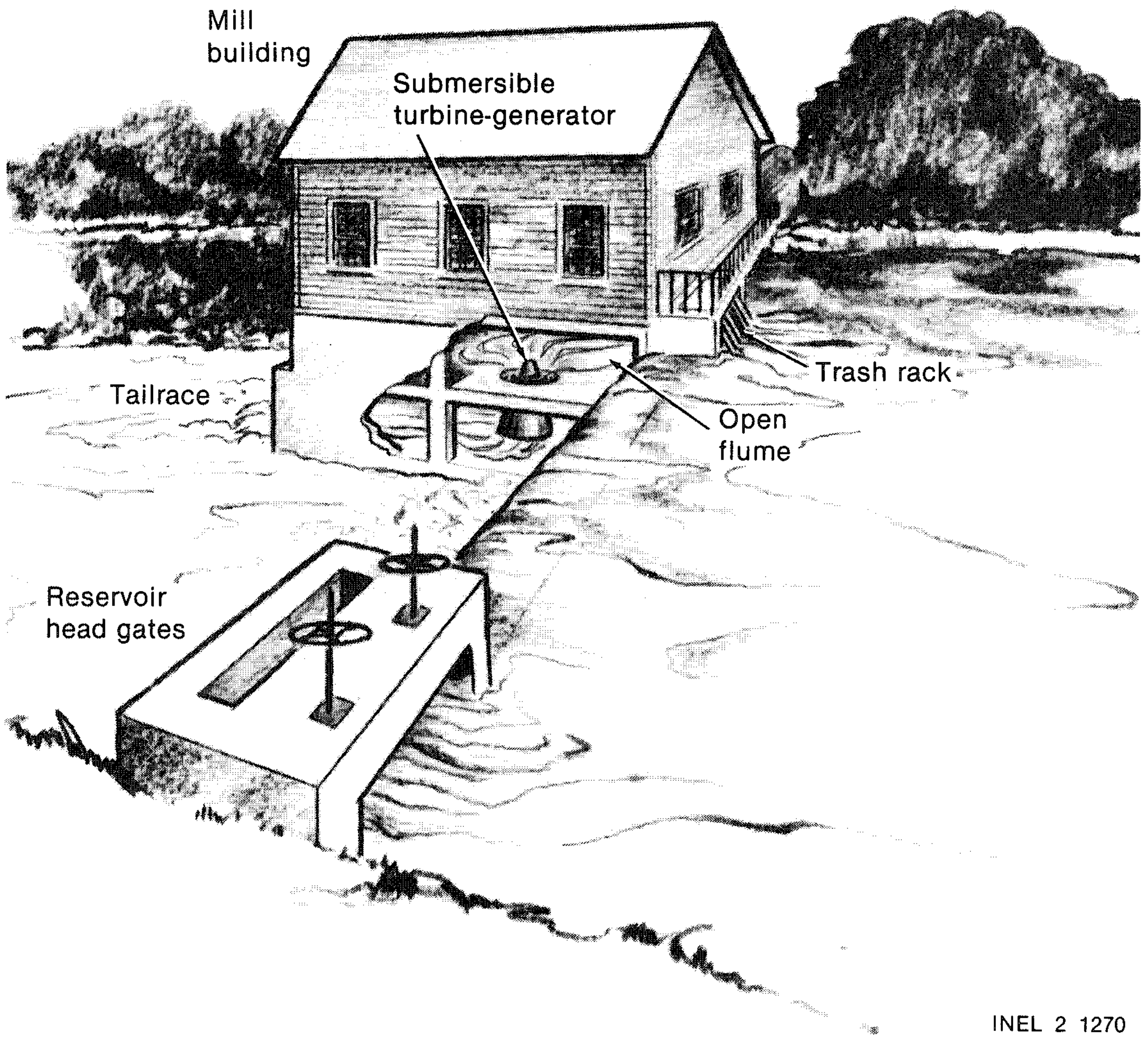


Figure 2-12. Installation at an old mill site.

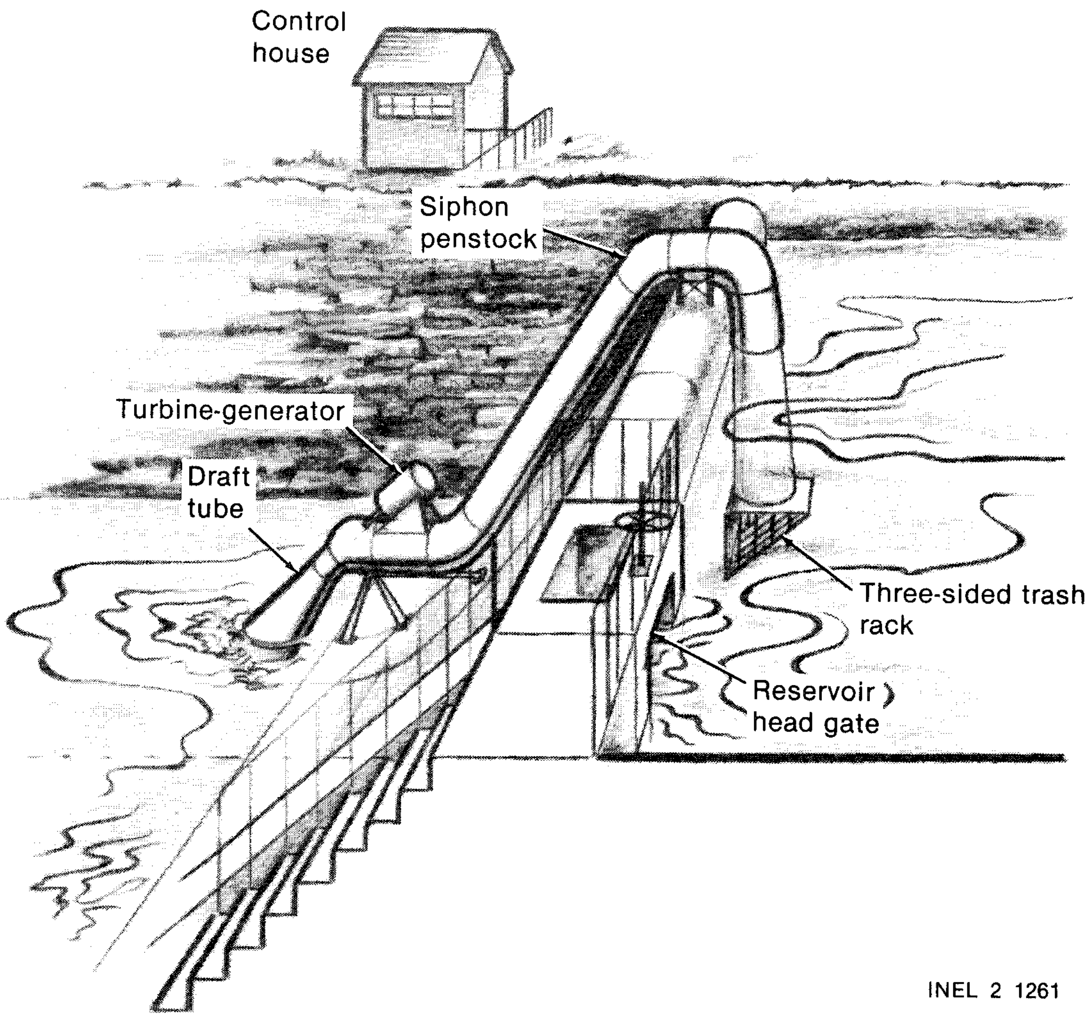


Figure 2-13. Siphon penstock at an existing dam.

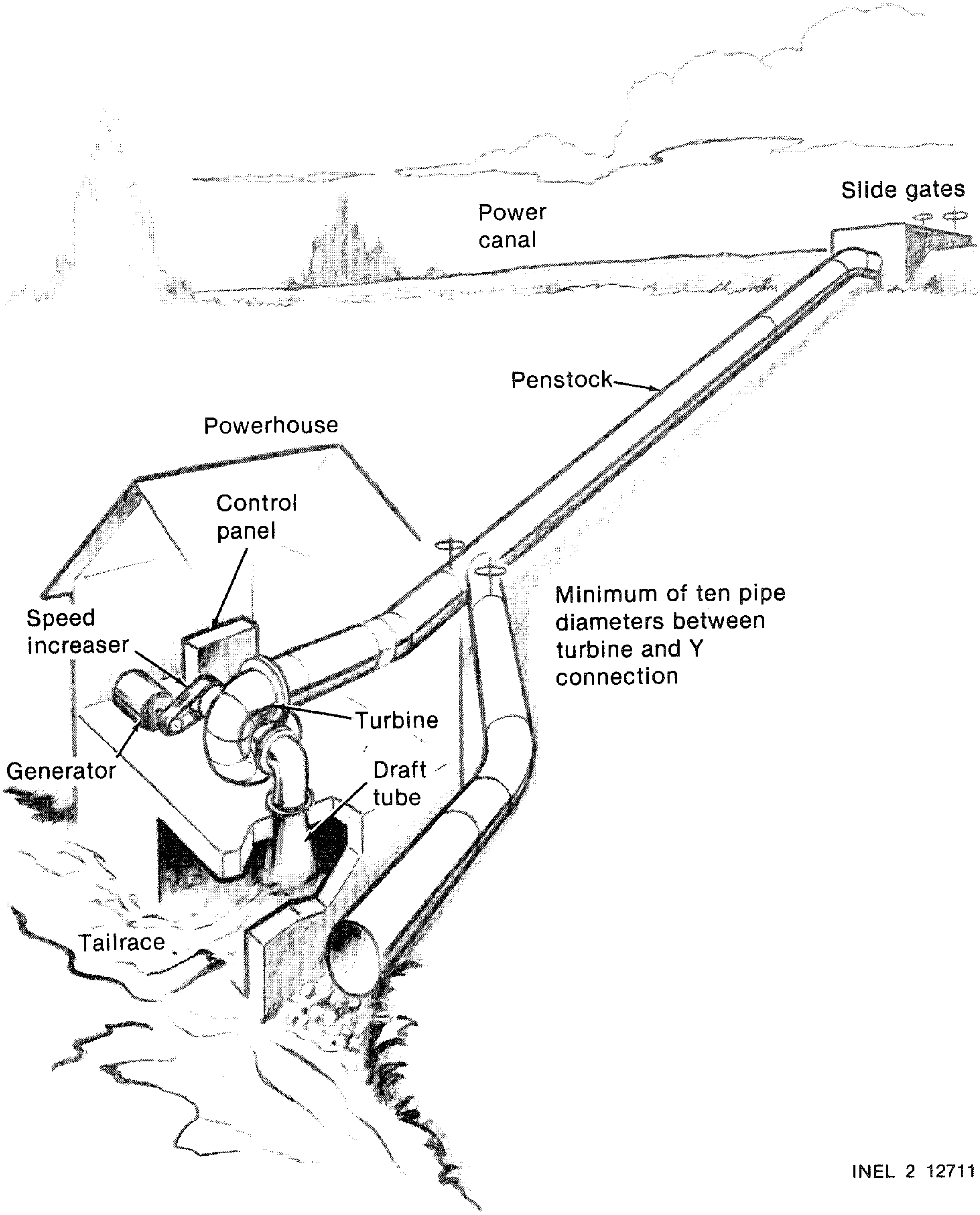


Figure 2-14. Installation at a canal.

Wastewater discharge from industrial or domestic treatment plants may have sufficient head and flow to be useful for hydropower. Those associated with such institutions may want to develop such a source. Private developers may be able to develop such a source, but the first hurdle is obtaining permission. Sources of this nature usually offer a steady flow, which helps to optimize turbine selection and minimize equipment cost. Figure 2-15 shows such an installation.

## 2.7 Typical Example Sites

Two site examples are presented in Appendix B. One involves developing an existing dam, and the other makes use of a natural, run-of-the-stream source. The specifications for these examples are given below. The developer is encouraged to determine which example most closely represents the site to be developed and to follow the details of the example as a guide for proceeding through the handbook.

### 2.7.1 Manmade Source

An existing dam is located on a small stream in the rolling hills of New Hampshire. The developer's site includes an old, retired gristmill. The mill and dam produce a 12-1/2-foot drop in the stream elevation. Upstream from the dam, the pool has filled in with gravel and silt, leaving it only 3 feet deep. The elevation of the pool is fairly constant, with the crest of the dam acting as the spillway. In a 20-foot wide gorge below the dam, the depth of the stream's normal flow varies from 26 inches in April to 8 inches in late August. Occasional spring rains will raise the stream to 3-1/2 feet. Twice in the last fifteen years the stream has flooded above the 5-foot gorge and inundated a lower pasture. During those floods, the stream was approximately 40 feet and 60 feet wide, respectively, and the depth of water in the pasture averaged 3 inches and 8 inches

The developer's residence and 75-head dairy operation are located near the old mill. The residence includes a washer, dryer, refrigerator, freezer, electric stove, hot water heater, and electric heat. The electric

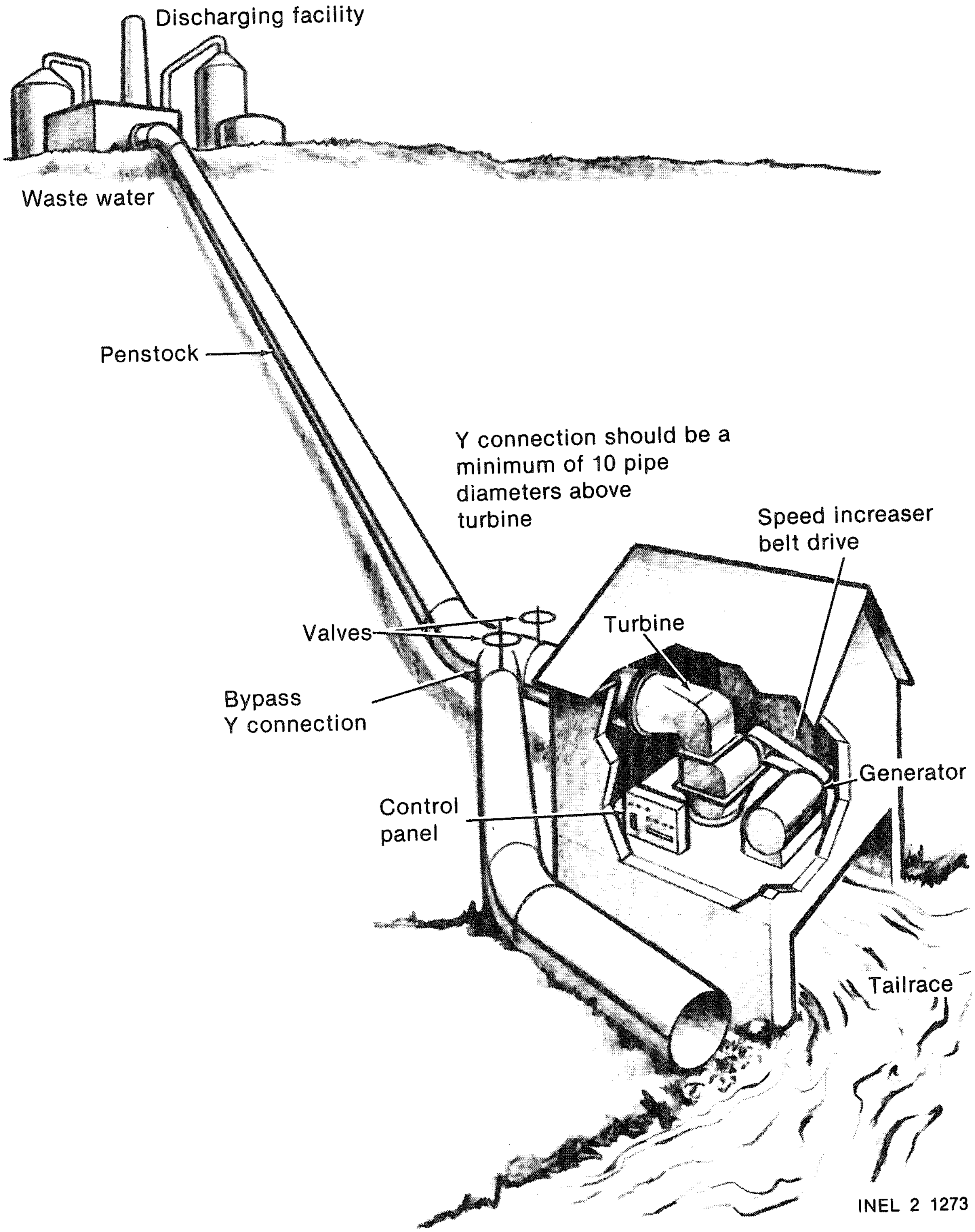


Figure 2-15. Installation using wastewater discharge.

utility's distribution line for the valley is within 300 yards of the mill. The developer hopes to supply his electrical needs and sell any excess power to the utility.

### 2.7.2 Run-of-the-Stream Source

The run-of-the-stream site is located in mountainous terrain in Washington. The average annual temperature range is from a high of 98°F to a low of -35°F. The stream flows from a narrow canyon that opens onto a high mountain meadow. The developer's property ends at the entrance to the canyon, which is the beginning of U.S. Forest Service property. A Forest Service road provides access to the canyon and is used for logging operations. The road crosses the stream and parallels it for approximately 1/4 mile before ascending into the canyon. According to a U.S. Geological Survey (USGS) contour map, the canyon floor rises approximately 440 feet in a mile.

The stream is fed by snow melt and small springs and contains small native fish. The size of the stream varies annually from 7 feet wide and 12 inches deep to 4 feet wide and 5 to 6 inches deep. Eight months out of the year the stream is usually at least 5 feet wide and 10 inches deep. At the location favorable for a powerhouse, high water markings are observed approximately 3 feet above the natural stream bed. At that height, the width of the stream would approach 25 feet.

Irrigation water rights are held by ranchers below the developer's site. Noncomsumptive water rights will have to be obtained by the developer.

The developer's primary objective is to provide power for two family dwellings that are currently satisfactorily supplied power from a 14-kW diesel generator. The dwellings each have electric water heaters, refrigerators, freezers, and use an electric resistance heater as a backup for wood heat. The dwellings commonly share a washer and dryer and are supplied with water by a 3/4 hp, submersible well pump typically energized 10% of the time. The developer also has a small shop with a table saw, drill press, grinding wheel, and other small tools that are used an average of 3 hours a day.

# 3. POWER POTENTIAL

This section shows you how to determine the amount of power you need and how to calculate the amount of power that potentially can be produced from your site. The needed power is referred to as required capacity, and the calculated power is referred to as design capacity. The design capacity is a function of head and flow and gives a quick indication of whether enough power can be produced to meet the developer's needs.

Before proceeding, you should have determined whether you are a Category 1 or Category 2 developer (Subsection 1.3) and if your hydropower source is manmade or run-of-the-stream (Subsection 2.6).

## 3.1 Power Required

In Section 2.0, you have become generally familiar with how electrical power can be produced from available water resources. Your next step is to determine how much power is needed for all of the electrical loads, such as lights, appliances, heaters, motors, etc., to be served by your development. The quantity of power that can be produced from a resource is the system capacity, measured in kilowatts. The quantity of power needed for all of the electrical loads to be served by your development is the required capacity, also measured in kilowatts. The system capacity must be equal to or greater than the required capacity, or system load. This subsection will familiarize you with fundamental power requirements such as typical household loads, metering, and nameplate data.

### 3.1.1 Typical Household Loads

To determine household power load, individual items should be checked to determine their rated power demand. The power demand can be found on the nameplate generally attached to the applicance or item of equipment. Where nameplates cannot be found, the values given in Table 3-1 can be used to estimate the power needed.

TABLE 3-1. TYPICAL HOUSEHOLD APPLIANCE LOADS

| Appliance | Power (W) | Average Hours of Use/Month | Total Energy Consumption (kWh/month) |
|---|---|---|---|
| Air conditioner | 800 to 1600 | 150 | 120 to 240 |
| Blender | 600 | 6 | 2 |
| Car block heater | 850 | 300 | 300 |
| Clock | 2 | 720 | 1 |
| Clothes dryer | 4600 | 19 | 87 |
| Coffee maker | 600 to 900 | 12 | 7 to 11 |
| Electric blanket | 200 | 80 | 16 |
| Fan (kitchen) | 250 | 30 | 8 |
| Freezer (chest, 15 $ft^3$) | 350 | 240 | 84 |
| Furnace fan | 300 | 200 | 60 |
| Hair dryer (hand-held) | 1200 | 5 | 6 |
| Hi-fi (tube type) | 115 | 120 | 14 |
| Hi-fi (solid state) | 30 | 120 | 4 |
| Iron | 1100 | 12 | 13 |
| Bathroom exhaust fan | 70 | 30 | 2 |
| Light (60 watt) | 60 | 120 | 7 |
| Light (100 watt) | 100 | 90 | 9 |
| Light (fluorescent, 4-ft) | 48 | 240 | 12 |
| Mixer | 124 | 8 | 1 |
| Radio (tube type) | 80 | 120 | 10 |
| Range | 8800 | 10 | 100 |
| Refrigerator (standard 14 $ft^3$) | 300 | 200 | 60 |
| Refrigerator (frost free 14 $ft^3$) | 360 | 500 | 180 |
| Sewing machine | 100 | 10 | 1 |
| Toaster | 1150 | 4 | 5 |
| TV (black and white) | 255 | 120 | 31 |
| TV (color) | 350 | 120 | 42 |
| Washing machine | 700 | 12 | 8 |
| Water heater (40 gal) | 4500 | 87 | 392 |
| Vacuum cleaner | 750 | 10 | 8 |
| Electric heater | | winter use | |
| 1 kW | 1000 | 150 | 150 |
| 1.5 kW | 1500 | 150 | 225 |
| 2 kW | 2000 | 150 | 300 |

TABLE 3-1. (continued)

| Appliance | Power (W) | Average Hours of Use/Month | Total Energy Consumption (kWh/month) |
|---|---|---|---|
| Furnace-electric | | | |
| 10 kW | 10000 | 150 | 1500 |
| 15 kW | 15000 | 150 | 2250 |
| 20 kW | 20000 | 150 | 3000 |
| Shop Equipment | | | |
| Water Pump (1/2 hp) | 460 | 44 | 20 |
| Shop Drill (1/4 in. 1/6 hp) | 160 to 250 | 2 | 0.3 to 0.5 |
| Skill Saw (1 hp) | 1000 | 6 | 6 |
| Table Saw (1 hp) | 1000 | 4 | 4 |
| Lathe (1/2 hp) | 460 | 2 | 1 |

Electrical appliances are rated in watts (or kilowatts), and electrical motors are usually rated in horsepower. During the tabulation of the household power demand, if motors are listed, the horsepower rating must be converted to kilowatts. Theoretically, to convert horsepower to kilowatts, the horsepower rating is multiplied by 0.746 (1 hp = 0.746 kW). However, to allow for the inefficiencies of electric motors and for other factors, you should use a factor of 1 hp = 1 kW when estimating the power demand for any motors on the household load list. Also, the starting current of a motor is typically six times the operating current. In other words, a 1 hp motor may require 6 kW to get started. This causes a momentary peak demand that you must account for when determining your system load.

The household appliance requirements listed in Table 3-1 are typical. The watts listed are approximate, and the average use per month will vary with climate, home insulation, and the user's personal habits. The first column lists the power each appliance requires when being used. The next column estimates typical monthly use of the appliances. The third column lists the energy consumption for the month and is simply the product of the first two columns divided by 1000 to convert to kilowatts [(power in watts ÷ 1000) x hours per month = kilowatt-hours per month].

Figure 3-1 is a chart to aid in determining daily electrical load requirements. Determining these requirements is especially important for Catagory 1 developers, who are interested in sizing their projects to meet the maximum power demand. Extra copies of the chart are provided in Appendix I. To use the chart, first list all of your electrical appliances and equipment in the left hand column, and enter the rated wattage for each item in the next column Then, monitor the use of all the items listed for a 24-hour period. Each time an item is used, enter the use on the chart in 15-minute increments. At the end of each hour, sum each increment and write the largest sum of that hour in the totals system at the bottom of the chart. Estimate night loads for the period when you are normally asleep. Figure 3-2 is an example of a filled out chart.

| Appliances or equipment | Watts | Hourly load schedule (Noon) | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| | | | | | | | | | | | | | | | | | | | | | | | | | |
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| | | | | | | | | | | | | | | | | | | | | | | | | | |
| Total hourly load in kW (W ÷ 1000) | | | | | | | | | | | | | | | | | | | | | | | | | |

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Figure 3-1. Daily electrical load chart.

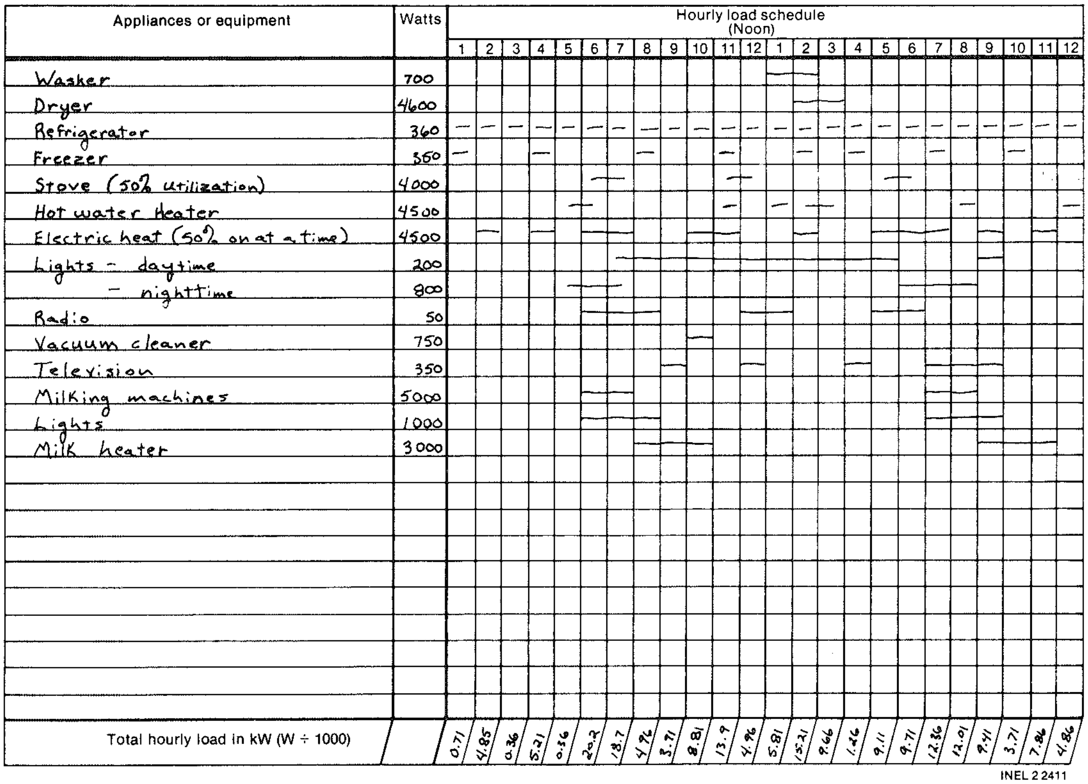


Figure 3-2. Sample of completed daily electrical load chart.

### 3.1.2 Metering

If a developer is connected to an electrical utility system, he can measure his daily, weekly, and monthly consumption by reading the electric meter. The electric utility measures the use of electrical power with meters. In general, each home will have one meter to measure all power consumption. Several types and styles of meters are used. One common type is shown in Figure 3-3. The figure shows two readings taken 30 days apart. The number of kilowatt-hours (kWh) used during the 30-day period is 1,660 (16,250 - 14,590, read from the small circular dials near the top of the meter). The average power used during the month is determined as follows:

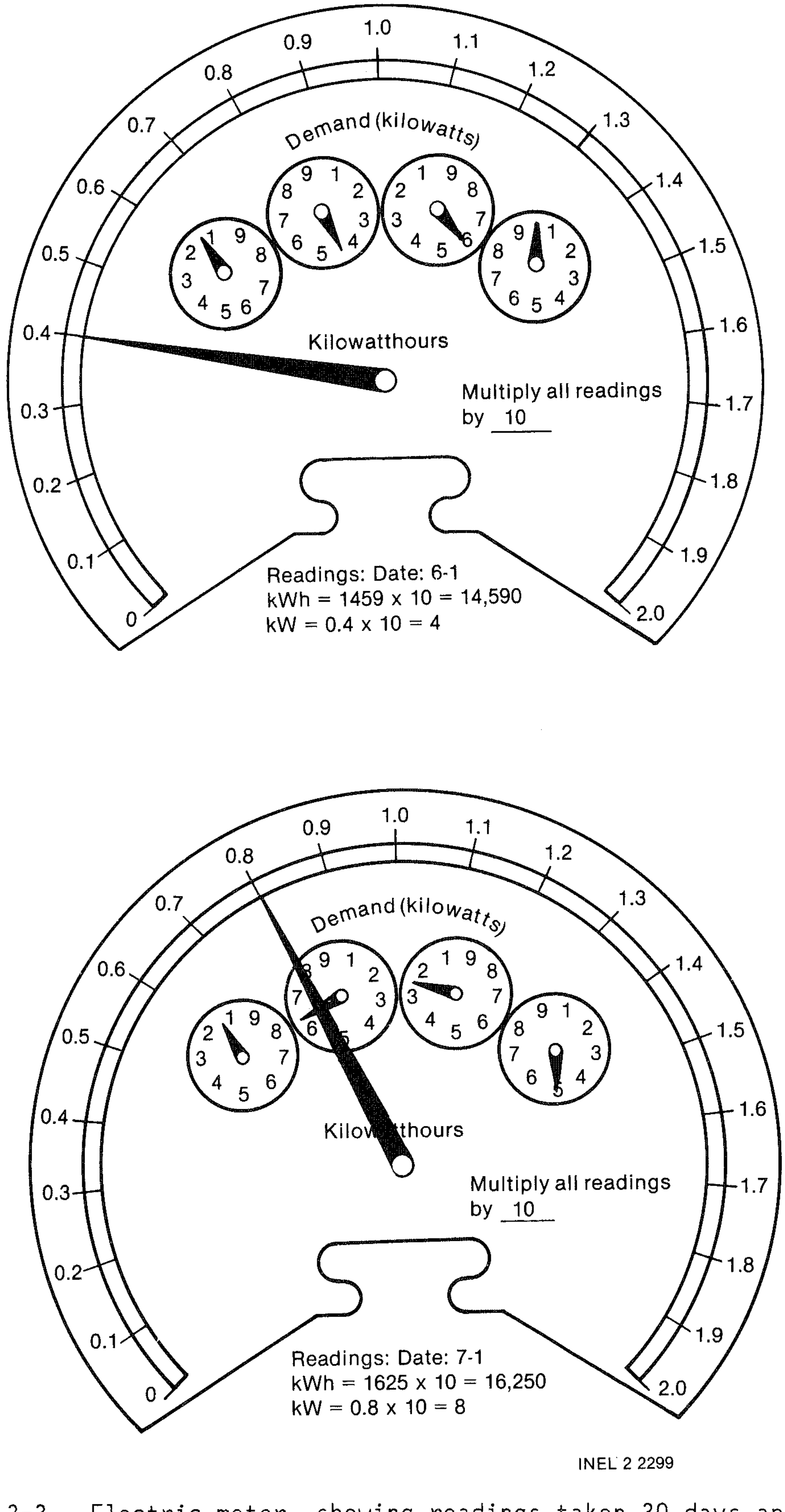


Figure 3-3. Electric meter, showing readings taken 30 days apart.

$$P_m = \frac{E_m}{24 \times D_A} \tag{3-1}$$

where

$P_m$ = average power for the month in kW

$E_m$ = total energy used for the month in kWh

24 = number hours in a day

$D_A$ = number of days in the month or measurement period.

From Figure 3-3:

$$P_m = \frac{1660}{24 \times 30}$$

$$P_m = 2.3 \text{ kW} \quad .$$

This is the average demand for the month. It does not represent the peak demand. The meter shown in Figure 3-3 also has a demand meter which can be read directly. For example, at the beginning of the month, the large pointer is at 0.4, indicating that the maximum demand was 4 kW (0.4 x 10). At the end of the month, the meter shows that at some time during the month the demand reached 8 kW. The meter will always read the maximum demand until it is reset by the utility.

The demand meter is used frequently by the utility as an important indicator. It measures kilowatts and indicates the maximum value of kilowatts required during a given time interval, usually 15 minutes. The utility usually reads and resets the demand meter for monthly billings.

If your electric meter does not measure demand, you can determine your maximum demand by reading the meter hourly. This method is not as accurate as using a demand meter, but it is good enough for estimating purposes. If electric motors are used, add the starting demand to the hourly reading.

Another method of determining maximum demand is to measure electrical use with a recording ammeter. This is a device that can plot amperage used versus time. If you use a recording ammeter, you should monitor each current-carrying conductor on equal time. This will allow an accurate measurement of the maximum current since each current-carrying conductor is not loaded equally. The power use in watts can then be determined from Equation (A6-5) for single-phase power or from Equation (A6-7) for three-phase power (see Appendix A-6).

$$P = E \times I \quad \text{(single-phase)} \tag{A6-5}$$

$$P = 1.73 \times E \times I \quad \text{(three-phase)} \tag{A6-7}$$

## 3.2 Inspection of Potential Hydropower Development

Next, you should conduct a site inspection. Although most will be familiar with site details, an inspection done with a few key points in mind may bring to light important issues previously overlooked. You should review the following outline and make notes on important issues. Also, before making the site inspection, review Subsection 2.6, Microhydropower Sources. Identify the type of source that most closely resembles the source for your site. Study the appropriate figure(s) and become familiar with the major components of your microhydropower system. Then, with these items in mind, conduct the inspection. After making the inspection, sketch the preliminary layout on a sheet of graph paper.

### 3.2.1 Manmade Sites

While the construction of a new dam for the sole purpose of developing a microhydropower site is not generally practical economically, there are literally thousands of existing dams, built for a variety of purposes, that may be attractive to the developer. The water level is often strictly controlled, so that a hydropower project at the site could use only the net inflow into the reservoir. These factors and their effect on your project should be determined early on. Some dams may be usable for only part of the year, which would also seriously affecting a project's economics.

The reservoir of many older dams will be partially filled with silt. Any impoundment with silting behind the dam will obviously have less reservoir capacity. The silt level may be a key factor in the dam's structural stability. Equilibrum of the dam, water, and silt may have been changing over the years. To remove the silt might upset the balance and cause a dam failure. Removing the silt also presents environmental problems because dredging will increase the silt load of the stream, and if the silt is removed and trucked away, disposal may present a further problem. Silt removal can be an expensive way to increase reserve capacity.

When reviewing the use of an existing dam, the following items should be considered:

- Dam structure

  - What is the state of repair?

  - How much work and material will be required to make the structure functional?

  - Does the dam have a spillway, and is it adequate?

  - How can the water be directed to the turbine?

  - Is the powerhouse part of the existing structure? If so, how much work is needed to repair it?

  - Can the height of the structure be increased easily? If so, what will be the effect upstream?

- Reservoir pool
  - What is the depth of the pool at the structure?
  - How much annual variation occurs in the pool? Will the variations change if you install a turbine that discharges water at a uniform rate?
  - How much debris is carried by the water, both on the surface and suspended in the water? Will the debris clog an intake trashrack easily?
- Construction features
  - Is the site easily reached for construction?
  - Will you have to divert the water? If so, what will be involved?
  - Are there any hazards near the construction site (overhanging power lines, etc.)?

### 3.2.2 Run-of-the-Stream Sites

The following items should be considered when evaluating a run-of-the-stream site.

- Identify one or two powerhouse locations (most powerhouses are located near the stream that supplies them with water).
  - How far will the power be transmitted?
  - Can you identify a high water mark?
  - Can a vehicle get to the site?

- Would there be any advantage to locating the powerhouse near the place where the power will be used?
  - How much shorter would the transmission distance be?
  - Could the penstock be shorter?
  - Could a tailrace be constructed easily to carry the water away from the powerhouse?
- Investigate potential penstock routing (the ideal penstock routing would be as short as possible, as straight as possible, and as steep as possible while still delivering the required flow and head).
  - Will the water rights, soil permeability, etc. allow the use of a power canal to shorten the penstock?
  - If the canal is a possibility, look uphill from the powerhouse location, identify the steepest slope to which a power canal can be run, and find several appropriate points on that slope for such a canal.
  - If a power canal cannot be built, locate the penstock intake near the stream.
  - Walk uphill from the powerhouse, identifying several areas that could be used for a forebay.
  - From these locations, what is the straightest or the shortest routing for the penstock?
- Walking along the proposed penstock routing, determine if there are logical areas where a forebay and penstock intake structure can be built.

- Can equipment be driven to the site (backhoe, cement mixer, or ready-mix truck, etc.)?

- How deep can the forebay be?

- If a power canal is used, can the canal be run level from the stream to the forebay?

- If the canal is more than 1/2 mile long, can a settling basin be built near the stream?

• Consider placement of diversion works.

  - Can the diversion works be set at right angles to the stream?

  - Can a backhoe be used in the area?

  - Who owns the property?

## 3.3 Determining Available Flow

Available flow is the flow that can be used in a microhydropower system to generate electricity. The flow available to generate electricity varies as the stream flow varies. In the spring, most streams are at their highest level, and more flow is available to generate electricity; in late summer, on the other hand, most streams are at their lowest level, and less flow is available to generate electricity. Category 1 developers, who require power at a constant level year round, should design their systems for the minimum available flow of late summer. They do not need the additional power that could be produced in the spring because of the larger flow available. Category 2 developers, however, who are interested in producing the most energy for the dollar invested, will use the larger flow available in the spring.

### 3.3.1 Flow Duration Curve

To determine available flow, engineers use statistical methods to project future stream activity from past stream flow records. The end product of such a hydrologic study is a flow duration curve. The curve is plotted on standard graph paper and shows the stream's average flow pattern. Available flow can be determined from this flow pattern. Figure 3-4 is an example of a flow duration curve. The scale on the left side of the graph measures flow in cfs, and the scale at the bottom of the graph measures the percentage of exceedance. For example, the value of flow shown on the curve above the 50% exceedance mark is 50 cfs, which means that, for this example, flow will equal or exceed 50 cfs 50% of the time during an average flow year. In curves of this type, the exceedance values at the bottom of the graph are always the same, while the flow scale on the left side of the graph is determined by the range of the flow pattern.

Since Category 1 developers are interested in the low-flow period of the year, the flatter portion near the bottom of the curve is of particular interest to them. Category 2 developers are normally interested in flows between 20 to 35% exceedance. The most economical design flow for Category 2 developers is usually in the range of 25% exceedance. The design flow is the available flow selected for use in sizing the microhydropower system.

Category 1 developers, who are interested only in the low-flow pattern of the stream, may be able to establish a value for available flow without developing a flow duration curve. To accomplish this, you should be completely familiar with the stream, especially with what it looks like during low-flow periods. Developers who feel sufficiently familiar with their streams can turn to Appendix A-2, "Estimating Minimum Stream Flow." Once again, this method is recommended only for those who have lived with a stream for a number of years and can accurately estimate the average annual low-flow mark on the stream bank.

The remaining Category 1 developers and all Category 2 developers should develop a flow duration curve. Developing such a curve may not be easy for the developer, and the first step may be to seek some assistance. One possibility is the U. S. Department of Agriculture, Soil Conservation Service (SCS). The assistance will vary from state to state depending on other priorities and personnel availability. You are encouraged to visit

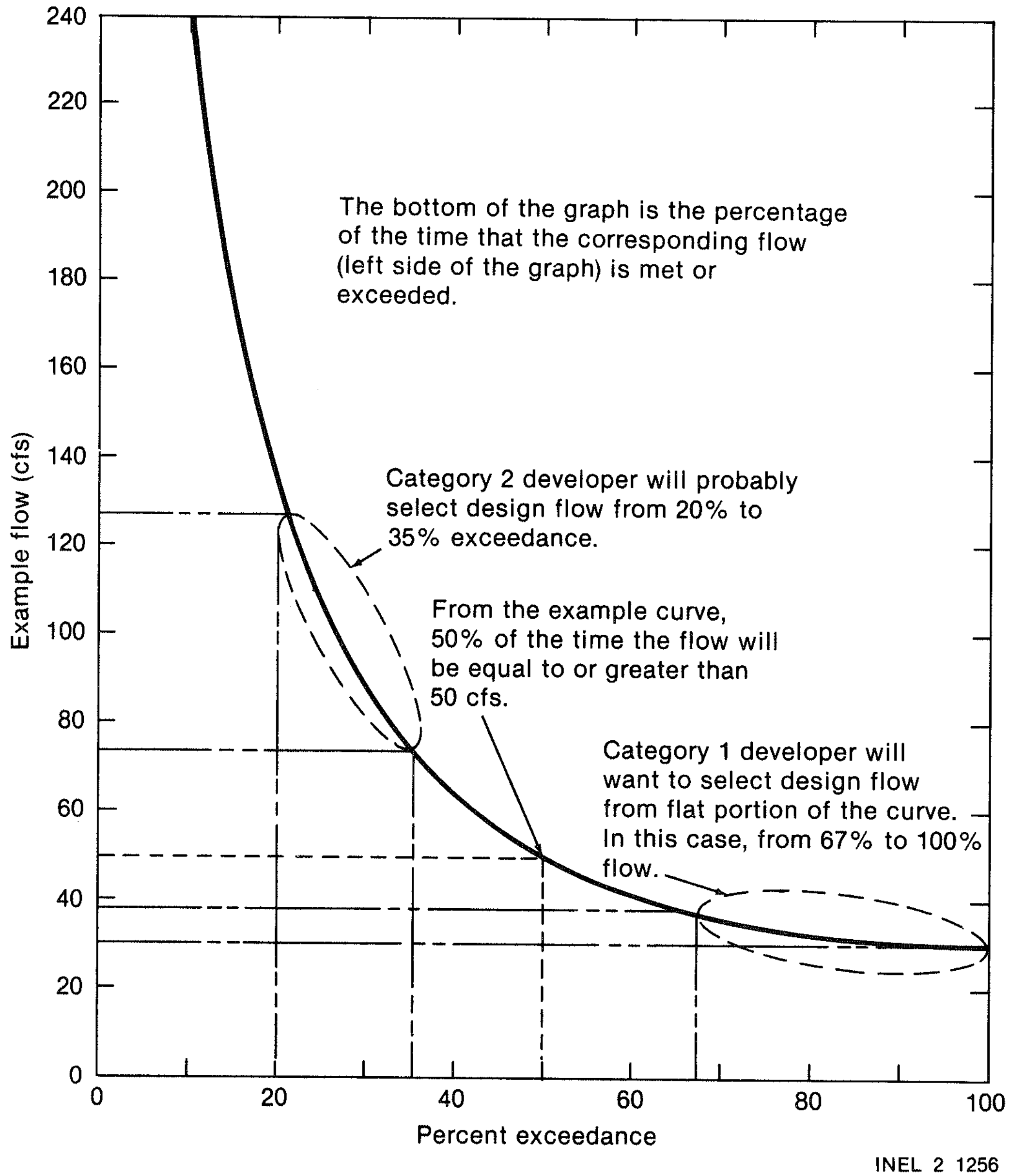


Figure 3-4. Flow duration curve.

the local county office of the SCS to determine what assistance might be available. They may perform the hydrologic study and even provide technical assistance with design and construction of intake or impoundment structures. Keep in mind which portion of the flow duration curve you are interested in. This will help the SCS determine how much assistance they might be able to offer.

### 3.3.2 Existing Stream Flow Records

If you are not able to obtain assistance and have to develop the flow duration curve yourself, you will have to collect additional information. First, you should gather information about the existing stream flow records in the area of the site. The initial step is to contact the U.S. Geological Survey (USGS). To locate the nearest USGS office, look in the white pages under U.S. Department of the Interior, Geological Survey, Water Resources Division, or write the appropriate regional office:

Northeastern Region

USGS
National Center, Mail Stop #433
12201 Sunrise Valley Dr.
Reston, VA 22092

Central Region

USGS
Field Center Location
Mail Stop #406, Box #25046
Denver Federal Center
Lakewood, CO 80225

Southeastern Region

USGS
Richard B. Russell Federal Bldg.
75 Spring St., S.W., Suite 772
Atlanta, GA 30303

Western Region

USGS
Field Center Location
345 Middlefield Road
Mail Stop #66
Menlo Park, CA 94025

Present the USGS with the following information:

- Section number, township, range, county, and state of the proposed site.
- The name of the stream on which the site will be located, and a reference to some easily identified landmark. Give the distance and direction from the landmark to the site.
- The name of any streams that feed into your stream above the proposed site.
- The name of the stream that your stream drains into, and the distance from the proposed site.

When you contact the USGS, request a "NAWDEX" for the county in which the site is located and the surrounding counties, and the "duration table from A9-69 program" for the gage that will most closely correlate to the flow at your site. Important: The gage must be active (currently operating) and have daily flow readings year round. There may be a minimum charge for the printouts. The NAWDEX, or National Water Data Exchange, is a listing of all federal and state and some private stream gages placed in the counties in question. Table 3-2 is a reduced copy of a NAWDEX printout for Hillsborough County, New Hampshire and for Stevens County, Washington. The NAWDEX printout lists the following:

- The agency that placed the gage
- The station number
- The station name and location
- When the gage started and stopped recording
- Whether the data is interrupted (rather than continuous)
  - y = yes
  - n = no
- The measurement (complete flow)
  - 1 = daily year round
  - 2 = daily seasonal
  - 3 = monthly year round
  - 4 = monthly seasonal
  - E = activity eliminated

TABLE 3-2. GAGING SITES IN HILLSBOROUGH CO, NH AND STEVENS CO, WA

| Organization Code | Organization Station Number | Station Name and Location | SW Begin Year | SW End Year | Interrupted Record | Complete Flow | SW Active Status |
|---|---|---|---|---|---|---|---|
| USCE | EM03 | Nubanusit Brook Below Edward MacDowell Dam | | | | | |
| USCE | E04 | Piscataquog River | | | | | |
| USCE | E05 | Piscataquog River Below Everett Dam | | | | | |
| USCE | | Edward MacDowell Dam NH | 1950 | | | 1 | Y |
| USCE | | Contoocook R AB Hopkinton LK HL3 | | | | | |
| USEPA | PMN004 | RGN 1 | | | | | |
| USEPA | 1-CNT | Connecticut River | | | | | |
| USEPA | 1-MER | Merrimack River | | | | | |
| USEPA | 3-6-NSH | Nashua, River | | | | | |
| USEPA | 330201 | Powder Mill Pond | | | | | |
| USEPA | 330202 | Powder Mill Pond | | | | | |
| USEPA | 330203 | Powder Mill Pond | | | | | |
| USEPA | 330501 | Glenn Lake | | | | | |
| USEPA | 330502 | Glenn Lake | | | | | |
| USEPA | 330601 | Kelly Falls Pong | | | | | |
| USEPA | 8-MER | Merrimack P. at Bedford, H. H. | | | | | |
| USGS | 01081900 | Town Line Brook Tributary near Peterborough, | 1971 | 1979 | | | N |
| USGS | 01082000 | Contoocook River at Peteborough, NH | 1938 | 1979 | Y | E | N |
| USGS | 01082500 | Edward MacDowell Reservoir at W. Peterborough | 1950 | | | | Y |
| USGS | 01083000 | Nubanusit Brook Near Peterborough, NH | 1920 | | Y | 3 | Y |
| USGS | 01083500 | Contoocook River Near Elmwood, NH | 1917 | 1924 | | E | N |
| USGS | 01084000 | North Branch Contoocook River Near Anthim, N | 1924 | 1970 | | E | N |
| USGS | 01084500 | Beards Brook Near Hillsboro, NH | 1945 | 1976 | N | E | N |
| USGS | 01090480 | Rays Brook at Manchester, NH | 1972 | 1979 | Y | | N |

TABLE 3-2. (Continued)

| Organization Code | Organization Station Number | Station Name and Location | SW Begin Year | SW End Year | Interrupted Record | Complete Flow | SW Active Status |
|---|---|---|---|---|---|---|---|
| USGS | 01090500 | Merrimack River at Manchester, NH | 1924 | 1950 | | E | N |
| USGS | 01090700 | Everett Lake Near East Weare, NH | 1962 | | | | Y |
| USGS | 01090800 | Piscataquog River BL Everett Dam, NR E Weare | 1963 | | N | 1 | Y |
| USGS | 01091000 | S Branch Piscataquog River Near Gufftown, N | 1940 | 1940 | N | E | N |
| USGS | 01091500 | Piscataquog River Near Guffstown, NH | 1936 | 1978 | Y | E | N |
| USGS | 01091950 | Bowman Brook Tributary Near Bedfore, NH | 1967 | 1969 | | | N |
| USGS | 01092000 | Merrimack R Nr Goffs Falls, Below Manchester | 1936 | | N | 1 | Y |
| USGS | 01093500 | Messabesic Lake Near Manchester, NH | 1941 | | | | Y |
| USGS | 01093610 | Merrimack River Tributary Near Merrimack, NH | 1967 | 1967 | | | N |
| USGS | 01093800 | Stony Brook Tributary Near Temple, NH | 1963 | | N | 1 | Y |
| USGS | 01093900 | Tucker Brook Near Wilton, NH | 1964 | 1973 | Y | | N |
| USGS | 01093910 | Tucker Brook Near Milford, NH | 1964 | 1972 | Y | | N |
| USGS | 01094000 | Souhegan River at Merrimack, NH | 1909 | 1909 | Y | E | N |
| USGS | 01094006 | McQuade Brook Near Bedford, NH | 1911 | 1979 | | | N |
| USGS | 01094008 | Babousic Brook at Merrimack, NH | 1910 | 1910 | | | N |
| USGS | 01094010 | Maticook Brook Near South Merrimack, NH | 1964 | 1972 | Y | | N |
| USGS | 01094020 | Maticook Brook Near Merrimack, NH | 1964 | 1973 | Y | | N |
| USGS | 01094040 | Chase Brook Near Hudson Center, NH | 1964 | 1972 | Y | | N |
| USGS | 01094050 | Chase Brook Near Litchfield, NH | 1964 | 1972 | Y | | N |
| USGS | 01094160 | Pennichuck Brook Near Nashua, NH | 1967 | 1969 | | | N |
| USGS | 01096502 | Nissitissit Brook Near Hollis, NH | 1971 | 1973 | | | N |
| USGS | 01096506 | Nashua River Near Hollis, NH | 1973 | 1963 | | | N |
| USGS | 01096507 | Nashua River at Nashua, NH | 1978 | | | | Y |
| USGS | 01096508 | Merrimack River at Nashua, NH | 1974 | | | | Y |
| USGS | 01096510 | Merrimack River Tributary at Hudson Center, | 1934 | 1972 | Y | | N |
| USGS | 01096520 | Old Maids Brrok Near Nashua, NH | 1964 | 1972 | Y | | N |
| USGS | 01096530 | Musquash Brook Tributary Near Hudson, NH | 1964 | 1972 | Y | | N |
| USNWS | 2-5702-N | Nashua NH On Merrimack R | 1939 | | Y | | Y |
| USNWS | 44386000NEEDED | MacDowell Dam NH | 1979 | | N | | Y |
| USNWS | 44389000NEEDED | Amoskeag Dam NH | 1978 | | N | | Y |
| USNWS | 44390000NEEDED | Baboosic BK at Merrimack NH | 1981 | | N | | Y |
| USNWS | 44391000NEEDED | Merrimack R at Nashua NH | 1979 | | N | | Y |
| USBPA | 12409000 | Colville R at Kettle Falls WA | 1970 | | | | Y |
| USEPA | 540112 | WPSS Northport Washington | | | | | |
| USEPA | 543182 | Columnbia R, at Northport WA | | | | | |
| USFS | 21001212 | East Fork Cedar Creek | | | | | |
| USFS | 21014206 | Pierre Creek | | | | | |
| USFS | 21014304 | Pierre Lake | | | | | |
| USFS | 21016201 | Cottonwood Creek | | | | | |
| USFS | 21016204 | North Fork Chewelah Creek | | | | | |
| USFS | 21017106 | Lake Gillette Swim Area | | | | | |
| USFS | 621001111 | Silver Creek | | | | | |
| USFS | 621001209 | Meadow Creek | | | | | |
| USFS | 621001210 | Smackout Creek | | | | | |
| USFS | 621001212 | East Fork Cedar Creek | | | | | |

TABLE 3-2. (Continued)

| Organization Code | Organization Station Number | Station Name and Location | SW Begin Year | SW End Year | Interrupted Record | Complete Flow | SW Active Status |
|---|---|---|---|---|---|---|---|
| USFS | 621016105 | Addy Creek | | | | | |
| USFS | 621016201 | Cottomwood Creek | | | | | |
| USFS | 621016202 | Sixmile Creek | | | | | |
| USFS | 621016203 | South Fork Chewelah Creek | | | | | |
| USFS | 621016204 | North Fork Chewelah Creek | | | | | |
| USFS | 621017102 | Lake Thomas Campground | | | | | |
| USFS | 621017103 | Gillette Recreation Area | | | | | |
| USFS | 621017104 | Lake Thomas | | | | | |
| USFS | 621017106 | Lake Gillette | | | | | |
| USFS | 621017206 | Deer Creek | | | | | |
| USFS | 621017207 | South Fork Mill Creek | | | | | |
| USFS | 621017208 | Middle Fork Mill Creek | | | | | |
| USFS | 621017213 | North Mill Creek | | | | | |
| USGS | 12399500 | Columbia River at International Boundary | 1893 | | Y | 1 | Y |
| USGS | 12399510 | Columbia R Auxil at Interna Bndry, Wash. | 1942 | | | | Y |
| USGS | 12399600 | Deep Creek Near Northport, Wash. | 1972 | 1976 | N | 4 | N |
| USGS | 12399000PLANNED | FY76 Change Operation OWDC32842 To | 1976 | | | 1 | Y |
| USGS | 12399800 | Deep C NR Northport WA | 1972 | | | 1 | Y |
| USGS | 12400000 | Sheep Creek NR Velvet Wash | 1929 | 1932 | Y | 2 | N |
| USGS | 12400500 | Sheep Creek Near Northport, Wash. | 1929 | 1948 | Y | 2 | N |
| USGS | 12400520 | Columbia River at Northport, Wash. | | | | | |
| USGS | 12404860 | Pierre Lake Near Orient | | | | | |
| USGS | 12406000 | Deer Lake Near Loon Lake, Wash. | 1952 | 1978 | N | | 1 |
| USGS | 12406500 | Look LK NR Looon Lake Wash | 1950 | | N | | Y |
| USGS | 12407000 | Sheep Cr at Loon Lake Wash | 1950 | 1959 | Y | 2 | N |
| USGS | 12407500 | Sheep Creek at Springdale, Wash. | 1952 | 1972 | N | 2 | N |
| USGS | 12407520 | Deer Creek Near Valley, Wash. | 1959 | | N | 2 | Y |
| USGS | 12407530 | Jumpoff Joe Lake Near VAlley, Wash. | 1961 | 1975 | Y | | N |
| USGS | 12407550 | Waitis Lake Near Valley, Wash. | 1961 | 1975 | Y | | N |
| USGS | 12407600 | Thmason Creek Near Chewelah, Wash. | 1953 | 1973 | | | N |
| USGS | 12407680 | Colville R at Chewelah, Wash | | | | | |
| USGS | 12407680PLANNED | Colville R at Chewelah, Wah | | | | | |
| USGS | 12407700 | Chewelah Creek at Chewelah, Wash. | 1956 | 1974 | N | 2 | N |
| USGS | 12408000 | Colville River at Blue Creek, Wash. | 1921 | | Y | 1 | Y |
| USGS | 12408195 | Leo Lake Near Tiger | | | | | |
| USGS | 12408200 | Patchen (Bighorn) C Nr Tiger, Wash. | 1953 | 1973 | | | N |
| USGS | 12408205 | Heritage Lake Near Tiger | | | | | |
| USGS | 12408210 | Thomas Lake Near Tiger | 1961 | 1966 | | | N |
| USGS | 12408214 | Gillette Lake Near Tiger | | | | | |
| USGS | 12408216 | Sherry Lake Near Tiger | | | | | |
| USGS | 12408300 | Little Pend Oreille River Near Colville, was | 1946 | 1976 | N | 4 | N |
| USGS | 12408300PLANNED | FY76 Change Operation OWDC01562 to | 1976 | | | 1 | Y |
| USGS | 12408400 | Narcisse Creek Near Colville, Wash. | 1953 | 1973 | | | N |
| USGS | 12408410 | Little Pend Oreille R at Arden, Wash | | | | | |
| USGS | 12408420 | Haller C Nr Arden Wash | 1959 | | Y | 2 | Y |

TABLE 3-2. (Continued)

| Organization Code | Organization Station Number | Station Name and Location | SW Begin Year | SW End Year | Interrupted Record | Complete Flow | SW Active Status |
|---|---|---|---|---|---|---|---|
| USGS | 12408440 | White Mud LK Nr Colville WA | 1961 | 1966 | | | N |
| USGS | 12408500 | Mill Creek Near Colville, Wash. | 1939 | | N | 1 | Y |
| USGS | 12408700 | Mill Cr at Mouth NR Colville Wash | 1959 | 1965 | N | 2 | N |
| USGS | 12409000 | Colville River at Kettle Falls, Wash. | 1921 | | N | 1 | Y |
| USGS | 12410600 | South Fork Harvey Creek NR Cedonia, Wash. | 1953 | 1973 | | | N |
| USGS | 12410650 | North Fork Harvey Creek NR Cedonia, Wash. | 1953 | 1973 | | | N |
| USGS | 12429800 | Mud Creek Near Deet Park, Wash. | 1953 | 1973 | | | N |
| USGS | 12433100 | Chamokane Creek Near Springdale, Wash. | 1973 | 1978 | Y | E | N |
| USGS | 12433200 | Chamokane CR Below Falls Near Long Lake, was | 1970 | 1968 | N | 1 | N |
| USGS | 41058000NEEDED | Grouse C WA | 1979 | | N | 1 | Y |
| USGS | 41059000NEEDED | Narcisse Creek WA | 1980 | | N | 1 | Y |
| USGS | 41060000NEEDED | Magee C NR Daisy WA | 1980 | | N | 1 | Y |
| USGS | 41061000NEEDED | Hunters C NR Hunter WA | 1980 | | N | 1 | Y |
| USGS | 470121117062501 | | | | | | |
| WA001 | 54A070 | Spokane River at Long Lake | | | | | |
| WA001 | 54A089 | Spokane R 2 M Below Ninemile Dam | | | | | |
| WA001 | 54A120 | Spokane R at Riverside State PK | | | | | |
| WA001 | 54A130 | Spokane R at Fort Wright Bridge | | | | | |
| WA001 | 59A070 | Colville River at Kettle Falls | | | | | |
| WA001 | 59A110 | Colville River at Blue Creek | | | | | |
| WA001 | 59A130 | Colville River at Chewelah | | | | | |
| WA001 | 60A070 | Kettle River Near Barstow | | | | | |
| WA001 | 61A070 | Columbia R at Northport | | | | | |
| WA013 | | Little Falls Power Station | 1910 | | | 1 | Y |

- The current status of the gage
  - y = active
  - n = not active.

Remember that the gage selected must be active (Status Category y) and must have a daily year round flow record (Measurement Category 1).

When you receive the NAWDEX, look for the gage closest to your site on your stream that has a daily reading and is currently operating. If the gage belongs to an agency other than USGS, contact that agency, reference the gage number, and inquire if they have "developed a flow duration curve or calculated exceedance values from their data." If they have, request a copy of the information, and have it available for future reference in Subsections 3.3.3.3 and 3.5.

The second item requested from the USGS is an A9-69 printout. The A9-69 program is a statistical analysis of the daily flow data. Your request must specifically ask for the "Duration Analysis." None of the other information available from A9-69 is needed for the handbook method of flow projections. Table 3-3 is an example of a duration table. The item of interest is the last column in the table, "Value Exceeded 'P' Percent of Time." These values will be used in Subsection 3.3.3.3 to develop a flow duration curve from the exceedance values.

### 3.3.3 Stream Flow Correlation

The next step is to correlate stream flow at the selected site to the flow reading at an existing gage. Two methods of doing this are used. The first, called "Flow Measurement Correlation," involves measuring flow at the proposed site, correlating the measured flow to flow data from a nearby gage, creating a flow duration curve from the gage data, and then modifying the flow scale of the curve to adjust to the flow measured at the proposed site. This method is discussed in this subsection.

The second method, called "Rainfall Runoff Correlation," which should require less time, involves using a map that shows geographic points where equal amounts of precipitation occur (called an isohyetgraph), determining the drainage basin area and the runnoff coefficient, creating a flow duration curve from the known gage data, and correlating this to the calculated average runoff or stream flow at the site. This method is discussed in Appendix A-3.

3.3.3.1 Flow Measurement. Most developers do not have flow data for their site. In fact, many have only a rough estimate of the present flow. Therefore, since the flow at the proposed site must be measured for the flow measurement correlation, the next step is to make this measurement. The measured flow can then be compared with the nearest gaging station that has exceedance data available and from which current daily flow readings can be received.

Flow should be measured accurately as close as possible to the proposed intake structure location. For existing dams, the flow can be measured just downstream or over the spillway. Collect four or more days of record, representing different flows. To get different flows, the days should not be consecutive. Disregard days with similar flows and unusually high-flow days that are a direct result of a local heavy storm in the drainage area of the site or of the gage with which the site is being compared. For each day in which measurements are taken, make a minimum of six measurements during the 24-hour period to ensure that average flow for the day is found. Calculate the average as follows:

$$Q_{avg} = \frac{Q_1 + Q_2 + Q_3 + Q_4 + Q_5 + Q_6}{6} \qquad (3\text{-}2)$$

where

$Q_{avg}$ = average of measurements made

$Q_1$ to $Q_6$ = individual flow measurements

6 = number of flow measurements.

TABLE 3-3. STATION NUMBER 12408500--DURATION TABLE OF DAILY VALUES FOR YEAR ENDING SEPTEMBER 30

Discharge-(CFS)
Mean
Mill Creek Near Colville, Wash.

| Class | Value | Total | Accum | Perct | Class | Value | Total | Accum | Perct | Class | Value | Total | Accum | Perct |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0.0 | 0 | 12784 | 100.0 | 12 | 21.0 | 630 | 5842 | 45.7 | 24 | 130.0 | 266 | 1239 | 9.6 |
| 1 | 4.0 | 9 | 12784 | 100.0 | 13 | 24.0 | 552 | 5212 | 40.8 | 25 | 150.0 | 175 | 973 | 7.6 |
| 2 | 4.7 | 43 | 12775 | 99.9 | 14 | 28.0 | 435 | 4600 | 36.5 | 26 | 170.0 | 231 | 798 | 6.2 |
| 3 | 5.4 | 105 | 12732 | 99.6 | 15 | 33.0 | 367 | 4225 | 33.0 | 27 | 200.0 | 163 | 567 | 4.4 |
| 4 | 6.3 | 265 | 12627 | 98.8 | 16 | 38.0 | 421 | 3858 | 30.2 | 28 | 230.0 | 141 | 404 | 3.1 |
| 5 | 7.3 | 345 | 12362 | 96.7 | 17 | 45.0 | 348 | 3497 | 26.9 | 29 | 270.0 | 109 | 263 | 2.0 |
| 6 | 8.5 | 692 | 12017 | 94.0 | 18 | 52.0 | 320 | 3089 | 24.2 | 30 | 320.0 | 76 | 154 | 1.2 |
| 7 | 9.9 | 421 | 11325 | 88.6 | 19 | 60.0 | 355 | 2769 | 21.7 | 31 | 370.0 | 86 | 78 | .6 |
| 8 | 11.0 | 1349 | 10904 | 85.3 | 20 | 70.0 | 335 | 2414 | 18.9 | 32 | 430.0 | 21 | 32 | .2 |
| 9 | 13.0 | 1938 | 955 | 74.7 | 21 | 81.0 | 273 | 2079 | 16.3 | 33 | 500.0 | 10 | 11 | |
| 10 | 16.0 | 830 | 7617 | 59.6 | 22 | 95.0 | 244 | 1806 | 14.1 | 34 | 580.0 | 1 | 1 | |
| 11 | 18.0 | 945 | 6787 | 53.1 | 23 | 110.0 | 323 | 1562 | 12.2 | | | | | |

VALUE EXCEEDED 'P' PERCENT OF TIME

V95 = 8.1
V90 = 9.5
V75 = 13.0
V70 = 14.0
V50 = 19.0
V25 = 50.0
V10 = 130.0

Three methods of flow measurement are presented below. Select the one that best fits the site where flow is to be measured. Each method gathers raw data that must be converted into cfs. A suggested form for recording daily cfs flow values is shown in Figure 3-5. Extra copies of the form are provided in Appendix I. To ensure a good correlation between the measured flow and a local gage reading, make a second series of flow measurements a month or two after the first series.

3.3.3.1.1 <u>Container Method</u>--The container method is suitable when the total flow can be diverted into a container of known size. This method can be used at small springs or streams, at dams that discharge through a pipe, and at industrial or domestic water discharges. The container must be big enough to measure all the flow even during high-flow periods.

FLOW MEASUREMENT TABLE

| Date | Time | Reading (time or depth) | Conversion factor | Computed flow, $Q_1$ (cfs) | Gage flow, $Q_2$ (cfs) | Correlation, $\frac{Q_1}{Q_2}$ |
|---|---|---|---|---|---|---|
| | | | | | | |
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Figure 3-5. Flow measurement table.

The method is simple. Discharge all the water through a pipe or similar device into a known volume, and record the time required to fill the container (Figure 3-6). From the volume and time, flow can be converted to cfs.

EXAMPLE: Assume that the 55-gallon drum shown in Figure 3-6 is filled in 1 minute and 14 seconds. Calculate the flow.

From conversion tables:

7.481 gallons (gal) = 1 cubic foot ($ft^3$)

1 minute (min) = 60 seconds (sec)

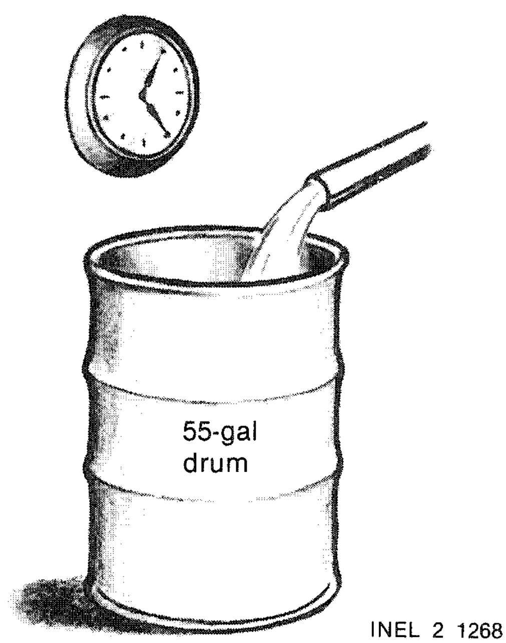


Figure 3-6. Measuring flow by the container method.

Volume in $ft^3$:

$$55 \text{ gal} \times \frac{1 \text{ ft}^3}{7.481 \text{ gal}} = 7.35 \text{ ft}^3 \text{ in the barrel} .$$

Time in seconds:

$$1 \text{ min} \times \frac{60 \text{ sec}}{\text{min}} + 14 \text{ sec} = 74 \text{ sec} .$$

From Equation (2-3):

$$\text{Flow} = \frac{\text{Volume}}{\text{Time}}$$

$$Q = \frac{V}{t} = \frac{7.35 \text{ ft}^3}{74 \text{ sec}} = 0.10 \text{ cfs} .$$

(NOTE: This flow would require a very large head to produce any significant power.)

Record this flow on the daily flow table (Figure 3-5).

3.3.3.1.2 Weir Method--A weir is a rectangular notch in a dam or similar structure forming a spillway that functions as a water meter. Once installed, a simple depth measurement can be accurately converted into flow. The method is practical for smaller streams where a temporary dam can be constructed. It can also be used on smaller existing dams where the water is continuously discharged over the spillway. The weir can be an integral part of a temporary dam constructed of tongue-and-groove lumber, or it can be a removable gate type made of metal plate. The downstream face of the weir must be beveled at least 45 degrees, and the bottom must be level when installed. The sides should also be beveled and should be at right angles (90 degrees) to the bottom. If the weir is constructed of wood, leave a 1/8-inch lip on the upstream face to prevent the wood from chipping (Figure 3-7).

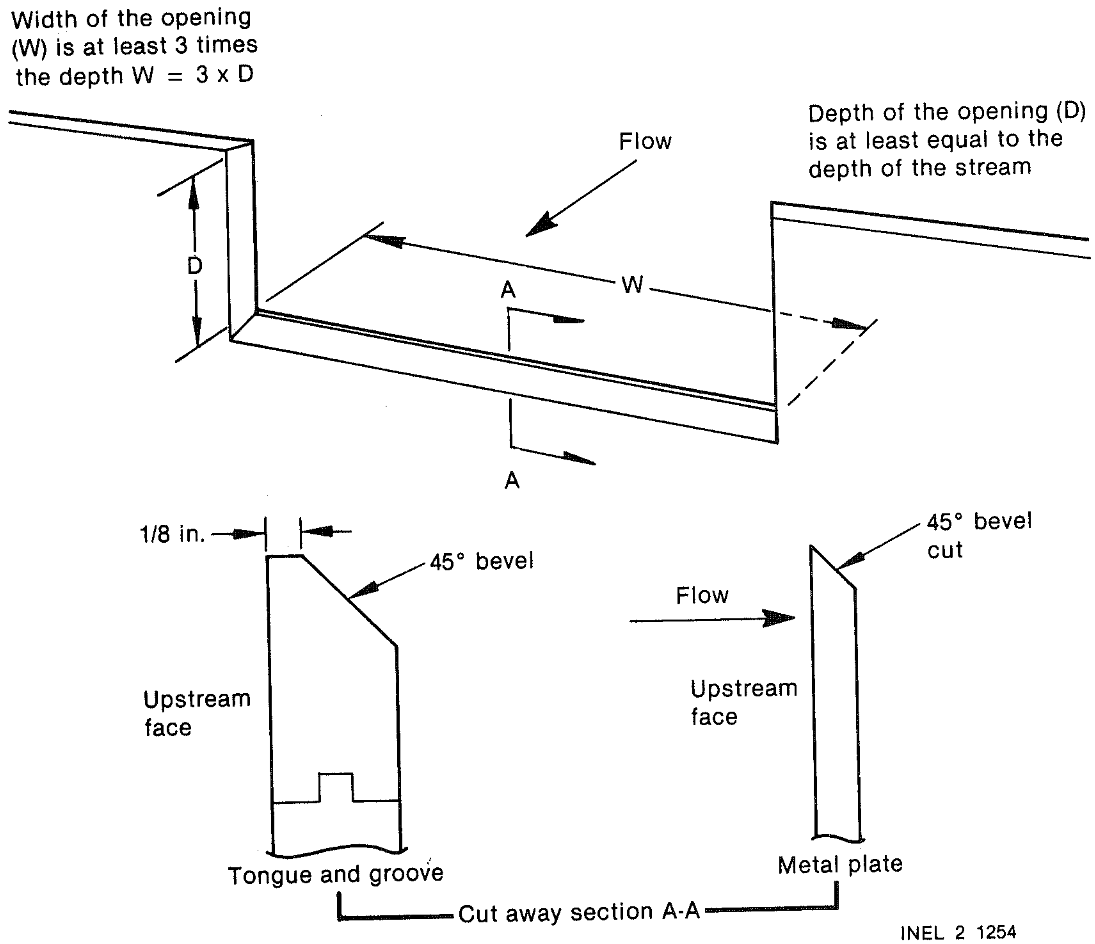


Figure 3-7. Weir details.

The type of temporary dam depends on the size of the stream and the required height of the dam. Suggestions for two dams are offered below: tongue-and-groove lumber for very small streams (Figure 3-8), and log crib for small streams (Figure 3-9).

Before building a dam, determine the size of the weir. To do this, select a convenient location along the stream to construct the dam. At that location, measure the deepest point in the natural stream bed. If the

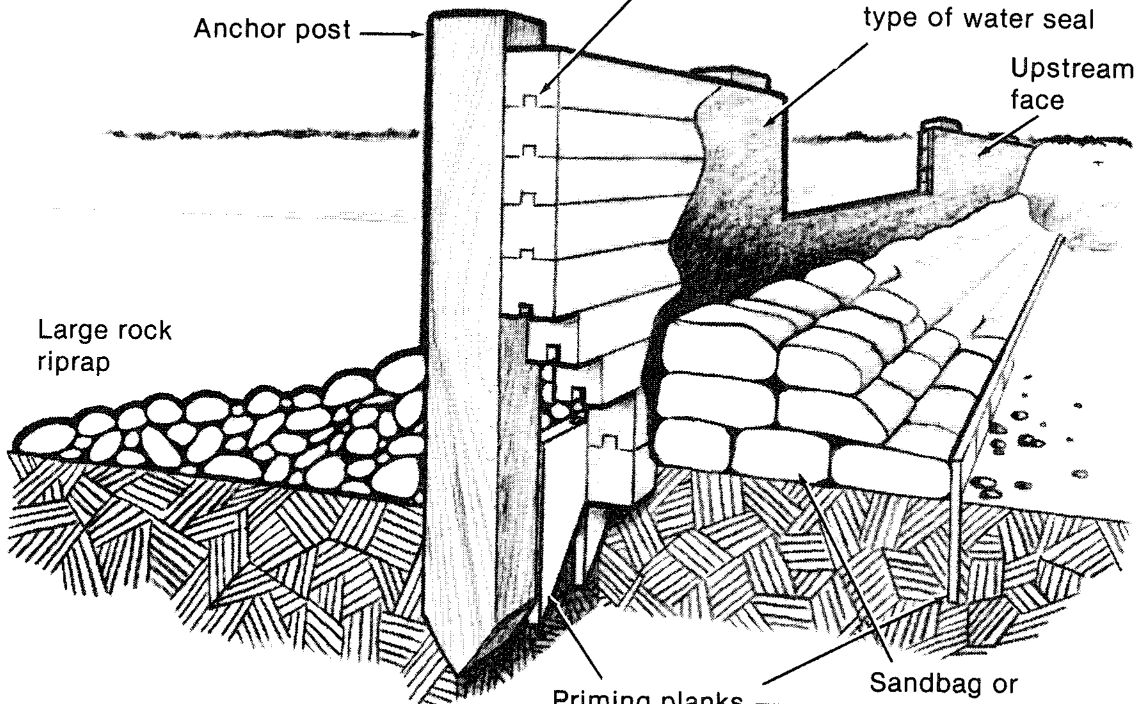


Figure 3-8. Tongue-and-groove lumber dam, with weir.

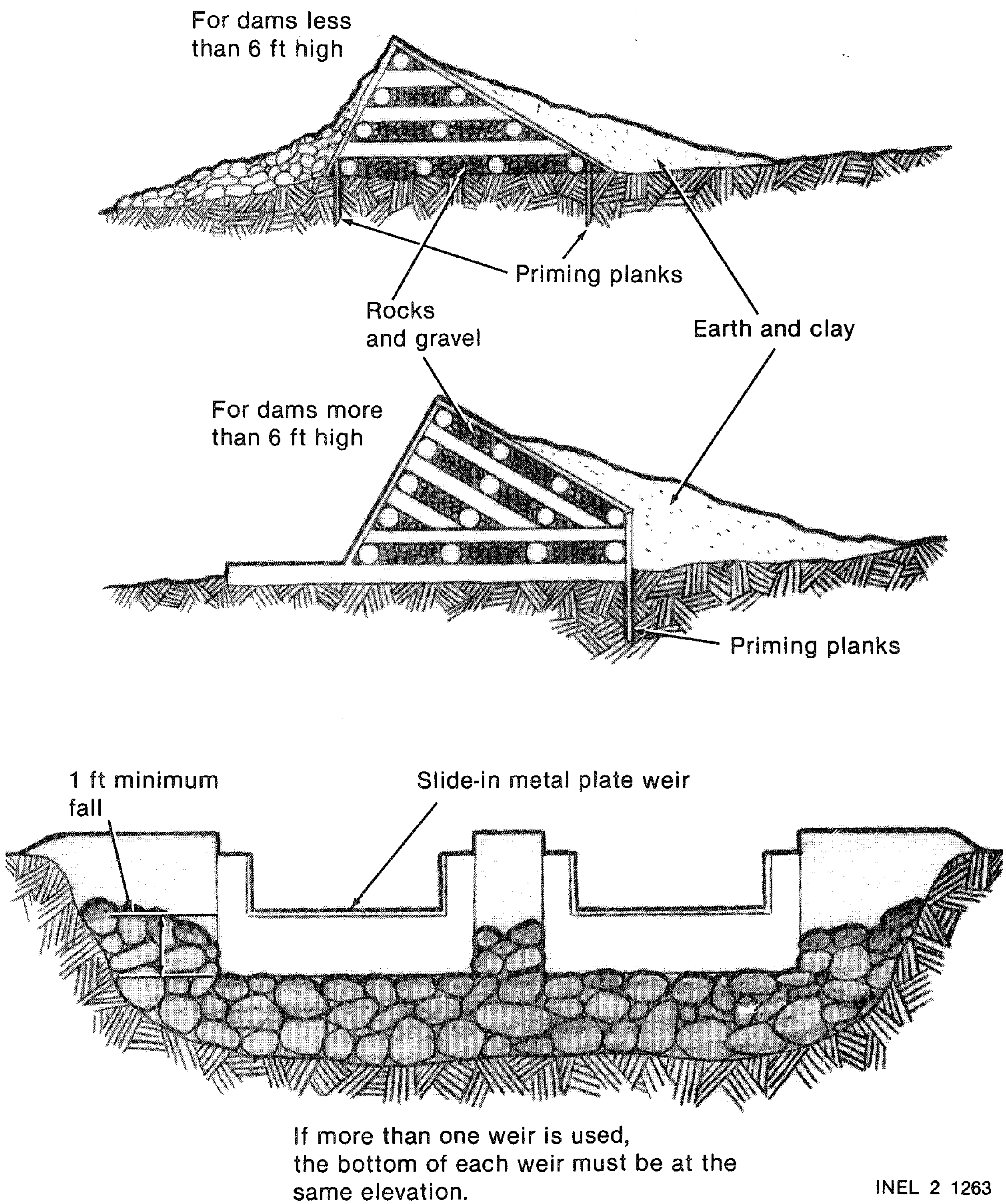


Figure 3-9. Log crib dams.

stream is unusually low at the time of measurement, estimate the additional depth for average flow. The depth of the weir notch should be at least equal to the depth of the stream. The width of the notch should be at least three times the depth (Figure 3-7).

The tongue-and-groove dam and weir is constructed of lumber spiked together. If possible, divert the stream around construction areas. Dig a trench across the stream perpendicular to the flow. This trench must be smooth enough so that the bottom piece of lumber can be leveled. Clay or earth can be used for leveling. Drive the downstream priming planks (see Figure 3-8) 2 to 3 feet deep into the stream bed to limit seepage under the dam. Priming planks are wooden boards, preferably tongue-and-groove, with one end cut to a point on one edge (Figure 3-10). They are driven into the soil so that the long pointed side is placed next to the previously driven plank. Then as each successive plank is driven, it is forced snug against the preceding board.[a] If the weir is a temporary installation, both upstream and downstream priming planks can be omitted.

Drive the timber anchor post into the stream bed until solid resistance prevents further driving. Shim between the post and the tongue-and-groove lumber while building the dam to maintain a vertical plumb on the dam.

After the lumber is in place and the weir notch is smooth, drive the upstream priming planks and waterproof the upstream face of the dam. Next, place sandbags or earth fill against the front face. Avoid placing the fill too close to the weir opening. Water turbulence upstream from the weir face will affect the measurement accuracy. Finally, at least 5 feet upstream from the weir, drive a post into the stream bed so that the top of the post is level with the bottom face of the weir. Use a carpenter's level to assure that the top of the post and the bottom face of the weir are level. NOTE: The post should be located so that it can be easily reached from the bank (Figure 3-11).

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a. Robin Saunders, Harnessing the Power of Water, Energy Primer, Portola Institute.

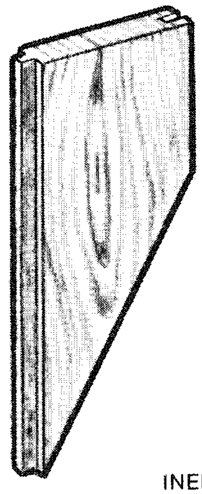

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Figure 3-10. Priming plank.

For crib dams, use several logs stacked together like a corncrib--hence the term "crib dam." A crib dam consists of green logs or heavier timbers stacked perpendicular to each other, spaced about 2 or 3 feet apart. Spike these together where they cross, and fill the spaces in between with rocks and gravel. Cover the upstream side, especially the base, with earth or clay to seal the edges. The priming planks should be driven 2 to 3 feet deep into the soil.

Protect the downstream face of the dam from erosion or undercutting wherever water will spill over. This is most important during times of heavy flow! The spillways can be made of concrete, lumber, or simply a pile of rocks large enough to withstand the continual flow. Crib dams can be built with the lower cross-timbers extended out to form a series of small water cascades downstream. Each cross-timber step should be at least as wide as it is tall.[a] Finally, as with the tongue-and-groove dam, drive a post into the stream bed at least 5 feet above the weir, and make the top of the post level with the bottom of the weir (Figure 3-11).

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a. Robin Saunders, Harnessing the Power of Water, Energy Primer, Portola Institute.

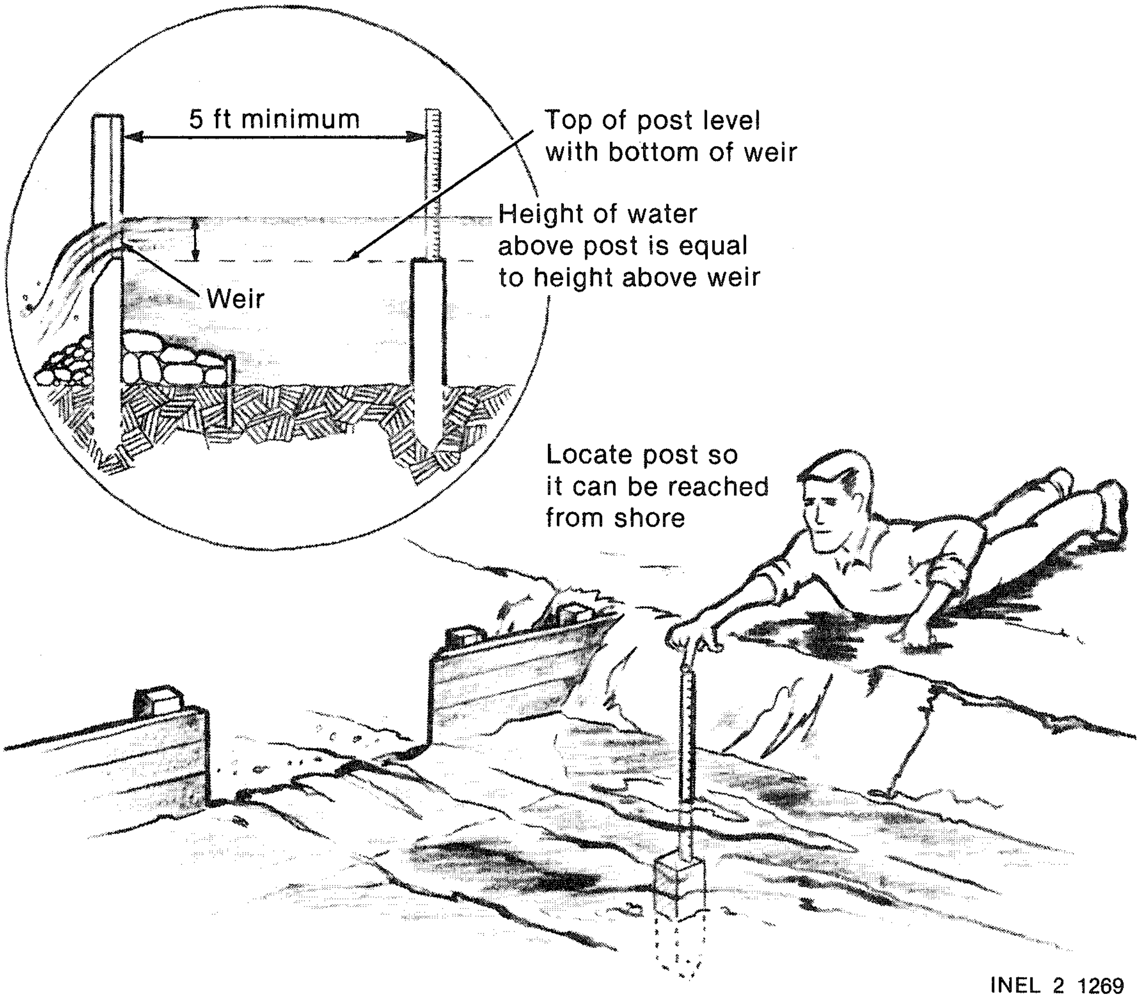



Figure 3-11. Relationship of weir and measuring post.

To measure the water depth above the weir, simply place a yard stick on the post and read the depth to the nearest 1/8 inch. The flow for measured depth is read directly from Table 3-4. This table lists flow for each inch of weir width. To convert to total flow, multiply by the width of the weir(s) in inches. Enter the resulting value in your flow table (Figure 3-5).

EXAMPLE: Assume that the weir is 3 feet wide and that the depth is measured at 4-3/8 inches.

TABLE 3-4. FLOW PER INCH OF WEIR WIDTH (cfs)

| Inches | 0 | 1/8 | 1/4 | 3/8 | 1/2 | 5/8 | 3/4 | 7/8 |
|---|---|---|---|---|---|---|---|---|
| 0 | 0 | 0.003 | 0.008 | 0.0015 | 0.0024 | 0.0033 | 0.0044 | 0.0055 |
| 1 | 0.0067 | 0.0080 | 0.0094 | 0.0108 | 0.0123 | 0.0139 | 0.0155 | 0.0172 |
| 2 | 0.0190 | 0.0208 | 0.0226 | 0.0245 | 0.0265 | 0.0285 | 0.0306 | 0.0327 |
| 3 | 0.0348 | 0.0370 | 0.0393 | 0.0415 | 0.0439 | 0.0462 | 0.0487 | 0.0511 |
| 4 | 0.0536 | 0.0561 | 0.0587 | 0.0613 | 0.0640 | 0.0666 | 0.0694 | 0.0721 |
| 5 | 0.0749 | 0.0777 | 0.0806 | 0.0835 | 0.0864 | 0.0894 | 0.0924 | 0.0954 |
| 6 | 0.0985 | 0.1016 | 0.1047 | 0.1078 | 0.1110 | 0.1142 | 0.1175 | 0.1208 |
| 7 | 0.1241 | 0.1274 | 0.1308 | 0.1342 | 0.1376 | 0.1411 | 0.1446 | 0.1481 |
| 8 | 0.1516 | 0.1552 | 0.1588 | 0.1624 | 0.1660 | 0.1697 | 0.1734 | 0.1771 |
| 9 | 0.1809 | 0.1847 | 0.1885 | 0.1923 | 0.1962 | 0.2001 | 0.2040 | 0.2079 |
| 10 | 0.2119 | 0.2159 | 0.2199 | 0.2239 | 0.2280 | 0.2320 | 0.2361 | 0.2403 |
| 11 | 0.2444 | 0.2486 | 0.2528 | 0.2570 | 0.2613 | 0.2656 | 0.2699 | 0.2742 |
| 12 | 0.2785 | 0.2829 | 0.2873 | 0.2917 | 0.2961 | 0.3006 | 0.3050 | 0.3095 |
| 13 | 0.3140 | 0.3186 | 0.3231 | 0.3277 | 0.3323 | 0.3370 | 0.3416 | 0.3463 |
| 14 | 0.3510 | 0.3557 | 0.3604 | 0.3652 | 0.3699 | 0.3747 | 0.3795 | 0.3844 |
| 15 | 0.3892 | 0.3941 | 0.3990 | 0.4039 | 0.4089 | 0.4138 | 0.4188 | 0.4238 |
| 16 | 0.4288 | 0.4338 | 0.4389 | 0.4440 | 0.4491 | 0.4547 | 0.4593 | 0.4645 |
| 17 | 0.4696 | 0.4748 | 0.4800 | 0.4852 | 0.4905 | 0.4958 | 0.5010 | 0.5063 |
| 18 | 0.5117 | 0.5170 | 0.5224 | 0.5277 | 0.5331 | 0.5385 | 0.5440 | 0.5494 |
| 19 | 0.5549 | 0.5604 | 0.5659 | 0.5714 | 0.5769 | 0.5825 | 0.5881 | 0.5937 |
| 20 | 0.5993 | 0.6049 | 0.6105 | 0.6162 | 0.6219 | 0.6276 | 0.6333 | 0.6390 |
| 21 | 0.6448 | 0.6505 | 0.6563 | 0.6621 | 0.6679 | 0.6738 | 0.6796 | 0.6855 |
| 22 | 0.6914 | 0.6973 | 0.7032 | 0.7091 | 0.7151 | 0.7210 | 0.7270 | 0.7330 |
| 23 | 0.7390 | 0.7451 | 0.7511 | 0.7572 | 0.7633 | 0.7694 | 0.7755 | 0.7816 |
| 24 | 0.7878 | 0.7939 | 0.8001 | 0.8063 | 0.8125 | 0.8187 | 0.8250 | 0.8312 |
| 25 | 0.8375 | 0.8438 | 0.8501 | 0.8564 | 0.8628 | 0.8691 | 0.8755 | 0.8819 |
| 26 | 0.8882 | 0.8947 | 0.9011 | 0.9075 | 0.9140 | 0.9205 | 0.9270 | 0.9335 |
| 27 | 0.9400 | 0.9465 | 0.9531 | 0.9596 | 0.9662 | 0.9728 | 0.9792 | 0.9860 |
| 28 | 0.9927 | 0.9993 | 1.006 | 1.013 | 1.019 | 1.026 | 1.033 | 1.040 |
| 29 | 1.046 | 1.053 | 1.060 | 1.067 | 1.074 | 1.080 | 1.087 | 1.094 |
| 30 | 1.101 | 1.108 | 1.115 | 1.122 | 1.129 | 1.136 | 1.142 | 1.149 |
| 31 | 1.156 | 1.163 | 1.170 | 1.178 | 1.184 | 1.192 | 1.199 | 1.206 |
| 32 | 1.213 | 1.220 | 1.227 | 1.234 | 1.241 | 1.248 | 1.256 | 1.263 |
| 33 | 1.270 | 1.277 | 1.285 | 1.292 | 1.299 | 1.306 | 1.314 | 1.321 |
| 34 | 1.328 | 1.336 | 1.343 | 1.356 | 1.358 | 1.365 | 1.372 | 1.378 |
| 35 | 1.387 | 1.395 | 1.402 | 1.410 | 1.417 | 1.425 | 1.432 | 1.440 |

From Table 3-4, the flow per inch of weir width is found to be 0.0613 cfs. Since the weir is 36 inches wide, the total flow is therefore:

$$36 \times 0.0613 = 2.21 \text{ cfs} \quad .$$

3.3.3.1.3 Float Method--The float method is recommended for larger streams where a temporary dam is not practical. The method is not as accurate as the previous two, but for large amounts of water, precise measurements are not as critical.

Equation (2-1) expressed flow as volume divided by time:

$$Q = \frac{V}{t}$$

The float method uses another definition for flow: Flow is equal to the area of a cross-section of the flowing water multiplied by the velocity of the flowing water--that is, the speed with which that cross-sectional area is moving (Figure 3-12). This subsection discusses how to determine area and velocity for this calculation.

$$Q = A \times v \qquad (3\text{-}3)$$

where

| | | |
|---|---|---|
| Q | = | flow in cfs |
| A | = | area in square feet ($ft^2$) |
| v | = | velocity in feet per second (fps) |

NOTE: Both Equation (2-1) and Equation (3-3) express flow in cfs.

To use the float method, you must determine two quantities:

- The average cross-sectional area of the stream
- The velocity at which the stream is moving.

To determine the average area, choose a length of stream at least 30 feet long (the longer the better) that is fairly straight, with sides approximately parallel. The stream should have a relatively smooth and unobstructed bottom. If there are large rocks in the bed or if the stream flow is irregular, you will have to apply an appropriate correction factor to the velocity.

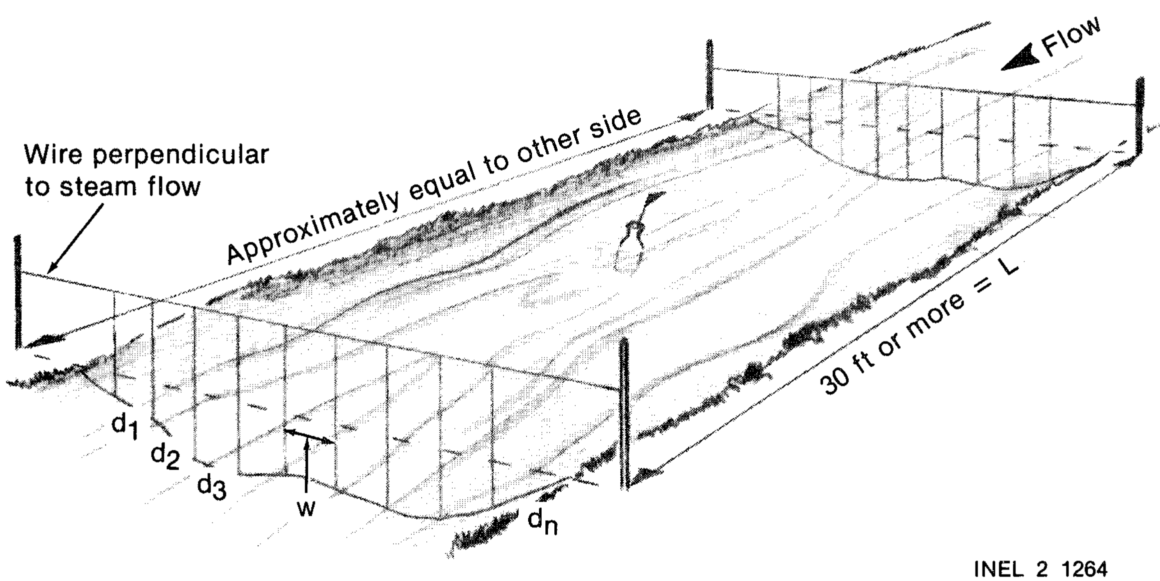



Figure 3-12. Float method for estimating flow.

To start, stake out a point at each end of the chosen length of stream and drive a post on each side of the bank at these points. Connect a wire or rope between the posts crossing the stream (Figure 3-12). Use a carpenter's line level to level the taut wire. Measure the width of the stream at each crossing. Divide the width into convenient equal segments (1 to 2 feet each). Cross the stream, tying a marker (string or ribbon) on the wire or rope to mark each equal segment. With a yard stick, measuring rod, or similar device, measure the depth of water at each marker. (It is usually easier to measure the distance from the stream bottom to the wire and then subtract the distance from the wire to the top of the stream.) Add up all the readings and multiply by the segment width.

$$D = (d_1 + d_2 + d_3 + \dots d_n) \tag{3-4}$$

where

D = sum of the depths measured

d = measured depth of the stream at each marker in inches

n = number of markers

EXAMPLE: Assume that the stream is 20 feet wide and that wires have been stretched across the stream in two places 45 feet apart. Find the sum of the depths measured.

First, divide each wire across the stream into 2-foot segments. Since 2 divides into 20 feet 10 times, use 9 markers to identify the segments, marking the first segment 2 feet from the shore and continuing across in 2-foot intervals. The last segment marker should be 2 feet from the opposite bank. Next, measure the depth of the stream at each marker, and add up all the depths measured.

$$D = d_1 + d_2 + d_3 + \dots d_9 = 168 \text{ in.}$$

Now, determine the cross-sectional area of the stream. To do this, multiply the sum of the depths measured (D) by the width of the individual segments (W) and divide by 144 to convert from square inches to square feet.

$$A = \frac{W \times D}{144} \tag{3-5}$$

where

$A$ = cross-sectional area in $ft^2$

$W$ = width of individual segments in inches

$D$ = sum of the depths measured in inches

$144$ = the number of square inches ($in^2$) in a $ft^2$

EXAMPLE:

$$A = 24 \text{ in.} \times \frac{168 \text{ in.}}{144}$$

$$A = 28 \text{ ft}^2$$

Repeat the process at the other crossing, add the two areas together, and divide by two to obtain an average cross-sectional area for the selected length of stream.

$$A = \frac{A_1 + A_2}{2} \tag{3-6}$$

where

$A$ = average cross-sectional area in $ft^2$

$A_1$ = cross-sectional area at the first crossing in $ft^2$

$A_2$ = cross-sectional area at the second crossing in $ft^2$

$2$ = number of areas added together

EXAMPLE: From the example above, $A_1 = 28\ ft^2$; assume that $A_2 = 34.5\ ft^2$

$$A = \frac{28\ ft^2 + 34.5\ ft^2}{2}$$

$$A = 31.2\ ft^2$$

Next you must determine the velocity of the stream flow. Make a float of light wood, or use a bottle that is weighted to ride like a piece of wood (Figure 3-10). A small flag can be put on the float so that its progress can be followed easily. Now set the float adrift upstream from the first wire. Time its progress down the stream with a stopwatch, beginning just when the float passes the first wire and stopping just as it passes the second wire. Repeat the measurement at least six times at various locations across the stream to obtain an average time. Perform this measurement on a calm day since wind will cause errors in your measurements.

$$T = \frac{t_1 + t_2 + t_3 + \ldots t_n}{n} \qquad (3\text{-}7)$$

where

| | | |
|---|---|---|
| T | = | average time in seconds |
| t | = | recorded time for each drift in seconds |
| n | = | number of drifts |

Since the water does not flow as fast on the bottom as it does on the surface, you must apply a correction factor (c) to the average time. For a straight stream with a smooth bottom, use 0.8. For a stream with large rocks on the bottom, use 0.6.

$$T_c = \frac{T}{c} \quad (3\text{-}8)$$

where

$T_c$ = corrected time in seconds

$T$ = uncorrected average time in seconds

$c$ = correction factor (no units)

EXAMPLE: Assume that the two wire crossings are 45 feet apart and that the bottom is smooth and uniform. Find the stream velocity by timing six drifts.

Measured drift times:

$t_1 = 23$, $t_2 = 26$, $t_3 = 22$, $t_4 = 25$, $t_5 = 23$, and $t_6 = 25$ .

From Equation (3-7):

$$T = \frac{23 + 26 + 22 + 25 + 23 + 25}{6}$$

$$T = \frac{144}{6}$$

$$T = 24 \text{ sec}$$

Assume that c = 0.8 (for smooth bottom). From Equation (3-8):

$$T_c = \frac{24}{0.8}$$

$$T_c = 30 \text{ sec}$$

Velocity is distance divided by time:

$$v = \frac{L}{T_c} \tag{3-9}$$

where

$v$ = velocity in fps

$L$ = distance between wires in feet

$T_c$ = corrected time in seconds

From Equation (3-9) (continuing the example):

$$v = \frac{45 \text{ ft}}{30 \text{ sec}}$$

$$v = 1.5 \text{ fps} \quad .$$

Both quantities needed to compute flow are now known and can be substituted into Equation (3-3), $Q = A \times v$.

EXAMPLE: $A = 31.2 \text{ ft}^2$ and $v = 1.5$ fps; find Q.

$$Q = A \times v$$

$$Q = 31.2 \text{ ft}^2 \times 1.5 \text{ fps}$$

$$Q = 46.8 \text{ cfs} \quad .$$

The float method is easier to set up than the weir method, but it is more difficult to make daily readings. Each time the depth of the stream changes, you must determine a new area and velocity. If the stream is used by others, it is not advisable to leave the wires or ropes across the stream. They should be taken down after each reading.

One suggestion that might simplify repeat measurements is to place a yardstick on a post and drive the post into the stream bed so that the yardstick can be read. Each time the measurements are made and the flow determined, record the depth on the yardstick. Then every time that depth appears on the yardstick, the flow is the same as previously determined, and since different flows are needed for correlation, another day should be selected to make further measurements.

In summary, select the best measurement method for the stream, locate the measurement station near the proposed location for the intake structure, and begin taking measurements.

3.3.3.2 Flow Computations. You should now have several days of flow measurement recorded. The next step is to determine from the gage previously identified as closest to your site the flow readings for the days on which your flow measurements were made. Record the gage readings in the appropriate column ($Q_2$) of the flow table (Figure 3-5), and compute the correlation by dividing the gage reading into the measured flow.

$$c = \frac{Q_1}{Q_2} \qquad (3\text{-}10)$$

where

$c$ = correlation number

$Q_1$ = measured daily flow in cfs

$Q_2$ = gage daily flow in cfs

If a good flow pattern relation exists between the site and the gage, the correlation value will be approximately the same for each day of measurement.

EXAMPLE: Assume that four measurements were made at the site and the flow was computed. Also assume that the gage readings have been received and recorded.

| Computed Flow, $Q_1$ (cfs) | Gauge Flow, $Q_2$ (cfs) | Correlation $Q_1/Q_2$ |
|---|---|---|
| 26.7 | 50.3 | 0.53 |
| 30.3 | 59.6 | 0.51 |
| 33.7 | 68.9 | 0.49 |
| 31.2 | 65.3 | 0.48 |

The variation in the correlation ranges from 0.53 to 0.48, or 0.05 variation. Since the variation is small, the correlation is good.

If the variation becomes greater than 0.15, the correlation between the site and the gage may not be sufficient to use the gage exceedance value to project stream flow at the site. A developer who cannot obtain a good correlation should use the procedure given in Appendix A-4, "Stream Flow Projections Where a Gage Correlation Does Not Exist."

3.3.3.3 Exceedance Value Flow Duration Curve. An example of a flow duration curve was presented in Figure 3-4. The curve was plotted from exceedance values obtained from the A9-69 printout (Table 3-3). The printout lists flow values for seven exceedance percentages (95, 90, 75, 70, 50, 25, and 10%). The 95% flow value means that that flow will be met or exceeded 95% of the time. The 70% flow will be met or exceeded 70% of the time, etc. If other than USGS exceedance values are used, the seven exceedance percentages may be different, but they can be used to develop a flow duration curve as easily as the USGS values.

The next step is to construct the flow duration curve. Figure 3-13 shows a reduced copy of standard 8-1/2 x 11 inch graph paper with 20 grid markings per inch. Paper with 10 divisions per inch could just as easily

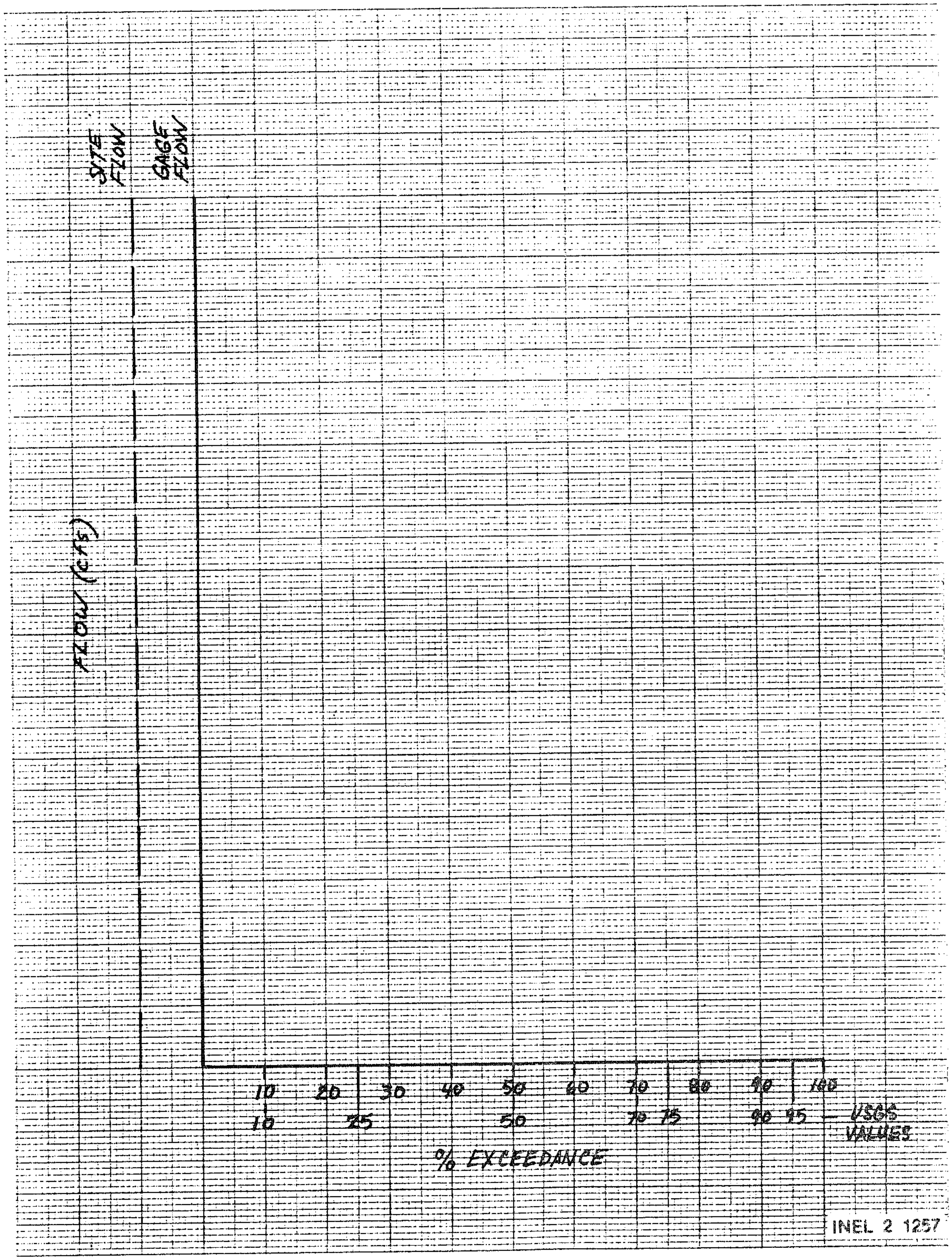



Figure 3-13. Form used for exceedance valve flow duration curve.

be used. The paper can be obtained at most office supply stores. Two coordinates are shown on the figure. The vertical coordinate is for the flow scales, and the horizontal is for the exceedance percentage. The seven USGS exceedance values are indicated below the horizontal axis.

The vertical axis is not scaled because the two scales that will be shown depend on the amount of flow to be plotted. The easiest method to determine the vertical scales is as follows. First, make the axis 6 to 8 inches long and divide it into convenient increments. For example, assume that the USGS 10% exceedance flow is 390 cfs. Make the axis 8 inches long, and find the next larger number above 390 into which 8 will divide evenly. Eight will divide into 400 fifty times. Therefore, make the increments 50 cfs per inch. Figure 3-14 is a flow duration curve for the exceedance values given in Table 3-3. Since in this example the 10% exceedance flow is 130 cfs, the vertical axis was made 7 inches long and a 20 cfs per inch increment was selected. Look at Table 3-3 for 95% exceedance; the flow is 8.1 cfs. To plot the first point, place one straightedge on the graph so that it passes vertically through the 95% mark, and another so that it passes horizontally through the 8.1 cfs flow level; mark the point where the two straightedges cross. You can use a draftsman's right triangle for this purpose. Repeat this procedure until the other six points are plotted. Connect the points together to form a curve. A drafting tool called a french curve will aid in drawing a smooth curve, but its use is not necessary. You can probably obtain satisfactory results by connecting each point with a straight line. The flow duration curve for the gage exceedance is now plotted.

Next, the scale for the site flow needs to be developed. Refer back to the previous subsection in which the correlation values were computed [last column of the flow table (Figure 3-5)]. Find the average correlation value by adding the numbers together and dividing by the number of readings.

$$C = \frac{c_1 + c_2 + c_3 + \dots c_n}{n} \qquad (3\text{-}11)$$

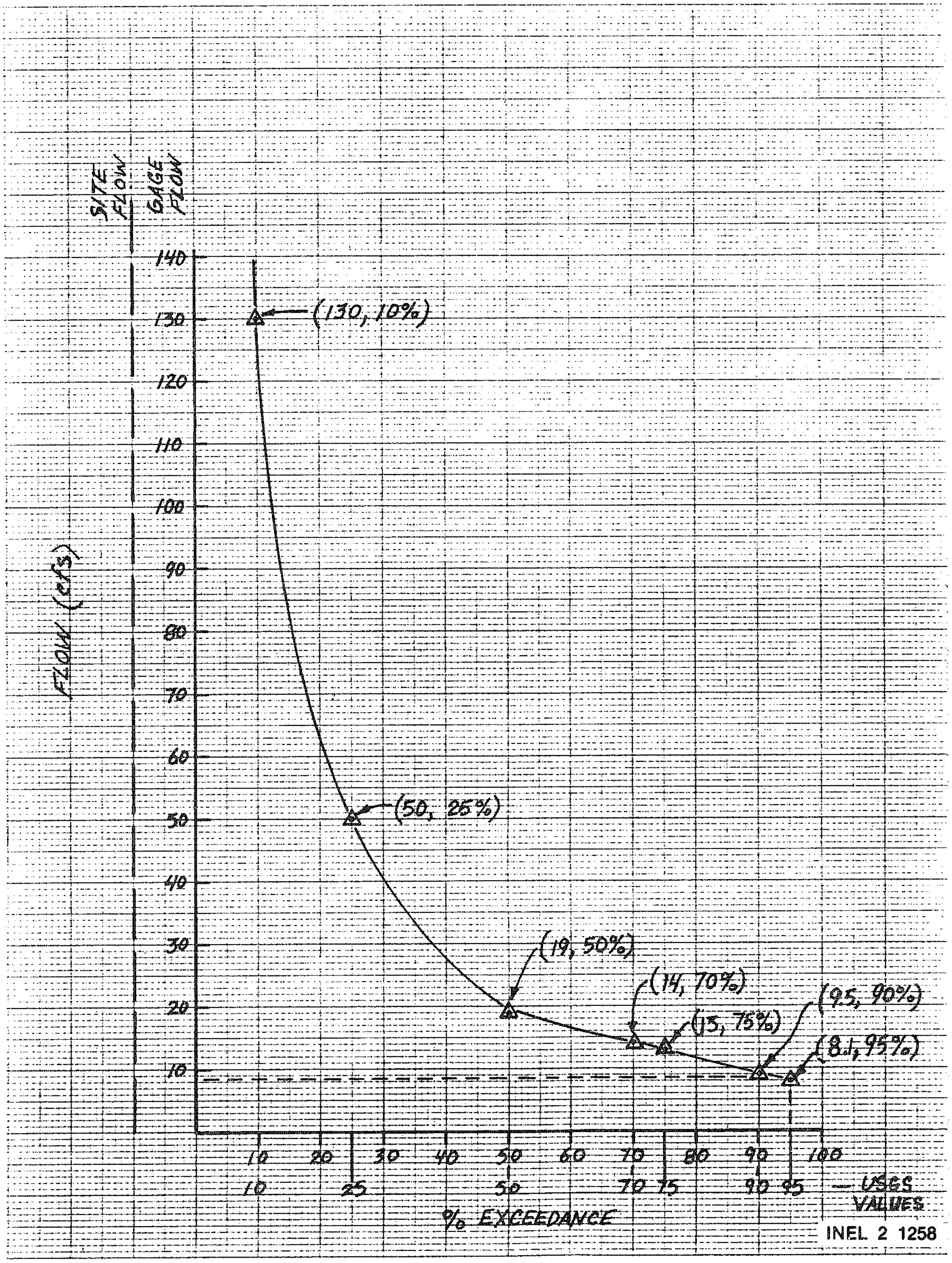



Figure 3-14. Flow duration curve for exceedance values in Table 3-3 (site flow scale missing).

where

$C$ = average correlation

$c$ = daily correlation value

$n$ = total number of correlations

For the Mill Creek example (Table 3-3, Figure 3-14), assume the following correlation values: $c_1 = 0.70$, $c_2 = 0.65$, $c_3 = 0.72$, and $c_4 = 0.74$.

From Equation (3-11):

$$C = \frac{0.70 + 0.65 + 0.72 + 0.74}{4} = \frac{2.81}{4}$$

$$C = 0.70$$

To find the site flow scale, multiply the gage scale by C. Thus, 10 cfs gage x 0.70 = 7 cfs site, 20 cfs gage x 0.70 = 14 cfs site, etc. Figure 3-15 shows the flow duration curve with the completed site flow scale. From the site flow scale, the minimum stream flow is around 5.7 cfs, and the 25% exceedance is 35.1 cfs.

Take another set of flow measurements a month or two after the first and check for correlation again. If the average correlation value is close to the first value, then the correlation is good and the curve (Figure 3-15) can be used for design. If the average correlation is not reasonably close (more than 0.15 difference), refer to Appendix A-4, "Stream Flow Projections Where a Gage Correlation Does Not Exist."

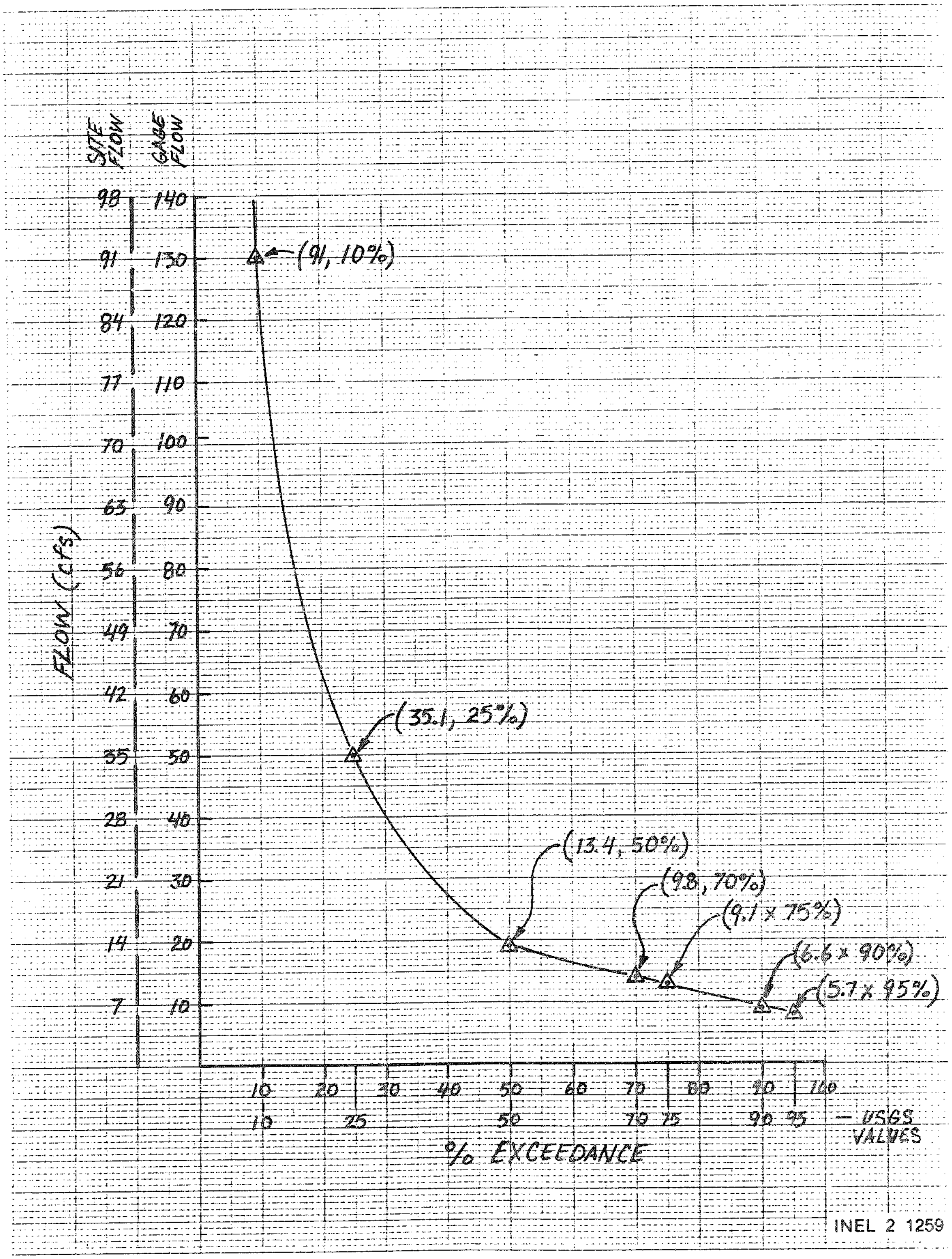



Figure 3-15. Flow duration curve with completed site flow scale.

## 3.4 Head and Distance Measurements

Before physically measuring head and distance, run-of-the-stream developers should proceed to Subsection 3.5.1, "Power Requirements and Minimum Flow Known; Calculate Head" to determine how much head is needed. After the head calculation, return to this section for the measurements.

Pool-to-pool head is the change in elevation measured in feet of vertical fall. If a power canal is used, the pool-to-pool head is only the change in vertical elevation from the water surface at the penstock intake to the tailwater surface elevation at the turbine. The change in elevation for the power canal does not count, since the canal is not under pressure. Head and flow are the two quantities used to compute power (Equation 2-2).

Some developers may want to hire a professional surveyor, who will use a survey level, rod, and steel chain, or even more sophisticated equipment, to measure head and distance. For the developer who wishes to do the work himself, this subsection gives suggestions on surveying.

If you have followed the instructions in Subsection 3.2, you have already made a preliminary site inspection, and run-of-the-stream developers have selected a penstock routing. You should also have determined the amount of head you require, or the amount available from the site. You are now ready to measure head and distance, starting from the proposed powerhouse location. After measuring pool-to-pool head and distance, make a sketch of the site showing the route and the distance.

### 3.4.1 Head Measurements

If the proposed powerhouse and penstock intake are near the stream, you can use the pressure method described immediately below to measure head. If they are not, you will have to use other survey methods.

3.4.1.1 Pressure Method for Measuring Pool-to-Pool Head at Run-of-the-Stream Sites. Head is measured in feet and represents pressure resulting from the weight of the water.

Weight of water = 62.4 pounds per cubic foot ($lb/ft^3$)

1 square foot = 144 square inches

Therefore:

$$\frac{62.4\ lb/ft^3}{144\ in^2/ft^2} = 0.433\ psi/ft\ of\ head$$

1 foot of head = 0.433 pounds per square inch (psi)

The pressure method uses this relationship to measure head. A pressure gage, which can be purchased for $10 to $20, is connected to the bottom end of a pipe or hose, and the static pressure in the pipe or hose is read on the gage. Static pressure means that the water is not flowing in the hose at the time of measurement.

This unique adaptation of the pressure-head principle is perhaps the simplest method for measuring head in a stream that changes elevation fairly rapidly. The only equipment required is a hose and a pressure gage. The gage should range from 0 to 10 psi if the head measurement will not exceed 20 feet for any single measurement. The gage should be accurate to at least the nearest 1/4 psi (0.1 psi accuracy preferred).

Starting with the lower end of the hose at the proposed powerhouse tailrace location and working upstream toward the proposed intake location, place the hose along or in the stream, submerge and anchor the upstream end of the hose, and allow water to flow through the hose until all air is removed and the water flows freely (Figure 3-16). Connect the pressure gage to the lower end of the hose and record the pressure. The upstream end of the hose should not be pointed directly into the stream flow but should be at a 90-degree or greater angle to the flow. If it is pointed directly into the flow, the pressure gage, because of the velocity of the water, will give a reading slightly higher than the static pressure. While

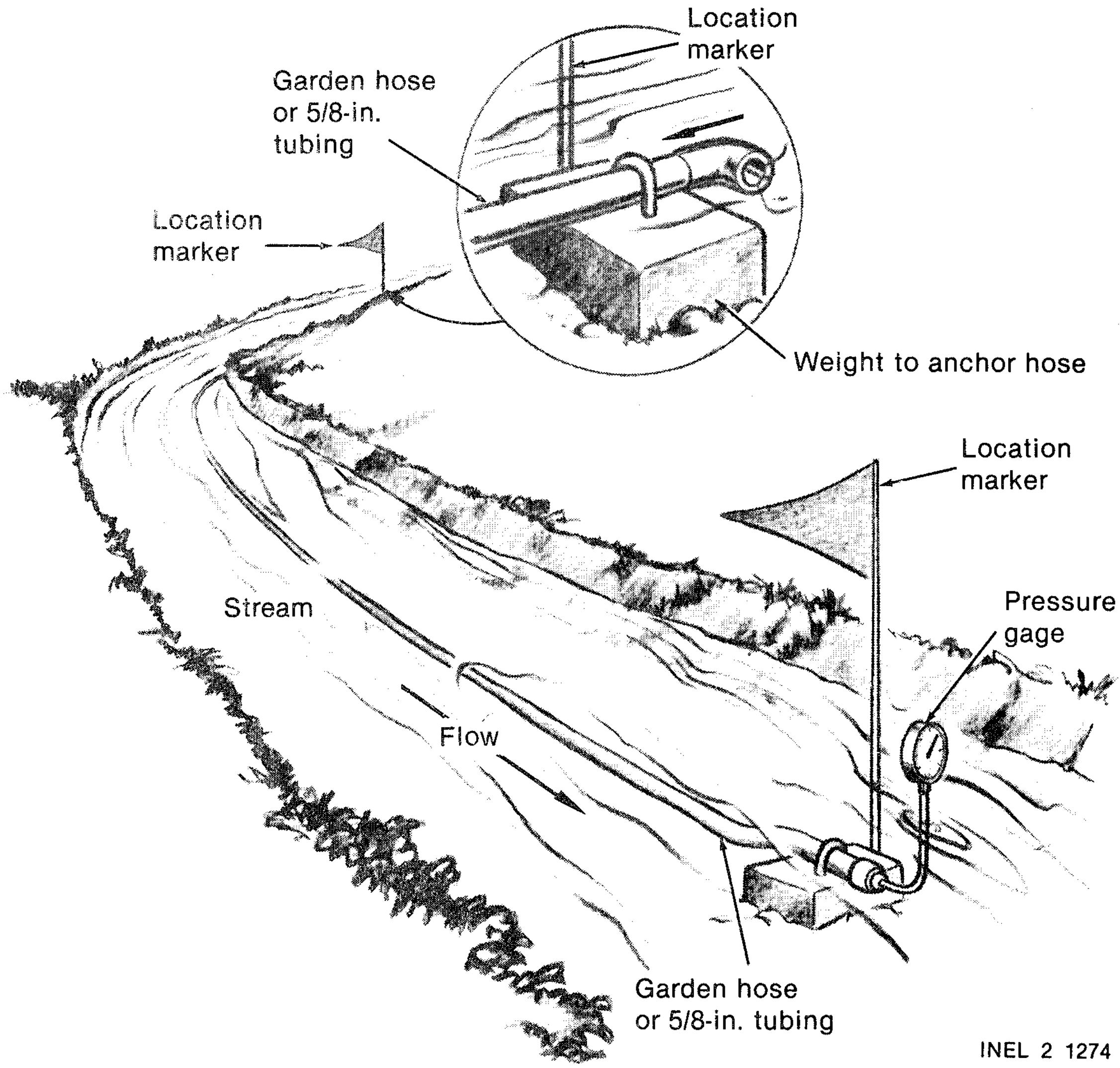



Figure 3-16. Pressure gage measurement of head for run-of-the-stream sites.

this error will not be great for a single reading, the cumulative effect of a large number of readings taken at the site will be an erroneous head measurement. Mark the location of the upper end of the hose so that the lower end can be placed there for the next reading. Continue the process until the length of stream in question is measured. Note: When making the last reading, coil the excess hose somewhere upstream from the gage; it won't affect the reading as long as all the air is removed. After taking the readings, add all the pressures and divide by 0.433.

$$h = \frac{P_1 + P_2 + P_3 \ldots P_n}{0.433} \qquad (3\text{-}12)$$

where

| | | |
|---|---|---|
| h | = | pool-to-pool head in feet |
| P | = | individual pressure measurement |
| n | = | number of measurements |
| 0.433 | = | pressure per foot of head in psi |

EXAMPLE: Assume that four readings were taken--$P_1$ = 5.6 psi, $P_2$ = 4.8 psi, $P_3$ = 6.1 psi, and $P_4$ = 5.9 psi. Find the head.

$$h = \frac{5.6 + 4.8 + 6.1 + 5.9}{0.433} = \frac{22.4 \text{ psi}}{0.433}$$

$$h = 52 \text{ ft} \quad .$$

NOTE: The hose(s) should be at least 100 feet or longer so that pressure readings are greater than 1 or 2 psi. Also, the fewer the readings, the smaller the error.

Compare the measured head with the required head, and adjust the location of the proposed intake point as needed to obtain 5 to 10% greater pool-to-pool head to allow for system losses.

3.4.1.2 Level Survey to Measure Head at Run-of-the-Stream Sites, Canal Drops, and Industrial Discharge Sites. If a power canal is going to be used, a level survey method will have to be used to measure the change in elevation from the powerhouse to the penstock intake. Although the level surveying procedure is independent of the equipment used to make the survey, the various types of equipment will be discussed first, and then the procedure.

The best equipment for making such a survey is a survey level (Figure 3-17) and rod designed specifically for surveying. With a few minutes of instruction, you can easily set up and level the level. A construction rental business should have such equipment available. Other possible sources of equipment are building contractors or the state highway department.

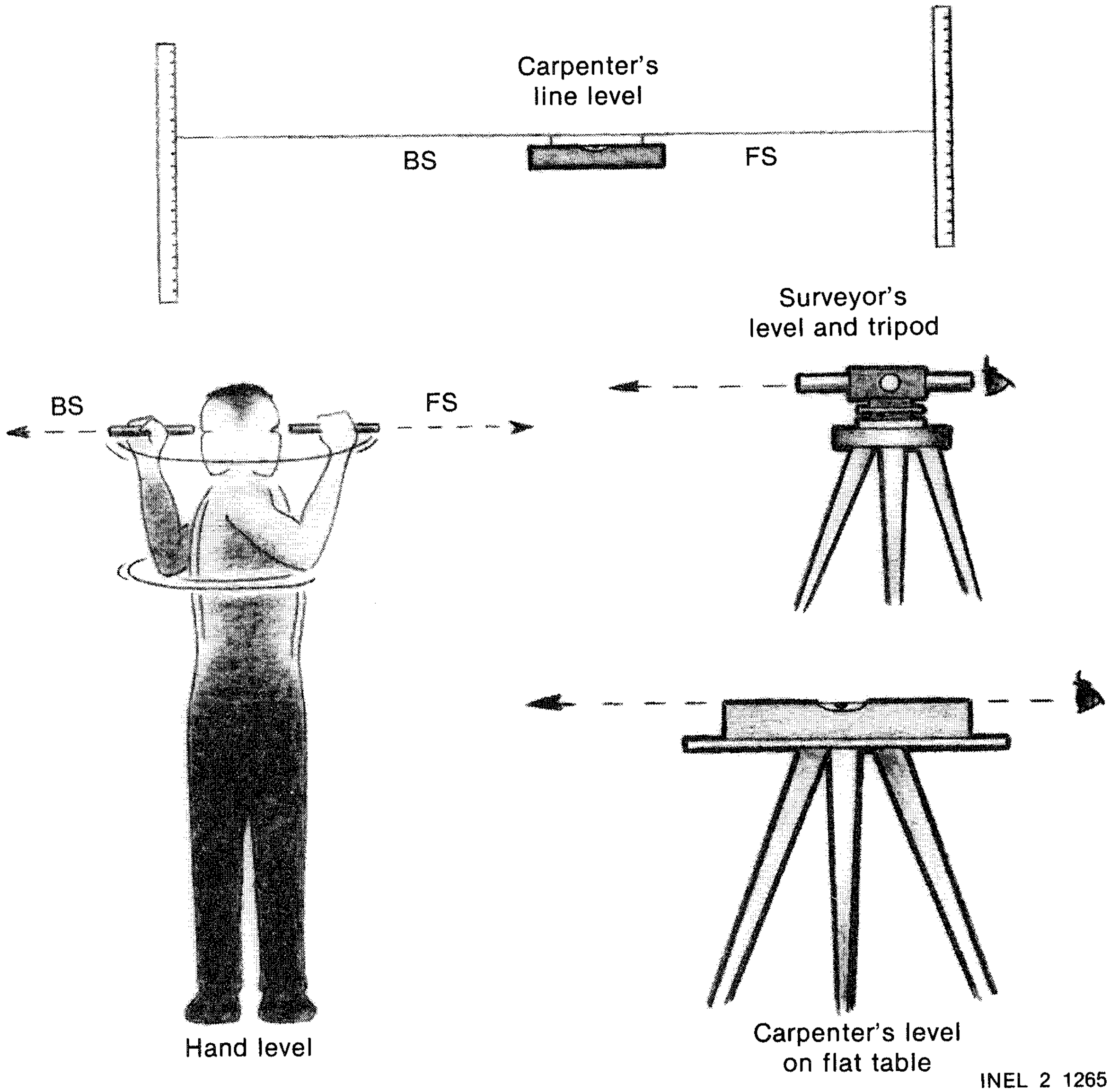


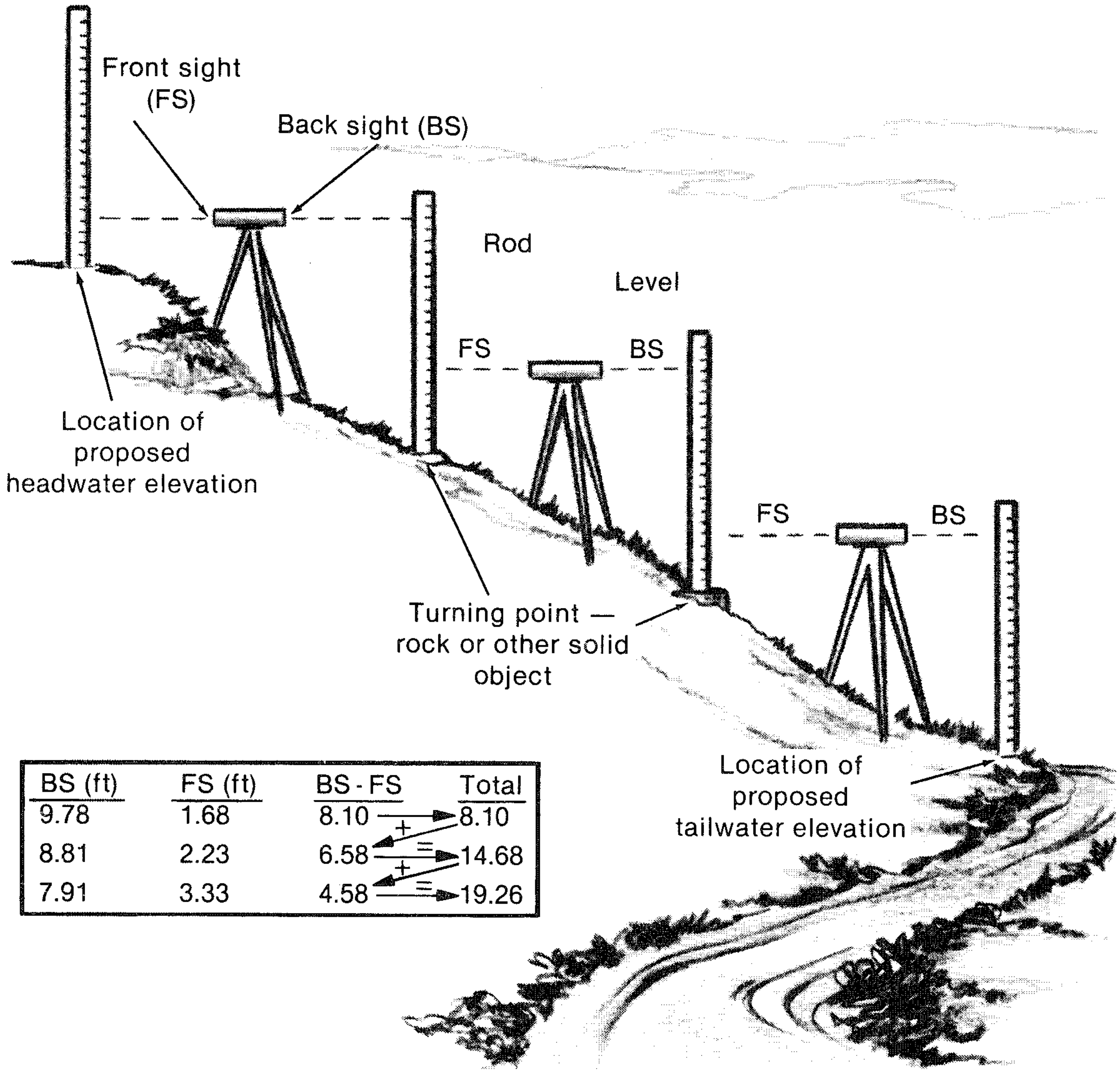

Figure 3-17. Types of level.

The surveyor's hand level is the next-best alternative. The hand level works on the same principle as the survey level, except that the person holding the level becomes the tripod. The person holding the level should stand facing at a right angle to the survey line (Figure 3-17) and should look over one shoulder to the back sight (BS) and the other shoulder to the front sight (FS) without moving the feet when shifting from one sight to the other.

A third type of level is a carpenter's level placed on a table or similar device. To take a reading, sight across the top of the level (Figure 3-17).

The rod is a straight, rectangular pole on which a measurement scale is placed. A regular survey rod is divided into feet and hundredths of a foot. A tape measure or three yard sticks placed on a 10-foot 2 x 4 or 2 x 2 will work. The person holding the rod can assist the reading of the rod by using a pointer that moves up and down the scale until the correct number is pointed to. To make sure that the rod is held vertically, hold the rod facing the level and slowly rock the rod back and forth toward the level until the minimum measurement is read.

The locations for the powerhouse and the penstock intake structure were preliminarily determined in Subsection 3.2. Use the same locations for the initial level survey. Place the measuring rod on a rock or similar solid object at the waterline at the powerhouse location. Place the level in the line of sight between the rod and the intake location. Level the level and read the height off the rod (Figure 3-18). The first reading to be taken is the height of the level above the stream. Record the reading and note it as BS (back sight). The person with the rod now proceeds uphill past the level and places the rod on a solid rock (or something similar) called the turning point. The rod should be faced downhill toward the level. The person with the level sights uphill toward the rod. Make sure that the level is still level before reading the rod. After checking, read




| BS (ft) | FS (ft) | BS - FS | Total |
|---|---|---|---|
| 9.78 | 1.68 | 8.10 | 8.10 |
| 8.81 | 2.23 | 6.58 | 14.68 |
| 7.91 | 3.33 | 4.58 | 19.26 |

Figure 3-18. Level survey method of head measurement.

the height on the rod and record the reading as FS (front sight). Subtract the FS reading from the BS reading and record the difference. Now, move the level uphill past the rod and along the path previously chosen. After the level is set up and leveled again, sight downhill toward the rod, which has been turned around to face the level. Read the height and record the reading as a BS. Continue in this manner, alternately moving the rod and level up the hill. After each FS measurement, subtract the FS measurement from the preceding BS measurement and add the results to the previously recorded differences. For run-of-the-stream sites, continue measuring elevation until the required head is reached.

3.4.1.3 Survey Methods for Manmade Dams with Low Head. Dam sites may have the problem of a fluctuating head. A method should be developed to measure head when measuring flow. You can use the pressure method described in Subsection 3.4.1.1 if a pipe penetrates the dam and a blind flange with a gage can be placed on the pipe. Otherwise, use the level survey method described in Subsection 3.4.1.2 to establish an initial head. Figure 3-19 shows a method for daily head measurements.

How to use the data being gathered depends on the fluctuation pattern. If the head fluctuation is seasonal, the average flow and head will have to be computed for the season instead of annually. In such situations, low flow usually corresponds to low head. Head at high flow should also be considered, since high flow may increase the tailwater elevation, thus reducing the head. You should pay particular attention to the low- and high-flow seasons to determine how much power (if any) can be produced during those periods.

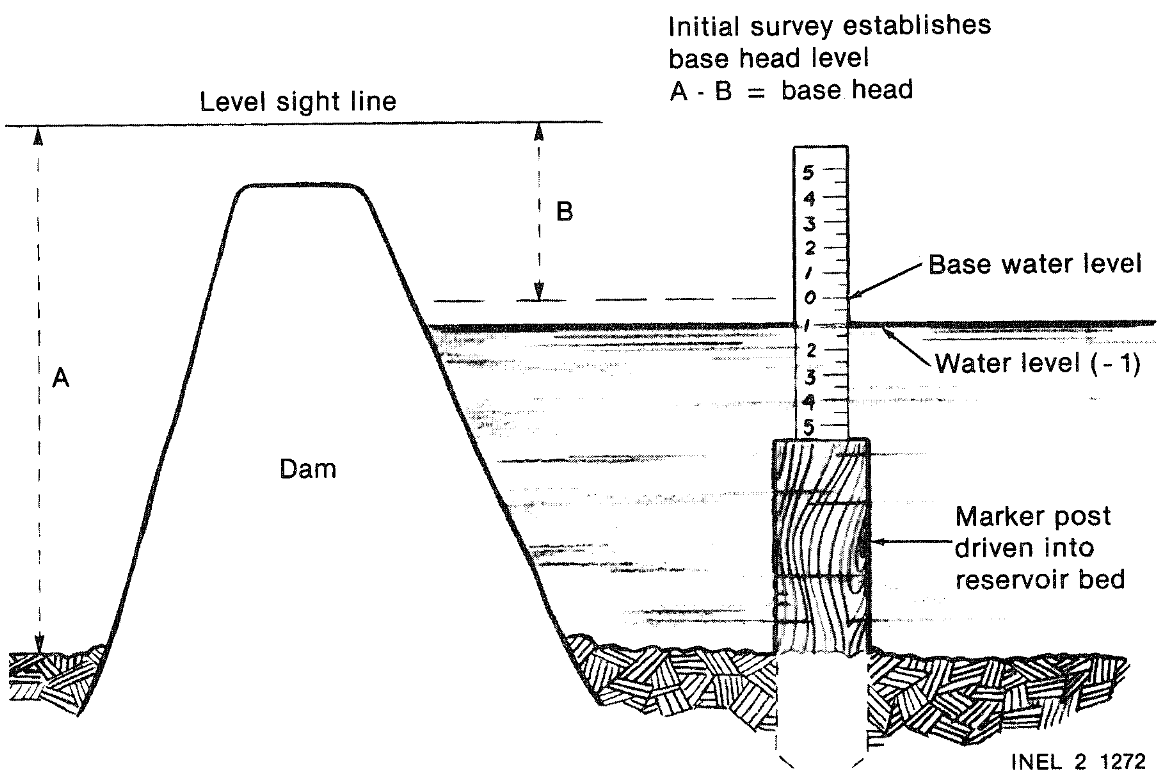



Figure 3-19. Method for daily head measurements at dam site.

If the head fluctuation is erratic, which is typical of small impoundment structures, you can use the average head for the rough first cut at calculating power. Once you have selected a flow for the preliminary design, you should estimate the effect of that flow on the reservoir drawdown. If your turbine uses less flow than is normally discharged, the effect is minimal. However, if the turbine uses more than the normal flow, the drawdown will be faster, and if the turbine's flow requirement is high enough, it may have to cycle on and off to prevent total drawdown.

### 3.4.2 Distance Measurement for Run-of-the-Stream Site

For run-of-the-stream sites, the distance from the intake structure to the powerhouse must be known to determine the length and size of the penstock and estimate the cost. As mentioned in Subsection 3.2, it is a good idea initially to lay out the system on a USGS contour map. Once the locations for the intake power canal, penstock intake, and powerhouse are proposed, select the shortest unobstructed distance between these locations. Remove brush and similar material, stake the proposed locations, and measure the distance along the line staked. A 100-foot steel tape is preferred so that the tape can be pulled tight (5 to 10 pound pull) for each measurement. Start at one location and measure to the other, recording each measurement. A common mistake in distance measurement is forgetting to record a measurement. One way to keep this from happening is to have both persons record each measurement and to make sure that the correct number is recorded before moving to make the next measurement. The total distance between the points is the sum of all the measurements.

## 3.5 Determining Design Capacity, Head, and Flow for Category 1 Developers

The information in this subsection is for Category 1 developers, those who want to be energy independent. The hydropower system will only be designed to supply the power needs of the developer. Category 2 developers are referred to Subsection 3.6, "Determining Design Capacity, Head, and Flow for Category 2 Developers."

If you are a run-of-the-stream developer, you should already have determined your power requirements and the minimum stream flow and, for manmade sources, measured the available pool-to-pool head. In this subsection, you will use the power requirements and minimum stream flow to determine the design head for your project.

The basic power equation [Equation (2-2)] will be used for all calculations.

$$P = \frac{Q \times h \times e}{11.81} \tag{2-2}$$

where

| | | |
|---|---|---|
| P | = | power in kW |
| Q | = | flow in cfs |
| h | = | head in feet |
| e | = | efficiency (assumed to be 60%) |
| 11.81 | = | constant of conversion |

To calculate design head, the equation is rewritten to solve for head. If the value of e is 60%, the design nead calculated will be the pool-to-pool head of your site since this value of e includes penstock losses.

$$h = \frac{11.81 \times P}{Q \times e} \tag{3-13}$$

To calculate flow if head and power are known, the equation is rewritten to solve for flow:

$$Q = \frac{11.81 \times P}{h \times e} \quad . \tag{3-14}$$

The flow value used for the first series of calculations should be the minimum flow. On most flow duration curves, the 95% exceedance site flow can be used for the minimum flow value. Be sure to use site flow, not gage flow. If the minimum flow will not produce the power required, or if it requires too large a head to produce the required power, you can estimate the relative economic benefit of the system from other sections of this book and determine from that whether or not to proceed with the development. There are four different calculations to use, depending on what is known and what needs to be computed.

- o Power requirements and minimum flow are known; calculate head.
- o Head and flow are known; calculate design capacity.
- o Head and power requirements are known; calculate minimum flow.
- o Head and flow vary; calculate design capacity.

Select the appropriate calculation, and follow the procedure given below.

### 3.5.1 Power Requirements and Minimum Flow Known; Calculate Head

From Equation (3-13), compute head. Use the previously determined values for power requirement (P) and minimum flow (Q). Determine if the calculated head is reasonable from the standpoint of physical installation and length of penstock (Subsection 3.2). If the head is reasonable, go to Subsection 3.4 to measure head and stake out the intake system. Then use the flow and head as design points, and proceed to Section 4 to select equipment.

If the calculated head is too large for the site, go to Subsection 3.4 and measure the maximum reasonable head. After the maximum head is determined, go to Subsection 3.5.3 and calculate flow for the site with a fixed head and a known power requirement.

### 3.5.2 Head Fixed and Flow Known; Calculate Design Capacity

In Subsection 3.1, you determined how much power was required to meet your needs. When head and flow are used to compute power, the power is called design capacity. Equation (2-2) is used to compute design capacity for a given head and flow.

$$P = \frac{Q \times h \times e}{11.81} \; .$$

Compare the rated capacity with the power required. If the rated capacity is greater than the required power, you can use Equation (3-14) to compute a lower design flow, using the power required as the design capacity.

$$Q = \frac{11.81 \times P}{h \times e} \; .$$

The power value used in Equation (3-14) should be the power required for your development. The head is still fixed. The design points are now defined for power, head, and flow. Proceed to Section 4.0 to select equipment.

If the calculated design capacity is less than the estimated power required, then the site will not produce all the energy needed year round. Use the power required as the value for the design capacity, P, and go to Subsection 3.5.3 to compute minimum flow requirements and determine what percentage of the year the required power will be available.

### 3.5.3 Head and Power Requirements are Known; Calculate Minimum Flow and Percentage Exceedance

Use Equation (3-14) to compute minimum flow.

$$Q = \frac{11.81 \times P}{h \times e}$$

Once flow is determined, refer back to the flow duration curve and locate the flow value on the vertical axis for site flow. If you used the method in Appendix A-2 to estimate minimum stream flow, you should return to Subsection 3.3 to create a flow duration curve.

When you have located the flow value on the vertical axis of the flow duration curve, draw a horizontal line to the right to intersect the curve. Where the line intersects the curve, draw a vertical line down to the percentage exceedance scale (horizontal axis). Read the percentage exceedance value where the vertical line intersects the axis. The percentage exceedance means that the system has the potential to supply the required power that percentage of the year.

EXAMPLE: Assume that the maximum practical head at the site is 30 feet and that the required power is 12 kW. Use Figure 3-15 as the flow duration curve. Find the minimum flow and the percentage exceedance.

Using Equation (3-14):

$$Q = \frac{11.81 \times P}{h \times e}$$

$$Q = \frac{11.81 \times 12}{30 \times (0.60)}$$

$$Q = 7.9 \text{ cfs} \quad .$$

Assume that a minimum stream flow requirement of 3 cfs has been established by the state to sustain fish habitat. Therefore, the minimum design flow in the stream would be:

$$7.9 \text{ cfs} + 3.0 \text{ cfs} = 10.9 \text{ cfs}$$

Round to 11 cfs.

On the flow duration curve, Figure 3-15, estimate the location of 11 cfs on the site flow scale, draw a horizontal line to intersect the curve, and then draw a vertical line to intersect the flow exceedance scale. The vertical line should be at 60% on the flow exceedance scale. Thus, the required power will be produced 60% of the year.

Turbines will continue to generate some power at flow ranges as low as 35% to 55% of the design flow. The amount of power for the percentage of design flow depends on the turbine-generator unit. Units will always work most efficiently at design flow and capacity. Developers in this situation should consult with various manufacturers to determine how much power can be produced in the lower flow ranges.

### 3.5.4 Head and Flow Vary; Calculate Design Capacity

For sites where both head and flow vary, determining the size of the turbine and the average power potential becomes the most difficult. You should construct a flow duration curve if you have not already done so.

Head fluctuation can be either seasonal, corresponding to average flow, or erratic, controlled by sources other than average flow.

If head becomes too small (less than 5 to 10 feet, depending on the turbine type), no power can be generated. Any period of time with less than minimum head cannot be considered for power generation.

3.5.4.1 Seasonal Head Fluctuation. With seasonal head fluctuation, the head should be related to flow. From the flow duration curve, mark off average head for seasonal flows. Follow the procedure in Subsection 3.5.2 to calculate design capacity for each season. You will have to measure head and flow several times each season to ensure that the relationship between head and flow is known. Using the head and flow relationships, determine the point where the design capacity most closely equals the power required. At that point, determine what portion of the year power production can be expected (Subsection 3.5.3).

If the design capacity is more than the power required, even at low head and flow, go to Subsection 3.5.3 to calculate design flow, and then proceed to Section 4.0 to select equipment.

3.5.4.2 Erratic Head Fluctuation. Erratic head fluctuation can have a number of causes, including a reservoir that is too small, discharge that is too large for the size of the reservoir, or control of the flow by other interests such as irrigation, etc.

The procedure for calculating rated power depends on how much the head fluctuates and how often. The first step is to calculate design power for the smallest head and lowest flow. If the design capacity is near the power required, use the head and flow as preliminary design points and proceed to Section 4.0. If the design capacity is considerably less than the power required, continue to increase the value for head and flow as they increase for the site until the design capacity equals the required power. At that point, determine how much of the year power would be available and decide if the project is worth while.

## 3.6 Determining Design Capacity, Head, and Flow for Category 2 Developers

The information in this subsection is for Category 2 developers, those who want to develop the maximum energy available from the site at the least investment.

In an effort to produce the maximum energy at the minimum cost, there are several effects that need to be pointed out. Normally, a larger hydroelectric plant produces a greater amount of energy, but, a larger plant costs more to construct and operate. Conversely, a small plant costs less to construct and operate, but it also produces less energy. Based on this, the best method of optimizing the project economics is to compare energy production costs in terms of a value per kilowatt hour (kWh). As an example, assuming all other economics are the same, two plants can be compared as follows:

- 1st Plant. A 50 kW plant that produces 263,000 kWh of energy per year at 60% plant factor (see Subsection 3.7) at an annual cost of $10,000 per year (including principal, interest, and operating and maintenance costs, as discussed in Section 7).

$$\text{Costs per kWh} = \frac{10,000}{263,000} = \$0.038 \quad .$$

- 2nd Plant. A 60 kW plant that produces 315,000 kWh of energy per year at 60% plant factor (see Subsection 3.7) at an annual cost of $15,000 per year (including principal, interest, and operating and maintenance costs, as discussed in Section 7).

$$\text{Costs per kWh} = \frac{15,000}{315,000} = \$0.0476 \quad .$$

If you can sell your power for $0.055 per kWh, the 1st plant will return 263,000 x (0.055 - 0.038) = $4471 per year, and the 2nd plant will return 315,000 x (0.055 - 0.0476) = $2331 per year. In this example, the smaller site represents the best investment for the developer since the profit margin is higher.

One method of determining the design capacity to produce the maximum amount of energy requires the use of a flow duration curve. It is recommended that the Category 2 developer rely on the turbine-generator manufacturer for the detailed energy production analysis and the costs for the economics.

The Category 2 developer can make a preliminary evaluation of the best design capacity and economics using the following "rule-of-thumb" method. The flow (Q) in the design capacity equation should be based either on flow at 25% exceedance on the flow duration curve or on the average annual flow, unless there are periods of zero flow. In that case, use the average during flow periods.

### 3.6.1 Head Fixed and Flow Known; Calculate Design Capacity

For many Category 2 developers, the head at the site will be fixed and cannot be altered because an existing dam is used. Given this fixed head and a known flow, the design capacity can be determined using Equation 2.2 as the basic equation and solving for $P_d$.

$$P_d = \frac{Q \times h \times e}{11.81}$$

where

| | | |
|---|---|---|
| $P_d$ | = | design capacity in kW |
| $Q$ | = | flow in cfs. Use the average annual flow or flow at 25% exceedance, whichever is greater |
| $h$ | = | head in ft |
| $e$ | = | efficiency (assumed to be 60%) |
| 11.81 | = | constant of conversion |

This preliminary design capacity should be used as a guide, since the type of turbine and the site characteristics will also affect the design capacity and economics. At this point, the turbine-generator manufacturer should be contacted as outlined in Subsection 4.2

### 3.6.2 Variable Head and Known Flow; Calculate Design Capacity

If the head at a potential microhydropower site is not fixed, and the objective is to produce the maximum amount of energy possible, then the head should be set as high as possible. The head ranges can be determined during the site inspection as discussed in Subsection 3.2. However, there are several items to consider when determining the best location for the intake structure, which is the point from which head is measured.

Items to consider when maximizing head for a Category 2 development:

- Generally, as the head increases, the cost of the turbine-generator equipment decreases.
- To gain additional head, additional penstock is required, which increases costs because of increased length and extra design requirements for pressure and safety.
- Site access and terrain may restrict additional head.
- Head affects the type of turbine that can be used. See Subsection 4.1.
- Addtional head may provide an installed capacity of greater than 100 kW, which may change the licensing requirements. See Section 8.

The Category 2 developer should select several heads on the basis of the site inspection and the considerations in this section and develop a range of heads for additional review.

Once the head ranges have been determined, then the design capacity can be determined from Equation (2-2). Calculate the design capacity for each head.

$$P_d = \frac{Q \times h \times e}{11.81}$$

where

$P_d$ = design capacity in kW

$Q$ = flow in cfs. Use the average flow or flow at 25% exceedance, whichever is greater

$h$ = head in ft

$e$ = efficiency (assumed to be 60%)

11.81 = constant of conversion

This preliminary design capacity should be used as a guide to contact the turbine-generator manufacturers, as described in Subsection 4.2

## 3.7 Determining Annual Energy

The energy potential is represented by the installed capacity operating for a period of time. This energy term is given in kilowatt-hours (kWh). If a power plant could operate continuously, the amount of energy produced in a year's time would be as follows:

Design capacity = kW x 8760 x 24 hr/day x 365 days/yr.

However, due to normal fluctuations in stream flow, high- and low-flow limitations on the turbine, and maintenance and down time, the plant will not operate at 100% capacity continuously. Therefore, a plant factor, which is the ratio of the average annual power production to the installed capacity of the plant must be introduced to estimate the average annual energy.

$$P_F = \frac{P_a}{P_d} \times 100 \qquad (3\text{-}15)$$

where

$P_F$ = plant factor, as a percentage

$P_a$ = average annual power production in kW

$P_d$ = design capacity in kW.

For example, in the Northeast, a plant factor of 50% to 60% has been found acceptable for small plants, while in the Northwest, a plant factor of 30% to 40% may be more practical. A plant factor of 50% is a good average to use for preliminary calculations. The estimated annual energy can be determined as follows:

$$E_A = P_F \times P_d \times 8760 \qquad (\text{Eq. } 3\text{-}16)$$

where

$E_A$ = annual energy in kWh

$P_F$ = plant factor expressed as a decimal

$P_d$ = plant design capacity in kWh

8760 = hours per year (24 hr/day x 365 days/yr)

Depending on the category of developer and the site characteristics, larger and smaller plant factors can be used in the annual energy calculation. Table 3-5 can be used to refine the plant factor.

TABLE 3-5. PLANT FACTOR BASED ON SITE CHARACTERISTICS

| | Type of Developer | |
|---|---|---|
| Site Characteristics | Category 1 | Category 2 |
| Constant head and flow | | |
| Does not vary more than 5% | 0.9 | 0.9 |
| Constant head and variable flow | | |
| Varies less than 30% | 0.8 | 0.8 |
| Varies between 30 and 50% | 0.6 | 0.6 |
| Varies more than 50% | a | 0.4 |
| Variable head and flow | | |
| Varies less than 30% | 0.5 | 0.5 |
| Varies between 30 and 50% | 0.3 | 0.3 |

a. The Category 1 developer sizes the installed capacity to match the lowest flow, and therefore the flow should not vary more than 50%.

In order to use this table, you must be able to calculate head and flow variation. These variations can be calculated as follows:

$$\delta Q = \left(1 - \frac{Q_l}{Q_d}\right) \times 100 \qquad (3\text{-}17)$$

where

$\delta Q$ = flow variation, expressed as a percent

$Q_l$ = flow during low-flow period, in cfs

$Q_d$ = design flow used for the installed capacity, in cfs

NOTE: Perform calculations in ( ) first.

$$\delta h = \left(1 - \frac{h_l}{Q_h}\right) \times 100 \qquad (3\text{-}18)$$

where

| | | |
|---|---|---|
| $\delta h$ | = | head variation, expressed as a percent |
| $h_l$ | = | head at the lowest point, in feet |
| $h_d$ | = | design head used for the installed capacity, in feet |

NOTE: Perform calculations in ( ) first.

EXAMPLE: Assuming a design flow of 10 cfs with a low flow of 8 cfs, and a design head of 100 feet with a low head of 96 feet, determine the plant factor.

$$\delta Q = \left(1 - \frac{8}{10}\right) \times 100$$

$$= (1 - 0.8) \times 100$$

$$= 0.2 \times 100$$

$$= 20\%$$

$$\delta h = \left(1 - \frac{96}{100}\right) \times 100$$

$$= (1 - 0.96) \times 100$$

$$= 0.04 \times 100$$

$$= 4\%$$

$P_F$ = 0.8, based on constant head and variable flow of less than 30%, from Table 3-5.

# 4. DESIGN, EQUIPMENT, AND SAFETY REQUIREMENTS

In this section, the following subjects are addressed.

- General discussion on turbines
- Contacting turbine generator manufacturers and suppliers
- Making a go/no-go decision and establishing design criteria
- Designing an intake system
- Designing penstock and valves
- Designing a powerhouse
- Designing a tailrace
- Selecting a generator and designing electrical equipment
- Designing a drive system and speed increasers.

The developer who has followed the instructions in the previous sections has accomplished the following:

- Has identified with the appropriate developer category.
  - Desires to be energy independent producing power for personal needs.
  - Desires to produce the most power for the dollar invested.
- Has identified the type of source.
  - Run-of-the-stream
  - Manmade.

- Has selected preliminary design head, or knows the head variation.
- Has measured flow and has selected a preliminary design flow.
- Has determined the power requirements.
- Has preliminary selection for location of intake structure and power house.
- Has measured length of penstock and transmission lines based on preliminary locations.

With this information, the next step is to contact various manufacturers and request additional information. A general discussion on turbines is provided next to aid in selecting the manufacturer whose turbines have the best potential of meeting the site criteria.

## 4.1 Turbines

This subsection presents a general discussion on the types of turbines, their areas of use, methods of regulating turbine speed, and design of draft tubes.

Hydraulic turbines are classified as impulse turbines or reaction turbines according to the process by which the water head and flow are converted to mechanical power. In impulse turbines, the head is converted to velocity in a stationary nozzle directed toward the turbine wheel, called a runner (Figure 4.1-1). The water jet from the nozzle is directed against curved buckets on the runner, which deflect the jet and reduce its velocity. The force resulting from this deflection is applied to the turbine runner, creating the turbine torque and power.

In reaction turbines, part of the available head may be converted to velocity within stationary parts of the casing, and the remainder or all of the head is converted to velocity within the turbine runner (Figure 4.1-2). The forces resulting from the velocity change act on the turbine runner, creating torque and power. In most cases, the impulse and reaction turbines in use today are the descendants of designs named after their inventors. Several of the more common types of hydraulic turbines and their areas of application are described below. Impulse turbines are used for higher head sites, usually with more than 60 feet of head. Reaction turbines are more appropriate for lower head sites.

### 4.1.1 Impulse Turbines

Impulse turbines are most suited for sites with relatively high head and low flow. This is because the high head and corresponding high water velocity concentrates the available water power into a small flow area. The concentrated power is most efficiently converted by directing one or more water jets against buckets on the runner. The runner deflects the jet and reduces its velocity. The best efficiency in impulse turbines, occurs when the speed of the runner is about 1/2 that of the water jet as it leaves the nozzle.

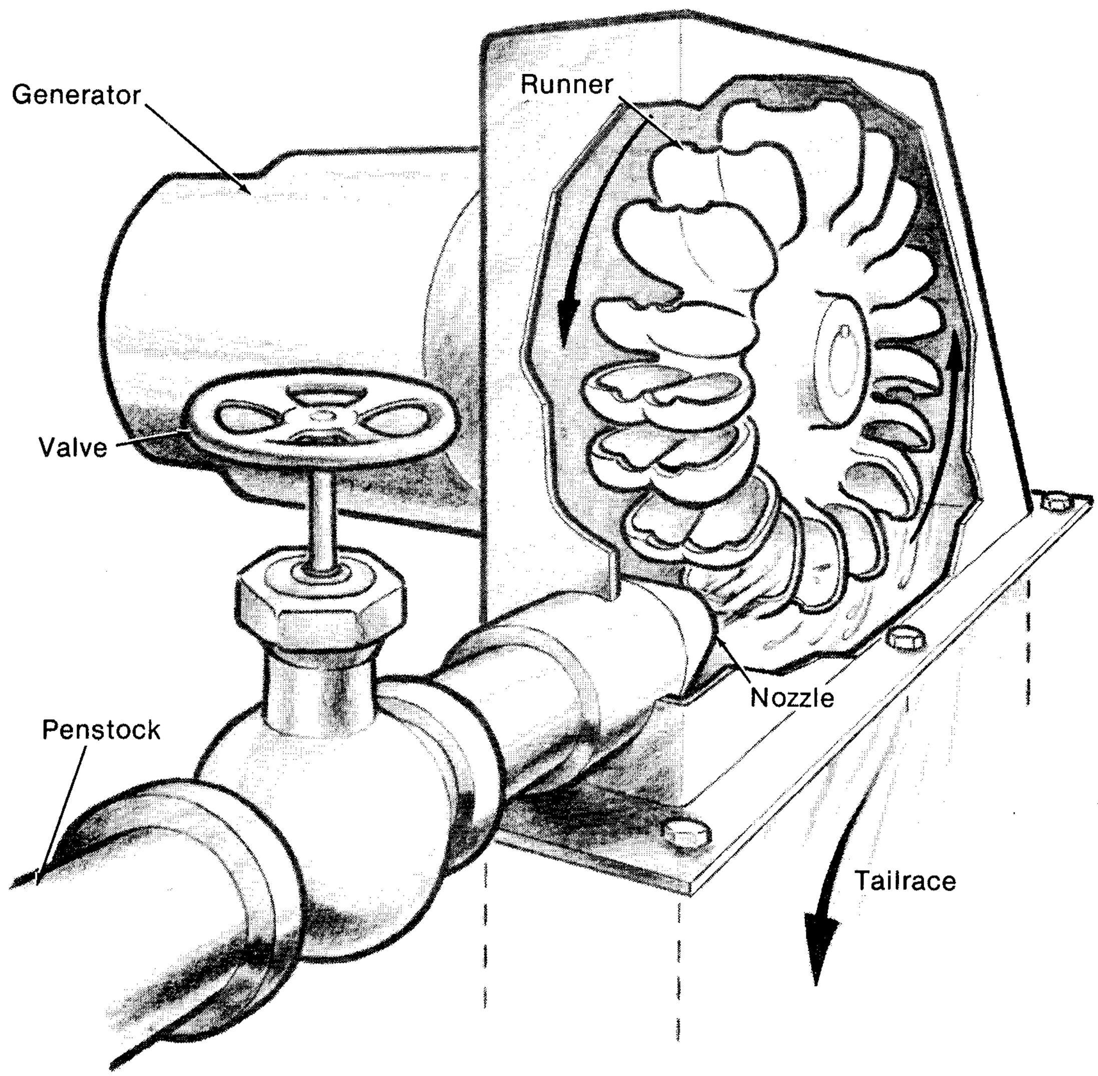



Figure 4.1-1. Impulse turbine (Pelton Wheel)

An advantage of impulse turbines over the reaction turbines is that since the head is converted to velocity in the stationary nozzles, there is no pressure drop across the runner. Consequently, no close-clearance seals are needed between the runner and the turbine housing. This makes the impulse turbines simpler to manufacture and maintain, and more tolerant of less-than-clean water conditions.

Impulse turbines are manufactered in three basic types: Pelton Wheel Turgo, and Crossflow. Each type is discussed below.

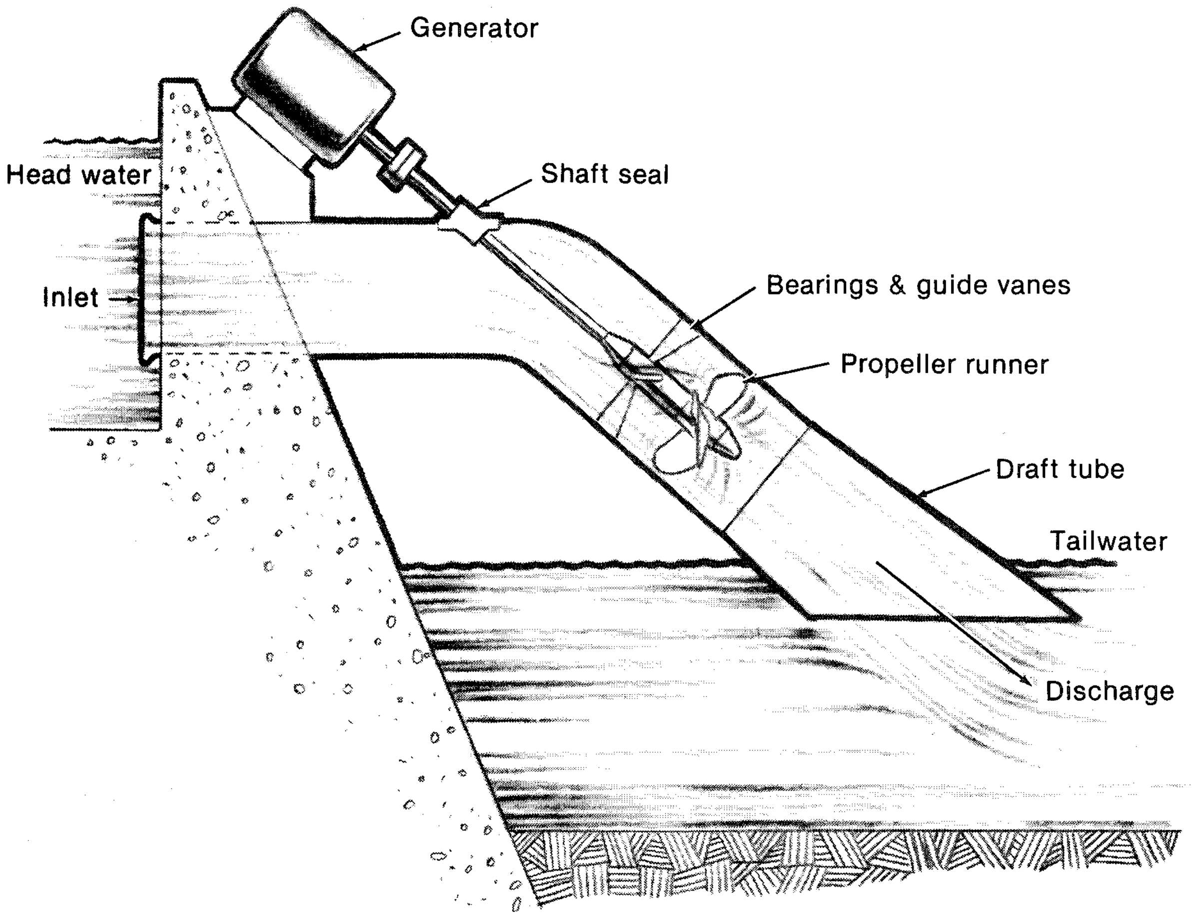



Figure 4.1-2. Reaction Turbine

4.1.1.1 Pelton Wheel Turbine. The best known impulse-type turbine is called the Pelton Wheel turbine, named after one of its inventors. This turbine, shown in Figure 4.1-1, has buckets on the runner that split the flow from the nozzle into two streams that are discharged from the sides of the runner. After the flow is diverted and split, the water drains from the turbine casing at a low velocity. An inherent limitation on the flow rate that a Pelton Wheel can handle is the size of water jet that can be efficiently diverted by the runner buckets. Several jets can be employed around the periphery of the runner to increase power. Under optimum conditions, a Pelton turbine can achieve up to 90% efficiency, due to the simple flow path through this type of turbine.

Application Guidelines--Pelton Turbines:

Head: 75 feet of head and up

Flow: Varies, but lowest of all turbines relative to head

Cost: \$300 to \$500/kW on suitable site. Cost per unit of output will decline as head increases. Peltons are uneconomic at low heads because limited water handling restricts output.

4.1.1.2 Crossflow Turbine. The crossflow (sometimes referred to as Banki) impulse turbine was invented to accommodate larger water flows and lower heads than the Pelton Wheel turbine. The crossflow turbine, shown in Figure 4.1-3, uses an elongated, rectangular-section nozzle directed against curved vanes on a cylindrically shaped runner. The water jet is slowed down in two stages, encountering the runner vanes twice as it passes through the horizontal runner. The elongated design of the runner and inlet nozzle increases the turbine flow capacity, which permits accommodation to lower heads. However, the more complex flow path through the crossflow turbine results in a lower efficiency, about 65%.

Application Guidelines--Cross-flow Turbines:

Head: 25 to 200 feet

Flow: Can be built to accommodate a wide range of flow as needed

Cost: \$500 to \$1,200/kW. Price varies with flow requirements, control systems used, and level of quality.

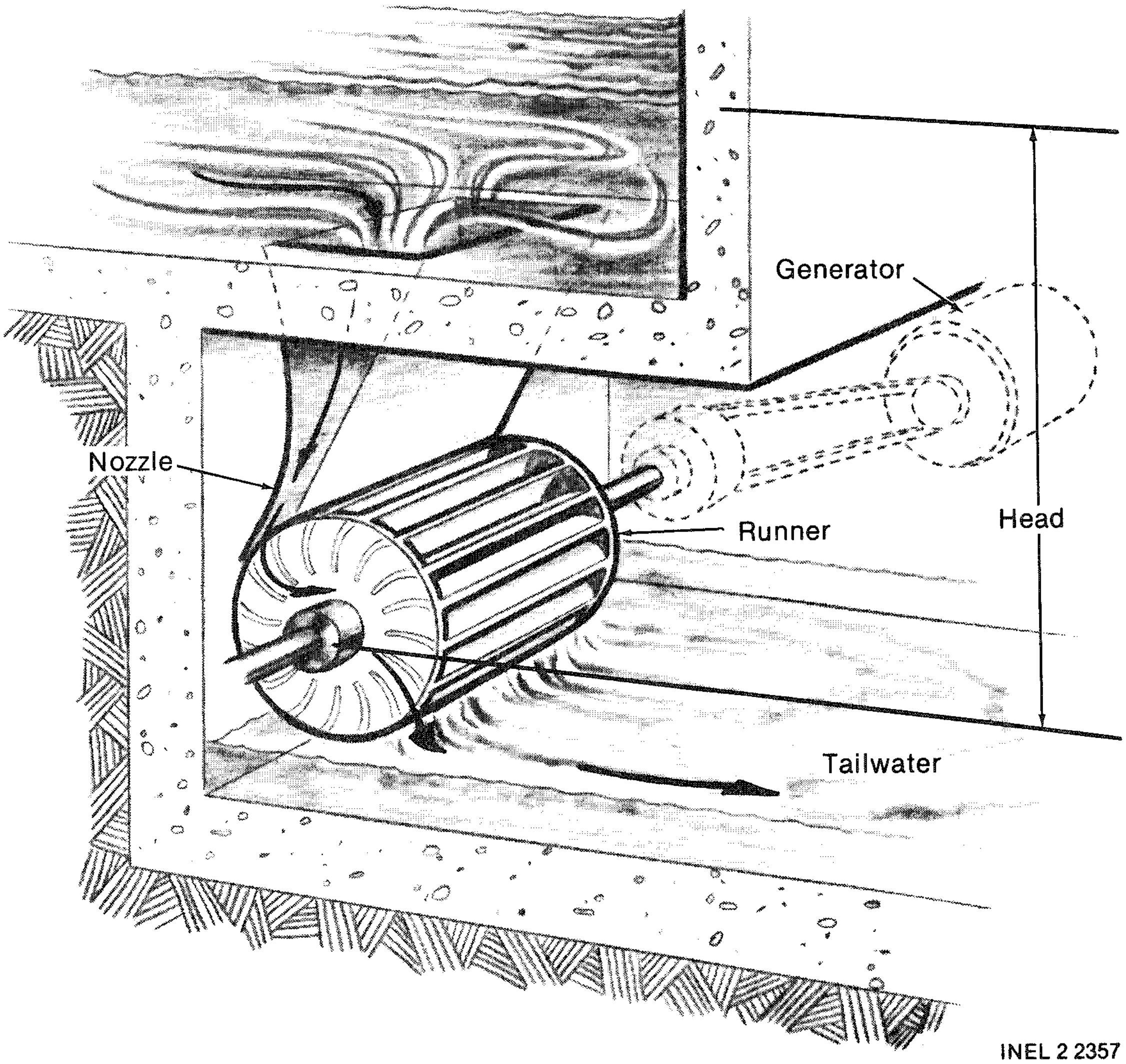



Figure 4.1-3. Crossflow turbine

4.1.1.3 Turgo Impulse Turbine. The Turgo impulse turbine, shown in Figure 4.1-4, is an impulse turbine that can accommodate more water flow than a Pelton turbine. More and larger nozzles can be placed around the circumference of the runner due to the flow orientation away from the nozzles. An additional advantage of the Turgo turbine is that for the same head and runner diameter, the speed is about twice that of the Pelton turbine. The Turgo can achieve efficiencies of 92%, and maintains high efficiencies with flows as low as 25% of design.

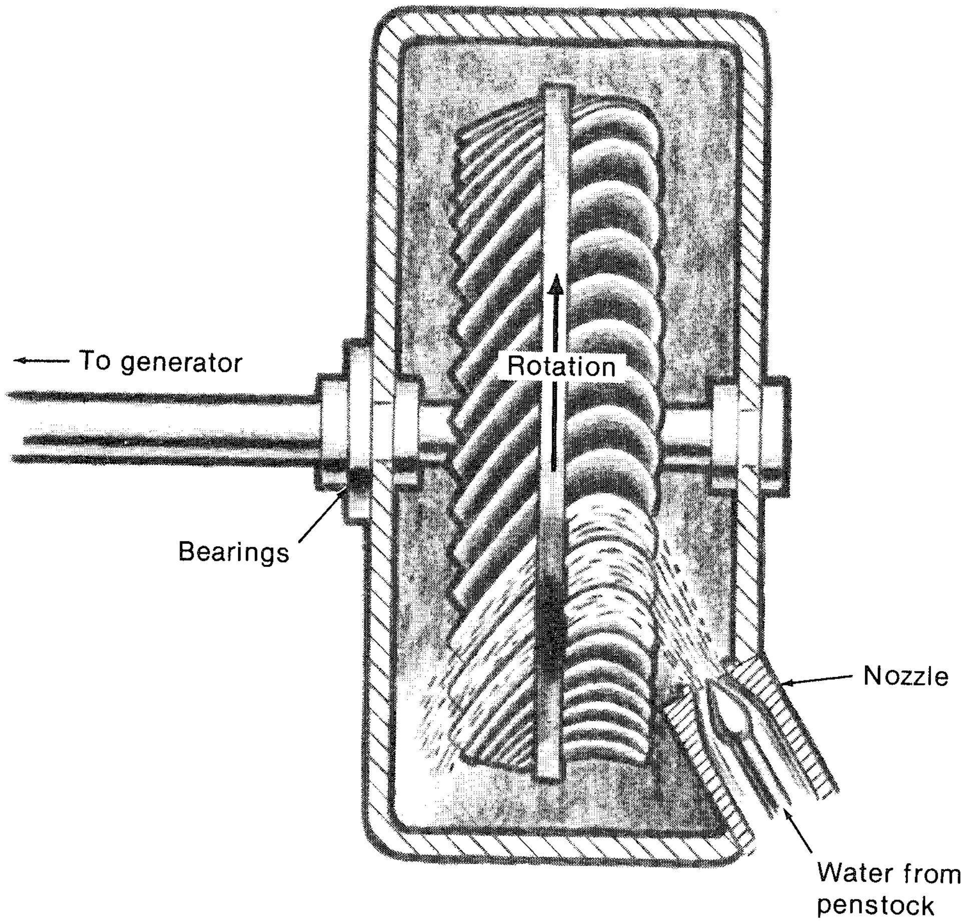



Figure 4.1-4. Turgo impulse turbine

Application Guidelines--Turgo Turbines:

Head: Comparable to Pelton--75 feet and up

Flow: For the same size runner, the Turgo will handle three times more volume than the Pelton. Also, for equal size flow, runner can be smaller and speed will be slightly more than twice that of the Pelton runner.

Cost: \$500 to \$700/kW. As with the Pelton, economics of the turbine improve with increased head.

### 4.1.2 Reaction Turbines

Turbines in which part or all of the head is converted to velocity within the runner are referred to as reaction turbines. The Francis turbine shown in Figure 4.1-5, named after its developer, is a reaction turbine. Propeller-type turbines are also reaction turbines that develop torque and power by imparting a whirl velocity to the water. A typical example is shown in Figure 4.1-2. Reaction turbines are more suitable than impulse turbines for lower head, higher flow applications, although there is considerable overlap in practical applications. In terms of mechanical design, an important feature of reaction turbines is that to maintain good efficiency, a close running clearance seal must be maintained between the runner and the casing. This is because reaction turbines operate with the

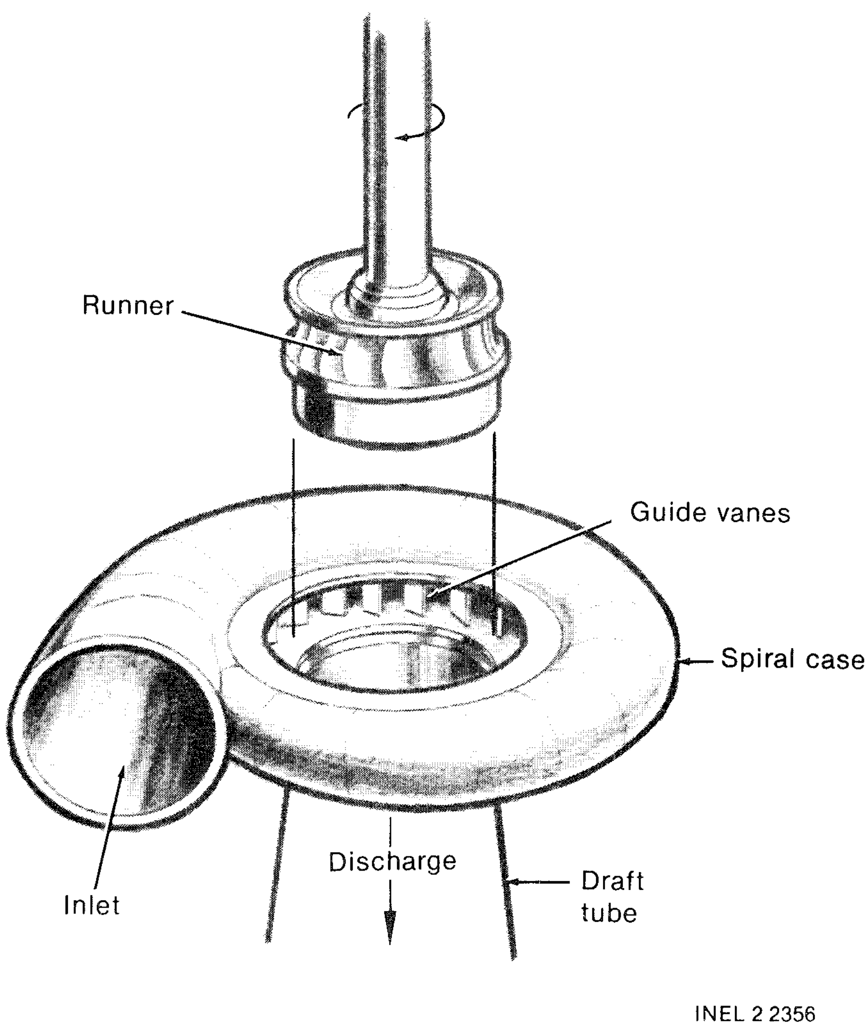



Figure 4.1-5. Francis reaction turbine

head applied across the runner, and leakage past the runner is lost power. For this reason, the performance and efficiency of reaction turbines is more likely to be degraded by entrained sand and silt in the water causing seal wear than is that of an impulse-type turbine. However, for low head applications, reaction turbines offer smaller turbine diameters and higher rotational speed than traditional impulse turbines. This advantage of a smaller runner for a given flow is offset by the fact that more flow is needed because of the lower head.

4.1.2.1 Francis Turbines. This design was first developed in the late 19th century. The Francis turbine has seen wide acceptance and is used in a full range of head and flow characteristics. Being a reaction turbine, the Francis uses both pressure and velocity to operate. Water is introduced radially--perpendicular to the shaft--at the entrance of the runner and turns 90 degrees within the runner to discharge axially--parallel to the shaft (Figure 4.1-5).

The flow is generally controlled by wicket gates. There are usually 12 to 24 wicket gates, and they are connected, by links to a gate ring to move in a coordinated fashion. The gates control flow and, alter the angle of that flow into the runner. The water in most modern Francis units is distributed to the gates and turbine via the spiral case. Note that the cross-section of the spiral case decreases as it moves around the runner because of the smaller volume of water. Not all spiral cases are shaped like this. It was common in earlier days to place the Francis turbine in the bottom of an open flume or box (Figure 2-12).

Francis turbines can be placed either horizontally or vertically and can be used on heads of 6 to 1,000 feet. Francis turbines can provide very good efficiency down to a flow of 50% of the design flow. For reasons relating to specialized design, and thus cost, Francis units have not been widely used in microhydropower installations in recent years.

Application Guidelines--Francis Turbines:

Head: 6 to 1000 feet

Flow: Design to suit--high volume with medium speed

Cost: $500 to $1,500/kW.

4.1.2.2 Propeller Turbines. There is a wide variety of turbine designs that have in common the use of a propeller-shaped runner. Only a few are applicable to microhydropower projects. Propeller turbines are reaction turbines, and most are axial flow, meaning that the water flow path is parallel to the turbine shaft. The runner resembles a boat propeller, although the two are in fact quite different. A boat propeller does not run inside a pressure casing, but a turbine runner does. Some people have successfully modified boat propellers for use as runner. The modifications usually consist of cutting off the curved end of the blade. Efficiency in the 50% range is not uncommon.

As with the Francis runners, this reaction turbine is full of water from the start of the penstock to the end of the draft tube. The runner rotates and power is extracted by the blade displacing water as the column of water moves through the turbine. Units designed for higher heads will have more blades while these used on lower heads, will have fewer blades. Blades on low head units will be set at a greater angle from the flow direction, while blades on high head designs will be set at a reduced angle.

Some propeller designs make use of preswirl vanes set upstream from the runner (Figure 4.1-2). The vanes give a tangential component to the column of water that increases the efficiency of the runner.

Fixed-blade propeller designs offer very good efficiency and high specific speed over a fairly narrow range of flow. Generally, as flow drops off, efficiency falls rapidly. The solution is to make the turbine adjustable in some way. Some designs adjust the angle of the guide vanes, some the blade angle of the runner, and some both. This adjustability is reflected in price increases. There are several types of turbines in the microhydropower range that use a simple fixed pitch propeller type runner. Fixed pitch units perform well at the design conditions, but suffer at other flow points. They are thus suitable for sites that offer constant flow conditions. Also, fixed pitch units often cost considerably less than the adjustable units.

Application Guidelines--Axial Flow (Propeller) Turbines:

Head: 6 to 100 feet

Flow: Design to suit--high volume with high speed

Cost: \$500 to \$1,500/kW. Low head will cost more but civil works are often less expensive.

### 4.1.3 Pumps Used as Turbines

When the flow in a centrifugal pump is reversed by applying head to the discharge nozzle, the pump becomes a hydraulic turbine. Pumps are usually manufactured in larger quantities and may offer a significant cost advantage over a hydraulic turbine. The potential advantage of using a pump as a turbine should be carefully evaluated by comparing cost, operating efficiency, and the value of the electric power produced with the same values for a traditional hydraulic turbine under the same head and flow conditions.

When a pump is used as a turbine, to operate at the rated pump speed, the operating head and flow rate must be increased over the rated head and flow rate for normal pumping operation. A common error in selecting a pump for use as a turbine is to use the turbine design conditions in choosing a pump from a catalog. Because pump catalog performance curves describe pump duty, not turbine duty, the result is an oversized unit that fails to work properly.

Since turbine performance curves for pumps are rarely available, you must use manufacturer's correction factors that relate turbine performance with pump performance at the best efficiency points. For pumps with specific speeds up to about 3500, these factors vary from 1.1 to 2.5 for head and flow and from 0.90 to 0.99 for efficiency (for a discussion of specific speed, see Appendix A-7). At this point, you should know your site's head and flow from worked performed in Section 3. These values are the turbine performance characteristics and must be converted to pump characteristics in order to properly select a pump. This is done as follows:

$$Q_p = \frac{Q_t}{C_Q}$$

$$H_p = \frac{H_t}{C_h}$$

$$e_t = e_p \times C_E$$

where

$Q_p$ = capacity of the pump in gpm

$Q_t$ = capacity of the turbine in gpm (site flow)

$C_Q$ = capacity correction factor

$H_p$ = head of the pump in feet

$H_t$ = net effective head of the turbine in feet (site net effective head)

$C_h$ = head correction factor

$e_t$ = turbine efficiency at best efficiency point

$e_p$ = pump efficiency at best efficiency point

$C_E$ = efficiency correction factor.

Note that the head used for the turbine (site head) is the net effective head and not the pool-to-pool head (see Subsection 2.2). You will have to size your penstock (see Subsection 4.5) and do preliminary design work on your intake structure (see Subsection 4.4) before you can calculate the net effective head.

Once you have determined $Q_p$ and $H_p$, you can review manufacturer's pump curves and select a pump that has these characteristics at best efficiency and operates at the desired speed.

In most cases, pump manufacturers treat correction factors as proprietary data. When these factors are not available, you will have to contact the pump manufacturer and supply head, capacity, and speed data so that he can select the proper pump.

Stepanoff[a] gives a method for approximating the transformation of pump characteristics to turbine characteristics. His analysis assumes that the efficiency as a turbine is approximately equal to that obtained when operating in the pumping mode. The correction factors are:

$$C_Q = \frac{1}{e_p}$$

$$C_h = \frac{1}{e_p^{\,2}}$$

$$e_p = e_t$$

a. A. J. Stepanoff, Centrifugal and Axial Flow Pumps, 2nd Edition, John Wiley and Sons, Inc., New York, 1957.

This method provides a very rough approximation. It is known from tests that different pumps operating as turbines have operated at higher and lower efficiencies than the best efficiency of the pumping mode. It appears that the wide variations in pump geometry affect some performance characterisitc while leaving others relatively unaffected. The end result is that relationships between pump performance and turbine performance of pumps are difficult to correlate in generalized formulas. You should contact the manufacturer if you are serious about using a pump as a turbine.

Since pumps are not specifically designed for reversed flow or for coupling with generators, consideration must be given to determining if the pump and generator bearings can support the reversed loads. This is particularly important in the case of vertical shaft pumps, which normally transfer their shaft weight and hydraulic thrust load to a thrust bearing in the drive motor. In this case, the generator must be designed for vertical mounting and have a thrust bearing capable of supporting the thrust loads. The pump manufacturer or a consulting engineer must be contacted to estimate vertical pump shaft loads when a pump is operated as a turbine.

Another possibility is to use a vertical shaft pump with a 90-degree gear box (Figure 4.1-6). The photo is of a 50-kW plant using a spring flow of 7.1 cfs and 140 feet of head. The plant was originally built to power the equipment in a gravel pit without an intertie to the local utility. Total plant costs were approximately $40,000. It is capable of producing power valued at more than $1,000 per month. The right-angle gear case is the type normally used in a well-pump installation. The generator was obtained used from a Caterpillar motor-generator plant.

The range of flowrate over which a pump can provide efficient turbine performance at constant speed will usually be more limited than that of a hydraulic turbine. This is because pumps have no provision for a regulating or diversion valve in their discharge (inlet for turbine operation) flow passage. Hydraulic turbines, on the other hand, have flow regulating wicket gates designed to restrict the flow. Obviously, it would be very difficult and expensive to modify a pump casing to install a

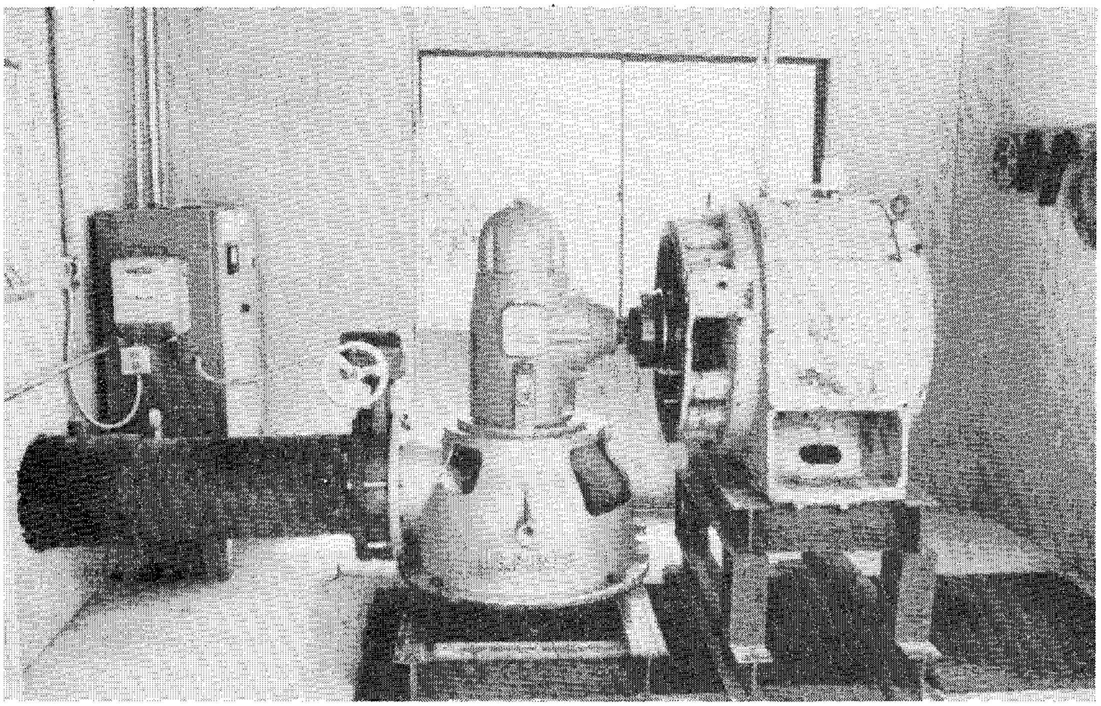


Figure 4.1-6. Vertical shaft pump used as a turbine, with 90-degree gear box.

regulating valve. A throttling valve installed upstream from the pump would not serve the same purpose, because the velocity produced at the valve would be dissipated in the piping and be unavailable for producing power in the pump (turbine). Using a pump as a hydraulic turbine should be restricted to situations where the flow is constant.

### 4.1.4 Turbine Application

The best way of obtaining the most efficient and reliable turbine, generator, controls, and auxiliary equipment is to obtain a preengineered package from a competent, experienced supply firm (see Subsection 4.2). The supply firm, given the site data and user requirements, should have the engineering capability and practical experience to select and assemble compatible equipment. Since a given supplier generally does not have knowledge of or access to all the suitable turbines that may be available, obtain proposals and quotations from several suppliers. Provide each supplier with the same site data and power requirements. The considerations involved in selecting a turbine for a given site are outlined below. This information is provided as a guide in turbine selection, and must not be considered a substitute for the detailed engineering needed to support a high-quality hydropower installation.

Particular combinations of site head and flow dictate the type of turbine that will efficiently produce power. For conditions where different types of turbines overlap, the selection process should be based on a comparison of equipment costs and performance quotations from competing manufacturers of several suitable turbines. In general, the turbine offering the highest shaft speed for the given head and flow should result in the lowest equipment cost.

If the head and flow allow the use of either impulse or reaction type turbines, the selection should be based on an evaluation of the following factors:

- If the water is sand or silt laden, an impulse turbine should be favored to avoid performance loss due to wear in the reaction turbine seals.
- If the turbine must be located some height above the tailwater level, a reaction turbine with a draft tube at the outlet should be favored to make use of the maximum head available (see Subsection 4.1.7).
- If the head and flow rate can be maintained relatively constant (which should be the case for most Category 1 developers), using a pump with reverse flow as a turbine should be considered since the initial cost and availability may be advantageous.
- A turbine that turns fast enough for direct coupling to the generator shaft should result in a more compact installation and less long-term maintenance than one coupled through drive belts or a transmission.

If the head and flow conditions indicate that a Pelton-type impulse turbine is most suitable, the tradeoffs between turbine size, speed, cost, and efficiency can be observed by comparing manufacturers' quotations for the turbine unit, drive system (direct coupling, V-belts, gear box, etc.), and generator. If the available water flow varies significantly over the period of time that power is needed, the use of a spear type regulating valve (see Subsection 4.5.5.5) built into the turbine nozzle should be evaluated in terms of cost and efficiency gains. Valving the flow to several nozzles may provide adequate flow and power control.

If the flow rate is at the upper end of the impulse turbine range, the crossflow (Banki) or Turgo-type impulse turbines should be evaluated. They offer higher speed than the Pelton Wheel, handle more flow, and do not require the close running seals needed by the Francis turbine and other reaction-type turbines. The crossflow turbine will be of particular interest to the individual who is capable of designing and building a turbine rather than purchasing a manufactured unit. The runner blades on a crossflow turbine have only cylindrical curvature and can be fabricated from sectors cut from common steel pipe. C. A. Mockmore and F. Merryfield[a] present the hydraulic theory needed to correctly design a crossflow turbine.

If the site conditions of relatively low head and high flow rate are suitable only for a reaction-type turbine, the choice is between the Francis turbine or a propeller-type turbine. Both of these types are available with movable gates to maintain good performance over a range of flow rates. As mentioned before, the Francis turbine uses movable inlet flow wicket gates, while the propeller turbine may use variable pitch runners or gates to adjust to changing flow conditions. The cost of controlled position runners and gates in these turbines is generally too high to make them feasible for microhydropower installations. Fixed geometry versions of the Francis and propeller types offer good performance over a limited range of flowrates.

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a. C. A. Mockmore and F. Merryfield, "The Banki Water Turbine," Oregon State College, Bulletin Series No. 25, February 1949.

### 4.1.5 Regulating Turbine Speed

With only a few exceptions, the electric power generated by a hydropower installation must be regulated to a frequency of 60 cycles per second (Hz) to be useful in powering motors and appliances. Notable exceptions would be using a turbine for producing mechanical power to drive equipment directly, and electric generation for the purpose of space or water heating, or absorption refrigeration.

Maintaining constant electric power frequency and voltage requires that the turbine be operated at constant speed. In commercial hydropower installations, turbine speed control is performed by a governor that senses generator frequency and positions the turbine nozzle spear valve, wicket gates, or runner blade angle to maintain 60-Hz power. These methods of speed control are designed to maintain a high turbine efficiency over a wide range of flows and corresponding power output. The cost of conventional governor mechanisms may well be prohibitive for microhydropower installations.

It is possible to operate a microhydropower installation interconnected to a power utility without benefit of a speed governor. Such installations are manually brought on line, and once set, they will supply to the utility the amount of power that corresponds to the hydro energy supplied to the turbine. Speed control of such an installation is inherently maintained, for normal operating limits, by the natural characteristics of the system after it is connected to the power utility. The operation can continue indefinitely, provided that variations in water flow and head stay within reasonable limits. In effect, the power utility is providing speed control for this mode of operation.

If the power plant is to be operated without a connection to a power utility, as in a Category 1 installation, then some form of speed regulation is probably a necessity.

A less costly method of speed regulation for small turbines and pumps used as turbines is to use an electronic load control device. Electronic load control allows stand-alone generation of regulated 60-Hz power. These devices sense the power frequency and adjust a variable fraction of the electric load to maintain the turbine-generator speed constant at 60 Hz. The range of power output that is varied to maintain constant turbine speed depends on the amount of water power that is available, and the variations in electric load demanded from the unit. These factors must be defined and analyzed by a supplier of electronic regulation units to determine if this method is feasible and economic for your installation. The power dissipated by the controller to maintain constant speed can be put to use for space or water heating, absorption type refrigeration, or rejected as heat by resistors placed in the turbine water flow stream.

These methods are explained in more detail in Subsection 4.8.

The conventional speed regulator discussed at the beginning of this subsection is a mechanical governor that controls inlet water flow to the turbine. A relatively recent development is the use of electronic speed sensors and microprocessors to control inlet water flow. This type of speed regulation should be more economical than the conventional governor system but may be difficult to purchase from conventional equipment suppliers. Some microhydropower developers have been very successful using innovative methods of speed control.

### 4.1.6 Turbine Setting

The setting of a reaction turbine in relation to the minimum tailwater elevation can have a significant impact on the life of the turbine. Improper turbine setting can lead to the phenomenon known as cavitation, which results in pitting of the runner. In reaction turbines, reduced pressures occur in the hydraulic passages as the fluid is accelerated to high velocities, and vapor bubbles form in the flowing stream. When these bubbles are then carried into a region of higher pressure, they can collapse rapidly. If this collapse occurs adjacent to the runner surface, it results in the removal of a small amount of the metal, and this process, if allowed to continue, accelerates with time. Thus, the cavitation that causes this type of damage to a turbine is to be avoided.

Excessive cavitation damage can be avoided by setting the horizontal centerline of the turbine's runner a specified distance above or below the tailwater elevation. The correct distance should be supplied by the turbine manufacturer and should be closely adhered to. From an equipment standpoint, a deep setting is better because it provides sufficient pressure at the runner discharge to allow the use of smaller, higher speed turbines, and therefore lower cost units, without excessive cavitation. The civil costs will be greater with the deeper setting because of additional excavation and more powerhouse work. A balance should be maintained between the civil costs and the equipment costs as determined by the turbine setting. For a further discussion on turbine setting, see Appendix A-7.

The setting of impulse turbines in relation to the tailwater elevation is not critical for prevention of cavitation. Impulse turbines are generally located as close as possible to the tailwater elevation to use as much of the available head as is possible, but they must run free of any tailwater interference.

### 4.1.7 Draft Tube

This section discusses draft tubes for reaction turbines. If you have an impulse turbine, the contents of this section are not important to you. Reaction turbines (Francis, propeller, and pumps-as-turbines) operate with the flow path completely filled with water. This allows the turbine to be mounted above the tailwater, and still use the full available head by means of a draft tube. A draft tube is a conical pipe, straight or curved depending on the turbine installation, that maintains a column of water between the turbine outlet and the downstream water level. Water leaves the turbine runner at a relatively high velocity, constituting a substantial portion of the total energy available. To recover this energy efficiently, the velocity must be reduced gradually and friction losses minimized. If the velocity is not reduced, the water will spill out the end of the turbine outlet into the tailwater, and the energy contained in the flowing water will be dissipated as turbulence in the tailrace.

The draft tube outlet must remain below the water surface at all water levels to prevent air from entering the tube and displacing the water column. It is the velocity of the water in the draft tube that acts, when reduced, as a suction head on the turbine runner. This suction head can be enhanced by converting part of the flow velocity to pressure within thge draft tube. This requires a tube of expanding flow area, with the diameter at the tube outlet about two times the diameter at the inlet, where it attaches to the turbine. The angle between the opposite walls of the expanding draft tube should be between 7 and 20 degrees to give optimum pressure recovery. The design of the draft tube for a commercially manufactured turbine should be approved by the manufacturer.

The majority of microhydropower sites will use straight conical draft tubes, (see Figures 4.1-2 and 4.6-4). For preliminary layout purposes, the draft tube outlet diameter should be twice the turbine runner diameter and the length of the tube should be four times the runner diameter. The bottom of a vertical conical draft tube should not be closer than one runner diameter to the bottom of the tailrace. The tailrace width for a vertical conical draft tube should be four turbine runner diameters, but can be only two diameters wide for a horizontal draft tube.

There are other draft tube designs that use curved sections to reduce the amount of excavation required. These are elbow or "S" shaped and should be designed by the turbine manufacturer on the basis of his model tests.

## 4.2 Contact Turbine-Generator Manufacturers and Suppliers

Microhydropower developers will want to select a standard turbine and not undergo the expense of having a turbine specifically designed to meet the site characteristics. To select a standard turbine-generator unit that will generate the most energy for the dollar, you should contact various suppliers and request them to recommend the unit that they feel best fits your site. Use the form that follows to provide the information the manufacturer will need. Provide as much information as possible to aid the manufacturers and suppliers in determining which unit to recommend. Pictures and drawings should be included if available. Identical information should be supplied to all manufacturers and suppliers so that you can evaluate the responses fairly. A listing of manufacturers and suppliers is included as Appendix F, and additional forms can be found in Appendix I.

To aid your understanding of the form, a narrative description referring to the major headings of the form is provided below.

### I. REASON FOR DEVELOPMENT

By choosing the most appropriate category, you are telling the manufacturer or supplier why you desire to produce power. If you state that you want to be able to generate power independent of the utility, you will be able to generate power when the utility service is interrupted or even if disconnected from the utility. This tells the manufacturer or supplier that he must supply a more expensive synchronous generator, along with speed regulating equipment. (Generators are discussed in Subsection 4.8). If you state that you don't mind being dependent on the utility, then the recommended generator will probably be an induction generator, which will use a less expensive motor starter for switchgear.

## II. TYPE OF SOURCE AND AMOUNT OF HEAD

Choose the category that best describes your type of source (Subsection 2.6), and give the available head (Subsection 3.4). If you are a run-of-the-stream developer and you have a fixed head, list that head. If you are not sure you have selected the best head, list the ranges of head available so that the manufacturer or supplier can select the head that best fits his equipment.

If the site is an existing dam which has a fluctuating head (Subsection 3.5.4), describe the characteristics of the fluctuation. In the additional comments, include an explanation of how the head fluctuation corresponds to flow variations.

## III. AMOUNT OF FLOW

If you have developed a flow duration curve, be sure and include a copy of the curve. This will help the manufacturer or supplier to optimize his turbine selection. If you have estimated flow on the basis of an average monthly flow value, then include that information. If you have a source, such as a canal, that flows only during part of the year, be sure and include that information.

## IV. PERSONAL POWER NEEDS

List the results you obtained from Subsection 3.1.

## V. ADDITIONAL INFORMATION

Include any additional information that might aid the manufacturer or supplier to evaluate your site requirements. Call the utility to learn how far the nearest substation is from your site. This distance will determine whether or not an induction generator can be used.

If you are developing an existing site with a turbine already installed and you would like to consider using it again, write the original turbine manufacturer to obtain information. In addition to the site data included above, send the following information, if available:

Name of the site
Name of the original turbine purchaser
Date the turbine was purchased
Contract number
Name plate data
Drawing numbers of the turbine.

After receiving the responses back from the manufacturer or supplier, proceed to the next section to make a preliminary cost estimate, decide whether or not to proceed, and establish design criteria.

MICROHYDROPOWER TURBINE GENERATOR
INFORMATION REQUEST

______________________
(DATE)

GENTLEMEN:

I am interested in installing a microhydropower system. The following site specifications are supplied for your evaluation. Please review the specifications and answer any appropriate questions concerning your equipment.

My Name: ______________________ Address: ______________________
Phone No. (   ) ______________________ ______________________
Project Name: ______________________ ______________________

I. REASON FOR DEVELOPMENT

(Check One)

___ 1. I am interested in supplying my own electrical needs. I do not plan to intertie with a utility. Therefore, I will require a synchronous generator.

___ 2. I am interested in supplying my own electrical needs. When my needs are less than the energy produced, I would consider selling to a utility. However, I want to be able to generate power independent of a utility. I therefore require a synchronous generator and speed control equipment.

___ 3. I am interested in supplying my own electrical needs. I want to be able to sell excess power to a utility. An induction generator is acceptable since I do not care to generate power independent of the utility.

___ 4. I am interested in generating as much power as possible for the dollar invested. However, I want a synchronous generator so that I can generate power if the utility service is interrupted.

___ 5. I am interested in generating as much electrical power as possible for the dollar invested. I am not interested in generating independent of the utility.

## II. TYPE OF SOURCE AND AMOUNT OF HEAD

(Check One)

___ 1. The site is a run-of-the-stream site and can have a pool-to-pool head from __________ to __________ feet.

___ 2. The site is an existing dam and has a constant/variable pool-to-pool head of __________ to __________ feet.

___ 3. The site is a canal drop/industrial waste discharge and has a pool-to-pool head of __________ feet.

## III. AMOUNT OF FLOW

(Check One)

___ 1. The flow values are based on the attached flow duration curve.

___ 2. The flow value is based on a minimum stream flow of _____ cfs. This is because my objective is to supply my energy needs as much of the year as I can.

___ 3. The flow is available _____ months out of the year and is fairly constant at ______ cfs.

___ 4. The flow values are based on monthly averages in cfs:

| | | | | | |
|---|---|---|---|---|---|
| Jan. | __________ | May | __________ | Sept. | __________ |
| Feb. | __________ | Jun. | __________ | Oct. | __________ |
| Mar. | __________ | Jul. | __________ | Nov. | __________ |
| Apr. | __________ | Aug. | __________ | Dec. | __________ |

___ 5. Other: See V-9, Additional Information.

## IV. PERSONAL POWER NEEDS

A copy of the daily load use table is attached. The daily peak load is estimated to be ________ kW. Major electrical equipment is listed below.

| | |
|---|---|
| ______________________________ | ______________________________ |
| ______________________________ | ______________________________ |
| ______________________________ | ______________________________ |
| ______________________________ | ______________________________ |
| ______________________________ | ______________________________ |
| ______________________________ | ______________________________ |

The voltage I need is ________, and is single/three phase.

## V. ADDITIONAL INFORMATION

1. Site location and stream name ______________________
______________________________________________
______________________________________________

2. Name of local utility ______________________
Distance to nearest substation is ________ miles.

3. The quality of the water is usually clear/murky/silt laden/muddy.

4. Site elevation is ________ feet.

5. Annual average temperature variation is from __________ to ________ °F.

6. A sketch of the site is/is not included.

7. Existing structures or equipment that should be used, if possible, include ______________________
______________________________________________
______________________________________________

8. The proposed diameter and length of the penstock are (leave blank if not known): ______ inches in diameter, ______ feet in length.

9. Additional information to be considered ______________________
______________________________________________
______________________________________________
______________________________________________
______________________________________________
______________________________________________
______________________________________________

The information returned to you by the manufacturer will assist you in developing your site and preparing a final design specification (Subsection 4.3.2) or bid package (Subsection 5.1.2). The following narrative is provided to aid in understanding the information returned by the manufacturer or supplier:

I. GENERAL COMMENTS

This section allows the manufacturer to state exactly what scope of information is included in the package. It also allows him to specifically exclude equipment, make recommendations, or qualify any of the information he has included.

II. RECOMMENDED EQUIPMENT SUPPLIED BY COMPANY

This section provides a listing of the equipment, type, and manufacturer included on the form.

III. EQUIPMENT SPECIFICATION

This section provides the detailed information on each item of equipment to be furnished. The manufacturer will have made some assumptions and performed calculations in order to supply this informaion. You should study it carefully since you will have to use it in your design. The power production of the turbine-generator will be based on a net effective head (see Section 2.2) that the manufacturer has calculated from information you have given him or he has assumed. Your final design should confirm that your site can provide this head. The rating of the generator may be higher than the power needed by you. In this case, the manufacturer has probably selected a standard generator. Since generators come in specific sizes, he will pick one which is the nearest to your needs but higher.

The turbine setting (reaction turbines only), size of powerhouse, and weight of the unit are other items you will need to know in the design of your site. The annual energy production calculated by the manufacturer may differ from your calculations, but it will be a more accurate number, particularily if you have provided him with a flow duration curve and the seasonal head variation.

## IV. COST OF EQUIPMENT RECOMMENDED

The cost provided by the manufacturer will be for the equipment listed in Section II of this form. Unless a bid is attached, you should use this price only for cost estimating purposes. Before ordering the equipment, ask the manufacturer for a firm bid.

## V. ADDITIONAL INFORMATION

This section is to allow the manufacturer to offer recommendations or other services. It also provides the delivery time for the items listed on the form. This information is needed when preparing the construction schedule (see Subsection 5.1.5).

(TO BE COMPLETED BY MANUFACTURER/SUPPLIER)

Name of Company ______________________________
Address ______________________________
______________________________
______________________________

Phone No. (   ) ____________ Date ____________

I. GENERAL COMMENTS ______________________________
______________________________
______________________________
______________________________
______________________________

II. RECOMMENDED EQUIPMENT SUPPLIED BY COMPANY

1. Turbine Inlet Gate or Valve

Manufacturer ____________________
Type ____________________
Model No. ____________________

2. Turbine

Manufacturer ____________________
Type ____________________
Model No. ____________________

3. Generator

Manufacturer ____________________
Type ____________________
Model No. ____________________

4. Is load diverter/governor included? Yes/No.
Yes: Type ____________________
Model No. ____________________
No: Type recommended ____________________
Model No. ____________________

5. Additional items supplied with package, e.g. transformer, protection devices, auxiliary equipment. Provide manufacturer, dimensions, operating characterisitcs

______________________________
______________________________
______________________________
______________________________
______________________________
______________________________
______________________________

## III. EQUIPMENT SPECIFICATION

For a net effective head at the turbine of _____ feet and a flow of _____ cfs, the generator will provide an output of _____ kW (assumed power factor of _____). This will result in a computed efficiency of ____%. Based on the information provided, the annual energy production is calculated to be ________________ kWh.

1. Turbine:

   ________ rpm at recommended head and flow.
   Diameter of runner __________ and type __________.

2. Generator:

   Operating rpm ________.
   Overspeed allowance ________%
   Voltage ________ single/three phase.
   Rating _____________________ single/three phase.
   Power factor _______________

3. Speed increaser

   Type _________________
   Ratio ________________________________
   Rated input horsepower _______________
   Service factor _______________________

4. Draft tube, if used:

   Length _____________
   Elevation from turbine runner centerline to tailwater at lowest water level _____________
   Outlet area ______________
   Inlet diameter __________

5. Is unit assembled on equipment frame? Yes/No

   Yes: Dimensions of frame ___________________

6. Recommended powerhouse minimum dimensions:

   Length _______ ft
   Width ________ ft
   Height _______ ft.

7. Weight of assembled unit __________ lb
   Shipping weight ___________________ lb
   Wetted weight _____________________ lb
   Recommended mass of equipment pad _________________ lb.

8. Is shutoff valve or gate provided ahead of turbine? Yes/No

No: Recommended size ___________
Type ______________________
Manufacturer _______________.

9. Recommended spare turbine-generator parts

________________ ________________
________________ ________________
________________ ________________
________________ ________________

10. Expected operating life with normal maintenance and operating conditions: ________ yrs.

11. Turbine warranty provisions included? Yes/No

12. Is cooling water required for the generator, speed increaser, and/or lubrication system? Yes/No

Yes: Flow ________ at ________ temperature, ________ system
Flow ________ at ________ temperature, ________ system
Flow ________ at ________ temperature, ________ system

13. Recommended powerhouse ventilation ___________ cfm.

14. With the information provided, the minimum output for the unit would be __________ kW at __________ head and _________ cfs flow. The maximum output for the unit would be ________ kW at __________ head and _______ cfs flow.

15. Diameter of turbine inlet __________ in. and outlet ________ in. or outlet dimensions ___________ in. by ___________ in.

16. Lightning protection is/is not provided.

IV. COST OF EQUIPMENT RECOMMENDED

(Choose Appropriate Answer(s))

1. The cost estimate is/is not based on a complete unit cost.

2. The cost delivered to the site is _________.
Bid is/is not attached.

3. On the basis of information provided, the cost of the equipment recommended should approach __________ delivered to the site. (This is not a bid.)

4. The cost estimates are good until ________________.

V. ADDITIONAL INFORMATION

1. Recommended material and equipment not furnished by company.

   Penstock: Size ______________ Material _____________
   Valves _____________________
   Electrical equipment _________________________________
   Additional items _____________________________________
   _______________________________________________________
   _______________________________________________________

2. Delivery time for packages ___________________________

3. Recommended design considerations _______________________________
   _____________________________________________________________________
   _____________________________________________________________________

4. Additional services provided by Company (i.e., financing, complete design, installations, etc.).
   _____________________________________________________________________
   _____________________________________________________________________
   _____________________________________________________________________
   _____________________________________________________________________

5. Specific Comments ____________________________________________
   _____________________________________________________________________
   _____________________________________________________________________
   _____________________________________________________________________

6. Please provide a list of three or four developers with addresses who have installed and operated your units.
   ______________________________ ______________________________
   ______________________________ ______________________________
   ______________________________ ______________________________
   ______________________________ ______________________________
   ______________________________ ______________________________
   ______________________________ ______________________________

## 4.3 Go/No-Go Decision and Design Criteria Selection

Before proceeding, this is a logical place to make the second go/no go decision. If the decision is to proceed with the project, then the design criteria should be selected.

### 4.3.1 Go/No-Go Decision

This decision will be based on economics; therefore, you need to make a preliminary cost estimate. Evaluate the responses received from the manufacturer inquiries. The evaluation should be based on dollars for kW of installed capacity and dollars per kWh of energy production.

Category 1 developers who don't want to sell to a utility should look at the dollar for kW capacity. Remember, the higher the head the less expensive the turbine generator unit, but the more expensive the penstock. A later paragraph will tell you how to adjust for civil cost. Category 2 developers and those Category 1 developers who plan to sell to the utility should compare cost to the energy production (kWh), since the financial return is based on kWh sold to the utility. A unit that costs less per kW capacity may produce a lot less energy (kWh) and thus may not be as good a buy. Therefore, compare all responses in accordance with the procedure presented below to select the best economic alternative.

Before proceeding, look at the manufacturer's information sheets. Be sure that you are evaluating equivalent items. In other words, if one manufacturer or supplier is supplying a complete unit including governor or load controller, etc., and the other is supplying a turbine or a generator, the two costs cannot be compared without adding the additional cost to the second machine.

For a preliminary cost estimate, a rule of thumb is that the civil cost (i.e., structures, earthwork, penstock, transmission line, etc.) should be less than or equal to the machinery cost. Therefore, to make a rough estimate of construction cost, take the equipment cost and multiply by 2. If you are using an existing flume with very little civil work, the estimate can be reduced. However, if the site will require an extra long penstock (1000 feet or more), or a lot of earth work, or anything else out of the ordinary, the civil cost estimate should be increased. After adding the machinery and civil cost together, round to the nearest $1,000 for a construction cost estimate.

The following items should be added to the construction cost estimate: 10% for administration cost (legal fee, taxes, permits, etc.), and 25% contingency to cover any uncertainty that may not be known or considered in the estimate. These should be added as follows:

- Take the estimated construction cost and multiply by 10%. Add the results to the construction cost.

  EXAMPLE: Assume a 15-kW site with estimated construction cost of $31,000.

  $31,000 x 0.10 (10%) = $3,100

  Adjusted cost estimate = $31,000 + 3,100 = $34,000 (rounded to nearest $1,000).

- Take the adjusted cost estimate and multiply by 25%. Add the results to the adjusted cost estimate to determine the total preliminary cost ($C_p$).

  EXAMPLE: Adjusted cost $34,000

  $34,000 x 0.25 (25%) = $8,500

  Total preliminary cost estimate = $34,000 + $8,500 = $C_p$ = $42,000 (rounded to nearest $1,000).

The total preliminary cost estimate is now determined. Next, divide the estimated cost by the kW capacity of the site.

EXAMPLE: $42,000 preliminary cost estimate and 15 kW capacity

$$\frac{\$42,000}{15 \text{ kW}} = \$2,800 \text{ per kW.}$$

The cost of a microhydropower installation will probably range between $1,000 and $4,000 dollars per kW installed capacity. Your estimate should be in that range.

CAUTION: This is a very rough estimate. It should only be used to decide if you are willing to invest that magnitude of dollars. The final estimate may vary up or down by 25% or more. The rest of this handbook will help you find ways to reduce the cost.

Category 2 developers will want to evaluate how much revenue can be recovered from the investment. To do that, take the annual energy (kWh) value, from the manufacturers returned form (III. EQUIPMENT SPECIFICATIONS), and multiply annual energy by 30 years.

$$E_T = E_A \times 30 \qquad (4.3\text{-}1)$$

where

$E_T$ = total estimated energy over 30 years in kWh

$E_A$ = manufacturer's estimated annual energy generation in kWh

30 = 30-year economic life of the site.

EXAMPLE: Assume that the 15-kW generator will produce 65,700 kWh annually. Find the total estimated energy over a 30-year period.

From Equation (4.3-1):

$$E_T = E_A \times 30$$

$$E_T = 65{,}700 \times 30$$

$$E_T = 1{,}971{,}000 \text{ kWh.}$$

Now, if you have not already done so, contact the utility to determine how much they are willing to pay for your power. They will quote a rate in mills per kWh. A mill is one tenth of a cent, 30 mills is 3 cents. So if the utility quotes 35 mills per kWh, they are actually quoting 3.5 cents per kWh. To determine your economics, take the total estimate energy production ($E_T$) times the mill rating.

$$R_T = E_T \times M_R \qquad (4.3\text{-}2)$$

where

$R_T$ = total estimated return in dollars

$E_T$ = total estimated energy production in kWh

$M_R$ = mill rating, in dollars per kWh.

EXAMPLE: The total estimate energy was computed to be $E_T$ = 1,971,000 kWh. Assume that the utility mill rating is 35 mill per kWh, and find the total estimated return.

35 mills = 3.5 cents = $0.035.

From Equation (4.3-2):

$$R_T = E_T \times M_R$$

$$R_T = 1{,}971{,}000 \times 0.035$$

$$R_T = \$68{,}985$$

$R_T$ = \$69,000 (rounded to nearest \$1,000).

It costs money to use money. If you had \$42,000, you could invest that money and earn at least three times that much in 30 years. Likewise, if you have to borrow \$42,000, it would cost you at least 3 times that much to use the money for 30 years. Therefore, the return on your investment (total estimated return = $R_T$) should be at least 3 times the total estimated cost ($C_p$).

$$R_T = 3 \times C_p \text{ (or more)} \qquad (4.3\text{-}3)$$

where

$R_T$ = return on investment

$C_p$ = total estimated cost.

In the example, $R_T$ should be at least 3 times larger than \$42,000, or \$126,000. Since $R_T$ = \$69,000, which is considerably less than \$126,000, the preliminary economics are not favorable for the site.

Assume that the estimate of the total cost is high by 30%. Would the site be economical then?

\$42,000 x (100% - 30% = 70%)

\$42,000 x 0.70 = \$29,000

New $C_p$ = \$29,000.

From Equation (4.3-3), $R_T$ = 3 x Cp (or more), and 3 x \$29,000 = \$87,000. Since \$87,000 is still larger than \$69,000, the site is probably not an economical investment. Unless something can be done to reduce cost or increase return, the site should be considered a no-go.

Another way for Category 2 developers to take a quick look at economics is to determine what mill rate ($M_R$) will be required to break even

(Investment = Return). To do this, take the total estimated cost ($C_p$), multiply by 3, and divide by the total estimated energy.

$$M_R^1 = \frac{1000 \times 3 \times C_p}{E_T} \qquad (4.3\text{-}4)$$

where

$M_R^1$ = required mill rate to break even

1000 = constant number to convert dollars to mills

3 = adjust total cost estimate

$C_p$ = total cost estimate in \$

$E_T$ = total energy generated in kWh.

EXAMPLE: Assume that the total estimated cost was $C_p$ = \$29,000 and that the total energy generated was 1,971,000 kWh, and find the mill rate required to break even.

From Equation 4.3-4:

$$M_R^1 = \frac{1000 \times 3 \times C_p}{E_T}$$

$$= \frac{1000 \times 3 \times \$29{,}000}{1{,}971{,}000}$$

= 44 mills per kWh, or 4.4 cents per kWh to break even.

In other words, a contract with a utility would have to be negotiated for more than 44 mills per kWh.

CAUTION: The approaches presented in this subsection (4.3.1) are very crude. They should only be used for the roughest estimation of econ- omic feasibility. If the economics look halfway reasonable, it is advisable to proceed to a detailed design and cost estimate.

### 4.3.2 Design Criteria Selection

If Subsection 4.3.1 has indicated that the project is worth pursuing and that the investment capital is in a range that you can handle, now is a good time to review the financial section (Section 7.0) and formulate a plan to pursue financing for the project. Also, the turbine selection should have been narrowed down to two or three manufacturers or suppliers. Before selecting the turbine-generator, contact the developers listed at the end of the Turbine-Generator Information Request form to see how their equipment is performing and if they have encountered any unusual problems. Get as much information as you can on the equipment. In particular, does the unit produce the power (kW) it is supposed to, and is the unit reliable? After gathering all the information you can, select the turbine-generator unit on which you will base your final design criteria.

# Design Specification

1. Net effective head of __________ feet, or pool-to-pool head range from _______ to _______.

2. The design flow is __________ cfs.

3. Turbine:

   a. Manufacturer ______________________________
   b. Supplier ______________________________
      ______________________________
      ______________________________
   c. Type ______________________________
   d. Model No. ______________________________
   e. Shaft speed at design head and flow _______ rpm
   f. Diameter of turbine inlet ____________ inches
   g. Diameter or dimension of outlet ____________ inches
   h. Setting of turbine at throat ____________ feet above minimum tailwater level.

4. Speed increaser type ____________________
   ratio ____________________
   input power ______________

5. Generator:

   a. Manufacturer ______________________________
   b. Supplier ______________________________
      ______________________________
      ______________________________
      ______________________________
   c. Type ______________________________
   d. Model No. ______________________________
   e. Operating speed ______________________________
   f. Voltage ____________ phase ______________

6. Wetted weight of equipment ____________________ pounds

7. Dimensions of equipment frame __________________ feet

8. Load diverter/governor:

   a. Manufacturer ______________________________
   b. Supplier ______________________________
      ______________________________
      ______________________________
      ______________________________
   c. Type ______________________________
   d. Model No. ______________________________

## 4.4 Intake System

The function of the intake system is to direct water into the penstock or the turbine inlet. The intake system must also prevent trash or other foreign material from damaging the turbine. This section discusses water quality and the major components of an intake system. The majority of the writeup is directed toward run-of-the-stream developers. Those with canal drops will be interested only in certain aspects of the run-of-the-stream material. Existing dams are addressed in Subsection 4.4.3. If you are using waste-discharge water, this subsection will probably not be appropriate since you are more than likely connecting to an existing discharge pipe.

The design of the intake system components depends on the amount of water the system must handle. That amount of water is the previously determined flow design criteria. Equation (3-3) showed that the amount of water is equal to the cross-sectional area of the stream times the velocity at which the stream is moving:

$$Q = A \times v \tag{3-3}$$

where

$Q$ = design flow in cfs (from Design Criteria in Subsection 4.3)

$A$ = cross-sectional area in $ft^2$

$v$ = velocity in fps.

In this section, you will determine the size of your intake system components. The size is nothing more than the cross-sectional area, A. Equation (3-3) can be rewritten to solve for A:

$$A = \frac{Q}{v} \quad . \tag{4.4-1}$$

Equation (4.4-1) will be used to design (size) the intake system.

Engineers use more sophisticated procedures for relating velocity to area for an open channel, but these procedures are beyond the scope of this handbook. Conservative approaches will be used to calculate the needed areas.

Working from the sketch made in Section 3, your next step is to determine what intake system components are needed and to size those components. The components required will depend on the source and quality of the water (how much silt the water carries).

### 4.4.1 Water Quality

Hydraulic turbines generally are designed for clean water, and they operate best when only clean water is run through them. Potentially damaging materials range in size from gravel and large sticks down to fine silt and sand. Very large items will cause immediate damage when contacting the spinning turbine runner, while the damage caused by silt and sand will usually occur over longer periods of time. Silt and sand suspended in water can wear away the internal surfaces of the turbine, resulting in declining turbine efficiency.

Very large material, such as pieces of wood or gravel, can be removed effectively with trashracks that do not allow material of a particular size or larger to pass. The clear spacing in the rack is largely determined by the maximum object size that will pass through the turbine without causing damage.

The removal of silt and sand requires a different approach, since a screen fine enough to filter sand would often be clogged and would thus be impractical. In general terms, the amount of material that can be suspended in water relates to velocity of water and size of particles.

Very fast water such as that present in a river at flood stage can keep large amounts of material in suspension. Because the microhydropower plant intake cannot filter the silt from the water, the system must be designed to sufficiently reduce the velocity of the water to allow the suspended material to settle out. This process involves the use of a forebay for a run-of-the-stream sources and a reservoir for a manmade source.

A generating system fed by clear springs usually will not require an extensive intake system. A simple forebay and penstock intake with trash-rack may be adequate. For a stream that drains cultivated land and has a high silt load, a larger forebay or a reservoir would be essential.

If you are uncertain about the water quality, take a clear bottle or canning jar, fill it with water from the stream, and let it stand to see what settles out. Noticeable settlement indicates that a forebay is advisable.

The ultimate test of an intake system is to remain functional after a large flood. A system that will stand the test of time and still deliver the design flow is designed well.

The design and the components of an intake system are as varied as the physical conditions of the site and the imagination of the developer. The material presented here should be used as a guideline. It represents the benefit of knowledge and experience gained by a number of engineers and manufacturers.

Further discussion on intake systems is divided into two types of sources:

- Run-of-the-stream sources and canal drops

- Manmade sources such as existing dams and industrial waste discharges, where appropriate.

### 4.4.2 Run-of-the-Stream Sources and Canal Drops

Run-of-the-stream and canal drop developers must take water from a flowing stream or canal and introduce the water into a penstock. The intake system needed to do this may include the following components (see Figure 2-10, Subsection 2.6).

- Stream-Diversion Works--Diverts the water from the stream into the intake system.

- Settling Basin--Located near the diversion works, and used to settle suspended material before the water enters the power canal. The basin is recommended when the power canal is 1/2 mile or longer.

- Power Canal--The power canal and intake canal carries water from the diversion works to the settling basin or the forebay.

- Forebay--A settling basin designed to settle out suspended material before the water enters the penstock.

- Penstock Intake Structure--Provides the transition from the forebay to the penstock. The structure also provides the framework for the trashrack.

- Additional Hardware--e.g., skimmers, trashracks, stop logs, and intake gates or valves. These are essential elements of an intake system.

The intake system described in this subsection includes all the components. Some layouts may be able to do without some of the components. These alternatives will be discussed.

After reading all the material on intake structures, it will be time to finalize the design, check the design against the natural terrain, and finally make a cost estimate. Before proceeding, review the sketch of the intake system made in Section 3. Are any changes to the preliminary design contemplated? If so, make a list of the things you are considering. After you are satisfied with the preliminary layout, proceed with this subsection.

4.4.2.1 Stream Diversion Works. The ideally designed diversion works will direct the design flow out of the stream while allowing the stream-carried debris to float on down the stream. The works must also function equally well in low flow and high flow. Where severe freezing occurs, the intake must be deep enough to prevent ice from restricting flow.

One form of diversion works or penstock intake is a dam across the stream. A dam or a check may perform well on a canal, but can be a real source of trouble on a stream. Unless properly engineered, a dam can easily wash out, taking the penstock with it. There is also increased liability to the developer if the dam washes out and causes water damage downstream.

Experience has shown that a diversion works set at 90 degrees in relation to the stream attracts the least amount of debris and is better able to withstand the force and erosion effects of flood waters (see Figure 4.4-1). Two modifications are made to the stream itself, Gabion weirs and deepening the channel at the intake. These and other features of the works are discussed below.

- Gabion Weir--Two weirs should be placed in the stream on the same side of the stream as the intake canal. To steer debris away from the intake, an upstream weir should be placed 50 to 100 feet above the diversion. This weir should extend across approximately 1/3 of the stream width. The upstream weir should be angled downstream approximately 20 to 30 degrees. The second weir should be downstream from the diversion at a distance approximately two to three times the intake canal width. The downstream weir should be perpendicular to the stream bank, extending across 1/2 the width of the stream. This weir facilitates the diversion of water. The weirs are simply piles of large rocks held together in bundles with chicken wire or something similar.

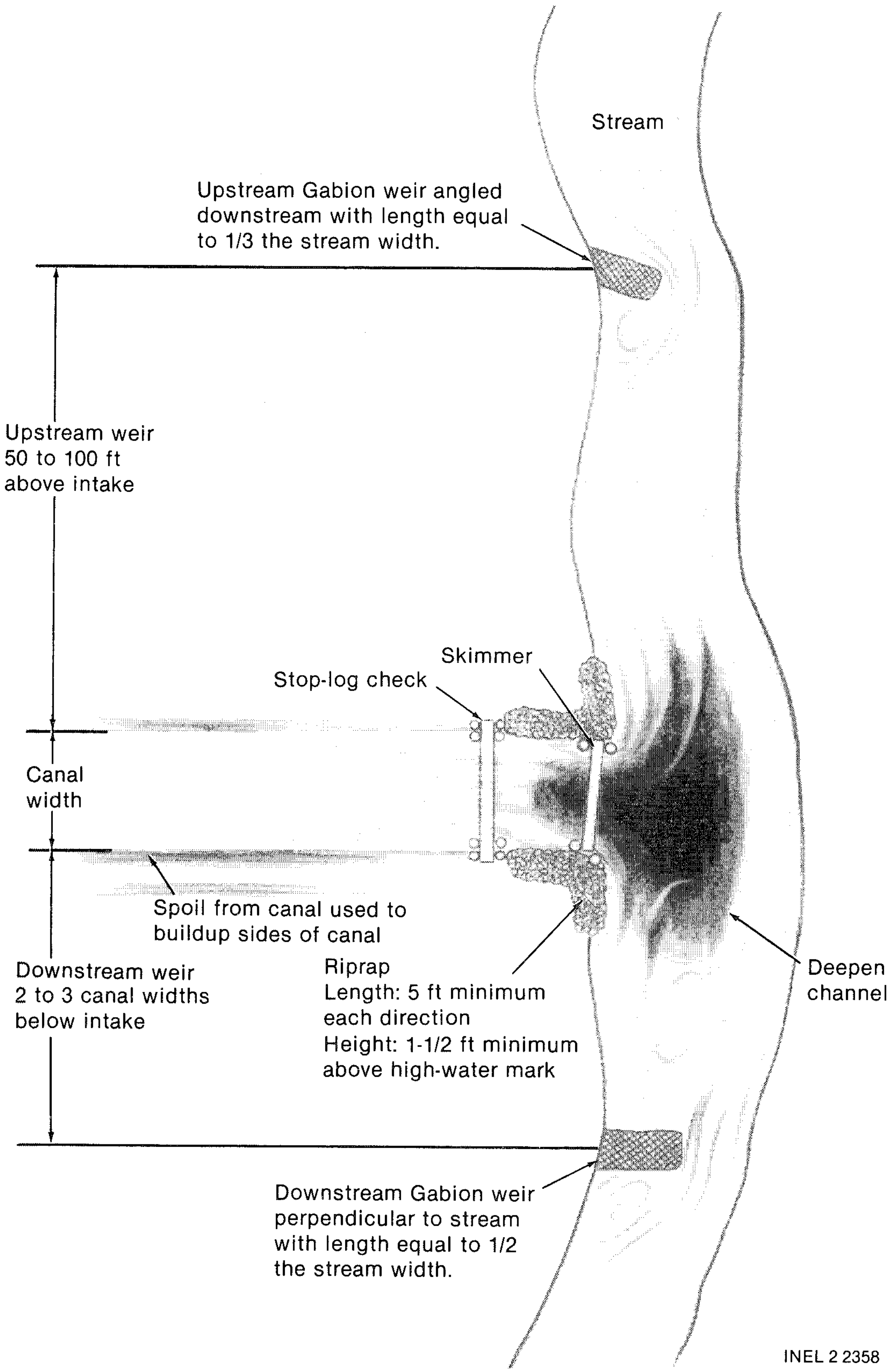


Figure 4.4-1. Typical diversion works.

The bundles are stacked on top of one another in pyramid fashion. The structures should be sized so that high water flows over the top of the weirs (see Figure 4.4-2).

- Deepening the Channel--The advantage of deepening the channel at the diversion is that the deeper pool reduces the velocity of the stream, limiting the amount of debris attracted to the intake. The deeper pool also reduces the effect of freezing. Ideally the dredging can be accomplished by a backhoe from the bank, and the deepened part can be cleaned out every few years.

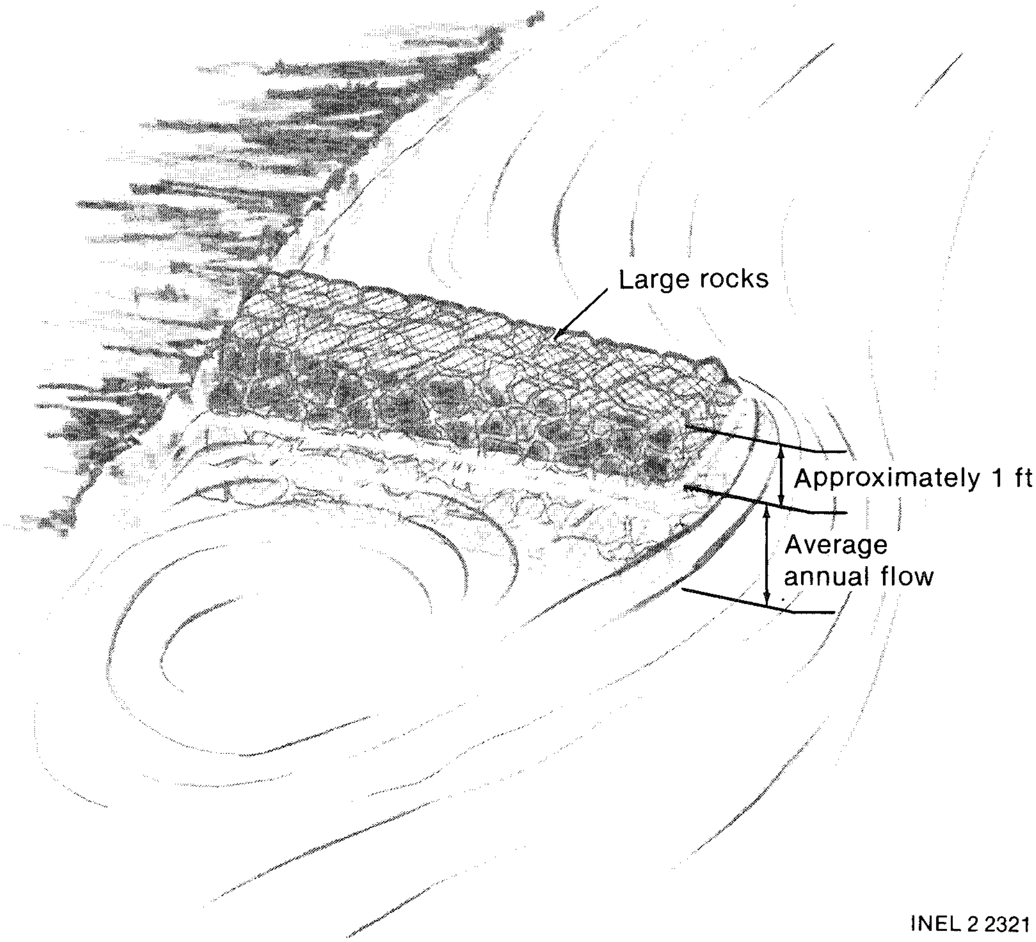


Figure 4.4-2. Gabion weir.

- Skimmer--At the entrance of the intake canal, a skimmer should be placed and angled slightly downstream. See Subsection 4.4.2.6.1 for design and installation considerations.

- Riprap--Riprap consists of large rocks placed along the bank to control erosion. If the material is available, the riprap should be constructed with 8-inch-diameter or larger rock. It is recommended that the riprap be placed at least 5 feet in each direction from the corner of the diversion (see Figure 4.4-1). Since the purpose of the riprap is to protect the intake structure from routine erosion and floods, it should be piled at least 1 foot higher than the high-water marks in the area.

- Berm--The berm consists of material dug from the canal, settling basin, and forebay. The berm should be the same height as the riprap.

4.4.2.2 Intake and Power Canal. The intake canal transports water from the stream to the settling basin or the forebay. The power canal is designed exactly like the intake canal and transports water from the settling basin to the forebay. These canals must be designed large enough to carry the design flow needed by the turbine. The recommended velocity in the canals is 2 fps. When the velocity and the design flow are known, Equation (4.4-1) can be used to calculate the cross-sectional area of the canals, provided that the recommended slope is maintained in the canal.

$$A = \frac{Q}{v} \quad . \tag{4.4-1}$$

Since the recommended velocity is 2 fps, the equation can be rewritten for canals:

$$A_c = \frac{Q}{2} \tag{4.4-2}$$

where

$A_c$ = area of canal in $ft^2$

$Q$ = design flow in cfs

$2$ = design velocity in canal in fps.

EXAMPLE: Assume that the design flow is 7.5 cfs; use Equation (4.4-2) to find the area of the canal.

$$A_c = \frac{Q}{2}$$

$$A_c = \frac{7.5}{2}$$

$$A_c = 3.75 \ ft^2.$$

Since the area of a canal is a product of width and height, two factors should be taken into account:

- The flow must be attracted into the canal even during low stream flow for Category 1, the intake at low stream flow must attract the design flow for Category 2, the intake must not attract the portion of stream flow that is required by the state to keep the stream alive (state-imposed minimum stream flow).

- The deeper the canal, the smaller the freezing problem--particularly if the flow in the canal must stop for some reason during cold weather.

These two considerations dictate that the bottom of the canal should be at least as deep as that of the natural stream. The exception to this rule would be if the diversion works were located at a naturally occurring deep pool in the stream (an ideal situation). In this case, the canal bottom should be well below the low-flow mark of the stream. For most cases, the canal bottom should be set at or below the natural stream bottom.

The actual flow of water into the canal is controlled by the demand of the turbine. A stop log wier check, described in Subsection 4.4.2.6.3, is also used to control flow and water level in the canal.

The dimensions of the canal can be determined with the following steps.

- Estimate the depth of the design flow for the natural stream in inches (see Figure 4.4-3): For Category 1, estimate the depth of the minimum annual low flow. For Category 2, estimate the depth of the design flow

- Use the estimated depth, along with the previously determined canal area, in Equation (4.4-3) to determine the canal width.

$$W_c = 12 \times \frac{A_c}{d} \qquad (4.4\text{-}3)$$

where

$W_c$ = width of canal in feet

$A_c$ = area of canal in $ft^2$, from Equation in (4.4-2)

$d$ = estimated depth of design flow in inches

12 = number of inches per foot.

EXAMPLE: Using the previous example, $A_c = 3.75\ ft^2$, assume that the design flow depth is 15 inches; use Equation (4.4-3) to find the canal width.

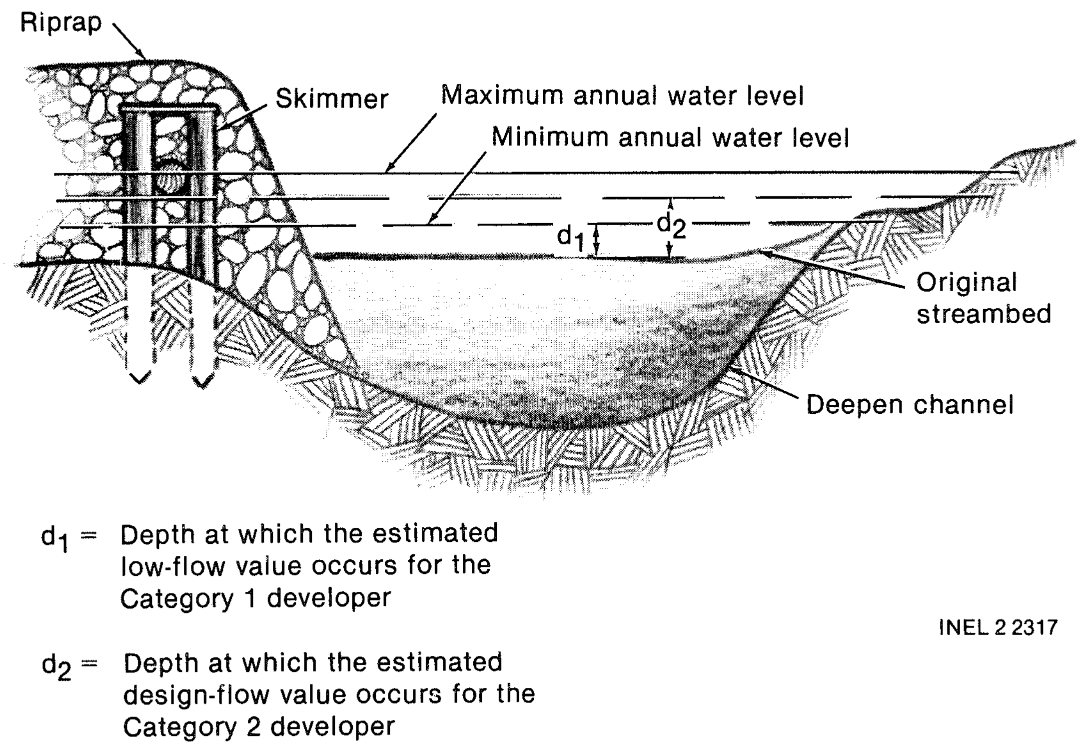


Figure 4.4-3. Estimating design flow for Category 1 and Category 2.

$$W_c = 12 \times \frac{Ac}{d}$$

$$W_c = 12 \times \frac{3.75}{15}$$

$$W_c = 3.0 \text{ ft.}$$

An actual canal will probably not have rectangular sides like the crosshatched area in the Figure 4.4-4. Therefore, the calculated width should be the bottom width of the canal. The sides of the canal should be cut back (angled out) so that the soil will stand freely without sluffing into the canal. The angle at which the side stands freely is called the "angle of repose." The additional area of the canal outside the rectangular crosshatch will compensate for the friction losses in the canal.

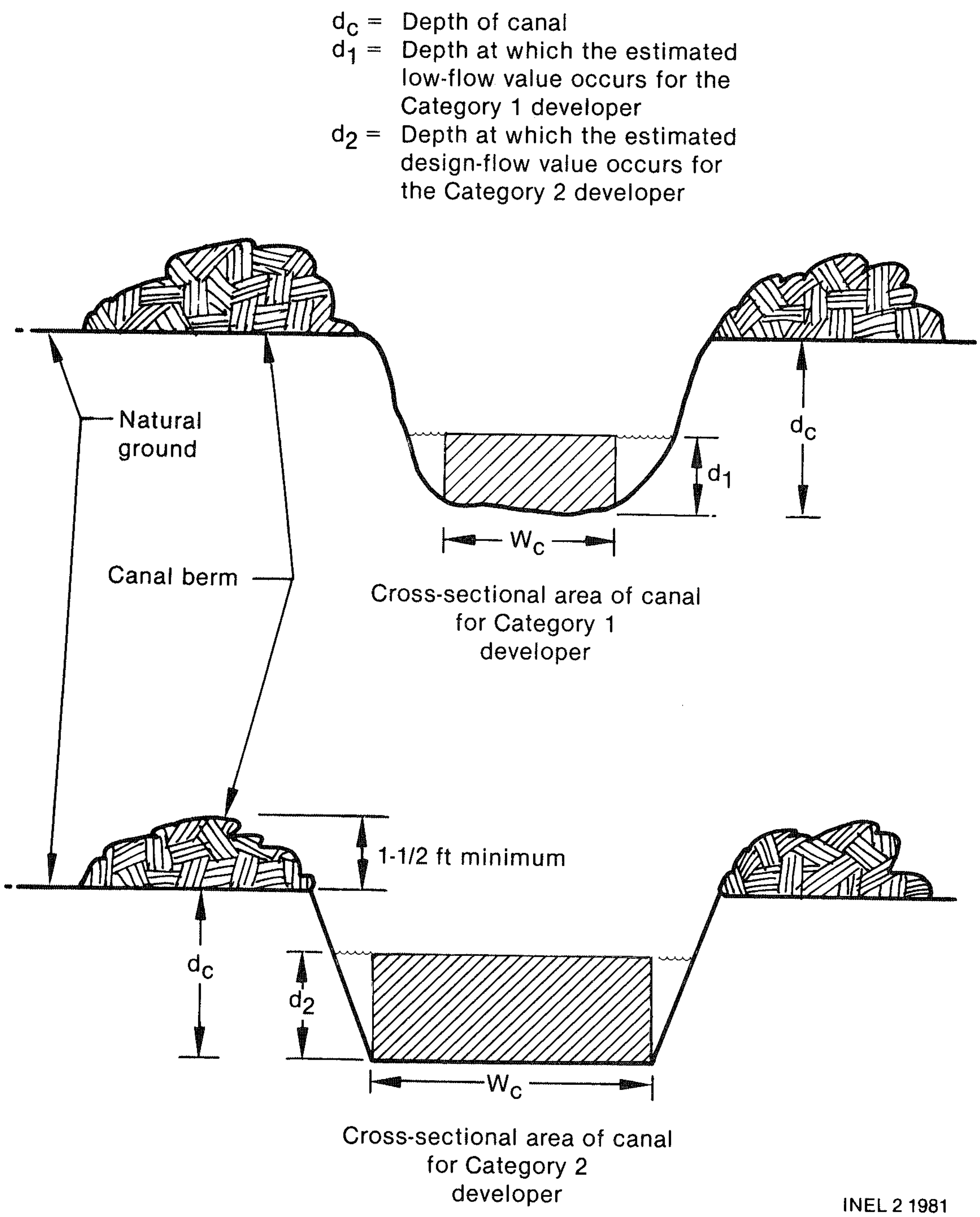


Figure 4.4-4. Canal cross-sectional area.

Short canals can remain fairly level. Longer canals will require a slight downward slope. Check with your local Soil Conservation Service to determine the required slope. Setting the grade is critical and should not be attempted without the aid of some type of leveling instrument (see Subsection 3.4).

CAUTION: If the power canal is long, snow melt or a heavy rainstorm may put more water in the canal than the turbine can handle. This situation could result in flooding the intake system. To reduce the effects of such a flood, the canal should be equipped with an overflow spillway (see Figure 4.4-5).

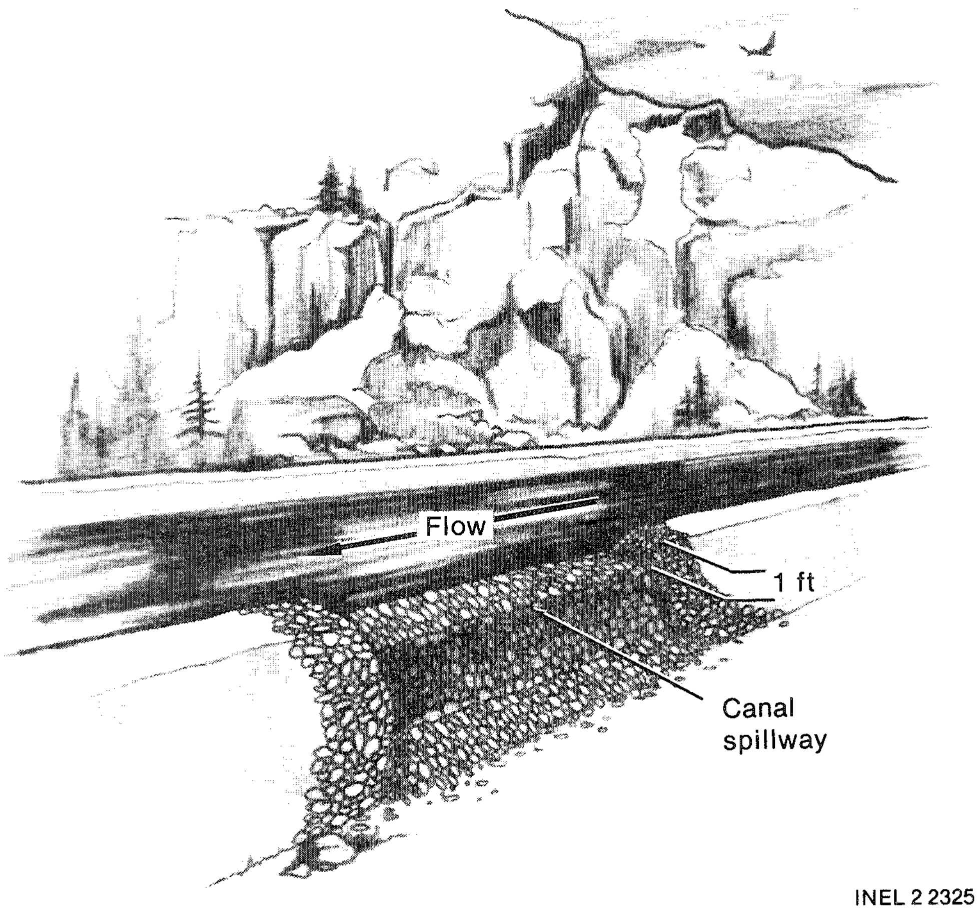


Figure 4.4-5. Canal spillway.

Also, it is very important that the top of the berm for the canal and forebay be kept at the same elevation as the top of the diversion structure, because if the penstock were closed, the water in the canal would seek the same elevation throughout the intake system (canal, forebay, etc). If the berm in the forebay were lower than the overall water level, the forebay would be flooded. If the canal is so long that it is not practical to keep the berm at the same elevation, a series of stop logs should be used to section off the canal.

After sizing the canal, make a sketch of the cross-sectional area of the canal. Note the width ($W_c$) and the total depth ($d_c$). The total depth is the distance from natural ground to the bottom of the canal. Figure 4.4-6 is a sketch of the example canal.

4.4.2.3 Settling Basin. A settling basin is recommended for sites where the power canal will be 1/2 mile or longer. The purpose of the basin is to prevent sediment buildup in the power canal. The basin slows the water down and allows the settlement of the larger material (fine sands, etc.) to occur in the basin. Periodically, the basin is flushed out through a cleanout pipe.

A good rule of thumb is to make the basin four times wider than the power canal, 2 to 3 feet deeper, if possible, and at least 90 feet long (see Figure 4.4-7). If this rule is followed and the power canal is designed for 2 fps, then the settling basin velocity will be less than 0.5 fps.

Thus,

$$W_s = 4 \times W_c \qquad (4.4\text{-}4)$$

where

$W_s$ = width of settling basin in feet

$W_c$ = width of canal in feet, from Equation (4.4-3).

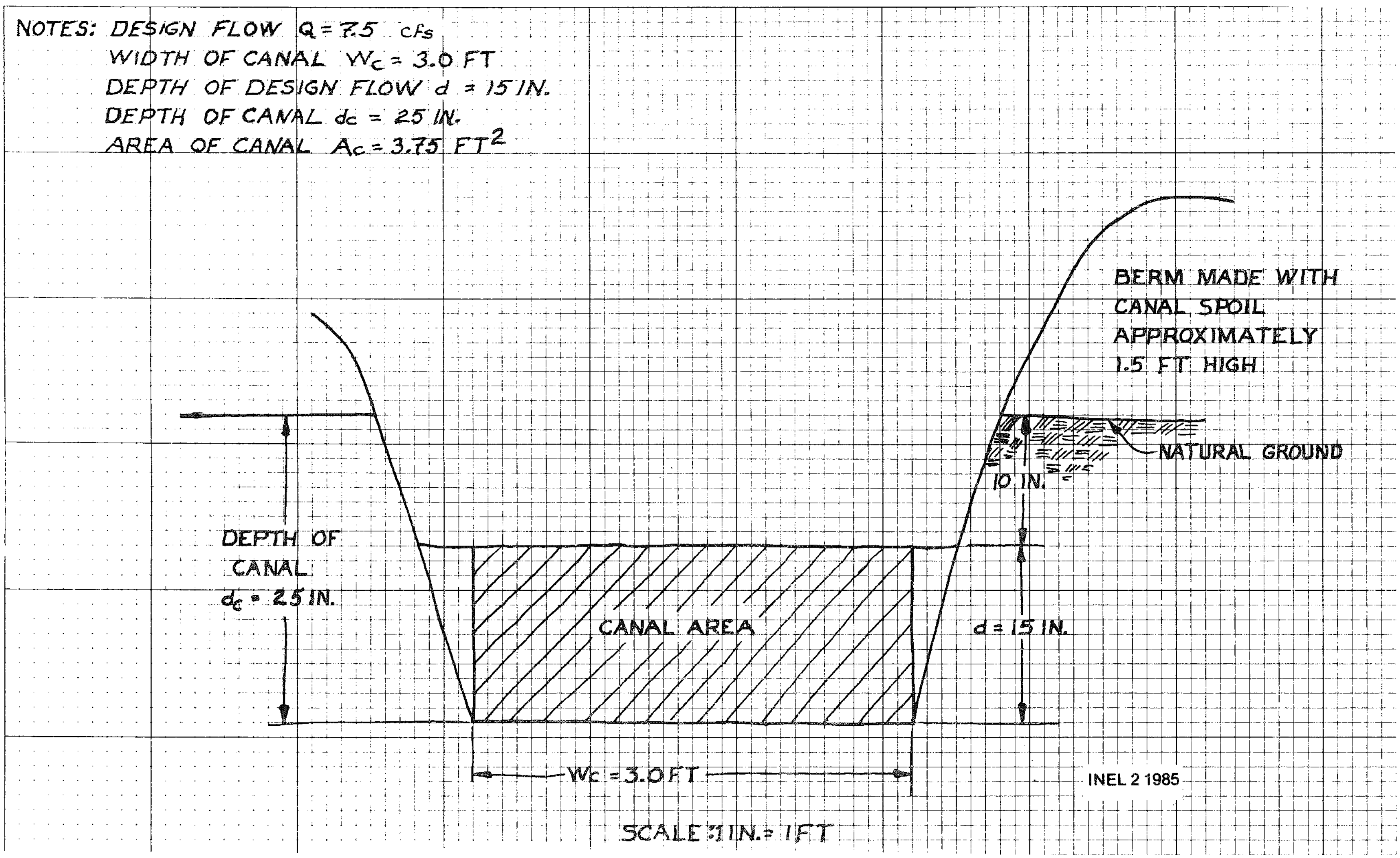


Figure 4.4-6. Sketch of canal cross-sectional area.

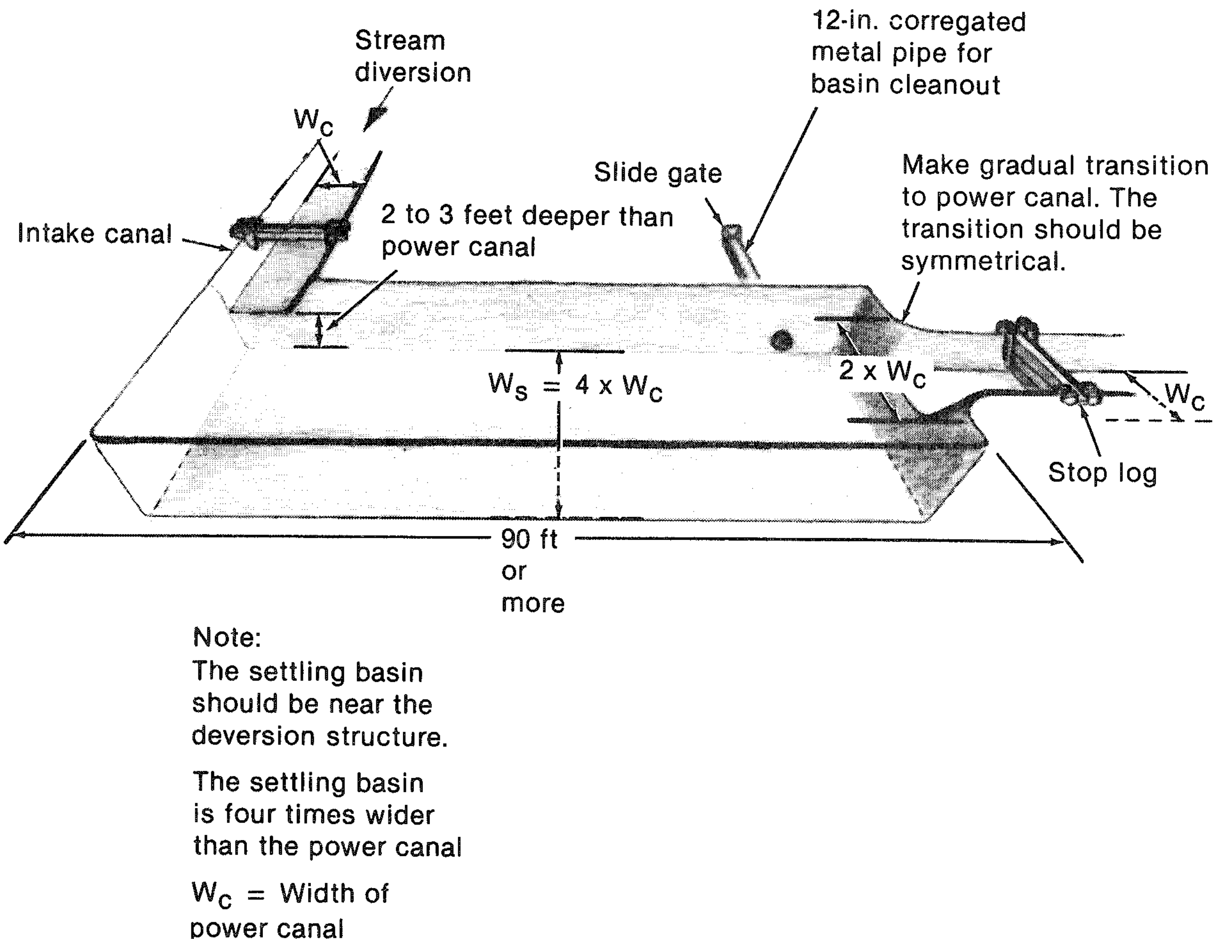


Figure 4.4-7. Diagram of settling basin.

The key to keeping the basin functioning is to maintain the slow velocity and large volume in the basin. To accomplish this, the basin should be equipped with a cleanout pipe. The pipe should be at least a 12-inch, corrugated metal pipe that drains from the bottom of the basin. To control flow in the pipe, some type of valve is required. A slide gate, as shown in Figure 4.4-8, is possibly the simplest. Some developers actually have the gate partially open to allow for continuous cleanout.

EXAMPLE: From the canal example, the width of the canal ($W_c$) = 3 feet. Use Equation (4.4-4) to find the dimensions of the basin.

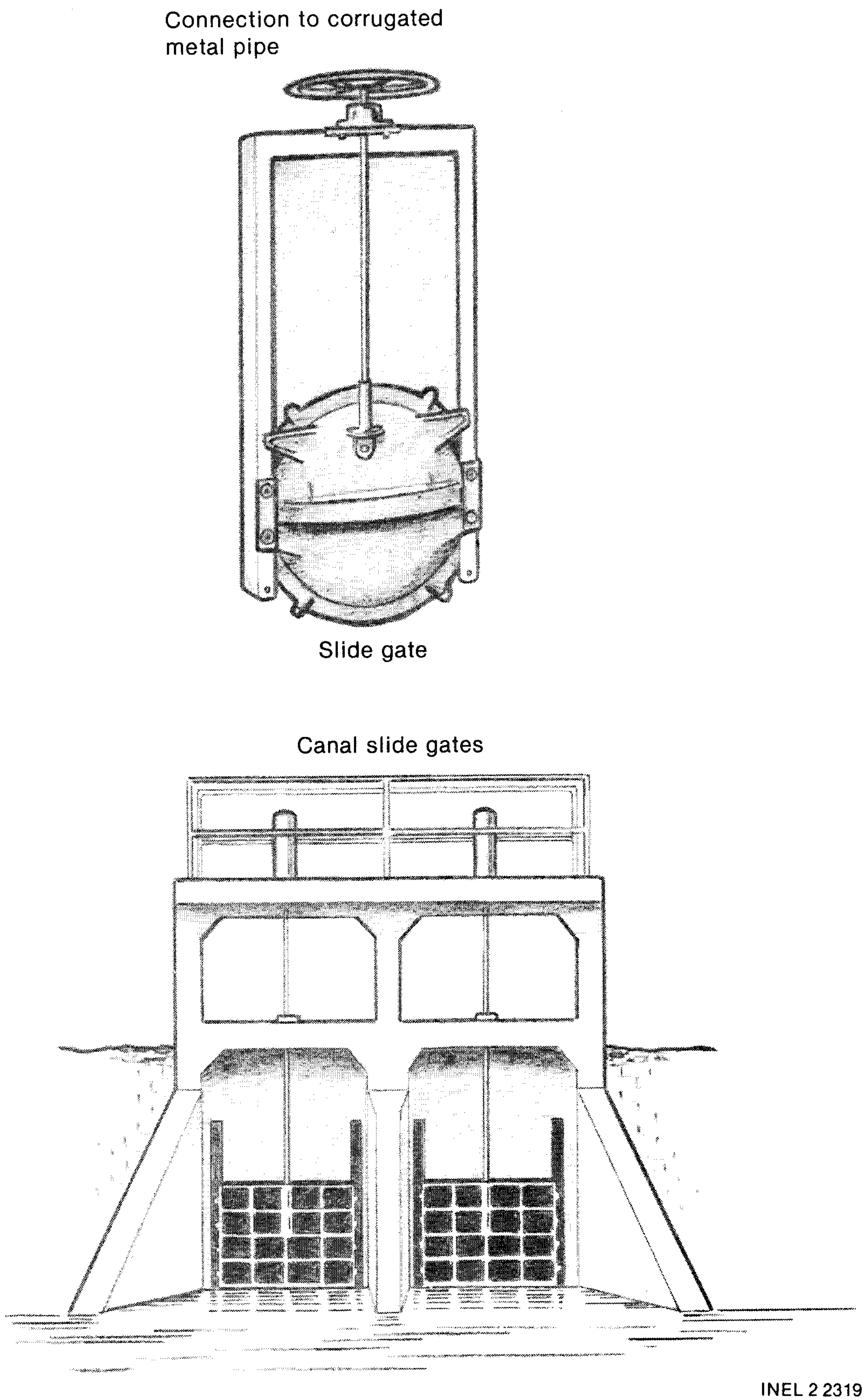


Figure 4.4-8. Flow control gates.

$$W_s = 4 \times W_c$$

$$W_s = 4 \times 3$$

$$W_s = 12 \text{ ft} \quad .$$

Thus, the basin should be 12 feet wide and 90 feet long.

After determining the basin dimensions, make a sketch of the basin. Figure 4.4-9 is a sketch of the example.

4.4.2.4 Forebay. Some type of forebay is required for all run-of-the-stream sites. The forebay is a settling basin to protect the turbine from suspended debris. The recommended velocity in the forebay is 0.25 fps; therefore, the cross-sectional area of the forebay should be eight times larger than that of the power canal (the velocity in the canal, 2 fps, is eight times larger). It is advisable to maintain a depth-to-width ratio of 1-to-1 (the depth should be equal to the width), as shown in Figure 4.4-10. This is not always possible in areas where rock, shale, boulders, or other obstructions limit the depth of excavation. In such cases, try to keep the area at least eight times larger than the canal.

Since the area of the forebay is eight times larger than the canal, Equation (4.4-5) can be written:

$$A_f = 8 \times A_c \tag{4.4-5}$$

where

$A_f$ = area of forebay in $ft^2$

$A_c$ = area of canal in $ft^2$, from Equation (4.4-2).

EXAMPLE: In the previous example, the area of the canal was computed to be $A_c = 3.75$ $ft^2$. Using Equation (4.4-5), determine the dimensions of the forebay.

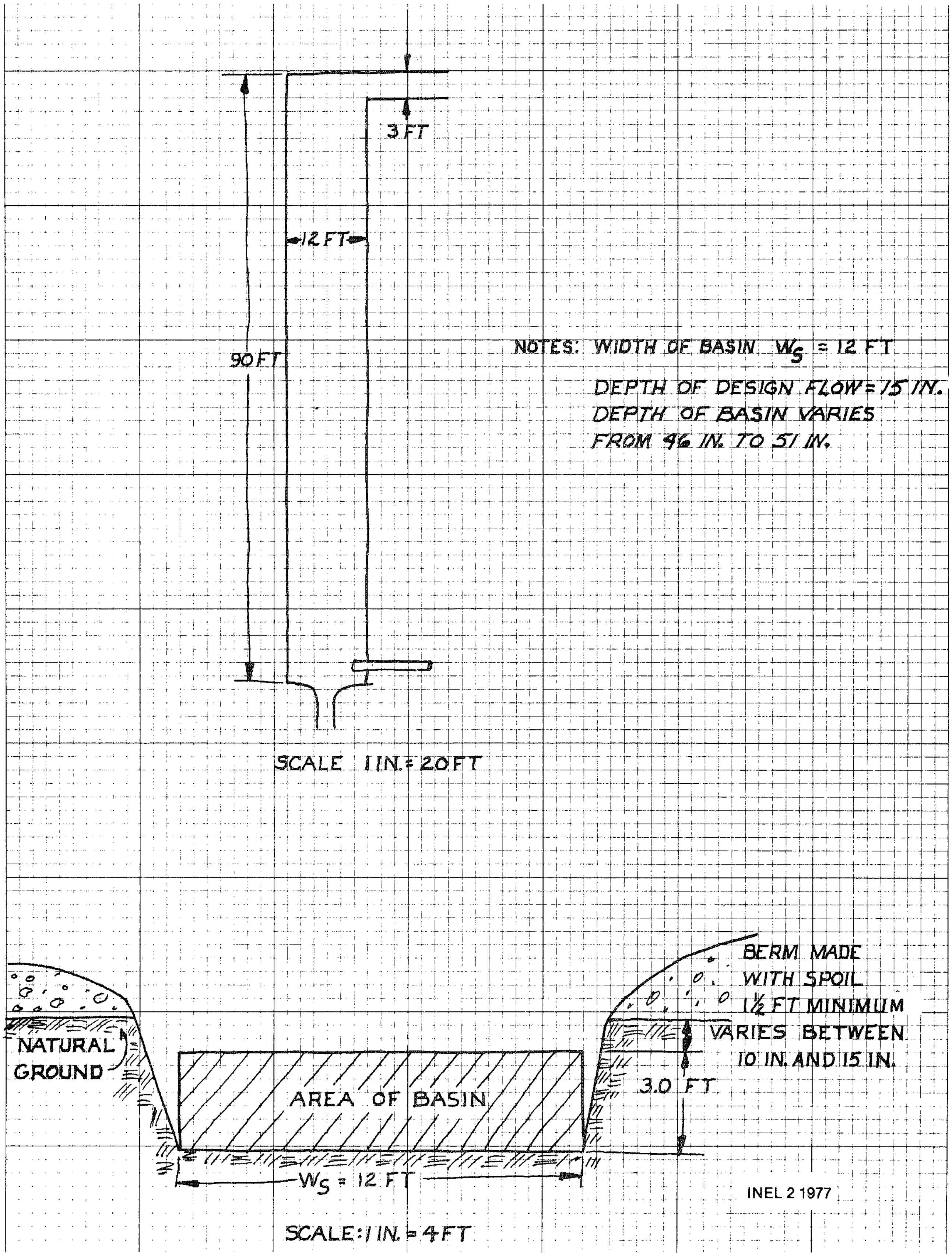


Figure 4.4-9. Sketch of settling basin.

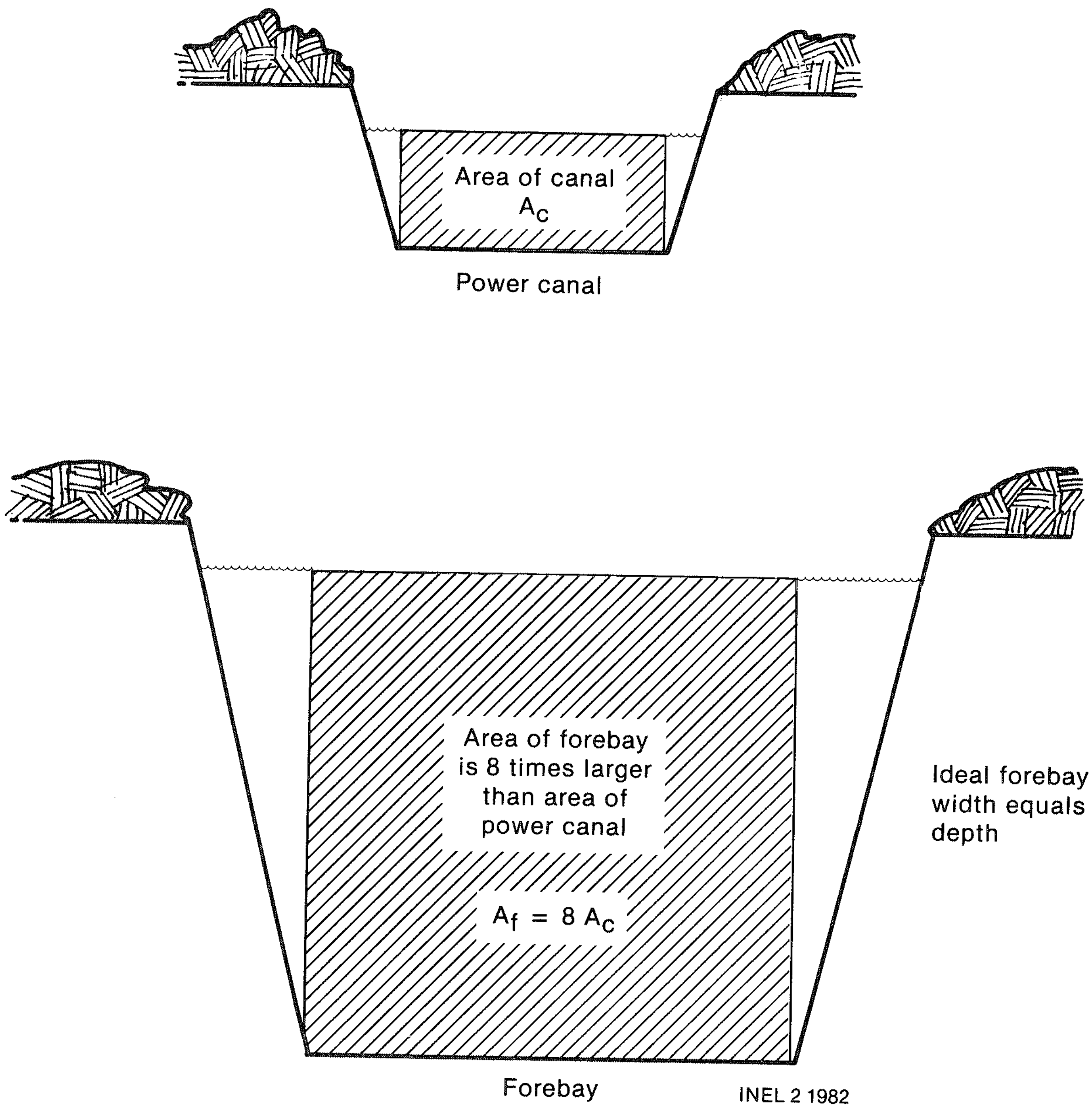


Figure 4.4-10. Diagram of forebay.

$A_f = 8 \times A_c$

$A_f = 8 \times 3.75$

$A_f = 30\ ft^2$.

NOTE: Since the ideal situation is to have the depth equal to the width, and since area is the product of width times the depth, take the square root ($\sqrt{\ }$) of $A_f$. If your calculator does not have a square root ($\sqrt{\ }$) function, find the square (product of a number multiplied by itself) that is closest to but larger than $A_f$. For example, 5 x 5 = 25 and 6 x 6 = 36; therefore use 6 x 6. Never use a width less than the canal width.

Assume in this example that you have a square root function on the calculator. The square root of 30 is 5.48. Therefore, the ideal forebay dimensions would be 5.5 feet by 5.5 feet.

Now assume that in the area where the forebay is to be placed the maximum available depth is only 4 feet; find the new forebay dimensions. Since area equals width times depth and depth is known, divide area by depth to get width:

$$W_f = \frac{A_f}{d_f} \qquad (4.4\text{-}6)$$

where

$W_f$ = width of forebay in ft

$A_f$ = area of forebay in ft$^2$, from Equation (4.4-6)

$d_f$ = depth of forebay in ft.

Therefore,

$$W_f = \frac{30}{4}$$

$$W_f = 7.5 \text{ ft} \quad .$$

The forebay should be oriented with respect to the penstock so that the penstock can be kept as straight as possible. In most cases where a power canal is used, the penstock takeoff will be placed at a 90 degree angle to the canal on the downhill side of the forebay.

The length of the forebay should be at least 45 feet to allow sufficient time for the fine sand, etc. to settle. If it is impractical to make the forebay 45 feet long, make the length as long as practical, and widen the forebay, if possible. The wider area will reduce the velocity and increase the settling time.

The forebay should also be equipped with a method for clean-out. The simplest method is to install 12-inch corrugated pipe through the downhill berm. The pipe should be placed on the bottom of the forebay. A slide gate should be placed on the pipe to control flow.

A skimmer should be placed in the forebay ahead of the trashrack. The skimmer should be angled to force the trash to the side of the forebay. The skimmer is discussed in Subsection 4.4.2.7, Additional Hardware.

4.4.2.5 Trashrack--Although the trashrack is actually part of the additional hardware, it is discussed separately at this point because the trashrack must be sized before the penstock intake structure on which it is mounted can be sized.

A trashrack is an essential element of any hydropower project. Microhydropower units in particular must be protected from trash carried by the water. The rack must strain unwanted material from the water and yet have enough openings to allow the design flow to pass through without significant loss of head. The rack must also be strong enough to withstand water pressure forced against it if the rack becomes completely clogged with trash. A trashrack mounted on the penstock intake structure is shown in Figure 4.4-11.

The design for microhydropower trashracks varies widely. In evaluating any design or your own creation, two key points should be kept in mind.

- The open, clear area of the rack must be large enough to allow the design flow to pass smoothly.

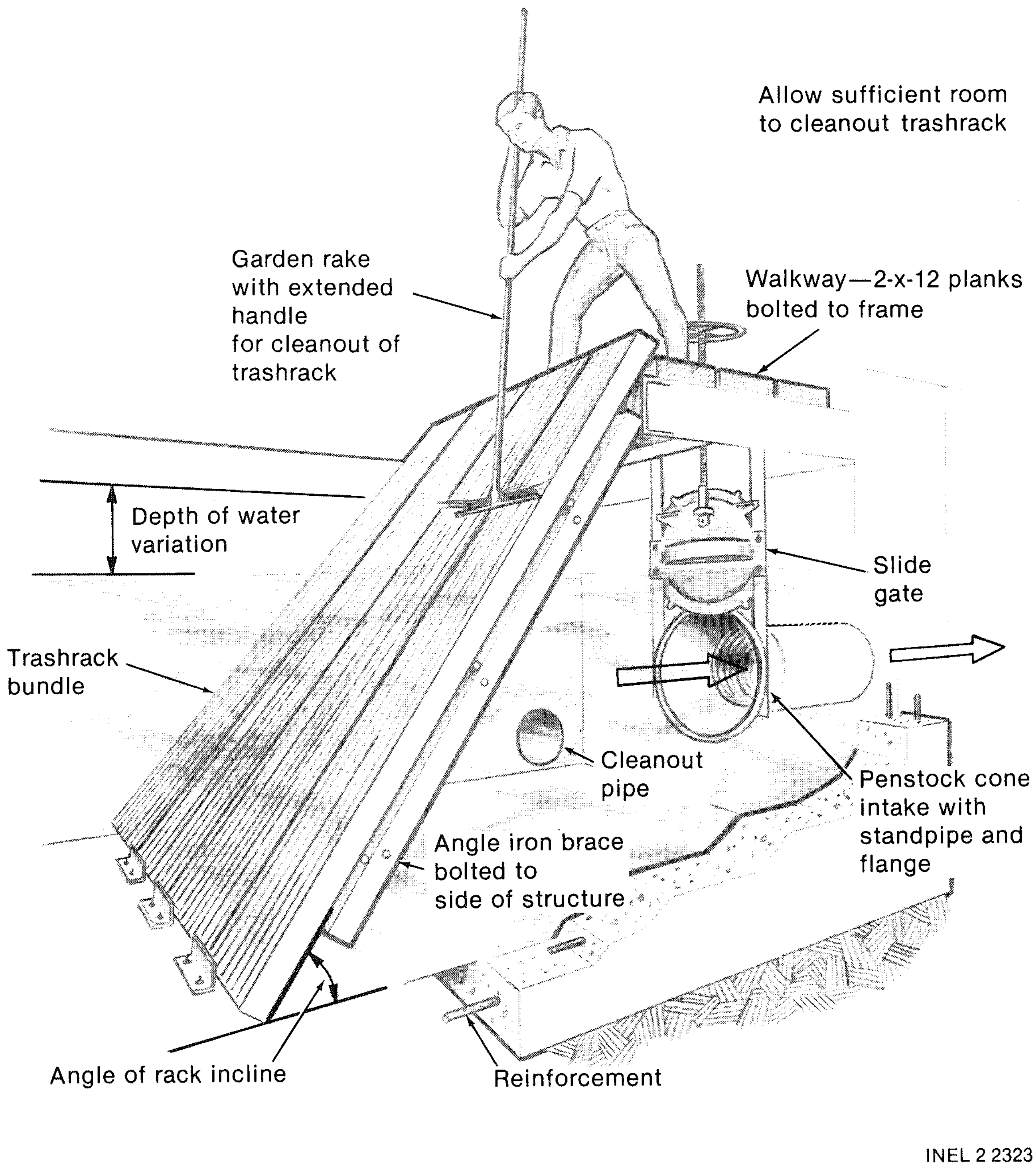


Figure 4.4-11. Penstock intake structure cross-section.

- The rack must be designed for easy, periodical cleaning without interfering with the operation of the turbine.

Figure 4.4-12 and 4.4-13 are photos of two vertical slide-in racks, set one behind the other. Figure 4.4-14 is a photo of a barrel-type trash-rack connected directly to the penstock intake.

The simplest trashrack is made of bundles that can be easily handled by one person. A typical bundle can be fabricated from 2- to 3-inch flat stock bars (strap metal), shown in Figure 4.4-15. Most racks should be made with bars 1/4-inch wide (very small ones can be made with 1/8-inch bars). The bars can be fabricated into bundles typically 12 inches wide with the bars placed vertical to the flow (see Figure 4.4-11). The length can vary according to the site criteria (usually less than 10 feet for ease of handling). The clear space between the bars is the area that must be designed to pass the design flow without causing significant head loss. For microhydropower projects, the spacing can range from 1/2 inch to 1 inch (see Figure 4.4-15). The smaller spacing is recommended for smaller turbine units. Racks fabricated into bundles in this fashion can be removed individually for repair, maintenance, etc. Keep a spare![a]

Because the design area is the clear area between the bars, sizing the trashrack is not as simple as finding the area of the power canal or the forebay. The area of the bars must be added to the design area to obtain the dimensions of the wetted area of the rack, the area submerged during normal design flow (see Figure 4.4-16). And since the rack is set at 45 or 60 degrees, the area is based on that incline angle. The steps involved are discussed in the paragraphs that follow.

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a. K. M. Grover, "Site Selection and Turbine Setting," (presented at Quito, Ecuador, August 1980), GSA International Corporation, Katonah, N.Y. 10536.

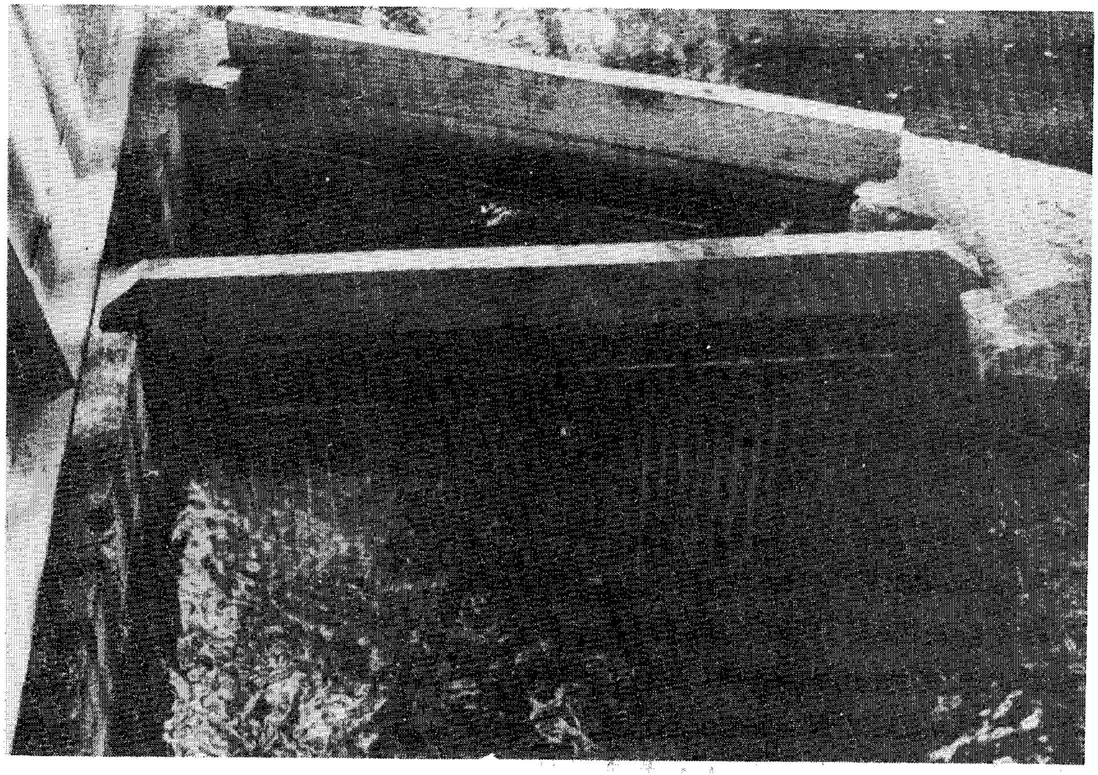


Figure 4.4-12. Vertical, slide-in trashracks.

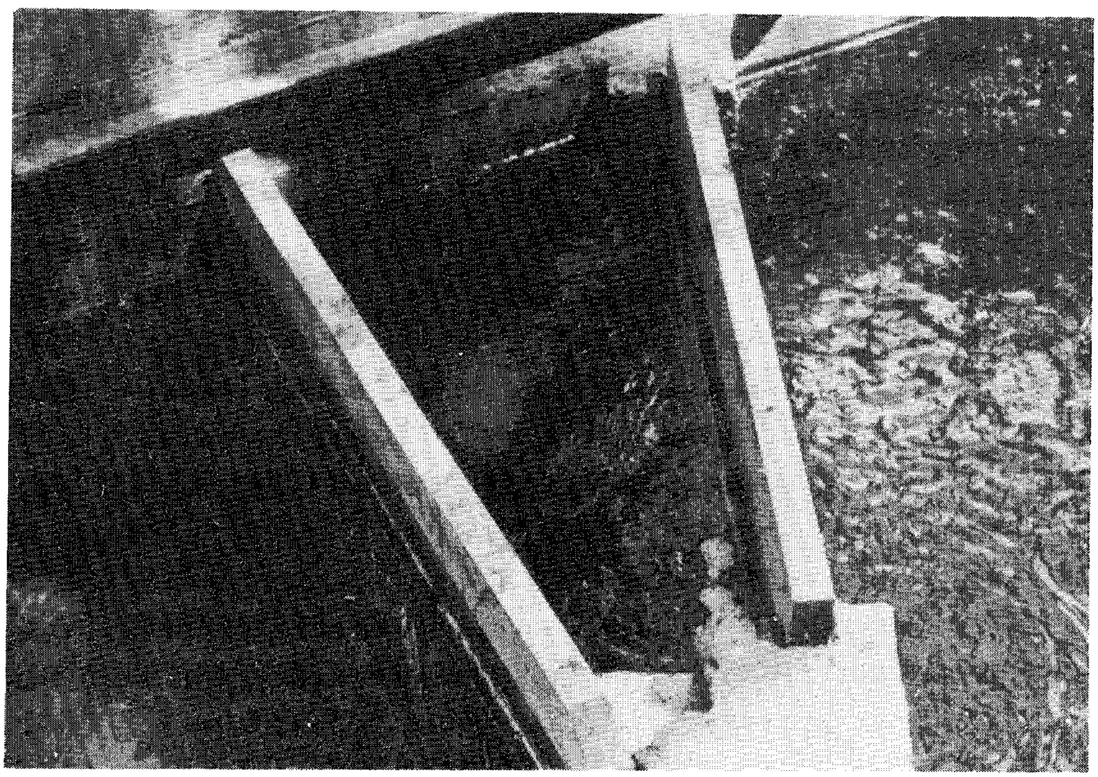


Figure 4.4-13. Vertical, slide-in trashracks.

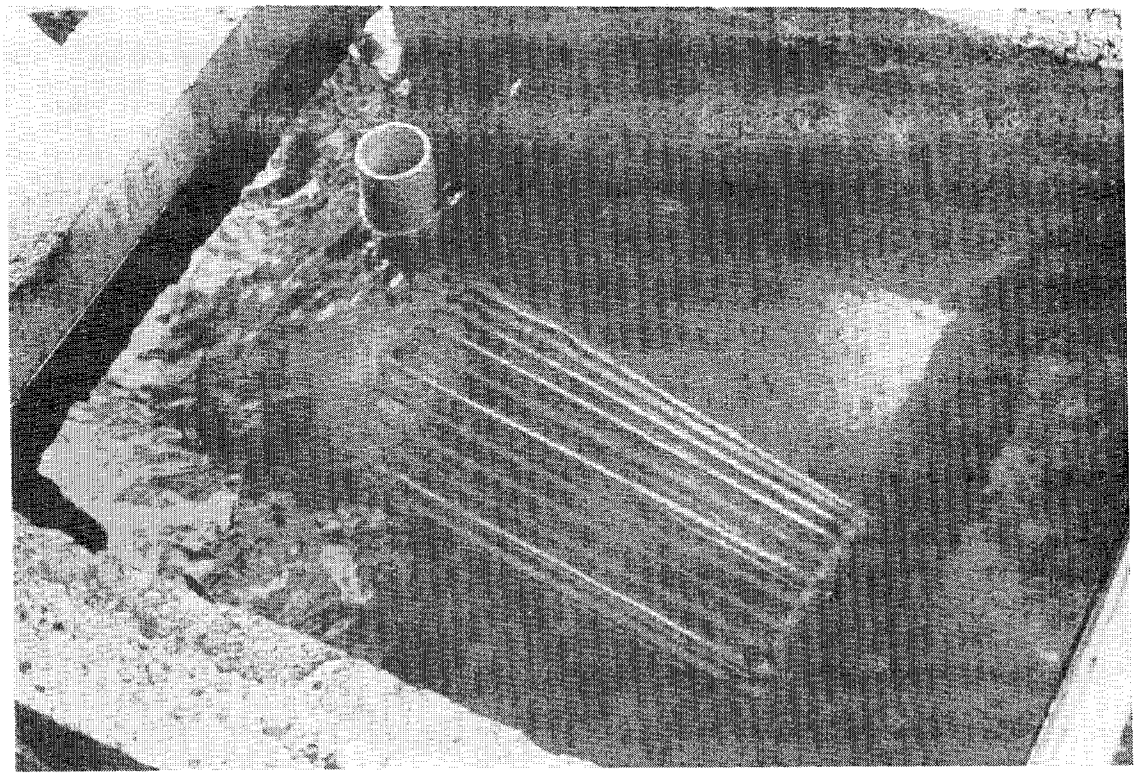
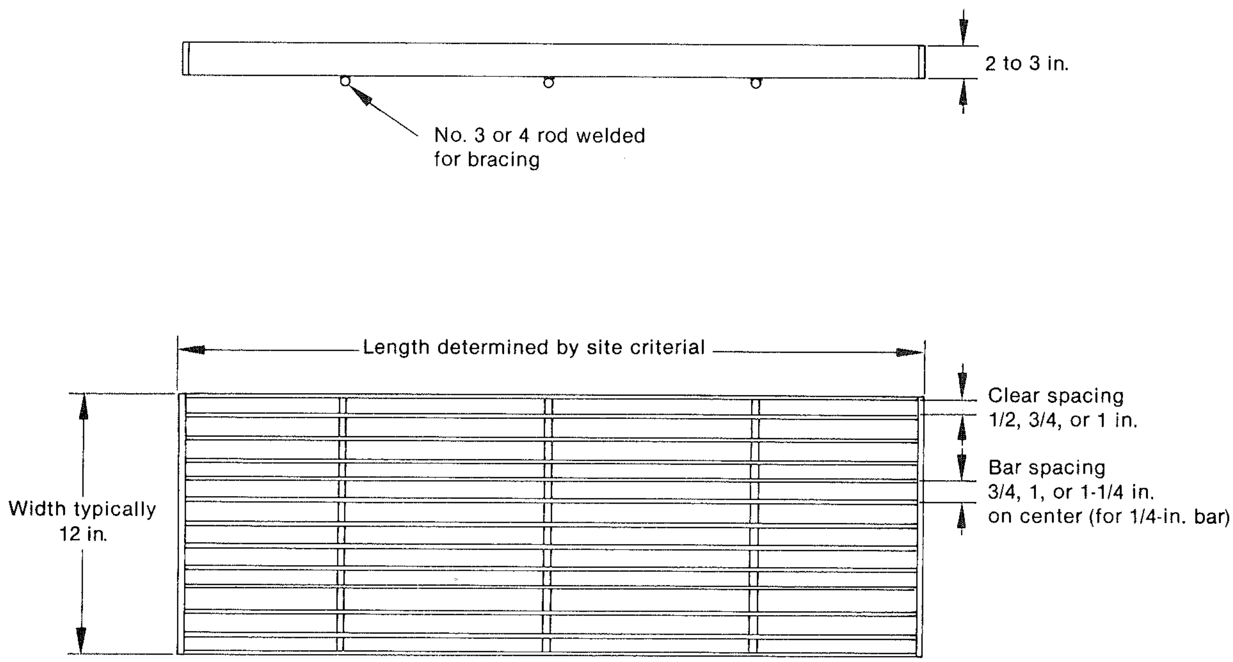


Figure 4.4-14. Barrel-type trashrack.




Typical trashrack bundle with 2- to 3-in. flat stock bars 1/4-in. wide. All weld construction with No. 3 or 4 rod braces.

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Figure 4.4-15. Typical trashrack bundle.

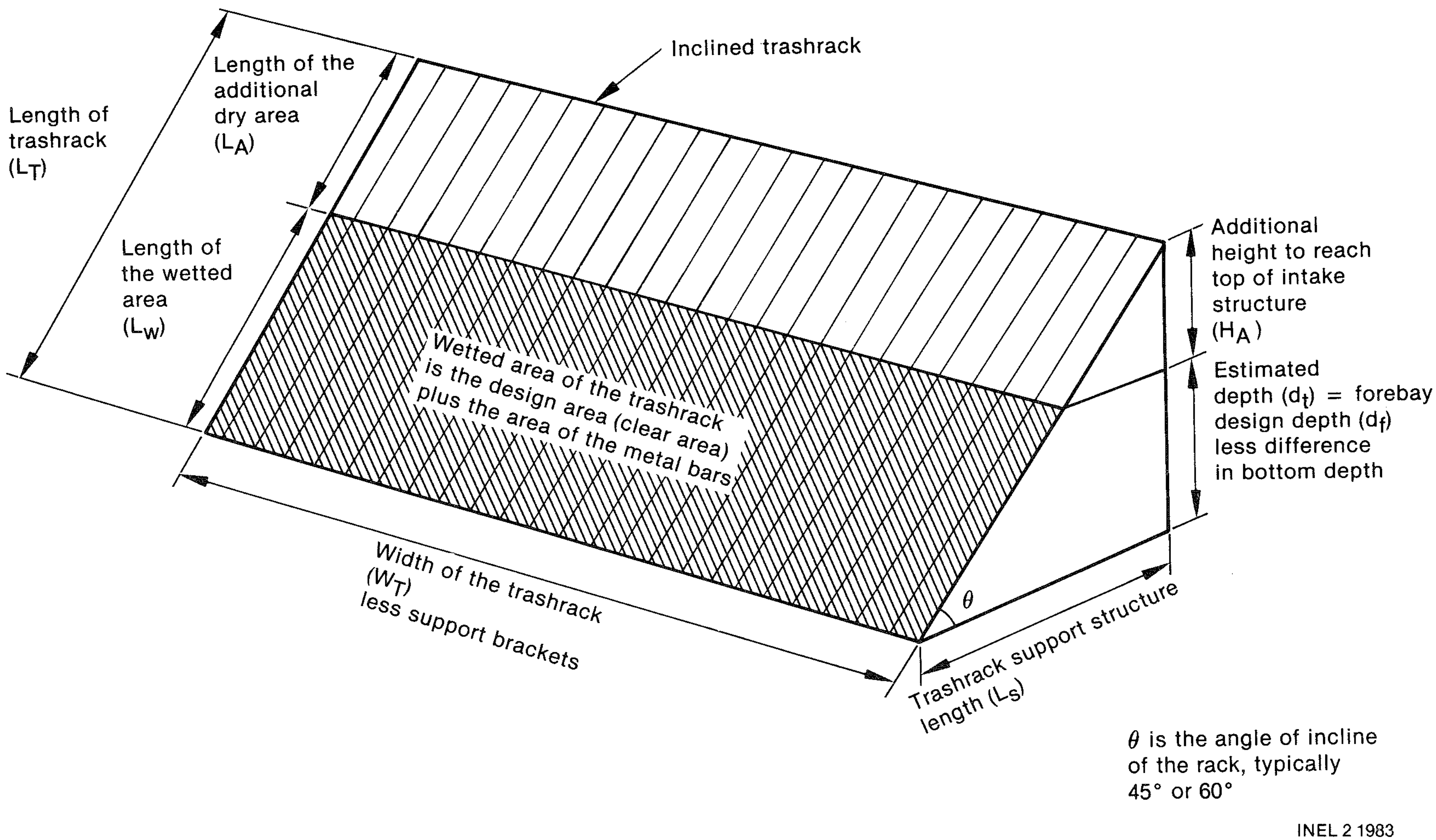


Figure 4.4-16. Trashrack dimension diagram.

4.4.2.5.1 <u>Trashrack Design Area</u>--The design area is the clear, open area in the rack through which the water passes. The design velocity for a trashrack is 0.5 fps. Therefore, Equation (4.4-1), $A = \frac{Q}{V}$, can be rewritten for V = 0.5 fps:

$$A_{DT} = \frac{Q}{0.5} \tag{4.4-7}$$

where

$A_{DT}$ = design area of trashrack in $ft^2$

Q = design flow in cfs

0.5 = velocity through rack in fps.

EXAMPLE: As in the previous example, the design flow is 7.5 cfs. Find the design area of the trashrack.

$$A_{DT} = \frac{Q}{0.5}$$

$$A_{DT} = \frac{7.5}{0.5}$$

$$A_{DT} = 15 \text{ ft}^2 \quad .$$

NOTE: Since 0.5 fps is four times smaller than the velocity of the power canal (2 fps), the design area for the rack is four times the area of the canal ($A_{DT}$ = 15 $ft^2$ and $A_c$ = 3.75 $ft^2$).

4.4.2.5.2 Length of Trashrack Wetted Area--The length of the wetted area is the length of the rack set in the incline (see Figure 4.4-16). To calculate the length, the depth of the water on the rack must be estimated. The water depth is equal to the depth of water in the forebay less any difference in elevation between the forebay bottom and the intake structure bottom. In most cases, the two bottom elevations will be the same; then the previously determined forebay depth ($d_f$) can be used directly.

$$L_w = \frac{d_t}{\sin \theta} \qquad (4.4\text{-}8)$$

where

$L_w$ = length of the wetted area of the trashrack in feet

$d_t$ = depth of water in the intake structure equal to $d_f$ (from Subsection 4.4.2.4) less the difference in elevation between the bottom of the forebay and the intake structure

$\sin \theta$ = trigonometric function of the angle of incline of the trashrack, usually 45 or 60 degrees.

Since the recommended angle of incline is either 45 or 60 degrees, and since $\sin \theta$ for these angles is 0.707 and 0.866, respectively, Equation (4.4-8) can be rewritten as follows:

$$L_w = \frac{d_t}{0.707}, \text{ for 45-degree incline} \qquad (4.4\text{-}8a)$$

$$L_w = \frac{d_t}{0.866}, \text{ for 60-degree incline} \quad (4.4\text{-}8b)$$

EXAMPLE: From the previous example, the depth of the forebay is 4 feet. Find the length of wetted trashrack for 45- and 60-degree angles.

From Equation (4.4-8a) for a 45-degree incline:

$$L_w = \frac{4}{0.707}$$

$$L_w = 5.6 \text{ ft} \quad .$$

From Equation (4.4-8b) for 60 degree incline:

$$L_w = \frac{4}{0.866}$$

$$L_w = 4.6 \text{ ft} \quad .$$

4.4.2.5.3 Nominal Width--The nominal width is the width of the design area (clear area). It does not consider the width of the metal bars. The following equation is used to compute the nominal width:

$$W_N = \frac{A_{DT}}{L_w} \quad (4.4\text{-}9)$$

where

$W_N$ = nominal width of the wetted area in ft

$A_{DT}$ = design area in $ft^2$, from Equation (4.4-7)

$L_w$ = length of the wetted area in ft, from Equation (4.4-8)

EXAMPLE: Assume from the above that $L_w$ = 4.6 ft and that $A_{DT}$ = 15 $ft^2$. Find $W_N$.

$$W_N = \frac{15}{4.6}$$

$$W_N = 3.75 \text{ ft.}$$

The nominal width must now be corrected to account for the width of the bars.

4.4.2.5.4 Width of the Trashrack--To find the width of the trashrack, the area of the metal bars in the rack must be added to the design area. This is difficult to determine since the total width is not known; therefore, the number of bars is not known. The only thing that is known is the area of the openings between the bars (design area). It was previously recommended that all racks be made with 1/4-inch bars (except for very small units, which can be made of 1/8-inch bars). For racks made with 1/4-inch bars, the following ratios can be used to compute the trashrack width:

- For 1/2-inch clearance between bars, the ratio is 3 inches of rack width for every 2 inches of opening: r = 1.50.

- For 3/4-inch clearance between bars, the ratio is 2 inches of rack width for every 1-1/2 inches of opening: $r = 1.33$.

- For 1-inch clearance between bars, the ratio is 5 inches of rack width for every 4 inches of opening: $r = 1.25$.

For very small racks, when 1/8-inch bars are used, only a 1/2-inch opening between bars should be allowed.

- The ratio is 5 inches of rack width for every 4 inches of opening: $r = 1.25$.

Now the width of the trashrack can be computed using the correct ratio:

$$W_T = r \times W_N \tag{4.4-10}$$

where

| | | |
|---|---|---|
| $W_T$ | = | width of the trashrack in ft |
| $r$ | = | ratio of total width to clear area width |
| $W_N$ | = | nominal width in ft, from Equation in (4.4-9). |

EXAMPLE: From the previous example, $W_N = 3.75$. Assume that the rack is constructed of 1/4-inch-wide bars with 3/4-inch openings. Find the width of the trashrack.

$$W_T = r \times W_N$$

$$W_T = 1.33 \times 3.75$$

$$W_T = 5 \text{ ft.}$$

4.4.2.5.5 Bundle Size--To size the rack bundles, you must use the wetted dimensions of the trashrack. ($L_W$ = length of wetted area, and $W_T$ = width of trashrack, as shown in Figure 4.4-16).

- Width of Bundle--Divide the trashrack width ($W_T$) into convenient widths, each approximately 12 inches wide. This sets the width of the bundle and the number of bundles. (Remember, it is advisable to have an extra bundle.)

- Length of Bundle--The length is the sum of the wetted length plus the extra length required to bring the rack to the top of the intake structure. Figure 4.4-11 shows the rack extended to the walkway so that debris can be easily raked onto the walkway.

To determine bundle length, go back to the forebay section (Subsection 4.4.2.4) and determine how high the top of the berm is above the design flow level. Add 6 inches to 1 foot to that distance so that the intake structure is above the berm. This distance is the additional height required for the bundle to reach the top of the intake structure (see Figure 4.4-16). The distance will have to be divided by the sin θ of the angle, as was the case for $L_W$ [Equation (4.4-8)]:

$$L_A = \frac{H_A}{\sin \theta} \qquad (4.4\text{-}11)$$

where

$L_A$ = additional length in feet

$H_A$ = additional height in feet

$\sin \theta$ = trigonometric function of the angle of incline of the trashrack, usually 45 or 60 degrees.

As with Equation (4.4-8), the equation can be rewritten for 45- and 60-degree inclines:

$$L_A = \frac{H_A}{0.707}, \text{ for 45 degrees} \tag{4.4-11a}$$

$$L_A = \frac{H_A}{0.866}, \text{ for 60 degrees.} \tag{4.4-11b}$$

EXAMPLE: Determine the length and width of trashrack bundles from the previous examples, where $W_T$ = 5 feet, and $L_W$ = 4.6 feet for a 60-degree angle of incline. The forebay berm is 2.5 feet above the design flow level.

Width of Bundle: 5 feet total; therefore make six bundles, each one foot wide (one extra bundle).

Length of Bundle: The design intake structure is 6 inches above the forebay berm; therefore, $H_A$ = 2.5 ft + 0.5 ft (6 in.) = 3 ft.

From Equation (4.4-11b) for a 60-degree incline:

$$L_A = \frac{H_A}{0.866}$$

$$L_A = \frac{3}{0.866}$$

$$L_A = 3.5 \text{ ft} \quad .$$

Thus,

$$L_T = L_W + L_A \qquad (4.4\text{-}12)$$

where

$L_T$ = total length of bundle in feet

$L_W$ = wetted length of bundle in feet, from Equation (4.4-8)

$L_A$ = additional length of bundle in feet, from Equation (4.4-11)

$L_T = 4.6 + 3.5 \text{ ft} = 8.1 \text{ ft.}$

Add a few extra inches so that the rack will be above the walkway, say to a total of 8.5 feet. Make the rack bundle 1 foot wide and 8 feet 6 inches long.

4.4.2.5.6 Trashrack Supports--Figure 4.4-17 shows some typical supports for the rack bundles. Each bundle can easily slide into its frame. The frame should be spaced with inside dimensions at least 1 inch wider than the bundle. This will help to ensure that the bundle does not bind in the frame.

The frame is constructed with 2- to 3-inch angle iron as shown in the figure. All connections are welded except where they are bolted to the intake structure.

4.4.2.5.7 Width of Trashrack Support Structure--The sum of the bundle widths plus the additional width required for support frames equals the width of the trashrack support structure.

EXAMPLE: From previous examples, $W_t$ = 5 feet; five bundles require six frames. Allow 2-1/2 inches additional width per bundle.

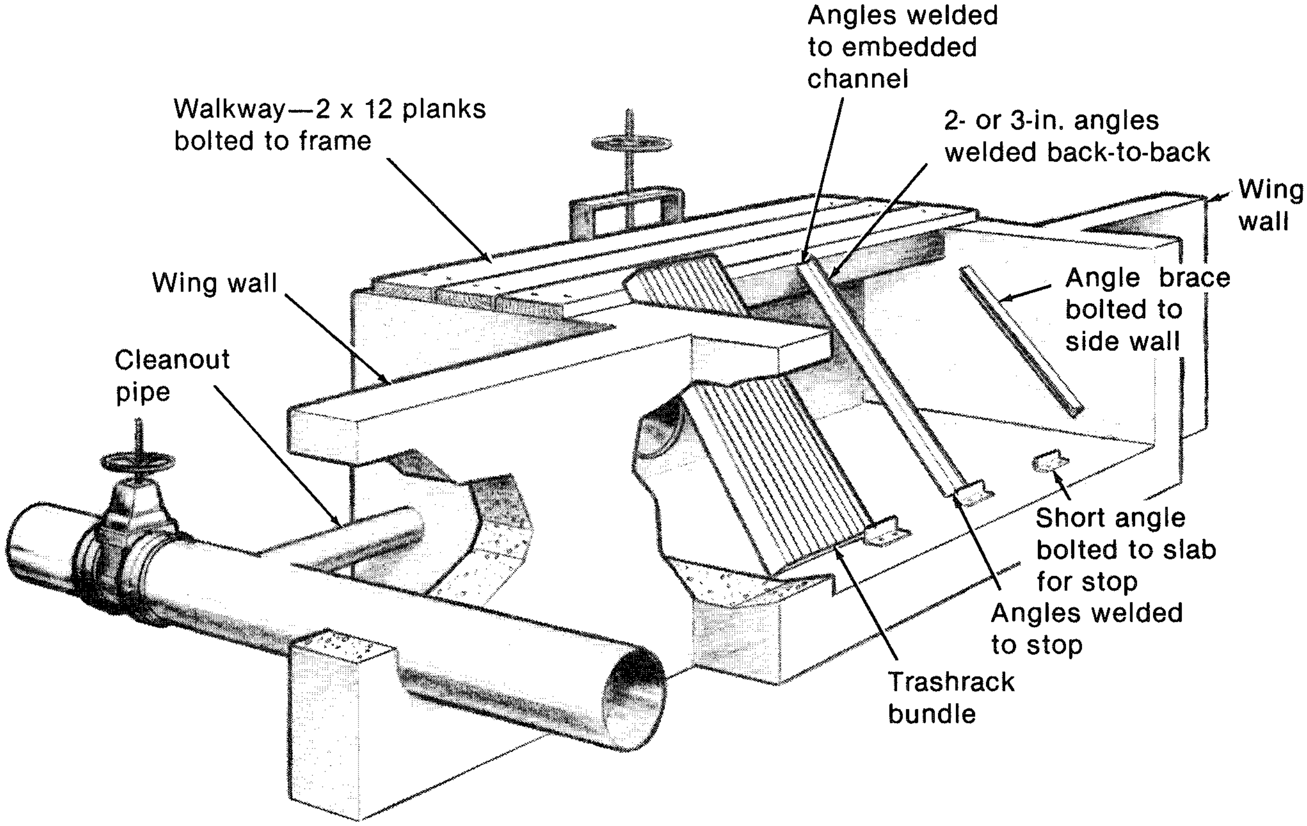


Figure 4.4-17. Penstock intake structure, showing typical trashrack supports, cleanout pipe, and wing walls.

Therefore, 6 x 2.5 = 15. Add an additional 15 inches to the width of the bundles. Thus, total width = 6 feet 3 inches.

4.4.2.5.8 Length of the Trashrack Support Structure--The length of the trashrack support structure $L_s$, in Figure 4.4-16, can be found by multiplying the total length of the trashrack by the cosine of θ.

$$L_s = \cos \theta \times L_T \qquad (4.4\text{-}13)$$

where

$L_s$ = length of intake structure required for trashrack in feet

$L_T$ = total length of trashrack in feet

$\cos \theta$ = trigonometric function of the angle of incline of the trashrack, usually 45 or 60 degrees.

Rewrite Equation (4.4-13) for 45- and 60-degree angles of incline:

$$L_s = 0.707 \times L_T, \text{ for 45 degrees} \tag{4.4-13a}$$

$$L_s = 0.5 \times L_T, \text{ for 60 degrees.} \tag{4.4-13b}$$

EXAMPLE: Assume that $L_T$ = 8.1 feet and that the angle of incline is 60 degrees. Find $L_s$ from Equation (4.4-13b).

$$L_s = 0.500 \times 8.1$$

$$L_s = 4 \text{ ft.}$$

4.4.2.6 Penstock Intake. The penstock intake provides a transition from the forebay to the penstock. The structure provides the following functions:

- Anchors the penstock
- Provides a framework for the trashrack and gates
- Diverts the water into the penstock.

The intake design will vary with each site. The size of the structure is dictated by the size of the trashrack; therefore, the trashrack dimensions must be determined as outlined in Subsection 4.4.2.5 before proceeding with the penstock intake.

The penstock intake structure generally will be constructed of concrete and should have steel reinforcement. If the structure is to be a large one, engineering services should be considered for design of the reinforcement. Figure 4.4-11 shows an example of a cross section of an intake structure. Points that should be considered in the design of a structure are:

- Bracing for the trashrack should be poured into, or attached to, the structural concrete.

- A walkway should be permanently attached above the rack to allow for cleaning of debris from the rack without interference of other equipment.

- A cleanout pipe is advisable.

- The penstock connection must be solidly mounted to the structure.

- Although not essential, a conical penstock intake provides a smoother water flow than does the butt end of a pipe and, consequently, loses less energy.

- The penstock intake should be far enough from the bottom of the structure to prevent the penstock from picking up debris off the bottom.

- The top of the penstock intake should be 1-1/2 pipe diameters below the low-water elevation. In areas where surface ice is a problem, the intake should be below the normal ice level.

CAUTION: The area at the top of the intake structure between the trashrack support and the structure's backwall should be sealed off at all times. The 2 x 12 wooden plank walkway shown in Figure 4.4-11 should be bolted down to prevent small animals or people from accidently falling into the water behind the trashrack and getting sucked into the penstock.

After the trashrack is sized, determine the dimensions of the intake structure. The width of the intake structure is the same as that of the trashrack support structure, which was determined in Subsection 4.4.2.5.7. The length of the intake structure is equal the length of the trashrack support [$L_s$, from Equation (4.4-13) in Subsection 4.4.2.5.8] plus the additional length required to allow an adequate trashrack cleanout area (see Figure 4.4-11). Usually 3 or 4 feet is sufficient for a working area.

EXAMPLE: From the examples in the trashrack section, the total width is 6 feet 3 inches; the length for rack supports is 4 feet. Find the dimensions of the penstock intake. Add 3 extra feet for cleanout to the length, for a total length of 7 feet; the width remains 6 feet, 3 inches.

The penstock intake should be constructed of reinforced concrete. Where the concrete will be poured in more than one step, use commercially available water stops in the concrete. Water stops are long strips usually made of rubber 8 to 12 inches wide. Half of the width is placed in the first pour and the other half is cast into the next pour. Lumberyards should have this material or something similar.

Water will tend to seep between the concrete and the earth fill around the structure. If the seepage is large enough, the earth fill will be washed away from the structure. To prevent this, increase the length of the seepage path by adding wing walls (see Figure 4.4-17). The length of the wall depends on the depth of the forebay. A rule of thumb would be to make the wall as long as the forebay is deep.

Make a sketch of the forebay and penstock intake for your site. Figure 4.4-18 is a sketch of the sample site, and Figure 4.4-19 is an artist's drawing of the same figure.

NOTE: For smaller intakes, the cleanout walkway can be located behind the back wall, as shown in Figure 4.4-19. (This feature is also shown in Figure 4.4-22.)

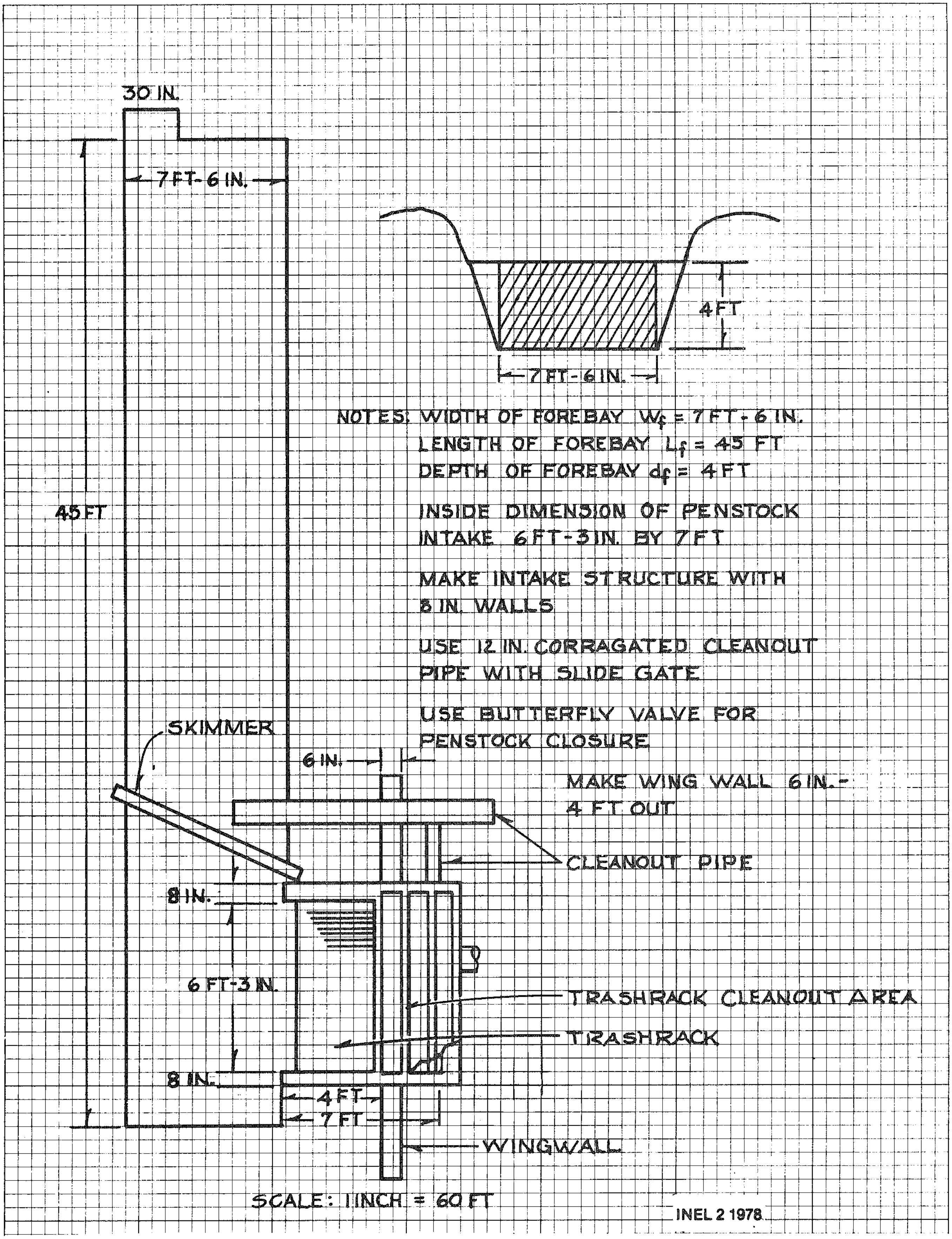


Figure 4.4-18. Sketch of forebay and penstock intake.

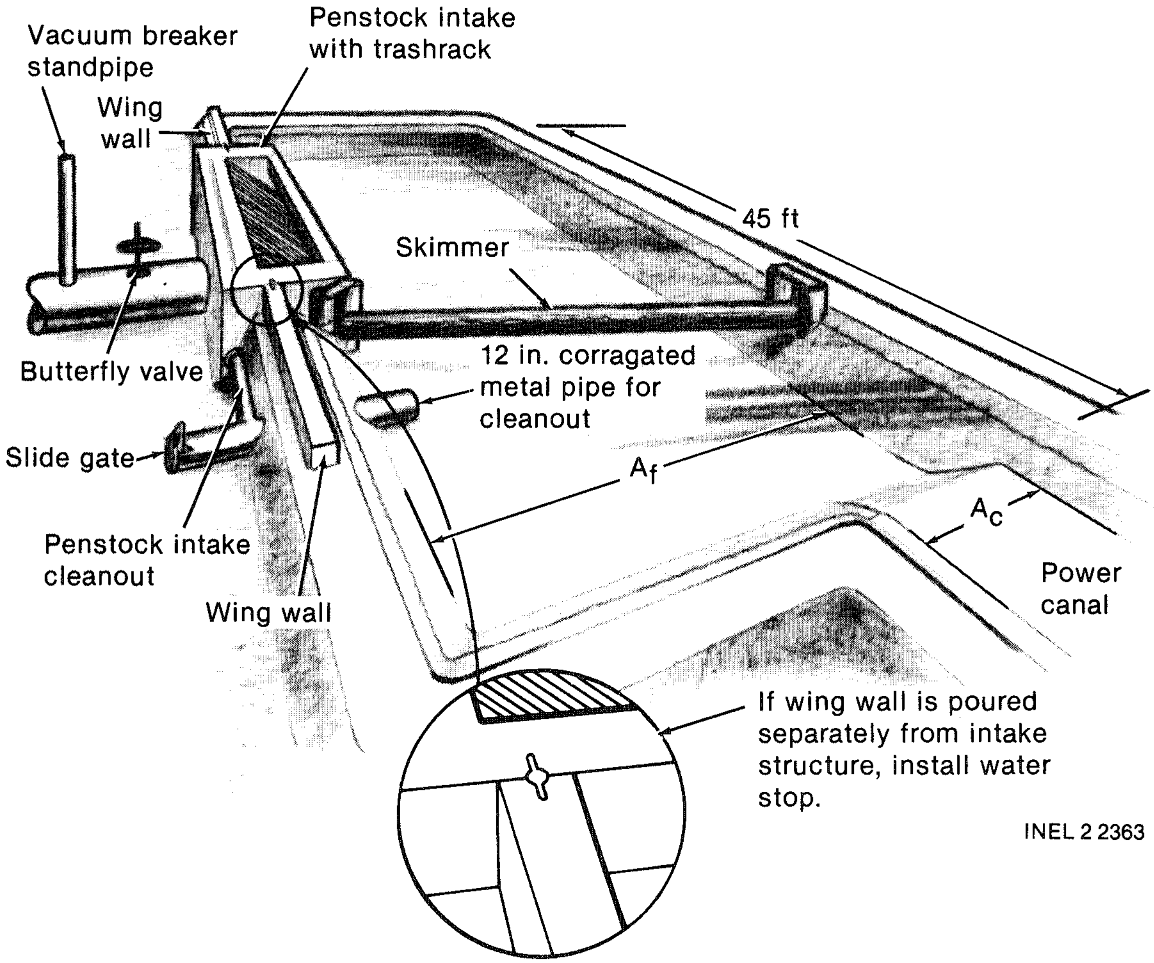


Figure 4.4-19. Forebay and penstock intake.

4.4.2.7 Additional Hardware. The following additional hardware items will be a part of most installations.

4.4.2.7.1 Skimmers--A skimmer is a floating log or something similar that skims trash floating on the surface, preventing further passage. At the diversion works, it prevents stream trash from entering the intake canal. In the forebay, it prolongs the timespan between trashrack cleanouts by diverting debris floating on the surface.

The skimmer should be set at angle to the stream flow. By angling the skimmer, the trash is forced to the downstream side and can easily be removed. Figure 4.4-20 shows a typical skimmer layout. The skimmer floats between two anchor posts on each side of the canal. Some authorities recommend that the skimmer be anchored down to prevent trash from working under it.

4.4.2.7.2 Stop Log Weir Check--In the canals at some location close to the entrances, a stop log weir check should be constructed. The check must be constructed properly so that the logs can be easily inserted in the case of an emergency. Two concepts for a stop log check are shown in Figure 4.4-21. One is made of logs and the other of poured concrete. Either method will serve the purpose equally well. The logs are stacked in the check so that they are readily available for use by pulling the pin. A canvas or a sheet of plastic placed in the canal upstream from the logs will settle against the logs and form a seal to stop any remaining seepage.

As a weir check, the water level in the canal downstream of the check can be controlled by raising or lowering the logs to restrict the canal opening.

4.4.2.8 Alternative Layouts. It is not always physically possible to construct an intake system as previously described. Whatever the configuration, the intake must take water from the stream and introduce it into the penstock. Figures 4.4-22 and 4.4-23 show two alternative intakes for a run-of-the-stream project in a narrow canyon.

### 4.4.3 Existing Dam Intakes

A suitable intake at an existing dam might be an open flume similar to that shown in Figure 2-12, where the water enters the flume through a trashrack, flows into the turbine, and exits through the tailrace. Another method is a penstock penetrating the dam; a third possibility is an open millrace (small wood or concrete-lined canal) that diverts the water to a water wheel or turbine intake.

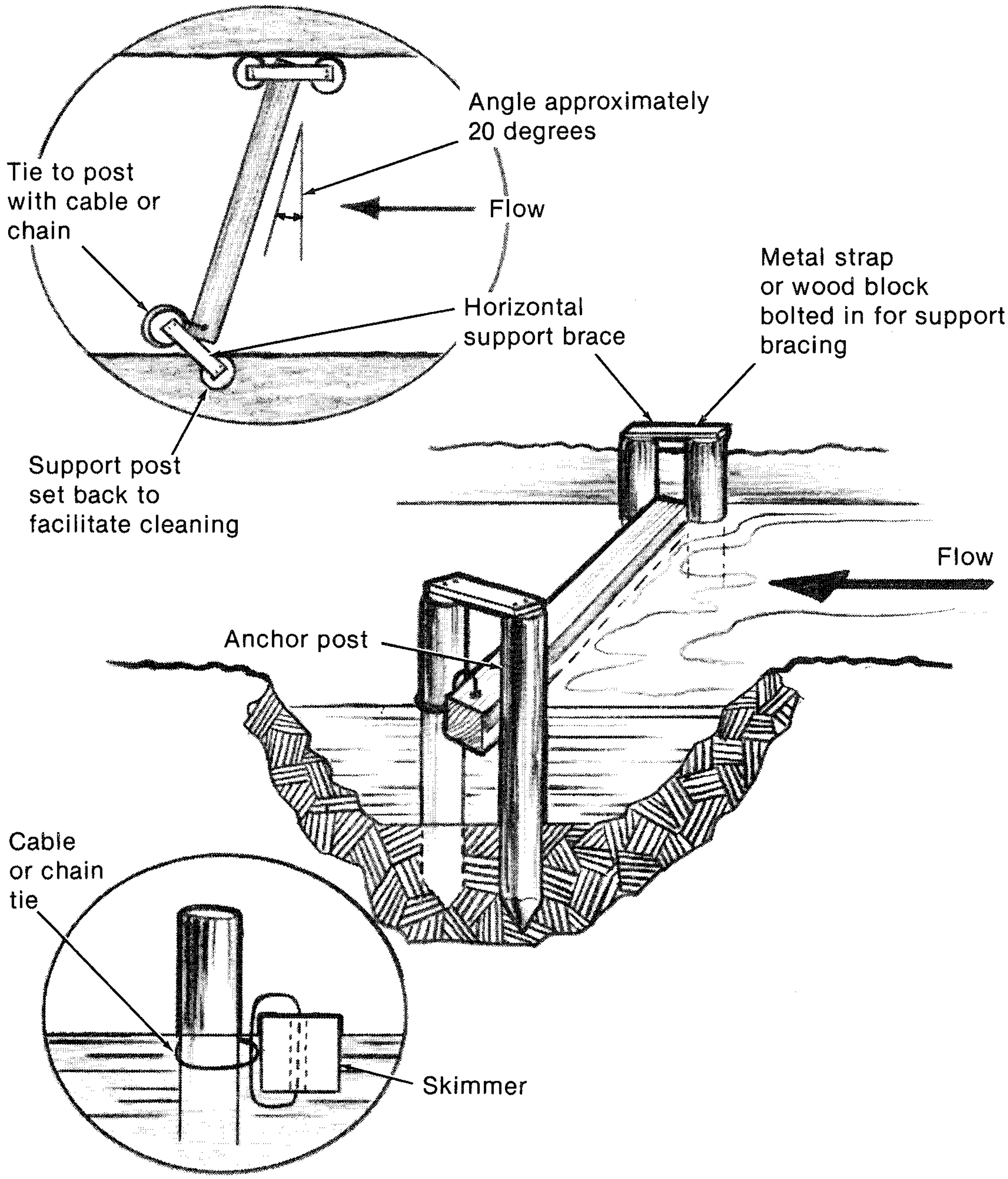


Figure 4.4-20. Typical skimmer layout.

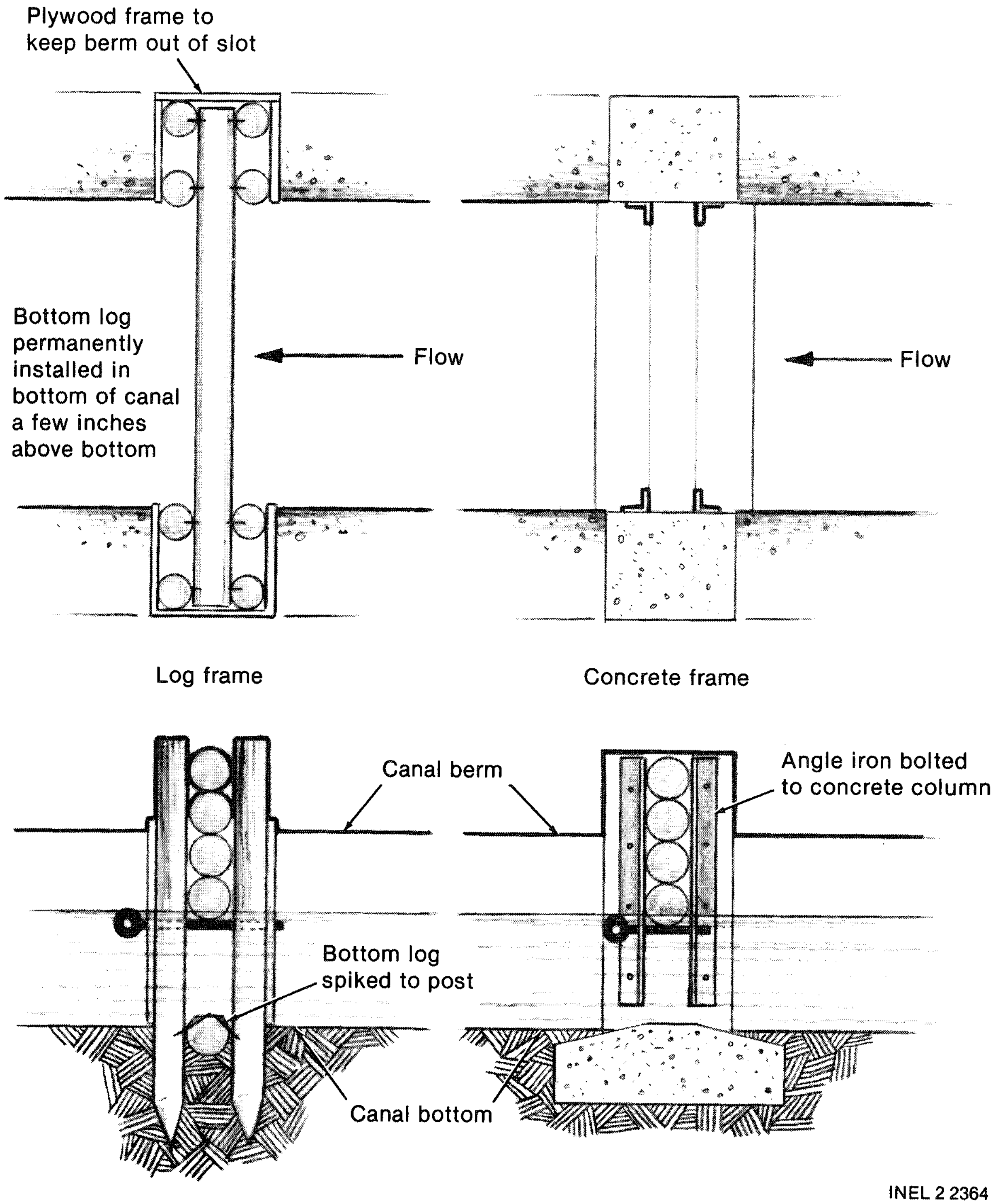


Figure 4.4-21. Stop log weir check configurations.

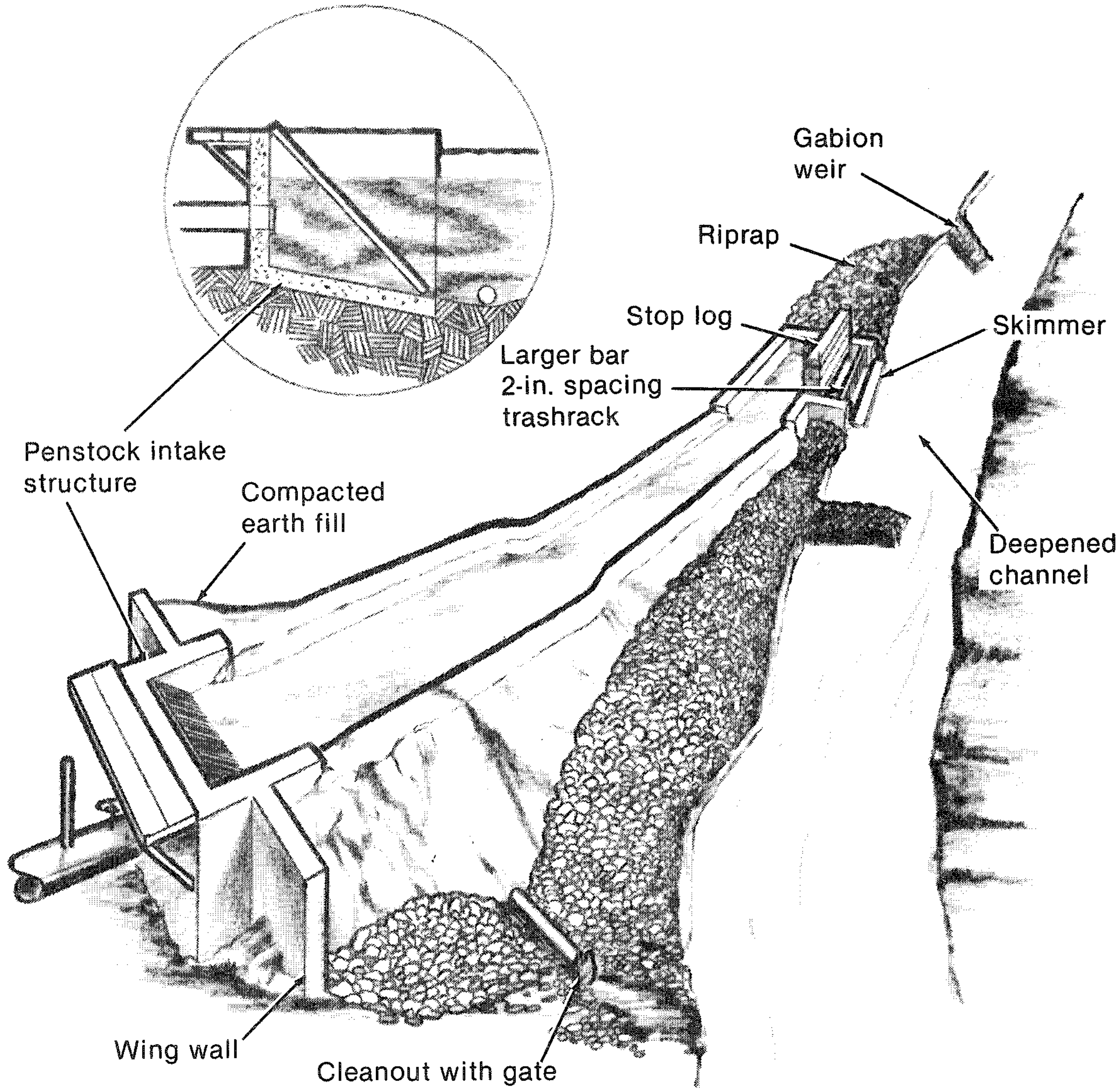


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Figure 4.4-22. Alternative layout for intake system.

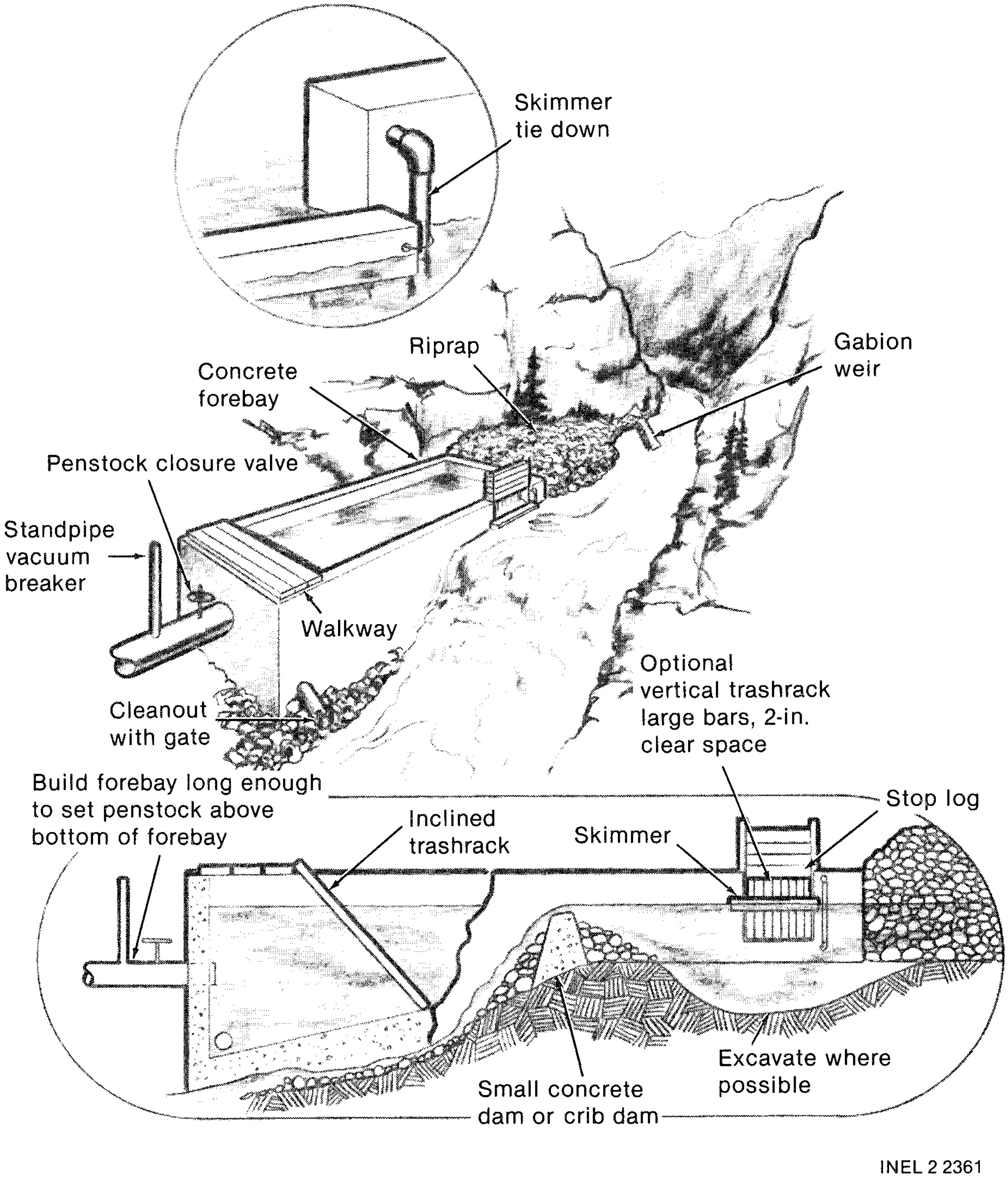


Figure 4.4-23. Alternative layout for intake system.

If the dam does not have one of the above features, some method will have to be devised to channel the water over, around, or through the dam in a form that can be used for power production. A siphon penstock is a possibility for moving water over the dam. To move it through the dam, the dam may have to be modified. Any modification to the structure of an existing dam should not be attempted without the direction of a professional engineer who has had experience in such modifications. Channeling the water around the dam would require a power canal that would take water out of the stream far enough above the dam so that the canal and penstock intake is above the dam crest. This type of system would use the run-of-the stream intake discussed in the preceding section.

4.4.3.1 Open Flumes. A dam with an open flume that dumps into a tailrace can have a turbine set between the two. There would be no penstock, but the system should include a trashrack (Subsection 4.4.2.5) and a stop log (Subsection 4.4.2.7).

4.4.3.2 Siphon Penstock. A siphon hydropower project works just like any other siphon. The penstock is run over the top of the dam, routed down the back of the dam, and connected to the turbine just above the tail water elevation. Siphons have a limit on how high they will lift water. If the lift is much more than 10 feet, a professional engineer should be consulted.

The siphon penstock should be located to one side of the dam to minimize exposure to floods. The siphon elbow (the bend that goes over the lip of the dam) may freeze in very cold climates because the pressure is lower at that point, and the freezing point will be slightly higher than normal. Most freezing problems can be solved by insulating the elbow.

The intake should be equipped with a trashrack similar to that in Figure 2-13. To compute the area of the rack, follow the procedures given in Subsection 4.4.2.6. If the first design for the rack results in a rack that is too large, the velocity can be increased to as high as 2 fps.

### 4.4.4 Design Layout

After you have decided on the type of intake, make a sketch or sketches of the system. As much as possible, make the sketches to scale and identify dimensions, materials, quantities, and anything else needed to ensure that all important points and cost factors have been considered.

If earth work is involved and you plan to hire the work done, the cost is generally based on an hourly rate or on the amount of material (earth) moved. The hourly rate and estimated number of hours will have to be obtained from a local contractor. If the cost is per yard, you should estimate the yards involved to verify the contractor figures.

$1\ yd^3 = 27\ ft^3$

To compute the volume in yards of material, first figure the volume in cubic feet, and then divide by 27:

$$V = \frac{L \times W \times d}{27} \qquad (4.4\text{-}14)$$

where

| | | |
|---|---|---|
| V | = | volume in cubic yards |
| L | = | length in feet |
| W | = | width in feet |
| d | = | depth of the excavation in feet |
| 27 | = | number of cubic feet per cubic yard. |

The volume of the material in a truck is 20 to 30% larger than the material volume was in the ground. The contractor will usually price the volume on the basis of what is in the truck. Therefore, if you buy material, multiply the computed volume from Equation (4.4-14) by 1.3 to estimate cost.

If concrete work is involved, the total number of cubic yards should also be computed. Small amounts of concrete can be prepared from ready-mix bags; for large amounts, consider either a portable concrete mixer or ordering direct from a ready-mix company.

Estimate the materials and cost, including labor cost. These figures will be added to the other costs determined in Section 5 to arrive at a total project cost estimate.