HYDROPOWER ENGINEERING HANDBOOK

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HYDROPOWER ENGINEERING HANDBOOK

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PREFACE

This book was written to fill a need in the hydropower industry; a text for mechanical, electrical, and civil engineers to develop background in the multidisciplinary field of hydropower development, and at the same time a reference book for the many practicing hydropower engineers. Because hydropower development involves so many disciplines, this book cannot serve as an all-encompassing handbook, but will, we hope, be the first place an engineer looks to find a short explanation of the particular phenomenon in question, some design and operational guidelines, and references to other, more specific publications. Thus, the *Hydropower Engineering Handbook* is midway between a text and a more traditional handbook, providing interdisciplinary information that addresses a wide variety of topics, relates them to hydropower engineering, and references other sources of information on a given subject. This approach has proved most successful in a four-day short course on hydropower development we have offered over the past eight years, and we hope will be successful as a handbook.

A number of handbooks on hydropower engineering ceased publication in the 1960s because of inexpensive oil and the subsequent decline of hydropower development. Those valuable reference books that hydropower engineers use to this day are currently available in only a few libraries and are not accessible to most engineers. Additionally, a significant amount of updating is needed and new techniques and instrumentation need to be described. Environmental impacts, for example, which only became a significant issue during the 1970s, today represent a major portion of any hydropower development project. The operation of hydropower systems for maximum benefit has also changed considerably in the past twenty years with the advancements in computer technology and improvements in optimization techniques. The instrumentation for plant maintenance has significantly greater capabilities and is more difficult to operate than thirty years ago. These are a few of the areas where new techniques predominate. Throughout the *Handbook,* each chapter incorporates newly developed procedures in the engineering of hydropower projects.

The contributors were selected for their expertise in the various disciplines that hydropower engineering comprises and their experience in applying the discipline to hydropower development projects. Brief identification of each, with addresses, is given under Contributors. They, as well as the editors, are eager to hear your response to the book, and questions or suggestions that you may have that can assist in improving the quality and applicability of the *Handbook*. The logic of the book follows the normal hydropower development sequence, beginning with preliminary investigations, equipment design and specification, site design, environmental impacts, and finally plant operation. The book can serve as a university-level text on hydropower, with numerous examples and case studies, as well as a reference text for practicing engineers.

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HYDROPOWER ENGINEERING HANDBOOK

CHAPTER 1

INTRODUCTION TO HYDROPOWER ENGINEERING

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Hydropower engineering encompasses many branches of engineering and other disciplines for the purpose of hydropower development. Mechanical engineering is involved in the design, manufacture, and selection of the turbine, bearings, valves, gears, governors, etc., needed to convert hydraulic to mechanical energy. Electrical engineering is involved in the design, manufacture, and selection of the generators, control systems, switchgear, transformers, etc., required to convert mechanical to electric energy. Civil engineering is involved in matters needed to place the machinery in position to extract the available hydraulic energy, such as: site hydrology; hydraulics; inspection and preparation; and dam, powerhouse, and conveyance facilities design and construction. The environmental impacts of a hydroplant are assessed and mitigated by ecologists and civil engineers. The economic analysis is performed by engineers, planners, developers, and others. An individual plant is operated and maintained under the supervision of mechanical and electrical engineers. Finally, the schedule and planning of operation to optimize power production within a system is performed by civil engineers (for reservoir operation) and electrical engineers (for complete system operation). Although these categories are not all-encompassing, and there is a substantial variation and overlap between disciplines, the variety of engineering activities which go into developing a hydropower facility is readily apparent.

In addition, hydropower development is an endeavor which is not very amenable to standardization, making each project an interesting engineering challenge. The engineering team must work with the conditions at a given site to develop a hydropower facility which is functional and economically sound. Because of the uniqueness of each site, a wide variety of dams, turbines, intakes, generators, fishways, etc., is found at various hydroelectric facilities. At each site there are numerous opportunities, often requirements, for innovative applications or designs. Hydropower development engineering is thus interesting and professionally rewarding, in spite of the many difficulties one can encounter in developing a given site.

1.1 HISTORY

Falling or flowing water has been used to perform work for thousands of years, the particular uses varying with the social and political conditions of the times. Although the Romans knew of waterwheels, these laborsaving devices were not used extensively until the fourteenth century [1]. Early tasks included grinding grain, sawing wood, powering textile mills, and later operating manufacturing plants. Mills or factories were located at the hydropower sites in order to directly utilize the available energy. The power output of these early plants, usually limited to 100 or so kW (134 hp), is compared with other power sources in Fig. 1.1 [2]. By the end of the eighteenth century, there were approximately 10,000 waterwheels in New England alone [1].

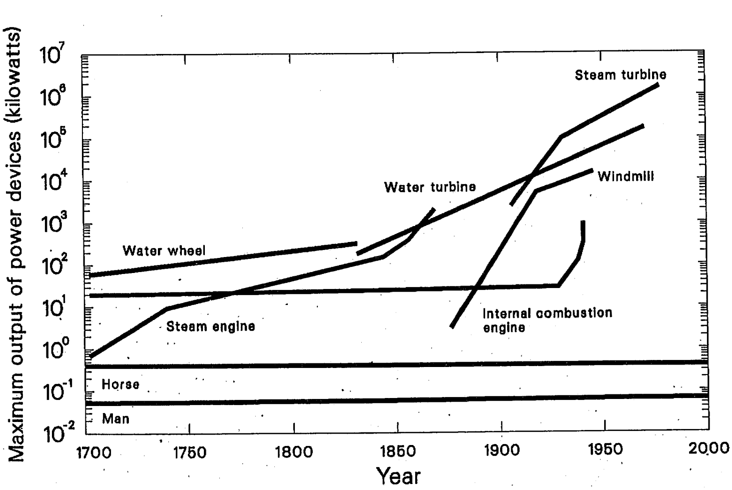


FIGURE 1.1 The maximum power output of selected power devices over the period 1700–1970. *(From Chappel [2].)*

During the nineteenth century, hydropower became a source of electrical energy. Although some form of hydroelectric turbine development can be traced back as far as 1750, Benoit Fourneyron is credited with developing the "first" modern turbine in 1833. A more extensive history of turbine development is discussed in Chap. 4. The first hydroelectric plant in the United States is usually documented as coming on-line September 30, 1882, in Appleton, Wisconsin. There is some dispute over this, however; Merritt [3] cites the Minneapolis Brush Electric Company as beginning operation of a hydroelectric plant some 25 days earlier. The generation of electricity from falling water expanded the need for larger hydroelectric plants because the energy did not need to be used on site.

The transmission of power over long distances became economical in the United States in 1901 with the installation of alternating current equipment at Niagara Falls in New York State, by George Westinghouse, further expanding the potential uses of hydropower.

As Fig. 1.1 indicates, the power capabilities of water turbines became larger as the need grew. In the 1930s, large dams and ever-increasing turbine capacities became the norm. The power capacity of steam turbines was also increasing rapidly, and the relative cost of electricity continued to fall. Finally, in the period 1940–1970, the cost of operating and maintaining older, smaller hydroelectric plants became greater than the income they could produce, and many were retired. This is seen in Fig. 1.2, where small hydropower capacity decreased as overall hydropower capacity climbed rapidly in the United States [4]. A similar trend occurred in European countries. Hydropower development in other parts of the world was insignificant before 1930, as indicated by world hydropower pro-

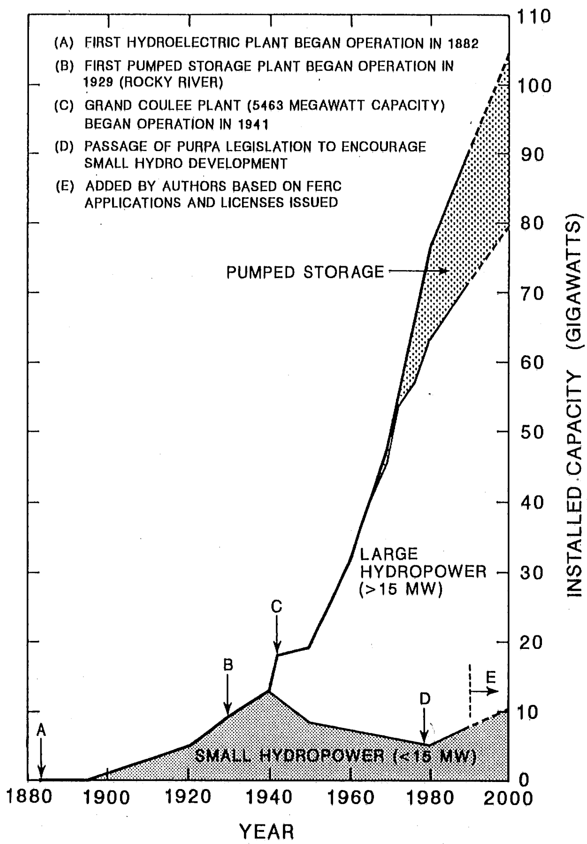


FIGURE 1.2 Installed hydroelectric capacity in the United States, 1882–2000. *(From Federal Energy Regulatory Commission [4].)*

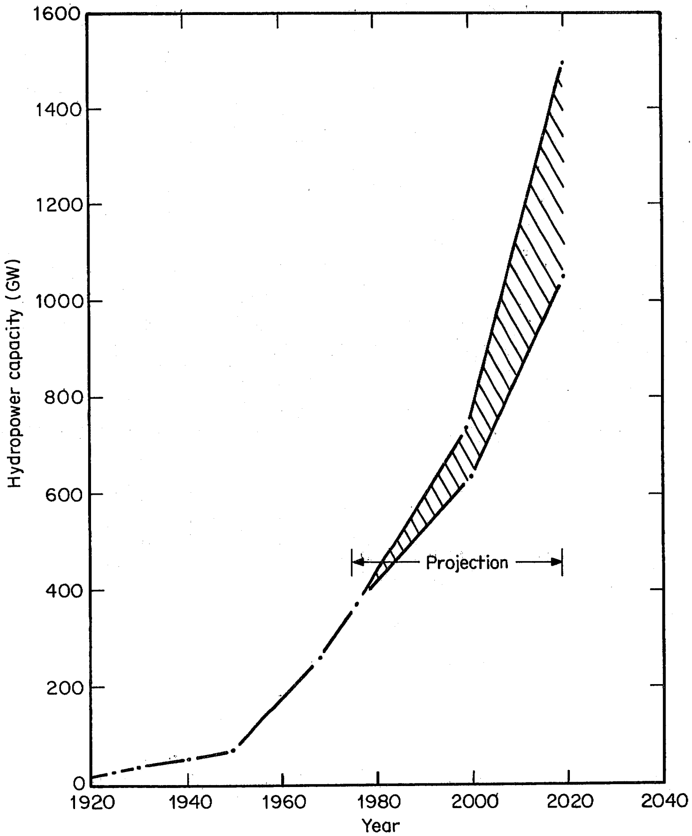


FIGURE 1.3 World hydropower production. Projection from 1983 World Energy Conference. Past production assembled from various sources.

duction (Fig. 1.3). The acceleration of worldwide hydropower development is projected to continue, although the rate of hydropower development in Europe, the United States, and Japan is expected to decrease, as discussed in the next section.

1.2 HYDROPOWER POTENTIAL

On a worldwide basis, hydropower represents approximately one-quarter of the total electrical energy generated. Predictions to the year 2000 indicate that this fraction will remain constant, while hydroelectric energy will grow by 85 percent over the 1979 generation levels [5]. According to Armstrong [6], there are approximately 2200 GW (2.95×10^9 hp) of developed and potential hydropower existing in the world. The total potential is that considered developable based upon physical, economic, and environmental considerations. It represents 12 percent of the total energy in the world's rivers [6]. The available and developed energy and

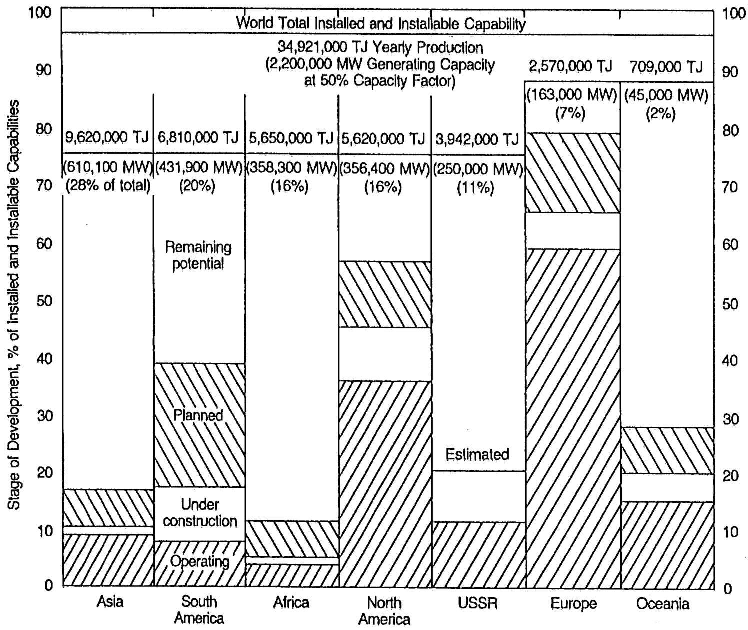
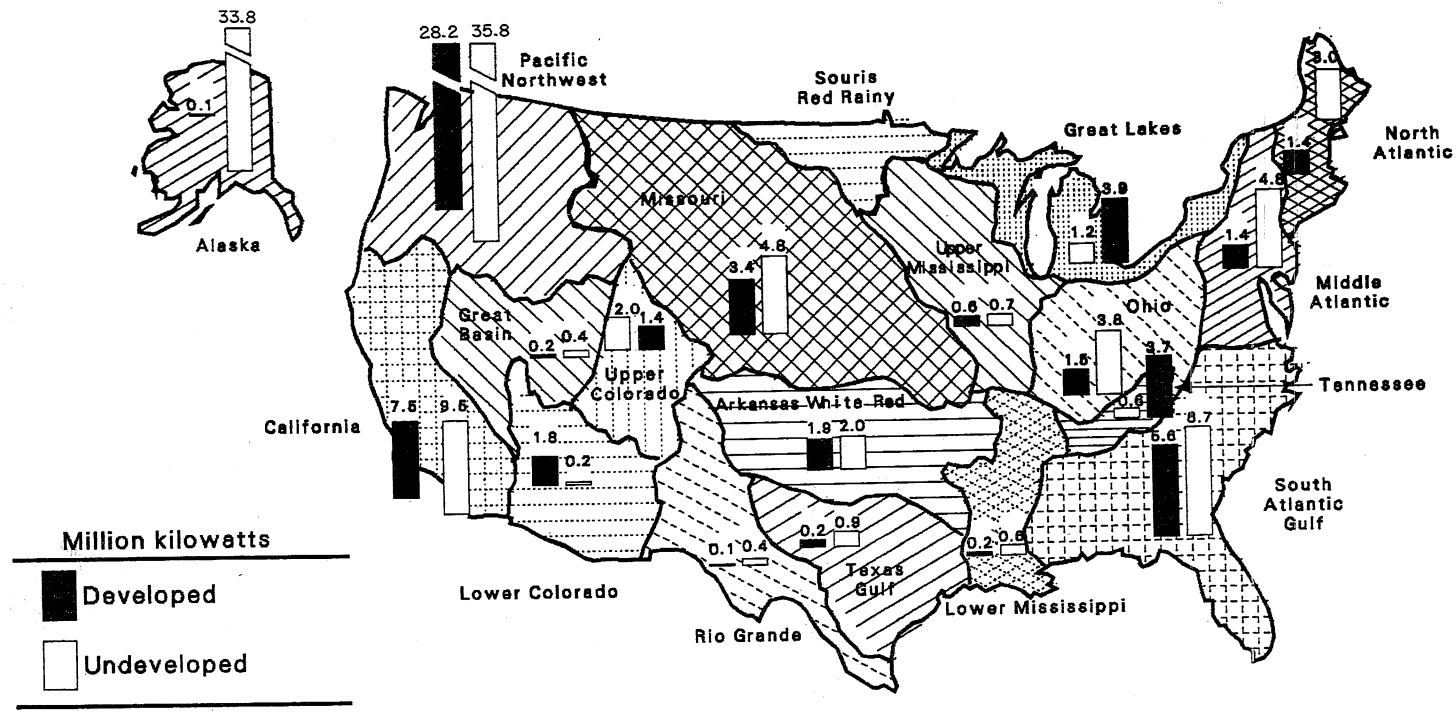


FIGURE 1.4 World hydropower resources as determined by the World Energy Congress, 1976. *(From Armstrong [6].)*

power (assuming a 50 percent capacity factor) in 1976 is given by continent in Fig. 1.4. The tremendous potential, planned or remaining, in Asia, Africa, and South America is apparent, amounting to half the world's total hydropower potential. On the other hand, hydropower developed or under construction in Europe and North America is at 65 and 47 percent of total potential, respectively. Most hydropower developed in these two continents in the future is likely to be pumped storage.

A 1983 study of the World Energy Conference (WEC) predicted that hydropower production would grow from 17 percent of the 2.2 million MW (2949 million hp) produced in 1976 to between 29 and 34 percent in the year 2000 and to between 48 and 68 percent in the year 2020 (Fig. 1.3). Thus hydropower capacity throughout the world is predicted to expand by approximately 350 percent in 44 years. *International Water Power and Dam Construction* [7] has recently completed a world survey of hydroelectric resources, which indicated that 549 GW (7.4 million hp) of hydropower had been developed by the end of 1988. This corresponds fairly well with the WEC projections.

An extensive effort in quantifying hydropower resources in the United States has been undertaken recently. The Federal Energy Regulatory Commission (FERC) [4] has classified developable hydropower potential by water resources regions, normally river basin, as shown in Fig. 1.5 and listed in Table 1.1. This study indicated that there are 1384 sites developed with a total capacity of 63,000 MW (84,450,000 hp) or a mean of 45.7 MW (61,260 hp) per site. There are 2093 undeveloped sites with a potential capacity of 111,000 MW (149 million hp), or a mean of 53 MW (71,045 hp) per site. Thus, the mean capacity of each site is not



The hydro potential of Hawaii is relatively small and thus is not shown

FIGURE 1.5 Conventional hydroelectric power developed and undeveloped—January 1, 1980, by river basin in the United States. *(From Federal Energy Regulatory Commission [4].)*

TABLE 1.1 Conventional Hydroelectric Power in the United States: Developed, Undeveloped, and Total Potential, January 1, 1980

By water resources regions

	Developed			Undeveloped			Total Potential		
Water resources region	No. of plants	Installed capacity (kW)*	Average annual generation (1,000 kWh)†	No. of sites	Installed capacity (kW)‡	Average annual generation (1,000 kWh)	Installed capacity (kW)	Average annual generation (1,000 kWh)	Percentage of total potential capacity now developed
North Atlantic	200	1,419,068	5,752,757	207	2,950,468	7,682,883	4,369,536	13,435,640	32.4
Mid-Atlantic	106	1,439,157	5,914,899	164	4,845,923	13,084,001	6,285,080	18,998,900	22.8
South Atlantic–Gulf	119	5,611,442	13,877,242	180	6,684,297	12,299,726	12,295,739	26,176,968	45.6
Great Lakes	203	3,876,391	24,175,432	126	1,199,702	3,678,558	5,076,093	27,853,990	76.3
Ohio	35	1,528,292	5,555,734	80	3,810,078	10,771,836	5,336,370	16,327,570	28.8
Tennessee	47	3,736,150	16,250,600	14	629,270	1,777,060	4,365,420	18,027,660	85.5
Upper Mississippi	99	605,476	3,116,689	51	687,000	2,910,378	1,292,476	6,027,067	46.8
Lower Mississippi	5	205,800	430,400	20	613,500	2,179,775	819,300	2,610,175	25.1
Souris–Red Rainy	8	13,600	68,000	1	250	1,000	13,850	69,000	98.1
Missouri	58	3,419,730	15,322,125	154	4,765,468	18,070,432	8,185,198	33,392,557	41.7
Arkansas–White–Red	23	1,851,912	5,336,046	57	1,975,050	4,501,990	3,826,962	9,838,036	48.3
Texas–Gulf	17	392,830	1,059,559	40	856,190	1,184,334	1,249,020	2,243,893	31.4
Rio Grande	4	65,408	233,521	18	369,900	1,039,300	435,308	1,272,821	15.0
Upper Colorado	21	1,351,522	5,857,895	58	1,963,150	8,102,400	3,314,672	13,960,295	40.7
Lower Colorado	16	1,815,063	6,228,900	11	190,400	980,000	2,005,463	7,208,900	90.5
Great Basin	54	180,520	642,750	24	385,565	771,630	566,085	1,414,380	31.8
Pacific Northwest	160	28,164,684	138,182,730	583	35,784,379	109,842,229	63,949,063	248,024,959	44.0
California	173	7,501,281	34,532,770	197	9,457,007	27,367,663	16,958,288	61,900,433	44.2
Alaska	21	122,102	503,242	106	33,347,305	174,577,849	33,469,407	175,081,091	0.3
Hawaii	15	18,152	104,400	2	35,000	229,000	53,152	333,400	34.1
Total United States	1384	63,316,580	283,145,691	2093	110,549,902	401,052,044	173,866,482	684,197,735	36.4

*1 kW = 1.341 hp.
†1 kWh = 1.341 hph.
‡Includes potential capacity additions or subtractions at existing plants.
Source: Federal Energy Regulatory Commission [4].

increasing significantly, the reason being that, in the United States, there will probably not be an extensive development of sites that require large dams. The *National Hydroelectric Power Resources Study* by the U.S. Army Corps of Engineers [8] is less enthusiastic. Using economic feasibility criteria, this study came up with 46,000 MW (61.7 million hp) that may be developed at 1948 sites, or a mean of 24 MW (32,171 hp) per site.

The largest hydropower facility in the world is currently the Guri Dam in Venezuela, completed in 1986. This dam's 10,000-MW (13,404,825-hp) generating capacity is the equivalent of 10 large nuclear power plants. Brazil and Paraguay are in the process of constructing a 12,600-MW (16,890,080-hp) hydroplant at the Itaipu, and China is studying the feasibility of an even larger project at Three Gorges. In all, projections are that approximately 170,000 MW (228 million hp) of large hydroelectric capacity will be added in developing countries between 1986 and 2000. More than half of this projected capacity increase will be in Brazil, China, and India [9]. The world's largest-capacity hydroelectric plants in 1988 are listed in Table 1.2 [10].

TABLE 1.2 World's Largest-Capacity Hydroelectric Plants as of 1988

Rank order	Name	Country or countries	Height above lowest foundation (m)*	Current rated capacity (MW)†	Year of initial operation
1	Guri‡	Venezuela	162	10,600	1968
2	Grand Coulee	USA	168	7,460	1942
3	Itaipu‡	Brazil/Paraguay	196	7,400	1983
4	Sayano Shushensk	USSR	245	6,400	1980
5	Krasnoyarsk	USSR	124	6,000	1968
6	La Grande 2	Canada	168	5,328	1979
7	Churchill Falls	Canada	32	5,225	1971
8	Bratsk	USSR	125	4,500	1961
9	Tucurui‡	Brazil	93	4,000	1984
10	Ust-Ilim	USSR	102	3,840	1977
11	Ilha Solteira	Brazil	74	3,200	1973
11	Brumley Gap	USA	78	3,200	1973
13	Xingo	Brazil	140	3,012	1987
14	Bennett, W.A.C.	Canada	183	2,730	1968
15	Mica	Canada	242	2,660	1976
16	San Simeo	Brazil	120	2,680	1979
16	Volvograd	USSR	47	2,563	1958
17	Paulo Alfonso IV	Brazil	33	2,460	1979
18	Cabora Bassa‡	Mozambique	171	2,425	1975
19	Chicoas'en	Mexico	261	2,400	1980
20	Gezhouba	China	47	2,340	1975
21	LaGrande 3	Canada	93	2,304	1982
22	Volga—V.I. Lenin	USSR	45	2,300	1955
22	Iron Gates	Romania/Yugoslavia	60	2,300	1970

*1 m = 3.28 ft.
†1 MW = 1340.48 hp.
‡Additional capacity planned or under construction.
Source: Mermel [10].

The potential for small hydropower (< 15-MW, or 20,107-hp) development is more varied, depending upon government policy. Small hydropower is engineering-intensive (and management-intensive), in that virtually all the issues of a large hydroplant are involved and must be dealt with at a fraction of the cost. Of course, the financial risks are not as great as with large-capacity plants, and so the required level of detail is significantly lower. Still, up-front costs of small hydropower, incurred before a decision is made to develop the site and obtain financing, are significant compared to the potential benefits. Thus, government encouragement in the form of grants, tax incentives, loans, etc., for the development of small hydropower sites is essential. Some of the countries where small hydropower has been developed successfully over the years are listed in Table 1.3 [11]. Other countries, such as the Philippines, Pakistan, Indonesia, Turkey, Peru, and the United States have instituted programs during the 1980s to encourage small hydropower. In fact, a World Bank survey of 100 developing countries found that 28 have small hydropower programs.

There are two other basic sources of hydropower, besides that which is extracted from the world's rivers—tidal power and wave power. Small tidal mills to provide mechanical power existed hundreds of years ago. However, the first hydroelectric tidal plant was developed in the 1960s on the La Rance estuary in northern France. Producing up to 240 MW (321,716 hp) of power, this tidal plant utilizes a dam across a cove mouth to form a pond. Sluice gates open to let water flow in during the rising tide, and then close with the returning tide as water is directed through a standard hydroturbine [9, 12].

The feasibility of tidal power depends upon the range of tide experienced and upon finding a location where an inordinately long dam does not need to be built. Thus, only a handful of tidal plants have been developed since the La Rance plant: a 10-MW (13,405-hp) plant (1986) and a number of smaller plants in China; an 18-MW (24,129-hp) plant (1984) at an existing flood structure at Annapolis Royal in Nova Scotia, Canada; and a 400-KW (536-hp) plant in the Soviet Union [8]. The turbines at Annapolis Royal are of interest because they can turn in both directions, thus capturing energy from both the incoming and the outgoing tides.

The United Kingdom is studying the feasibility of building a dam across the Severn estuary and generating up to 7000 MW (9,383,378 hp) of power (5 percent of the country's current electricity demand) with 192 hydroturbines. Canada is undertaking a similar study on the Bay of Fundy [9]. Both the economics and the environmental impacts of these proposed projects need to be assessed carefully.

TABLE 1.3 Small Hydroplants (< 10 MW)* in Selected Countries

Country	Year	Number of small (< 10 MW) plants	Total capacity (MW)	Plant size (MW)	Average percentage of generating capacity
Japan	1972	1,350	7,000	5.00	6
China	1979	88,000	5,400	0.06	16
Norway	1968		2,200		
France	1972	2,200	1,800	0.80	4
Finland	1975	175	380	2.00	5
Turkey	1973	110	70	0.64	

*1 MW = 1340.48 hp.
Source: Feder [11].

All tidal power projects are low-head, up to 36 ft (11 m). Since the power a project develops is proportional to net head, H, and since the cost of developing the power has been found to be approximated by $H^{-0.35}$, the cost/kW (cost/hp) produced is approximately proportional to $H^{-1.35}$. Obviously, one must carefully plan any low-head hydropower project.

1.3 TYPES OF DEVELOPMENT

The broad variety of natural conditions at hydropower sites has resulted in development of many different types of hydropower schemes. This is a disadvantage as far as engineering costs are concerned, but it provides a great deal of stimulus to those involved in hydropower development.

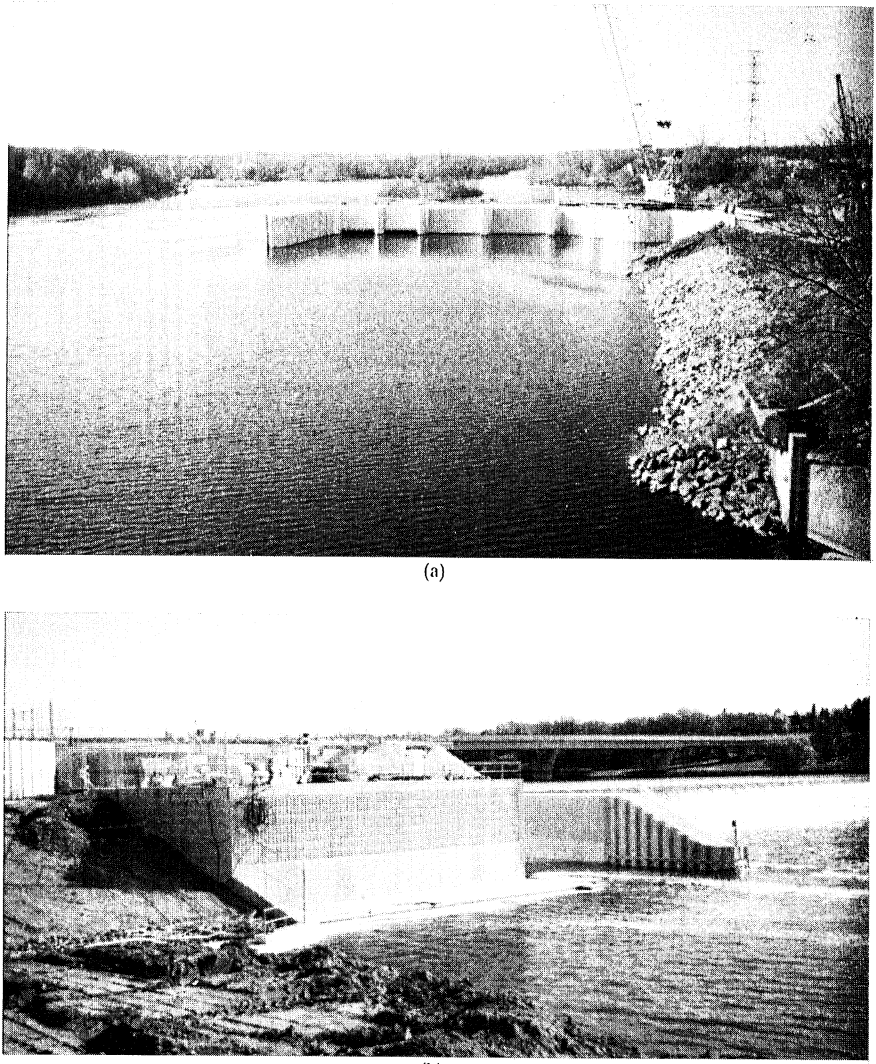
In a *run-of-the-river development*, a short penstock or dam directs the water through the turbines. The powerhouse is often an integral part of the dam, as shown in Fig. 1.6. The natural flow of the river remains relatively unaltered with the exception of oxygen content, as discussed below. A more complex development occurs at *diversion or canal projects*, where the water is diverted from the natural channel into a canal or long penstock, as shown in Fig. 1.7. This results in a significant change in the flow of water in a given reach of the river, sometimes for a considerable distance.

Storage regulation developments are defined as those in which an extensive impoundment at the power plant, or at the reservoir upstream of the power plant, allows for regulation of the flow downstream through storage. Water is stored during high-flow periods and is used to augment the flow during low-flow periods. This allows for a relatively constant supply of energy over the year. Significant storage is normally only used in large base-load plants. The word "storage" is used for long-term impounding of water to meet the seasonal fluctuation of water availability, whereas the word "pondage" refers to short-term storage of water, usually on a daily basis, to meet the diurnal variations in power demand.

Pump storage facilities are normally large developments in which water is pumped from a lower reservoir during off-peak hours when the cost of energy is low. The pumps are run in reverse as turbines during the hours of peak demand to augment the power supplied from other sources. Thus, the pumped storage facility serves the same function as a large battery. It is used to allow large base-load facilities to operate at continuous power output in cases where it is uneconomical to allow the power output of a large plant to fluctuate.

Tidal power plants have also been developed or considered. These are located in areas where there are large tidal fluctuations. Low-head turbines are used to harness the energy in the tidal cycle, and an entire bay or estuary is enclosed by a low dam.

Hydropower schemes can either be *single-purpose*, having as their only purpose the production of electricity, or *multipurpose*, having hydropower production as just one aspect of the total utilization of the facility. Multipurpose facilities include those in which hydropower is developed in conjunction with irrigation, flood control, navigation, and water supply. Hydropower plants are also categorized by the type of utilization. For example, a base-load plant is one in which the power is used to meet all or part of a sustained and constant portion of the electrical load. Energy from these plants which is available at all times is referred to as "firm power." The need for power varies during the day, and power requirements over and above the base-load requirement are met by peak-



(a)

(b)

FIGURE 1.6 The 8.4-MW (11,260-hp) St. Cloud hydropower facility is a run-of-river development with the powerhouse composing a portion of the dam. Net head is 17 ft (5.2 m). (*a*) Upstream view of the spillway and intakes into gates and powerhouse. (*b*) Downstream view of the powerhouse and spillway. *(Courtesy of the M. A. Mortenson Co.)*

load facilities. These are plants in which the electrical production capacity is relatively high and the volume of water discharged through the units can be changed readily to meet peak demands. Storage or pondage of the water supply is necessary for these load demands, since such plants can be started and stopped more rapidly and economically than fossil-fuel and nuclear power plants.

A *small hydropower facility* is defined herein as one which has less than 15-

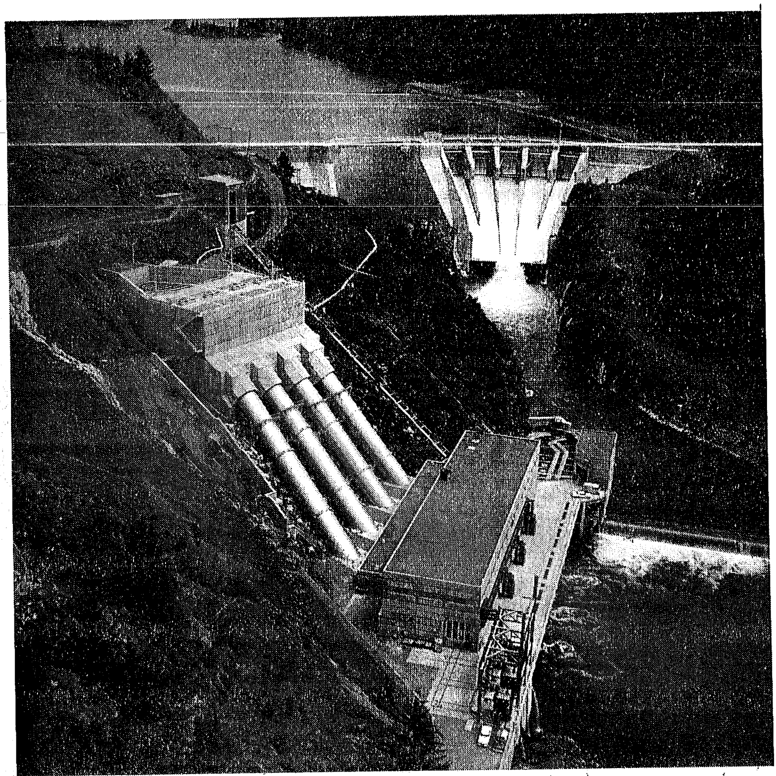


FIGURE 1.7 The Mayfield project on the Cowlitz River in Washington is composed of an arch dam and spillway (*background*), a power tunnel leading to surge tanks (*left*), and penstocks leading to a 122-MW (163,539-hp) powerhouse (*center front*). Notice how the layout maximizes efficient use of the terrain. *(Courtesy of Harza Engineering.)*

MW (20,107-hp) total capacity, such as that shown in Fig. 1.8. Minihydropower facilities are those less than 1 MW (1340 hp), and microhydropower facilities are those less than 100 kW (134 hp). Examples of mini- and microhydropower facilities are given in Figs. 1.9 and 1.10.

1.4 COMPONENTS OF HYDROPOWER FACILITIES

A typical hydropower facility consists of the following components:

1. The powerhouse structure and its foundation
2. Hydraulic conveyance facilities, which include the head race, headworks, penstock, gates and valves, and tailrace

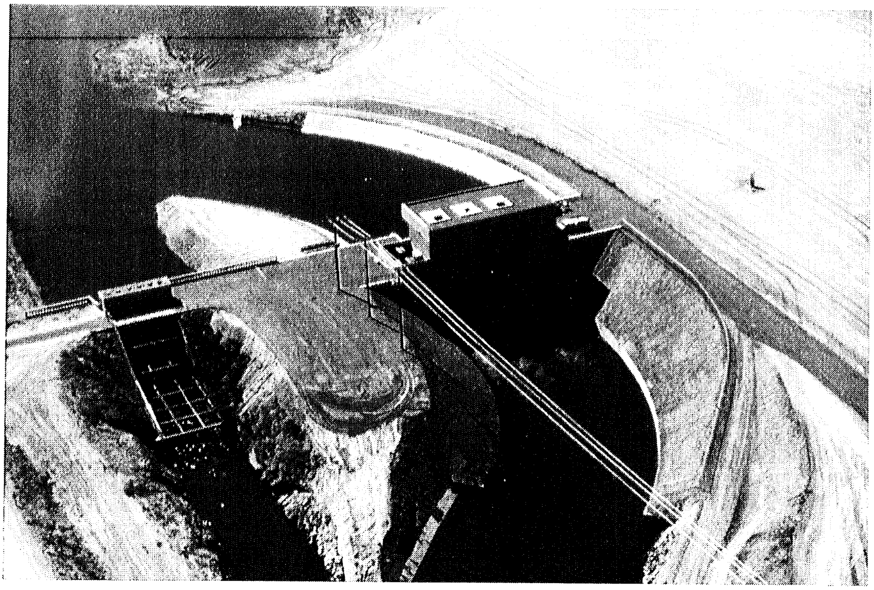


FIGURE 1.8 Turlock Irrigation Districts Drop No. 1 Project is located at the outlet of Turlock Lake Reservoir. The capacity of the facility is 3.26 MW (4370 hp), producing 12 GWh (160,857 hp) on the average per year. *(Courtesy of the Small Hydropower Demonstration Programs, U.S. Department of Energy.)*

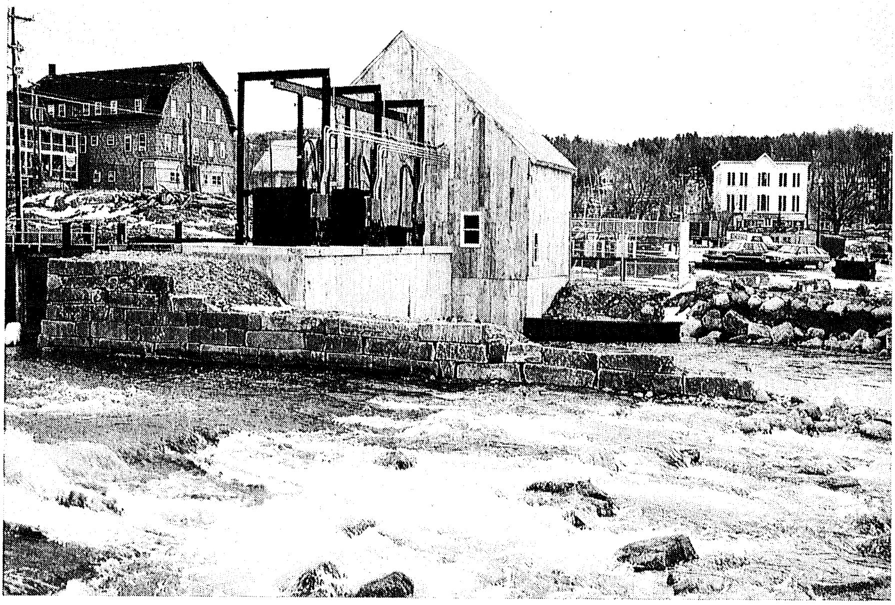
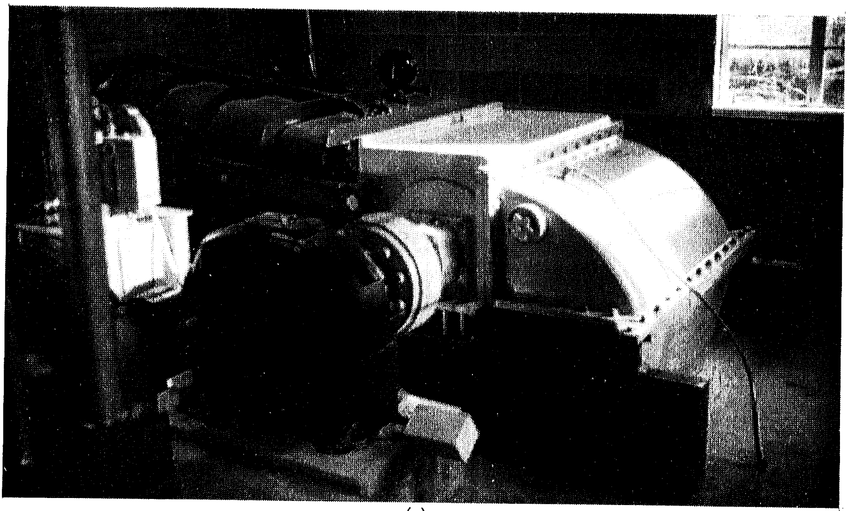


FIGURE 1.9(*a*) The Laconia, New Hampshire, minihydroelectric facility. Inside the cylinder gates here are three 160-kW submersible generator units shown in 1.9(*b*). Switch gear and controls are inside the adjacent structure in 1.9(*a*). *(Courtesy of Flygt Hydroturbines.)*

FIGURE 1.9(*b*) The Laconia, New Hampshire, minihydroelectric facility. Inside the cylinder gates shown in 1.9(*a*) are three 160-kW submersible generator units as shown here. Switch gear and controls are inside the adjacent structure in 1.9(*a*). *(Courtesy of Flygt Hydroturbines.)*

3. The turbine-generator unit, including guide vanes or wicket gates, turbine, draft tube, speed increaser, generator, and speed-regulating governor
4. Station electrical equipment, which includes transformer, switch gear, automatic controls, conduit, and grounding and lightning systems
5. Ventilation, fire protection, communication, and bearing cooling water equipment
6. Transmission line

In Chap. 6, Fig. 6.6 illustrates some standard types of hydropower facilities. One of the main features of this diagram is the variety of configurations that exist. The existence of many different types of plants allows the engineer to meet the specific requirements of given sites. This also places an additional burden on the engineer to arrive at an estimate of feasibility, since several different configurations must be studied.



(a)

(b)

FIGURE 1.10 Eastman Brook microhydroelectric facility (100 kW, or 134 hp) in Piermont, New Hampshire. (*a*) Flow turbine and gear box. (*b*) Penstock leading to powerhouse. *(Courtesy of Ben Rinehart, EG&G Idaho.)*

As discussed in detail in Chap. 4, the selection of the proper turbine for a given site depends on the head and flow available, as well as the use that is to be made of the facility. Because there is such a broad variation in available conditions in the field, many different turbine configurations have been developed and are available. It is essential that a hydropower engineer understand in detail the rationale for selection of hydroturbines. The other items listed, although not as central to hydropower production, are also crucial to a properly operating facility. A hydropower engineer should develop some knowledge and understanding of all hydropower facility components to assure that an economically optimized, properly integrated, and properly functioning facility is built and maintained.

1.5 HYDROPOWER DEVELOPMENT SEQUENCE

There is no set pattern which must be followed in developing a hydropower facility. The following typical hydropower development sequence, however, will give an idea of where the various levels of study fit in the development process.

1. *Power production screening.* The power production screening uses basic information such as the net head and flow available at the site to determine whether it is worthwhile to even consider hydropower production. Potential feasibility is determined from general criteria. One example of the type of general criteria which is used for power production screening is given as a case study in Chap. 2.
2. *Preliminary feasibility study.* The preliminary feasibility study is a limited investigation utilizing existing information to develop a preliminary indication of project feasibility. The basic purpose is to determine whether it is worthwhile to allocate funds and effort for a comprehensive feasibility study of the proposed project.
3. *Review of other potential development constraints.* There are many other potential constraints to hydropower development. Three examples are: (a) financing may not be obtainable, (b) severe potential environmental impacts may restrict development, and (c) local regulatory agencies or public opinion may be against development of the site. These and other potential development constraints should be considered before proceeding with a feasibility study.
4. *Permit applications.* A variety of permits must be obtained before hydropower development can proceed. In the United States, for example, it is possible to obtain a Federal Energy Regulatory Commission preliminary permit that retains the right of the permit holder to file for an FERC license without being preempted by another application. It usually takes 3 to 5 workdays to complete a preliminary permit application, once a preliminary feasibility study has been completed.
5. *Comprehensive feasibility study.* The objective of a feasibility study is to propose a viable project development, to make recommendations on development strategy, and to provide the basis for further studies required for licensing and permits. A feasibility study will normally include:
   - Hydraulic and hydrologic analysis
   - Formulation of project development alternatives
   - Cost estimates for equipment and construction
   - Analysis of plant operation strategies
   - Computation of expected energy production
   - Analysis of energy value and markets
   - Benefit/cost analysis
   - Financial analysis
   - Analysis of environmental impacts

- Analysis of socioinstitutional impacts
- A strategy for project implementation

6. *Application for operating permits and licenses.*
7. *Purchase negotiations.* Negotiations with potential purchasers should begin as soon as the project implementation decision has been made.
8. *Facility design.* The design study often includes the review of bids for equipment from manufacturers.
9. *Construction and installation.*
10. *Operation.*

A sample project implementation schedule for small hydropower projects developed by the U.S. Army Corps of Engineers [3] is shown in Fig. 1.11 [13]. It should be noted that, according to the expenditure pattern in Fig. 1.11, expenditures prior to construction and the purchase of equipment account for between 10 and 25 percent of the total project cost. Larger hydropower projects generally follow the lower curve, except that the total time required is normally greater (sometimes much greater) than 4 years.

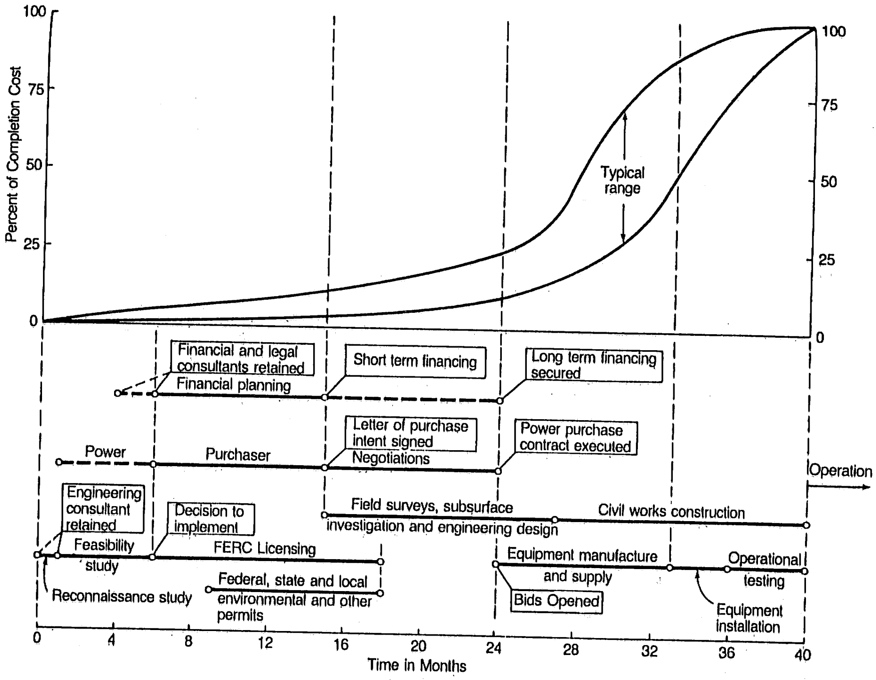


FIGURE 1.11 Typical project implementation schedule and expenditure pattern for a small (<15 MW) hydropower project. *(From U.S. Army Corps of Engineers [13].)*

1.6 ADVANTAGES AND DISADVANTAGES OF HYDROPOWER DEVELOPMENT

It is obvious that hydropower development is a renewable energy source, and that its renewability is a significant advantage. When hydropower development occurs at existing dam sites, the environmental impact is minimal or nonexistent. However, keep in mind that significant changes can occur when an existing dam site is developed for power generation. For example, water that would normally flow over a spillway, where a large amount of aeration would take place, is now funneled through turbines, where little or no aeration occurs. The resulting substantial difference in the rate of aeration at a given point in the river can have a notable influence on the dissolved oxygen content of a considerable reach of river. In certain areas, especially those near large municipalities, the environmental impact can be significant.

While the fuel costs of hydropower plants are negligible, their construction and capital equipment costs are usually substantially greater per unit of installed capacity than those of thermal power plants. This can be seen as either a benefit or a detriment. The economic feasibility of hydropower development is very sensitive to the *difference* between a discount rate, used to bring future income and costs to present value, and an assumed escalation rate, used to predict the future cost of electricity. The discount rate is taken as the loan interest rate, which is usually fixed. The escalation rate is an estimate, which can vary greatly. In a period of escalating inflation, for example, a hydroelectric project which is already in place looks like a very good investment.

When considering hydropower development within a given region, it is also important to look at the overall economics of that region. For example, it should be noted that hydropower development means that a substantially larger percentage of the investment capital can stay within a given region, since much of the developmental work can be done by local engineers and contractors. The more sophisticated coal-fired and nuclear power plants are designed and built by specialized constructors, which often means that large amounts of capital leave the local economy. In many instances, the same is true for the amount of capital necessary for fuel for thermal power plants. This substantial drain on the economy can be very significant. An example in the United States is the estimate that, in Minnesota, each $1 spent on out-of-state coal is equivalent in effect on the economy to $3 invested within the state. In addition, hydropower facilities require minimal maintenance and do not have the same requirements for skilled personnel as do the more sophisticated thermal power plants.

There are other advantages of hydropower, and especially of small hydropower. Many future possibilities in small hydropower development will depend on the economic climate. There is a definite market for small turbine technology. Should this market be developed, it can be expected that significant improvements, both in operational characteristics of turbines and in reduced cost, can lead to small hydropower facilities being more cost-effective. Cost savings of approximately 30 percent have already been realized in Europe by utilizing modular construction for small hydropower facilities. There are many other aspects of small hydropower that have not been explored as yet. In many cases in developing nations, the small hydropower station becomes the catalyst for the development of small manufacturing facilities. One case in point is electrolytic manufacturing of fertilizer [14]. It should also be pointed out that the current miniprocessor technology has developed to the point that it is feasible to operate a system of small hydropower plants on a completely automated basis, with only a traveling crew of workers needed for maintenance. In some cases, the number of small sites that are developable could supply the same amount of power as one large nuclear power plant, without the safety or security hazard normally involved with a large-scale development. If the total amount of power is distributed over several small plants, overall reliability can increase, since it is very unlikely that all plants would suffer an outage at the same time.

There are, however, also several disadvantages to small hydropower development. The most obvious is the fact that economy of scale does not prevail. This results in high initial cost for a relatively low installed capacity. In many cases, these plants are run-of-river; that is, their capability for generating power fluctuates wildly with the seasons, and this prevents a system of small power plants from acting as an equivalent base-load plant. In many areas of the world, peak power is available in late spring, whereas peak demand occurs in midsummer or midwinter. This mismatch of power need and availability can be quite serious.

In addition to the lost economy of scale, there are other disadvantages which relate to the head available. For example, low-head facilities are those in which the available head is less than approximately 66 ft (20 m). Since available power is proportional to the product of flow and head, larger amounts of flow must be

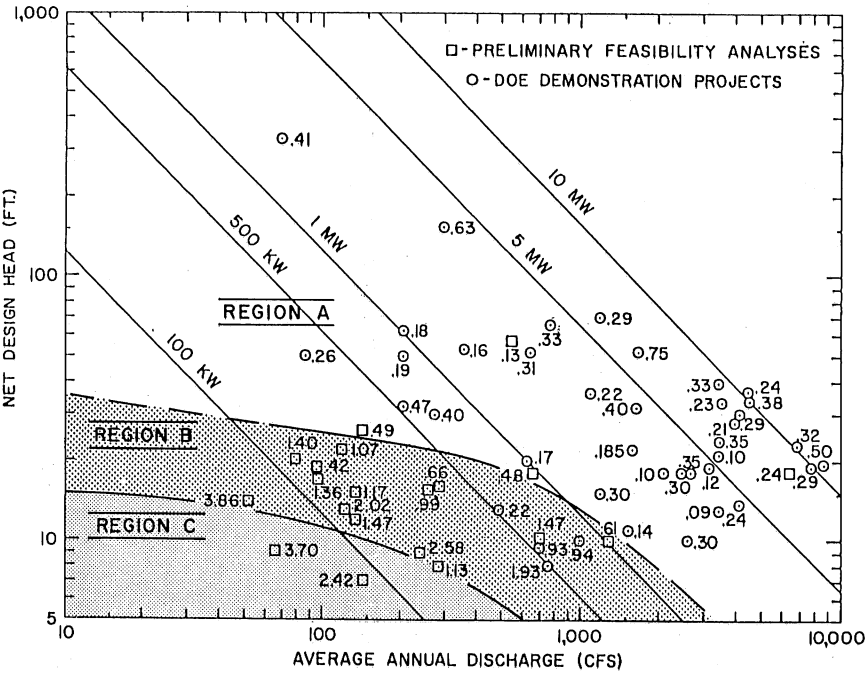


FIGURE 1.12 Feasibility regions utilized in a resource inventory for the State of Minnesota. The numbers given next to each point are relative cost (total project cost divided by average annual energy production) in 1981 \$/kWh (\$/hph), which were used to identify feasibility regions. *(From Gulliver and Garver [15].)*

handled in order to generate a given power level at lower head. This means that the size of the machine increases, producing a disproportionate increase in the cost for the amount of power developed. This problem also places a responsibility on the engineer for extremely careful design of inlet and outlet facilities since hydraulic losses which would be insignificant in higher-head installations become of major concern. The problem of the relative size and head is illustrated in Fig. 1.12, which is based on the results of some studies made at the St. Anthony Falls Hydraulic Laboratory and a compilation of U.S. Department of Energy (DOE) demonstration projects [15]. In each case, there is a number next to the symbol in Fig. 1.12. This is the *relative cost,* defined as the total project cost divided by the annual energy production in \$/kWh (\$/hph). To fix ideas, this would be the cost per kWh (hph) that would have to be received to pay off the project cost in 1 year. Relative cost is only an indicator of the benefit/cost ratio, the calculation of which requires a much more complex economic analysis. On the chart, we have three regions applicable to Minnesota, where 1981 energy prices were roughly \$0.035/kWh (\$0.026/hph): region A, in which the relative cost is less than 50 cents; region B, in which the relative cost is between 50 cents and \$1.50; and region C, in which the relative cost is in excess of \$1.50. You will notice that, as the size of the facility decreases, its relative cost becomes very sensitive to head. This is an important factor which must be kept in mind when considering development of a small, low-head hydropower facility.

1.7 SUMMARY

It can be said that the escalation of nonrenewable energy production costs, which occurred through the 1970s, has made hydropower more attractive. The future of hydropower development depends greatly on the ability of hydropower engineers to find innovative ways to reduce fixed costs of development while providing for a minimal environmental impact. It is the aim of this handbook to provide the background material for effectively meeting the demand for well-designed economical hydropower facilities.

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CHAPTER 2

PRELIMINARY STUDIES: HYDROLOGY, HYDRAULICS, AND COSTS

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2.1 BACKGROUND

The primary purpose of a preliminary investigation in hydropower development is to determine how much power is available at the site and how often it is available. Also of interest is the magnitude of floods which will occur if spillway capacity is an issue in the development (and it usually is). Flood frequency analysis will not be discussed here because it is well documented in a number of handbooks and textbooks [1, 2]. This chapter addresses the four issues which are of primary importance in terms of benefits and costs: flow duration, available head, single-purpose operation of one reservoir, and total project costs.

The power output of a hydroelectric plant is given by the equation

$$P(kW) = \frac{e\gamma Q(\text{ft}^3/s)H(\text{ft})}{737} = \frac{e\gamma Q(m^3/s)H(m)}{1000} \tag{2.1}$$

where P = generator output in kW
e = overall plant efficiency (a fraction)
γ = specific weight of water in lb/ft^3 (or N/m^3)
Q = flow through the turbine in ft^3/s (m^3/s)
H = net head across the turbine in ft (m)

For power output in horsepower, the conversion

$$P(\text{hp}) = \frac{P(\text{kW})}{0.746}$$

is convenient. This chapter will first address the availability of flow, which can vary from near zero to a very high value at most sites. The key questions are what percentage of the time a given flow will be available, and what flow will be available when the power and energy generated are most valuable, during peak demand periods. Determination of the available head, i.e., headwater and tailwater curves, will then be discussed, followed by a short description of the use of daily and seasonal pondage to increase the availability of flow, and some guidelines for preliminary cost estimates. Finally, case studies will be used to demonstrate the use of these principles for a given hydropower site.

2.2 DESCRIPTION OF WATERSHED HYDROLOGY

A watershed or drainage basin is the region contributing flow to a given location, such as a prospective hydropower site on a stream. Any watershed is composed of a wide variety of foliage, soils, geologic formations, streams, etc., and is therefore unique. No two watersheds are the same, and therefore comparisons of runoff from two separate watersheds and generalized relationships for watershed runoff are not very accurate. The data obtained from stream gauges in the watershed of interest should be used whenever possible.

The ratio of stream runoff to precipitation within a given watershed depends upon seven factors, which are illustrated in Fig. 2.1 and discussed below.

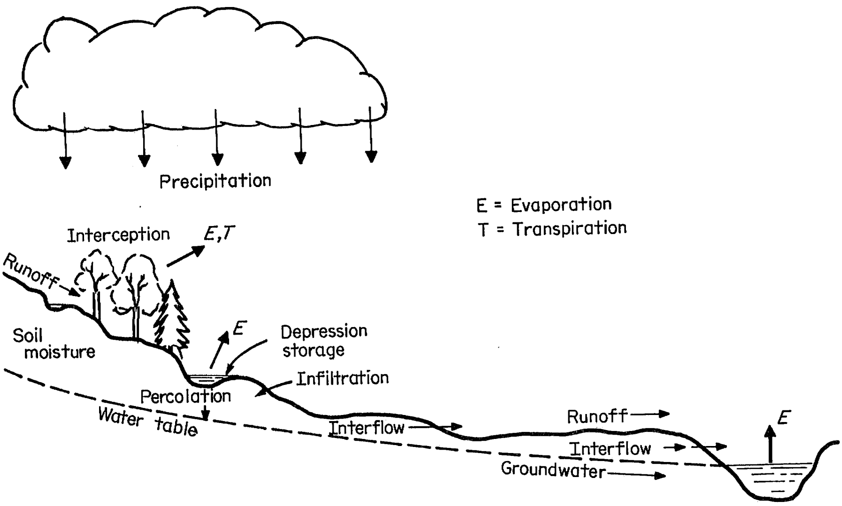

FIGURE 2.1 Processes affecting the relationship between precipitation and runoff.

Channel Precipitation

A small portion of any precipitation event will fall directly into the channel or stream. This immediately becomes runoff. It is grouped together with runoff from impervious areas, such as parking lots, streets, and buildings, because this runoff occurs rather quickly after precipitation.

Surface Retention

Surface retention is the portion of storm precipitation which does not run off or infiltrate into the soil. It is retained at the surface. It, therefore, represents precipitation which does not result in stream flow. It may be divided into three components, as follows:

Interception. Interception is the precipitation stored on vegetative cover, then evaporated. Interception accounts for between 10 and 20 percent of annual precipitation in a well-developed forest. The interception of crops varies greatly, however. Some approximate values for a 1-in (2.54-cm) storm are:

Vegetation type	%
Cotton	33
Small grains	16
Tobacco	7
Corn	3
Alfalfa	3
Meadow grass	3

Depression Storage. Depression storage is the rainwater retained in puddles, ditches, and other depressions in the surface. It occurs when the rainfall intensity exceeds infiltration capacity. At the end of the storm, water held in depression storage is either evaporated or infiltrated into the soil. The depression storage capacity of most drainage basins is between 0.5 and 2 in (1.3 and 5 cm) of precipitation.

A specific type of depression storage which is handled separately is a blind or self-enclosed drainage basin—that is, a portion of the drainage basin which does not drain into the stream network but is self-enclosed, usually with a lake, marsh, or bog at the center. Blind drainage basins are normally excluded from the hydrologic analysis of a hydropower site.

Surface Detention. Surface detention is attributed to a thin film of water covering the soil surface, which later evaporates or infiltrates into the soil. Since surface detention is relatively small, it is normally incorporated into depression storage.

Surface Runoff

Surface runoff is the precipitation that moves downslope along the soil surface until it reaches a stream or lake. It is primarily associated with flood events, although in larger watershed basins the effects of surface runoff can be felt for up to a month.

Infiltration

Infiltration is the passage of water through the soil surface. It usually implies percolation, which is the movement of water through unsaturated soil. Infiltration capacity depends upon soil porosity, moisture content, and vegetative cover. Sandy or highly organic soils will have a greater infiltration, largely due to increased porosity. A wet soil will have a lower infiltration capacity. Vegetation cover increases infiltration by retarding surface flow, increasing soil porosity with the root system, and reducing rain packing of the soil surface.

Soil Moisture

The precipitation which infiltrates into the soil will first be used to replenish soil moisture. Over time, the soil moisture is taken by plant root systems and eventually transpired from plant foliage as part of the photosynthetic process. Water used to replenish soil moisture will not appear as stream flow. Soils with a high percentage of decayed plant material have a large capacity to retain moisture.

Interflow

Interflow is water which infiltrates through the soil and moves laterally in the upper soil layers until it reemerges as surface runoff. A thin soil surface covering rock, hardpan, or plow bed will usually have large quantities of interflow. Interflow often emerges as a spring in riverbanks. It will not usually affect flood peaks, but will increase stream flow at a steady rate for some time after the peak.

Groundwater Flow

If the infiltrated water percolates downward until it reaches the water table (zone saturated with water), it will eventually reach streams as groundwater flow, which is the primary source of base flow for streams. Groundwater flow influences stream flow on a seasonal, rather than a weekly, time scale.

A schematic diagram of the segmentation of rainfall for an extensive storm in a relatively dry basin is given in Fig. 2.2 [3]. The shaded area indicates the quantity of rainfall which will eventually become stream flow. The general order in which the various types of flow reach the stream is as follows: channel precipitation, surface runoff, interflow, then groundwater.

We have identified seven parameters that influence the runoff process, which depend upon foliage, soil type, geologic formation, and watershed geomorphology. These parameters can vary greatly within a given watershed and will certainly vary between two distinct watersheds. It should be no surprise, therefore, to discover that two adjacent watersheds of similar drainage areas have entirely different runoff characteristics. This is the norm, rather than the exception, and caution should be observed when applying gauge data of one watershed to a hydropower site in a distinctly separate watershed.

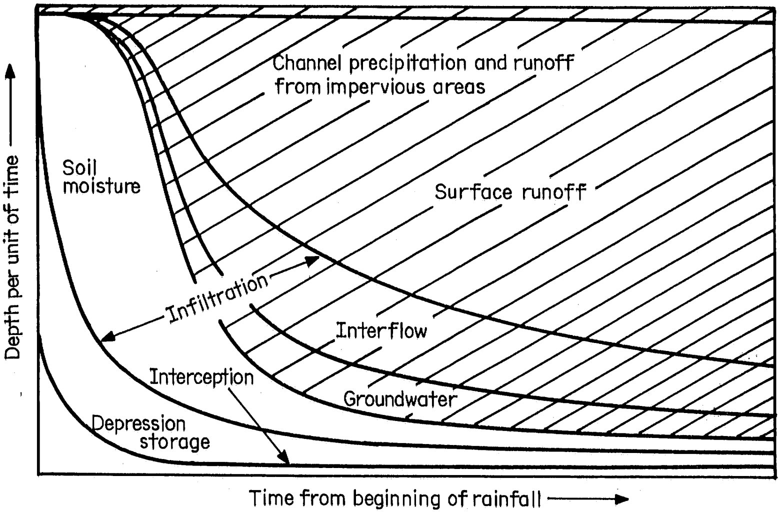

FIGURE 2.2 Schematic diagram of the segmentation of rainfall for an extensive storm in a relatively dry basin. The shaded area indicates precipitation that eventually results in a runoff. *(From Linsley, Kohler, and Paulhus [3].)*

2.3 FLOW DATA

Sources of Data

The accuracy of a hydrologic analysis depends greatly on the record of stream-flow data for the watershed. In many watersheds throughout the world, the stream-flow data are limited or nonexistent, and frequently precipitation data are not abundant either. When considering hydropower development in such a watershed, discharge measurements should be undertaken as soon as possible because any stream-flow data will reduce uncertainty in the prediction of available flow. Techniques to measure flow are reviewed in Iverson [4] and various manuals [5, 6]. Discharge measurements should be followed by the use of a hydrologic runoff model such as that described by Crawford and Thurin [7] and Crawford [8] to synthesize a flow-duration curve and estimate the flood flows which occur. A sensitivity analysis over the feasible range of input coefficients will also give a sense of the possible range of predictions which will occur.

If a potential hydropower site is in a region that has relatively good stream-flow data over a number of years, the estimates of available flow and flood discharge improve greatly. In the United States an extensive compilation of hydrologic data by the U.S. Geological Survey (USGS) exists that extends back to the turn of the century. Many countries throughout the world have similar stream-flow data collection services. The value of these data cannot be overstated. Synthesized discharges are a poor substitute. The techniques described herein will be based upon data availability similar to that in the United States.

Typically, a number of stage-discharge gauges are used to report daily discharge, in addition to partial-record stations where stream discharge is reported a few times per year. These partial-record stations can be quite helpful when large adjustments in the flow statistics of a gauge are required to represent the flow regime at a given dam site.

The agencies often undertake a significant analysis of the data at each gauge to give low-flow and peak-flow statistics as well as a flow-duration curve. In the United States, this information is available without charge at the local USGS District Office. If individuals wish to undertake their own analyses of the data from given stations, e.g., flow routing through a reservoir with various hydroelectric plant operational schemes, the District Office will produce a tape of the station data or transfer the data to a computer via telephone connections at a reasonable cost.

Quality of Data

The USGS classifies the quality of its discharge data as excellent (within ±5 percent), good (within ±10 percent), fair (within ±15 percent), and poor. These types of accuracies are common for river stage-discharge gauging stations. It is difficult to get a stage-discharge station to be more accurate than ±5 percent. In addition, the errors may have a significant bias (systematic error), in that they may not be entirely random, and the mean discharge will thus have a significant error. Thus, to predict the average annual energy generation at a hydropower site to better than ±5 percent is probably not possible.

A hydrologic analysis for a hydropower facility also assumes that future conditions may be approximately represented by what happened in the past. The risks of this assumption are inherent in any water project. For example, drainage of wetlands, deforestation, etc., can cause the drainage basin to take on characteristics entirely different from its past characteristics. There are means of testing the quality of the data, however, for some obvious inconsistencies.

Cyclicity and Trend

Engineers involved in water resources projects are well aware that sequences of wet and dry years form irregular cyclical patterns. These patterns are sometimes evident for some decades before losing their form. The characteristics of wet and dry years in runoff and precipitation determine several design and operation criteria. A typical design problem may be the determination of reservoir storage capacity for given flow regime and water demand. It is also important to study cyclical patterns in determining the firm capacity of a hydropower station, or to appraise the relatively short runoff records that one is often compelled to use in design of hydraulic engineering projects.

The causes and nature of cycles in precipitation and hydrologic data have been studied for many years. Various correlations have been attempted, including sunspot cycles and planetary period, without success. In the future there are also human influences to consider, such as the warming of the atmosphere due to CO_2 and other emissions. Existence of persistent trends and cyclicity may be detected from a 5-year running average of mean annual discharge. For the sample given in Fig. 2.3, the 5-year running average in 1910 is the average of the mean annual discharges for 1908, 1909, 1910, 1911, and 1912. Calculation of the value for 1911 includes 1913 and drops 1908. One immediate question is whether the long, dry period from 1920 to 1940 is an event which occurs once a century or once a

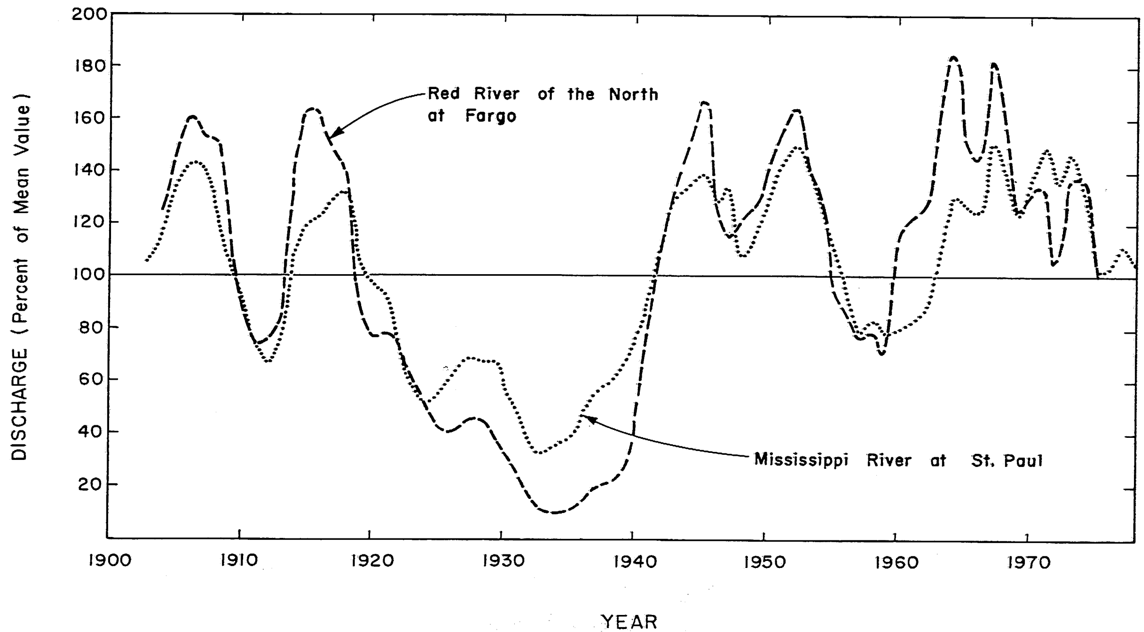

FIGURE 2.3 Cyclical variations in runoff for selected rivers in Minnesota.

TABLE 2.1 Comparison of Average Runoff for 20-Year, 30-Year, and 40-Year Periods with the Mean Annual Runoff for 76 Years of Record at Two USGS Stage-Discharge Gauges

Period of record	Deviation from the mean (%) Mississippi River at St. Paul	Red River of the North at Fargo
1911–1930	−18.7	−17.0
1921–1940	−66.2	−50.4
1931–1950	−27.9	−18.3
1941–1960	+19.5	+15.2
1951–1970	+31.4	+15.3
1931–1960	−15.4	4.5
1931–1970	+0.4	−1.5

millenium. If once a century, it probably should be included in the hydrologic analysis. Usually, it is included.

The next question is what length of flow record is required for a hydrologic analysis. To answer this, the two curves in Fig. 2.3 have been broken down into five 20-year averages, one 30-year average, and one 40-year average, as shown in Table 2.1. It is apparent that none of the 20-year periods gives an accurate discharge estimate. The period from 1931 to 1960 gives a relatively low error of 4.5 percent for the Red River of the North, but a large error of −15 percent for the Mississippi River at St. Paul. The 40-year period from 1931 to 1970 is satisfactory for both records. For most sites, between 30 and 50 years of record is required to avoid cyclicity problems.

Finally, what if you do not have the 40 or more years of records required? The 5-year running average of at least two gauges with a long record should be plotted. The drainage area of one gauge should be similar to that of the site. These two records can be used to adjust the shorter record of the gauge to be used for the site. For example, if a record from 1950 to 1970 is available near the Red River of the North at Fargo, those data are probably 15.3 percent too high, on average. Thus, all flows from the 1951–1970 record can be adjusted, i.e., reduced by 15.3 percent.

Mixed Population of Data

The flow characteristics of a watershed can be greatly altered by human activities such as construction of flow-regulating reservoirs, harvesting of lumber, forest fires, and urbanization. This can result in a mixed population of discharge data, meaning that the characteristics of the watershed, and therefore the data, have changed significantly. This change is very difficult to detect because of long-term runoff cycles. A large change would be indicated by a break in the running-average discharge which is not explained by weather. One means of finding such a break would be by comparison with the running average of a nearby gauge on a separate watershed. Sometimes a discontinuity in the trend of the data is more easily discerned from a double-mass curve, which is a plot of cumulative flow versus cumulative years.

2.4 AVAILABLE HEAD

The gross available head is an important parameter in hydropower feasibility and design. The power produced with a given turbine discharge increases linearly with head. In addition, the cost of equipment decreases roughly with head to the 0.35 power, as shown in Sec. 2.7. The power produced per dollar of construction cost is therefore proportional to $H^{1.35}$. It is for this reason that accurate calculation of headwater and tailwater curves can be very important at low-head hydropower sites. These calculations are complicated by the fact that both headwater and tailwater elevation increase with discharge, and not at the same rate.

Headwater Curve

Headwater elevation versus spillway discharge is a relatively simple comparison because a weir-type equation relates head above the spillway crest with the flow over the spillway, as follows:

$$Q = Cg^{1/2}LH^{3/2} \text{ or } H = \left[\frac{Q}{g^{1/2}CL}\right]^{2/3} \tag{2.2}$$

where Q = discharge over spillway
C = discharge coefficient
g = acceleration of gravity
L = length of spillway perpendicular to flow
H = height of headwater level above spillway crest

The value of C depends upon spillway type and shape, and may be found from hydraulic reference books such as Brater and King [9] or Bradley [10].

Note that Eq. 2.2 is for discharge over the spillway, rather than river discharge. Reservoir headwater elevation is normally kept at spillway crest elevation when all the river discharge is passed through the hydroturbines. When river discharge exceeds maximum turbine discharge, Eq. 2.2 is applied to the excess flow, which passes over the spillway.

Tailwater Curve

Developing a curve for tailwater elevation versus river discharge can be the source of a great deal of field work. There are a few means of developing a tailwater curve for a given site; in descending order of accuracy and reliability, these are:

- Develop the curve from existing data taken by dam operators or available from another source. These data are usually available at a U.S. Army Corps of Engineers lock and dam, for example.
- Use data from previous hydropower operation.
- Develop the curve from the computed backwater curve, i.e., HEC 2, developed by the Hydrologic Engineering Center (HEC), U.S. Army Corps of Engineers, Davis, California. An extensive survey of downstream reach for river cross

sections is required. If a flood-plain insurance study has been performed on the reach, the cross-sectional data are probably available.

- Assume uniform flow at one downstream cross section and use Manning's equation (or a similar open-channel flow equation) for rock-, gravel-, and earth-bed streams [11, 12]. This is a very poor last choice because it is rare that the flow is approximately uniform in streams and channels.

Regardless of which method is used, it is good to check the tailwater curve with at least one hard-data point if time and money permit. This can be done by surveying tailwater elevation and taking a cross section of velocities at one point in the stream. The velocity measurements may then be used to determine river discharge on that day. If there is a stage-discharge gauge nearby, the flow at that gauge may be adjusted to give river flow at the site.

2.5 FLOW-DURATION CURVES

General Description

A flow-duration curve gives the percentage of time a given flow has been equaled or exceeded for the period of record. A daily flow-duration curve, such as the example in Fig. 2.4, is developed by ranking all the daily flow data of record according to discharge, regardless of the sequence in which they occurred. The percentage of the daily flow data equal to or greater than a given flow measurement, termed the "percentage exceedance," is calculated. This flow measurement is then plotted versus the corresponding percentage exceedance, as in Fig. 2.4. Daily flow-duration curves are recommended over monthly flow-duration curves (using mean monthly flow), since they can be significantly different.

Use of the Flow-Duration Curve

The following illustration demonstrates use of the flow-duration curve in Fig. 2.4 for energy and power calculations of a strict run-of-river hydropower facility.

- Let us assume that we are sizing our turbines to run full 20 percent of the time. The turbine design discharge is then found by taking the intercept of the 20 percent exceedance ordinate and the flow-duration curve, and moving across horizontally to the river discharge abscissa at 7100 ft^3/s (200 m^3/s). This is the river discharge which is equaled or exceeded 20 percent of the time, and will be our turbine design discharge.
- Now assume that we will receive a capacity credit for all power available 80 percent of the time. For a strict run-of-river plant, this may be computed from the river flow available (equaled or exceeded) 80 percent of the time, or 2000 ft^3/s (57 m^3/s) from Fig. 2.4.

The average annual power and energy produced by the hydroelectric plant is found by constructing a *power-duration curve*. The power produced at each river discharge on the flow-duration curve is

$$P_i = \frac{e_i \gamma Q_i H_i}{737} \text{ (ft/s)} = \frac{e_i \gamma Q_i H_i}{1000} \text{ (m/s)} \tag{2.3}$$

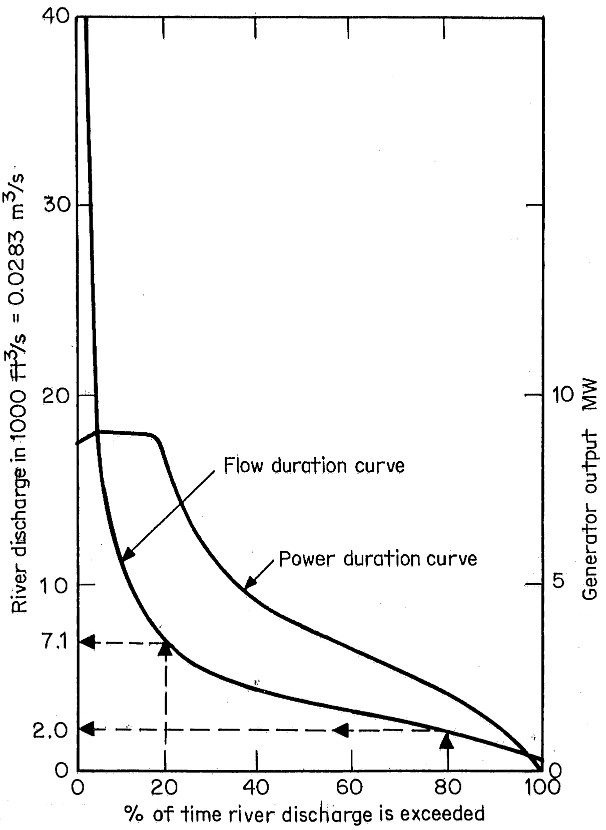

FIGURE 2.4 Flow duration curve at St. Cloud Dam. *(From Knowlton et al. [17].)*

where P_i = generator power production (kW or hp) river and turbine discharge at exceedance percentage i.
e_i = overall plant efficiency (fraction) with turbine discharge at Q_i.
Q_i = turbine discharge (ft^3/s or m^3/s) at percentage exceedance (Q_{20}, Q_{30}, ...). Turbine discharge equals river discharge except when river discharge exceeds maximum turbine discharge or other constraints on turbine discharge are encountered.
H_i = net head (ft or m) available with river flow at percentage exceedance i.

Since both efficiency and net head vary with turbine and river discharge, respectively, the power-duration curve may not be computed directly from the flow-duration curve. The power-duration curve in Fig. 2.4 was developed from the flow-duration curve, from net head occurring at a given flow, and from efficiency information for the turbines considered. The area under the power-duration curve will give the average annual energy production. The calculations can be made on a form such as that given in Table 2.2, which interpolates between 21 points on the power-duration curve with linear segments.

TABLE 2.2 Power Generation Worksheet

*Developed from the flow-duration curve, head information, and overall plant efficiency curves for the St. Cloud site. The alternative is five 3000-mm tubular turbines, with a full-scale discharge of 7770 ft³/s (220 m³/s) at a net head of 16.5 ft (5 m).**

Flow exceedance	0	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
River Q(ft³/s)	46,000	13570	9900	8400	7000	5500	4900	4250	3860	3500	3290	3000	2770	2500	2200	1930	1650	1400	1180	800	250
Net head H(ft)	16.0	16.8	16.6	16.4	16.0	15.8	16.0	16.1	16.2	16.3	16.3	16.4	16.4	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
Plant Q_P(ft³/s)	7651	7840	7794	7746	7000	5500	4900	4250	3860	3500	3290	3000	2770	2500	2200	1930	1650	1400	1180	800	250
e(%)	0.82	0.80	0.82	0.83	0.85	0.84	0.86	0.85	0.85	0.86	0.85	0.84	0.84	0.86	0.85	0.83	0.80	0.83	0.85	0.78	0
P(kW)	8492	8914	8975	8920	8054	6175	5704	4920	4497	4151	3856	3496	3228	3001	2610	2236	1842	1622	1400	871	0
ΔT(%)		5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5
E(GWh)		3.812	3.918	3.919	3.717	3.116	2.602	2.327	2.062	1.894	1.754	1.610	1.473	1.364	1.229	1.061	0.893	0.759	0.662	0.417	0.191

$\Sigma E_i = 38.86\ GWh$

*Data: $Q_i = Q_d\sqrt{H_i/H_d}$; $P = 0.0846\ QHe$; Q_d = design discharge; H_d = design head; E_i (GWh) = $0.00876(\Delta T/100)(P_i + P_{i+1})/2$; total energy = 38.86 GWh; 1ft = 0.305 m; 1 ft³/s = 0.0283 m³/s.

If the flow distribution at certain portions of the year (such as the peak demand period) is of importance, monthly flow-duration curves can provide valuable information. The monthly flow-duration curves may also be valuable to personnel from regulatory agencies who may have a larger in-stream flow requirement in spring and early summer than in the fall and winter. A monthly flow-duration curve is compiled with the flow data from a given month of the year, over all the years of record. Thus, a given station would have 12 monthly flow-duration curves. Figure 2.5 shows a summary of the flow-duration curves at a prospective hydropower site. The summary was compiled by taking 5 points off each monthly flow-duration curve.

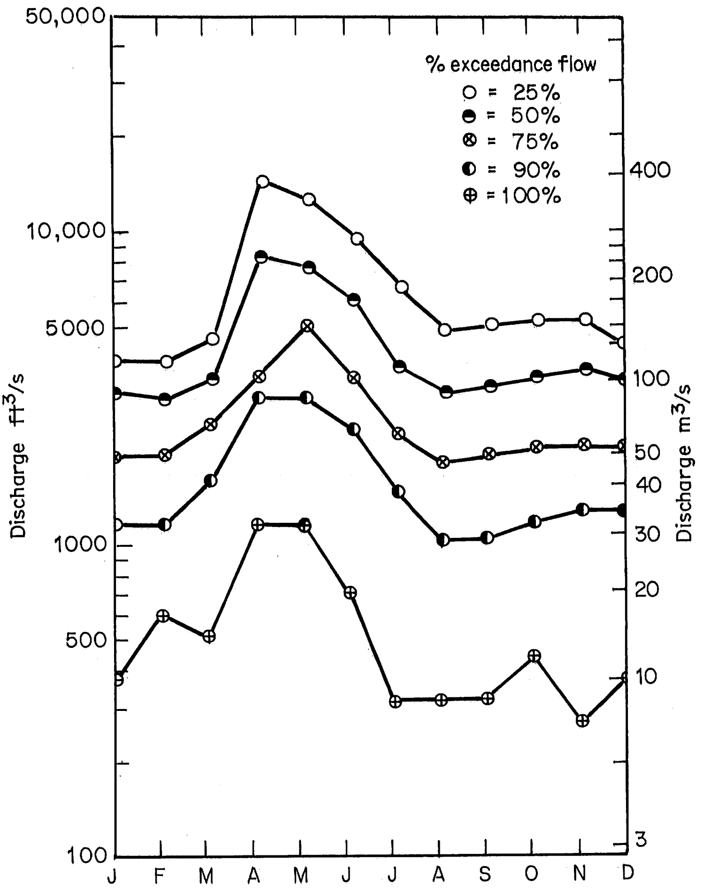

FIGURE 2.5 Summary of monthly flow duration curves at St. Cloud Dam. *(From Knowlton et al. [17].)*

Although the flow-duration curve is a valuable tool for estimating available flow at a hydropower site, it cannot be used to evaluate a peaking operation with more than 1 day's reservoir storage. Daily peaking, however, is easily incorporated into the flow-duration curve, as demonstrated below.

Daily Peaking Operation

Let us assume that a prospective hydroelectric plant will operate with a relatively simple daily peaking operation, with a fixed number of on-peak and off-peak hours each day. The goal of hydroelectric plant operation is to have as much flow available for generation as possible during on-peak hours, in order to maximize income. (The energy is usually much more valuable during on-peak hours.) A flow-duration curve (similar to that in Fig. 2.4) and the sequence of calculations shown below may be used to estimate the on- and off-peak turbine discharge at each percentage exceedance level.

Notation

Q_R = river discharge
Q_D = design discharge of turbine units
Q_{peak} = turbine discharge during on-peak hours
Q_{op} = turbine discharge during off-peak hours
t_p = on-peak h/day
t_{op} = off-peak h/day = $24 - t_p$
$\mathcal{V}$ = reservoir volume available for peaking operation
Q_{min} = minimum stream flow (set by regulatory agency)
$Q_{t,min}$ = minimum turbine discharge
H_D = design net head of turbine units
H = available net head

Calculation Sequence for Daily Peaking Operation

1. Set peak turbine discharge to utilize all available reservoir storage

$$Q_{peak} = Q_R + \frac{\mathcal{V}}{t_p(3600 \text{ s/h})}$$

2. Q_{peak}, obviously, cannot be greater than the maximum turbine discharge. If

$$Q_{peak} > Q_D\sqrt{H/H_D}, \text{ then } Q_{peak} = Q_D\sqrt{H/H_D}$$

NOTE: This equation assumes that the turbine discharge responds as an orifice to variations in net head. This is usually a good assumption as long as net head does not vary more than ± 25 percent from design net head.

3. The reservoir must refill during the off-peak period. Thus, if

$$t_p(Q_{peak} - Q_R) < t_{op}(Q_R - Q_{min})$$

then

$$Q_{peak} = \frac{t_{op}}{t_p}[Q_R - Q_{min}] + Q_R$$

4. Determine off-peak turbine discharge:

$$Q_{op} = \frac{24\,Q_R - t_p Q_{peak}}{t_{op}}$$

5. Off-peak discharge cannot be greater than the maximum turbine discharge. If

$$Q_{op} > Q_D\sqrt{H/H_D}, \text{ then } Q_{op} = Q_D\sqrt{H/H_D}$$

6. There can be no peaking operation if river flow is below minimum stream flow. If

$$Q_R < Q_{min}, \text{ then } Q_{peak} = Q_{min}, \text{ and } Q_{op} = Q_{min}$$

7. The turbines cannot operate below a given minimum discharge. If

$$Q_{op} < Q_{tmin}, \text{ then } Q_{op} = O$$

Or if

$$Q_{peak} < Q_{tmin}, \text{ then } Q_{peak} = O$$

The solution to this last constraint may be varied, based upon the turbines chosen, the flow-duration curve of the site, and the respective values of peak and off-peak energy.

As an exercise, assume that at the site with the flow-duration curve given in Fig. 2.4, the available reservoir storage is 90 million ft^3 (2.5 million m^3), the minimum stream flow is 1000 ft^3/s (28 m^3/s), and the daily on-peak period is a total of 6 hours. Four turbines with a total design discharge of 10,000 ft^3/s (284 m^3/s) and a single unit minimum discharge of 1000 ft^3/s (28 m^3/s) have been selected to match the minimum stream flow. We will ignore variations in net head on the first pass of the calculations. The flow-duration curve in Fig. 2.4 may then be used to develop two additional flow-duration curves, corresponding to peak and off-peak energy generation, as given in Fig. 2.6. A separate power-duration curve may be constructed from each of these flow-duration curves, and used to compute average annual on- and off-peak energy generation.

There is one major exception to the use of the flow-duration curve to extract daily on- and off-peak energy generation. The contract eventually signed for the sale of power and energy (or the schedule used) may be more complex than our simple on- and off-peak energy values. The marginal cost of electricity for a utility can vary greatly through any given day. If the purchase price of energy produced by the hydroelectric plant is tied to a marginal cost, for example, the simple on-, off-peak calculations given above are an approximation. A more detailed operational analysis with daily flow routing and a past history of energy values may be more accurate. This entails significantly more effort, and is briefly described in Sec. 2.6.

Development of a Flow-Duration Curve

Although a flow-duration curve is usually available for the stage-discharge gauging stations in a region, it is unusual for one of these gauges to be located pre-

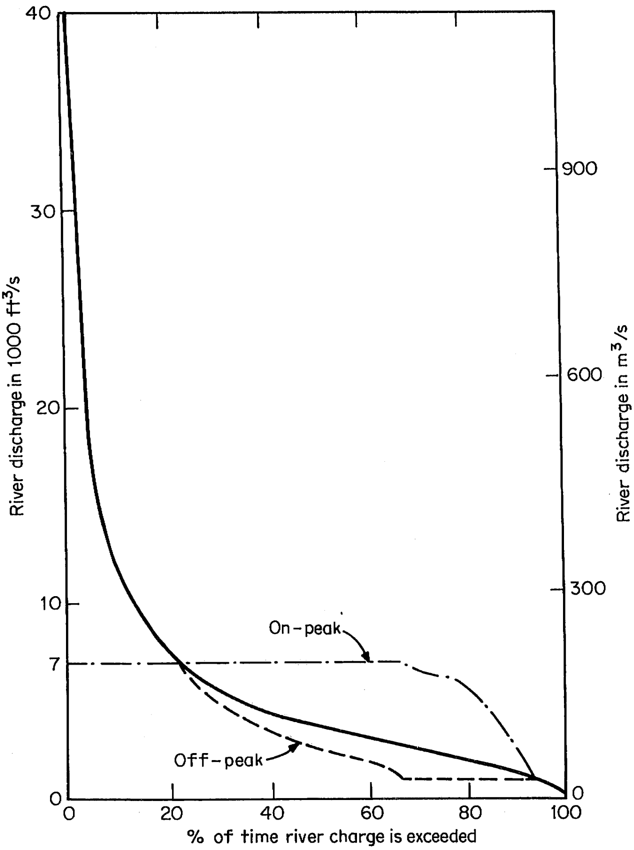

FIGURE 2.6 Flow duration curve for St. Cloud Dam, including daily peaking example.

cisely at the hydropower site of interest. There is, however, often a gauge located on the same river or a downstream river with a drainage area (DA) containing the site's watershed. The data from one or more gauges may then be adjusted to represent that of the site. The author's preference is the simple relationship which adjusts the gauge flow-duration curve in order to represent flow duration at the site.

$$Q_{\text{site}} = \left[\frac{\text{drainage area of site}}{\text{drainage area of gauge}}\right]^n Q_{\text{gauge}} \tag{2.4}$$

where n typically varies between 0.6 and 1.2. The value of n chosen depends upon the application and the data available. Some guidelines are as follows:

1. If the DA site is within 20 percent of the DA gauge ($0.8 \leq$ DA of site $\div$ DA of gauge ≤ 1.2), use $n = 1$. The estimated discharge at the site will probably be within 10 percent of the actual discharge, which is normally sufficient.

2. If the DA site is within 50 percent of the DA gauge, consider whether the data of two gauges can be combined. In addition, when a weighted average between upstream and downstream gages is possible, the following linear interpolation may be applied:

$$Q_{\text{site}} = \frac{(\text{DA}_{\text{gauge1}} - \text{DA}_{\text{site}})Q_{\text{gauge1}} + (\text{DA}_{\text{site}} - \text{DA}_{\text{gauge2}})Q_{\text{gauge2}}}{\text{DA}_{\text{gauge1}} - \text{DA}_{\text{gauge2}}} \tag{2.5}$$

The daily flow data from the two gauges should be used to compile a new set of daily flow data for the site. A flow-duration curve is compiled from the new flow data.

For this case, comparing watersheds may be helpful because Q_{site} may be off by 30 percent. The existence of self-enclosed drainage areas in either the site or the gauged watershed must be considered because they usually will not add to either peak flow or base flow.

If there is a partial-discharge station near the site, it can give an indication of the proper value of n. The ratio of partial discharge to gauge discharge on the same day versus gauge discharge is plotted. The average of these values may be used to estimate n.

3. If the DA site is only within 80 percent of the DA gage, the recommendation is to do everything listed above, if possible. In addition, when discharge versus DA is plotted for all gauges in the watershed basin, e.g., Figs. 2.7 and 2.8, a value of n may be indicated by these data. Then one may ask whether the drainage area is relatively wet or dry.

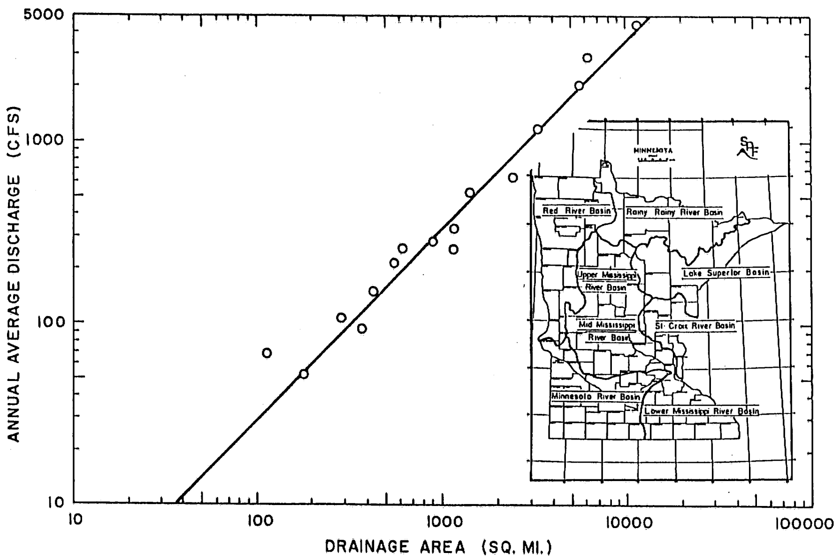

FIGURE 2.7 Average annual discharge versus drainage area for USGS stage-discharge gauges in the upper and mid-Mississippi River basins, Minnesota. (1 ft³/s = 0.0283 m³/s; 1 mi² = 2.56 km².) *(From Gulliver and Garver [23].)*

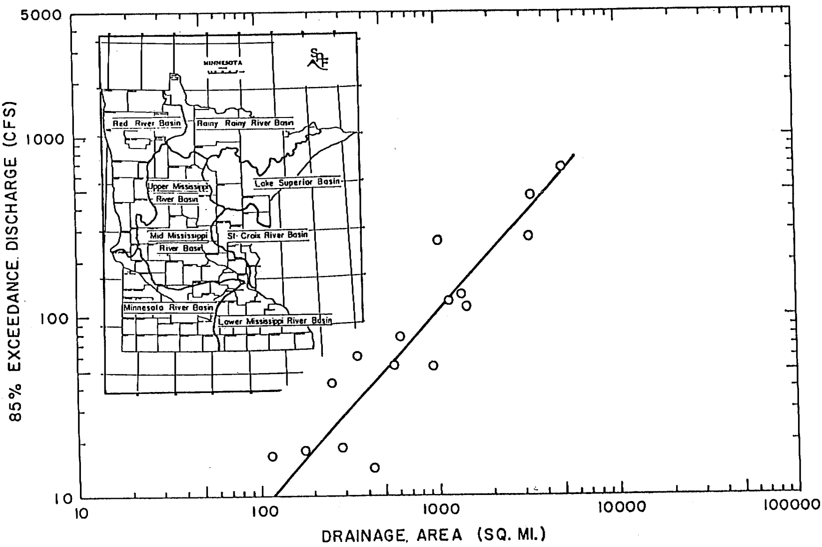

FIGURE 2.8 Stream discharge which is equaled or exceeded 85 percent of the time versus drainage area for USGS stage-discharge gauges in the upper and mid-Mississippi River basins, Minnesota. (1 ft^3/s = 0.0283 m^3/s.)

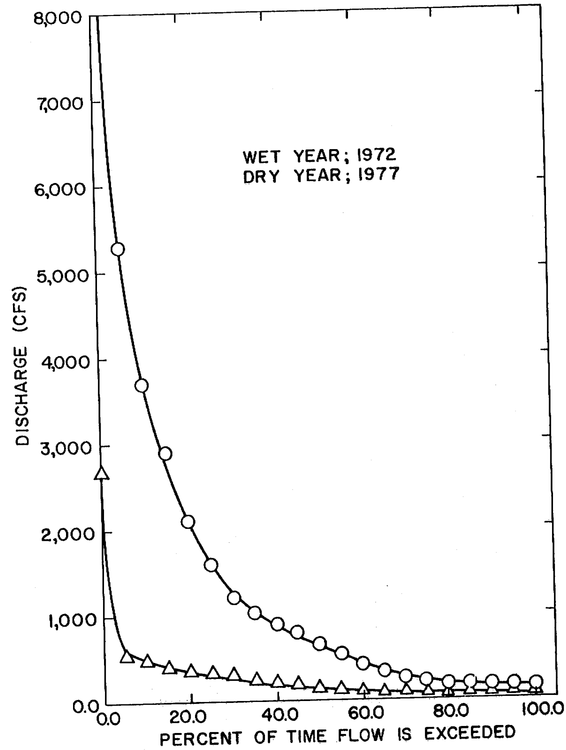

FIGURE 2.9 Flow duration curve for the wettest and driest years of record at the Kettle River Dam. (1 ft^3/s = 0.0283 m^3/s.) *(From Gulliver, Knowlton, and Garver [18].)*

Another possibility is to compare watersheds in more detail, i.e., the percentage of lakes (more lakes indicate greater base flow), and near-surface geological formations (sand, gravel, and sandy loam, and sandstone formations will transmit groundwater and increase base flow; clay, granite, and sandy clay will not appreciably transmit groundwater and will decrease base flow).

Just as there are long-term cycles in average annual flow, the discharge in the first year of operation will probably not be that of the flow-duration curve. Figure 2.9 gives the wettest and driest years of an 11-year record, which indicates what may be expected in the first year. This information is important in a financial analysis of cash flow, because the project developer must have sufficient funds (or an insurance policy) to cover the potential negative cash flow in a dry year.

2.6 OPERATIONAL ANALYSIS OF HYDROPOWER FACILITY

If the hydroelectric plant has the use of considerable reservoir storage or if the income generated by peaking is highly variable, the anticipated energy production and income are best estimated with an operational analysis which uses daily flow information in its true time sequence. For multipurpose reservoirs which may have a flood control, irrigation, or navigation function, operational analysis can be a complex problem, as described in Chap. 11. If the reservoir is not multipurpose, the problem may be solved with a relatively simple computer program which uses a given period of record and an operational scheme to route stream flow into and out of the reservoir, and to compute energy generation and expected income on a daily basis. This may be achieved in the following steps:

1. Obtain one or more representative periods of record; remember that it is important to choose one period of record which represents the mean.
2. Develop an operational procedure for the hydroelectric plant, including a minimum flow release, the allowable flux in pool elevation, and the daily peaking procedure. A simple peaking procedure might be that described in Sec. 2.5.
3. Route daily flows through the reservoir and hydroelectric plant to determine peaking capability, energy generation, income, etc. The routing routine will keep track of the volume of water, as follows:

Reservoir inflow from flow record	→	Reservoir storage from water surface elevation	→	Outflow through operational procedure	→	Power production, energy generation, and income

The optimum operational procedure depends upon the constraints and objectives, and can be quite elusive. If the objective function constraints are relatively complex, an optimal control algorithm [13] is suggested. (This is also discussed in Chap. 11.) Hydroelectric plant operations over a sequence of single periods are optimized individually, to maximize the benefit (or income) B from hydropower operation during that period plus the future value of the stored water, FV. The future value of the stored water is the sum of $B + FV$ for the next time period. Thus all single periods are linked. The inflow to the reservoir must be predicted, and of course, the storage volume of the reservoir is limited. The solution finds the operational plan (releases) that maximizes benefit for a given set of the future value of stored water divided by volume storage terms; then iterates the set of future value terms until global optimum benefit is found.

For multiple reservoirs, successive linear programming may be used, in which the objective is to maximize the value of generated electricity plus the water left in storage at the end of the planning period. The objective function for total benefits is assumed to be piecewise-linear, and an optimum solution is found. Then a new linearized approximation is developed at the most recent solution, and an optimum is again found until the final operational procedure is achieved.

In developing an operational procedure, one may also want to consider the stochastic nature of the flow; i.e., it is useful to remember that the future is not an exact replica of the past. Uncertainty in the forecasts must be taken into account. One means of achieving this would be to generate a number of flow forecasts, each with a given probability of occurrence, and to optimize an operational routine that considers all the forecasts and their probability. (Again, this is discussed in Chap. 11, in more detail.)

2.7 METHODOLOGY FOR COST ESTIMATES

Preliminary investigations typically involve a total-project cost estimate. The purpose of this section is to provide guidelines for equipment and total-project cost estimates at the preliminary level.

The total-project cost estimate is based upon the concept developed by Gordon and Penman [14] and Gordon [15] and modified by Gulliver and Dotan [16] which incorporates a single equation for equipment cost, a site factor for the total project cost, and a weighting factor to represent the possible range of site factors. The equation for total equipment cost which was developed has been updated to July 1987 by the Producer Price Index for machinery and equipment, and is given below:

$$C_T = 10{,}600\ \mathrm{kW}^{0.82} H_R^{\ -0.35(\mathrm{ft})} = 16{,}100\ \mathrm{kW}^{0.82} H_R(m)^{\ -0.35} \tag{2.6}$$

where C_T = equipment cost in U.S. dollars as of July 1987
kW = total plant capacity in kW (1 kW = 1.341 hp)
H_R = rated head in ft (m)

Equation 2.6 results in satisfactory equipment cost estimates (± 20 percent) for a plant capacity range from 50 to 40,000 kW (67 to 5364 hp) and a hydraulic head range from 12 to 330 ft (4 to 100 m).

Civil works (construction) costs are an important, undefined variable in hydropower development because the costs are very site-specific. An estimate of the cost of a hydropower project, therefore, requires prior knowledge of the civil works costs associated with the site. To give consistency to these estimates, Gordon and Penman introduced the concept of a *site factor*, which is the total project cost divided by the total equipment cost for a hydropower project. If a site factor for a given site has been estimated, total equipment cost from Eq. 2.6 may be multiplied by site factor to determine total project cost.

Gulliver and Dotan [16] redefined the site factor concept to incorporate only the component costs which are present in all hydropower projects, such as head race, headworks, powerhouse, tail works, tailrace, engineering costs, and management costs. It does not include the cost of a penstock, diversion works, dams, spillways, transmission lines, and remote-access facilities—factors which can greatly escalate project cost, but are not present in many hydropower projects and should be determined separately. Cost curves and cost-related information for these specific items are given in Chaps. 6 and 7.

The newly defined site factor for various projects is plotted against design capacity in Fig. 2.10. These data were used to formulate the following observations:

- A site factor of 1.5 adequately represents situations in which a new unit is added to an existing powerhouse without extensive modification.
- An existing powerhouse which requires some major repairs will typically have a site factor between 1.5 and 2.0.
- Site factors for locations with no existing powerhouse fall between the two envelope curves given in Fig. 2.10. The lower envelope curve has a constant value of 2.0. The upper envelope curve has a constant value of 3.0 if plant capacity is greater than 5 MW (6702 hp). If plant capacity is less than 5 MW, the upper envelope curve is given by the equation

$$SF = 9.8\ \text{kW}^{-0.14} \tag{2.7}$$

where SF = site factor
kW = plant capacity (of all new units) in kW (1 kW = 1.341 hp)

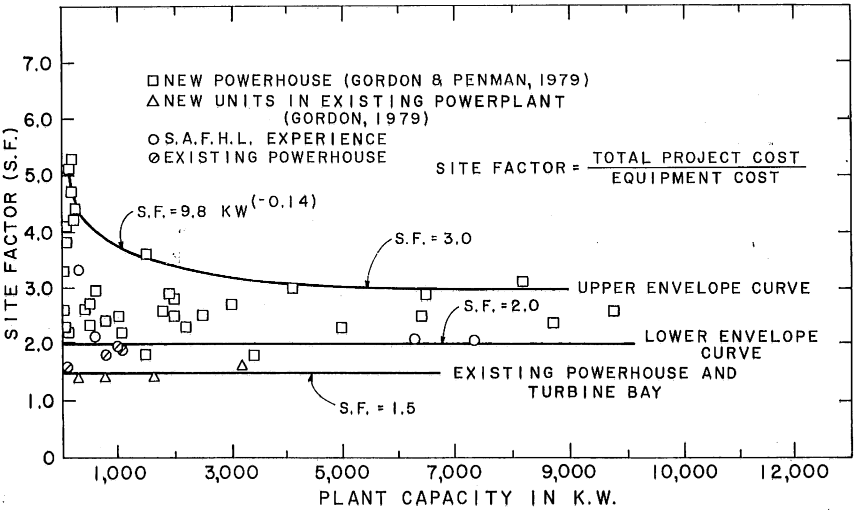

FIGURE 2.10 Site factor for various hydropower projects versus plant capacity and associated envelope curves. *(From Gulliver and Dotan [16].)*

Except in extreme cases, the site factor for a new powerhouse should be between the upper and lower envelope curves. A choice of the precise site factor value between the envelope curves requires knowledge of the site and engineering judgment. The method described above, however, provides a foundation for such decisions.

In order to further facilitate the incorporation of civil works and other costs into total project costs, a *weighting factor* was established as a replacement for site factor. The weighting factor is the fractional distance between the upper and lower envelope curves. A weighting factor of 1.0 will give a site factor on the upper envelope curve. A weighting factor of 0.0 will give a site factor equal to 2.0, which is the lower envelope curve. Site factor is then given by the following two equations:

$$SF = 2.0 + WF, \text{ kW} > 5000 \tag{2.8}$$

$$SF = 2.0 + WF(9.8 \text{ kW}^{-0.14} - 2.0), \text{ kW} < 5000 \tag{2.9}$$

where WF = weighting factor for site. (1 kW = 1.341 hp.)

The weighting factor can be assigned to a proposed site by comparison with completed projects, regardless of the plant capacity. Thus, a project with a weighting factor of 0.8 will have a site factor of 3.4 for a 1000-kW (1341-hp) project and a site factor of 4.5 for a 100-kW (134.1-hp) project. Use of a weighting factor has been found to give a more consistent and reliable estimate of total project cost than choosing a site factor for projects below 5 MW (6702 hp).

Following are two examples which illustrate how weighting factors may be estimated.

1. The Orwell Dam on the Ottertail River in Minnesota is owned by the U.S. Army Corps of Engineers. The average annual flow is 305 ft^3/s (8.7 m^3/s), with a gross head of 33 ft (10 m). The dam is a rolled earth-fill type with structural height of 47 ft (14 m), side slopes 1:3, and a top width of 20 ft (6 m).

   The construction of the powerhouse will, therefore, include a relatively high cofferdam upstream of the dam, concrete walls for the headrace, cutting through the earth embankment which will involve extensive excavation, and a relatively long tailrace—all this for approximately 1-MW (1304-hp) capacity. All the above-mentioned factors result in a weighting-factor estimate of 0.9.

2. Lock and Dam No. 2 on the upper Mississippi River is a navigational dam owned by the U.S. Army Corps of Engineers. The average annual flow is 10,700 ft^3/s (3031 m^3/s), with a gross head of 12 ft (3.7 m). The dam consists of 20 tainter gates, a 100-ft-long (30.5-m-long) fixed-crest spillway, a main lock, and a riverward lock which is no longer in use. The Corps of Engineers has indicated that a powerhouse may be located in the riverward lock, the fixed-crest spillway, or both.

   In this particular case, there is no need to build a headrace or tailrace; the existing foundations and possibly walls can be used, and a cofferdam may not be required upstream. With these reductions in civil works, the estimated weighting factor is 0.0.

It is important to emphasize that the project cost-estimating method described herein was developed for preliminary assessments, to be used when a first approximation of the cost is required.

2.8 CASE STUDIES

Lock and Dam No. 5 (Upper Mississippi River)

The hydrologic and hydraulic analysis is probably simplest on navigational locks and dams, such as those operated by the U.S. Army Corps of Engineers. We found this to be true for Lock and Dam No. 5 on the upper Mississippi River. All the locks have an operational plan which includes a headwater curve. In addition, tailwater elevation is recorded every day by lock operators. This information was correlated with daily mean stream flow to give the tailwater elevation curve shown in Fig. 2.11. Only a few elevations were plotted for periods when the flow was relatively steady, although the daily observations extend back to 1934. Note that a ±1-ft (±0.3-m) variation in tailwater discharge was common at the lower discharges.

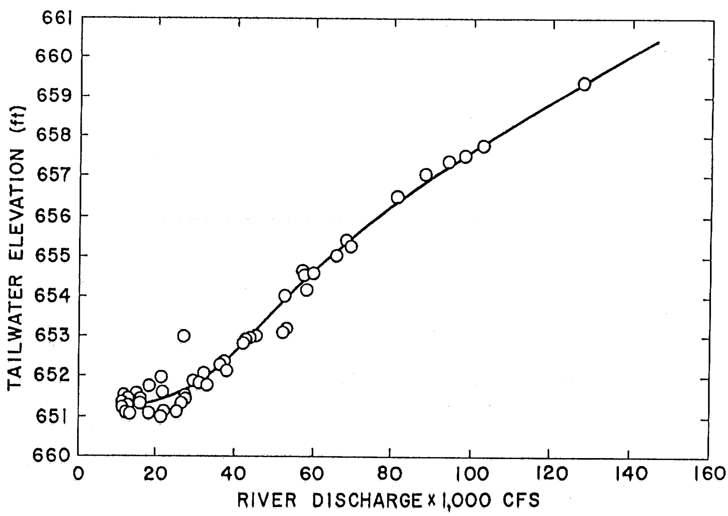

FIGURE 2.11 Tailwater elevation curve for Lock and Dam No. 5, upper Mississippi River. (1 ft = 0.305 m; 1 ft^3/s = 0.0283 m^3/s.)

The site is located between two U.S. Geological Survey stage-discharge gauges. Because the drainage area of the downstream gauge is very close to that of the site, only one gauge was used, and it was adjusted for drainage area to give a flow-duration curve.

The lock and dam is a low-head structure very similar to Lock and Dam No. 2, described in Sec. 2.7. The weighting factor was chosen as 0.0 and the site factor was 2.0. An installed capacity of 8 MW (10,724 hp) could be achieved at an estimated total project cost of \$32 million. The site has been developed recently even with the high cost (\$4000/kW, or \$2984/hp) because the plant could run at full output most of the time.

St. Cloud Dam

The St. Cloud Dam on the Mississippi River is shown in Fig. 2.12. It is composed of an unregulated overflow spillway and a small earth embankment. Average annual flow at the site is approximately 4970 ft^3/s (141 m^3/s), and hydraulic head is about 17 ft (5.2 m). A previous hydroelectric plant was removed in 1969 when a new dam was constructed.

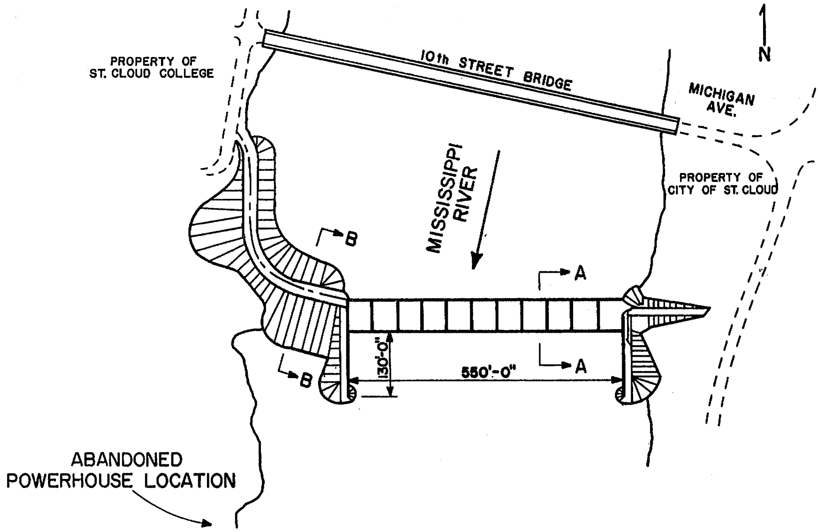

FIGURE 2.12 Plan of Mississippi River at St. Cloud Dam. (1 ft = 0.305 m.) *(From Knowlton et al. [17].)*

The drainage area of the site is 13,200 mi^2 (33,792 km^2) and the nearest gauging station at Royalton (upstream) has a drainage area of 11,600 mi^2(29,696 km^2). This gauge would account for 87 percent of the site's drainage area, and probably would give a fairly accurate representation of the flow when scaled up according to drainage area. The Sauk River, which enters the Mississippi River just above the site, is also gauged, however, accounting for a 925-mi^2 (2368-km^2) drainage area. For this reason, the U.S. Geological Survey was asked to combine the daily flows of the two gauges and transfer the information to our computer. The following equation was then used to estimate discharge at the site [17]:

$$Q_{\text{site}} = \frac{(Q_{\text{Royalton}} + Q_{\text{Sauk R}})\text{DA site}}{\text{DA Royalton} + \text{DA Sauk R}}$$

This daily flow information was used to develop a flow-duration curve and 12 monthly flow-duration curves. The combined gauges accounted for 94 percent of the site's drainage area.

A tailwater curve, given in Fig. 2.13, was prepared by operators of the previous hydroelectric plant. There were no developments downstream which would invalidate that curve, and so it was used directly. In addition, field surveys indicated that the curve is sufficiently accurate. The headwater curve assumed flow over an *OG* crest where

$$Q_{\text{spillway}} = Cg^{1/2}LH^{3/2}$$

and $C = 0.68$ for this shape of crest [9].

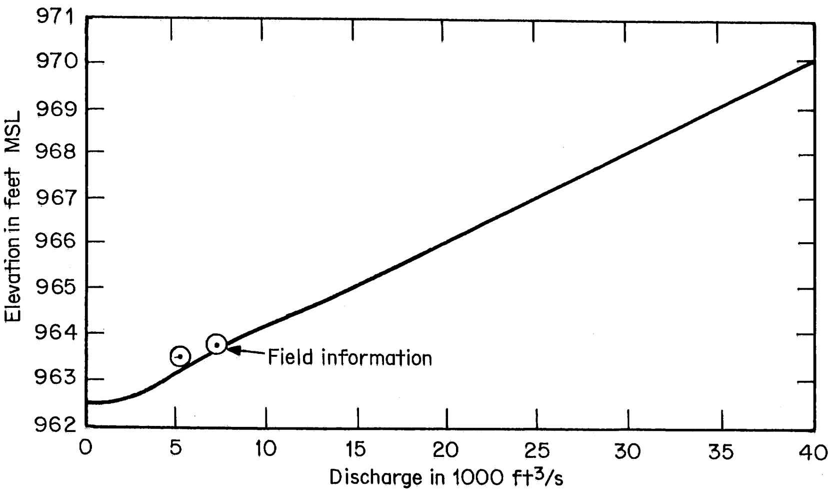

FIGURE 2.13 Tailwater elevation curve for the St. Cloud Dam. (1 ft = 0.305 m; 1 ft^3/s = 0.0283 m^3/s.) *(From Knowlton et al. [17].)*

The earth embankment could be excavated for a hydropower facility. An upstream cofferdam could be avoided; however, fairly substantial walls would be required for the intake region. A weighting factor of 0.2 was chosen. Since the site is greater than 5 MW (6702 hp) the site factor was 2.2. The city of St. Cloud has recently installed two 4.2-MW (5630-hp) pit turbines into a new powerhouse.

Kettle River Dam

The Kettle River Dam is located on the Kettle River, a State-designated Wild and Scenic River with approximately 20-ft (6-m) head and an average annual discharge of 722 ft^3/s (20.4 m^3/s). The dam has an abandoned, partially destroyed powerhouse which will, nevertheless, reduce construction costs (Fig. 2.14). A U.S. Geological Survey stage-discharge gauge is conveniently located 900 ft (274 m) downstream from the dam, and therefore no adjustment to the gauge flow-duration curve was required [18].

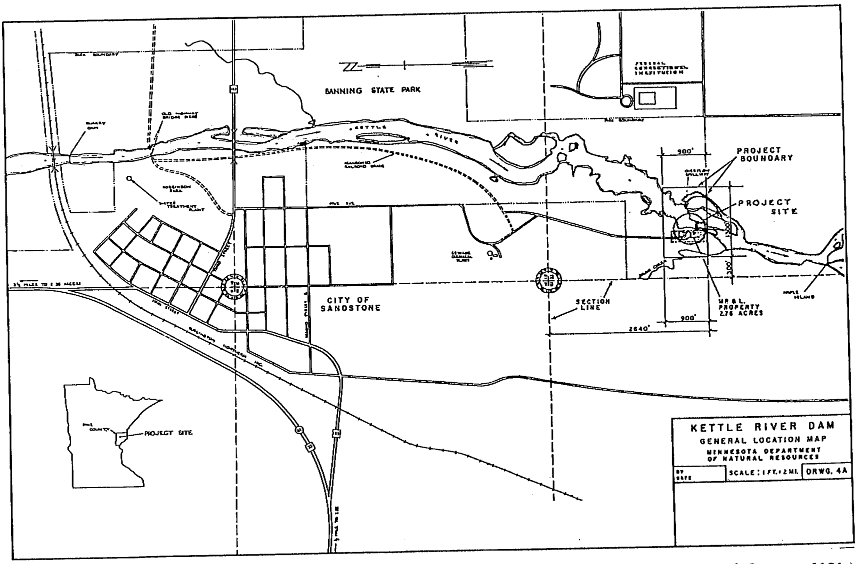

FIGURE 2.14 Site plan for the Kettle River Dam. *(From Gulliver, Knowlton, and Garver [18].)*

The dam has a main spillway and an overflow spillway at elevations 956.3 ft (291.5 m) and 958.5 ft (298.6 m), respectively. Therefore, the headwater curve is given by

$$Q_{\text{spillway}} = C_1 g^{1/2} L_1 (HW - EL_1)^{3/2} + C_2 g^{1/2} L_2 (HW - EL_2)^{3/2}$$

where C_1 and C_2 = discharge coefficients
EL_1 and EL_2 = spillway crest elevations
L_1 and L_2 = lengths of the two spillways

The main spillway has an unusual shape, but discharge coefficients for a similar shape were found in Brater and King [9]. The overflow spillway is a broad-crested weir.

The stage-discharge relationship at the U.S. Geological Survey gauge was used as a tailwater curve. Manning's equation was used to incorporate the change in elevation between the powerhouse and the gauge. Uniform flow was *not* assumed by this procedure.

It was decided instead that Manning's equation be used to estimate the frictional head loss over 900 ft (274 m) of river. At 3000 ft^3/s (914 m/s), only 2 in (5 cm) in elevation was added to the U.S. Geological Survey curve to develop a tailwater curve.

Even though there is an existing, partially destroyed powerhouse, the project has a relatively small potential capacity of between 300 and 1100 kW (402 and 1475 hp).

Any change to an existing structure can be expensive relative to equipment costs in this size range. Therefore, a weighting factor of 0.0 was chosen for the site. The site has not been developed even though the development cost was relatively low (\$1800/kW, or \$1342/hp, as of 1981) because the region currently has an oversupply of power.

Granite Falls Dam on the Minnesota River

The city of Granite Falls is a municipal utility with its own distribution system, meeting its demand peaking of approximately 6000 kW (8046 hp) through a hydroelectric facility and outside suppliers.

The hydropower plant (Fig. 2.15) consisted of two vertical Francis turbines, supplying a total of 450 kW (603 hp). Average annual flow is approximately 704 ft^3/s (214 m^3/s), with the river being typical of an arid region, i.e., having large floods and low base flow.

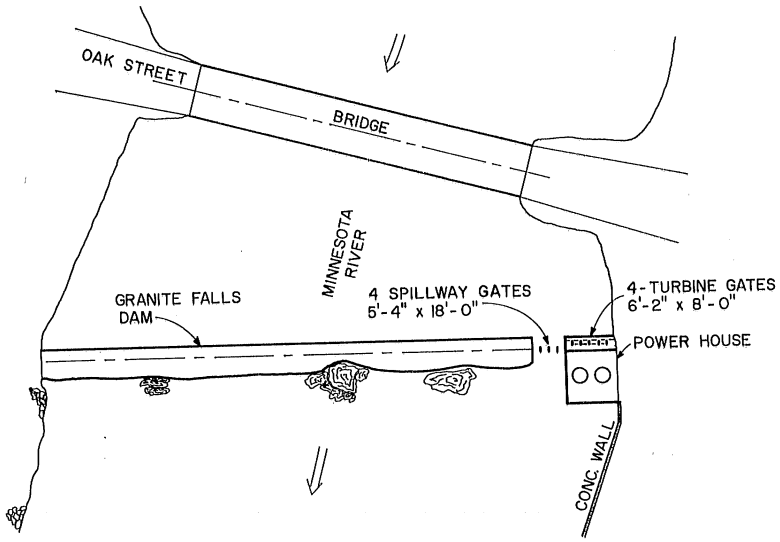

FIGURE 2.15 Plan view of Granite Falls Dam. (1 ft = 0.305 m; 1 in = 2.5 cm.) *(From Gake et al. [19].)*

All preceding information, including the existing turbine ratings, indicated that the net head available at the site is approximately 21 ft (6.4 m). In a field survey it was found that the flashboard elevation is 1 ft (0.3 m) below the value given in previous design plans, and that the normal tailwater elevation is 2 ft (0.6 m) above the previously assumed value. This is a loss of 3 ft (0.9 m) or 15 percent in the net head available at the site [19].

A flood insurance study has been performed previously for the river reach, so the cross-sectional data were obtained and HEC 2 (a standardized computer program for water surface profiles in natural channels developed by the Hydrologic Engineering Center in Davis, Calif.) was run for the downstream reach to give a tailwater curve (Fig. 2.16). It was found that:

- Tailwater elevation is controlled predominantly by the spillway crest of the Minnesota Falls Dam, 4 mi (6.4 km) downstream.
- If normal flow had occurred in the reach, there would be no difference between the tailwater curves with and without flashboards at Minnesota Falls.
- Normal flow calculations would not have been accurate.

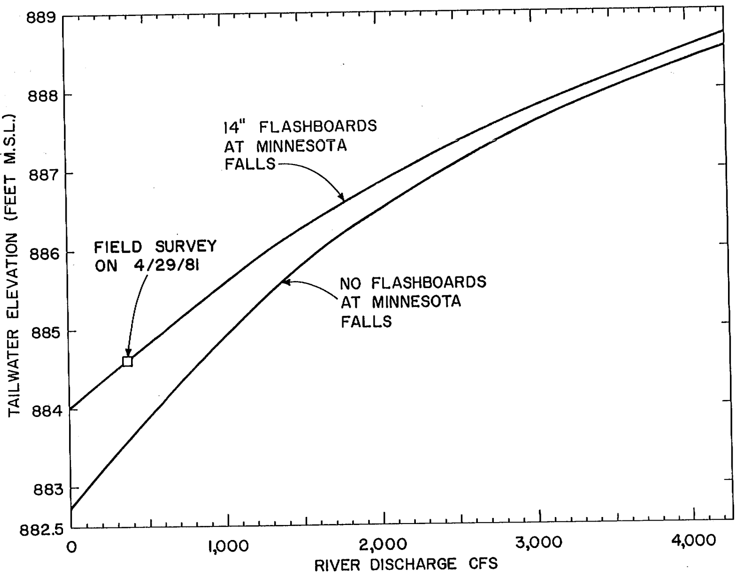

FIGURE 2.16 Tailwater elevation versus river discharge for the Granite Falls Dams. (1 ft = 0.305 m, 1 ft^2/s = 0.0283 m^3/s.) *(From Gake et al. [19].)*

- A field survey indicated that the tailwater curve is sufficiently accurate at flows near 400 ft^3/s (11 m^3/s).

Subsequent to this study, the city of Granite Falls decided to expand the facility by adding a 710-kW (952-hp) fixed-blade, fixed-vane propeller turbine. This was supplied with an automatic on/off control and automatic throttling of the two existing units. The three units will be automatically controlled by computer software designed to share peak demand as much as possible, given the reservoir volume available and demand load characteristics [19].

Lanesboro Dam

The Lanesboro Dam on the south branch of the Root River in Minnesota included a 250-kW (335-hp) hydropower facility (shown in Fig. 2.17), which was operated until 1978. Flow-duration information was developed as part of a feasibility study on whether to rehabilitate, replace, or abandon the existing equipment [20].

The drainage area of the Lanesboro Dam is 297 mi^2 (477.8 km^2). The only gauge which includes this basin has a drainage area of 1270 mi^2 (3251 km^2) and a drainage area ratio of 0.23. There are, however, a partial-discharge station at the dam with seven discharge measurements, an adjacent basin with very similar drainage characteristics, and a gauge at the confluence with a 615-mi^2 (1574-km^2) drainage area. The ratio of discharges between the two adjacent basins on the 7 days of measurement were computed and plotted on Fig. 2.18, with an average ratio of 0.61. This was felt to be a very good correlation because there was very

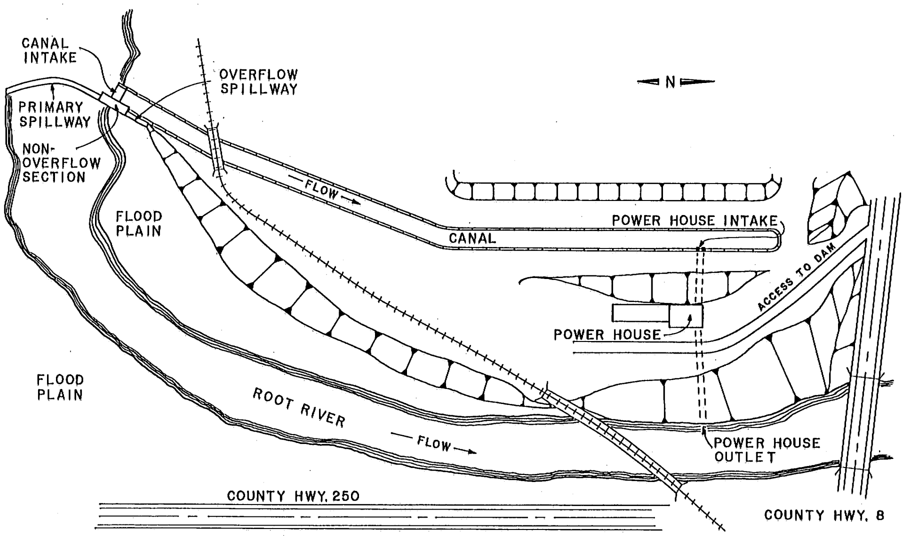

FIGURE 2.17 Overall plan of the Lanesboro Dam and hydropower facility. *(From Gulliver, Gake, and Renaud [20].)*

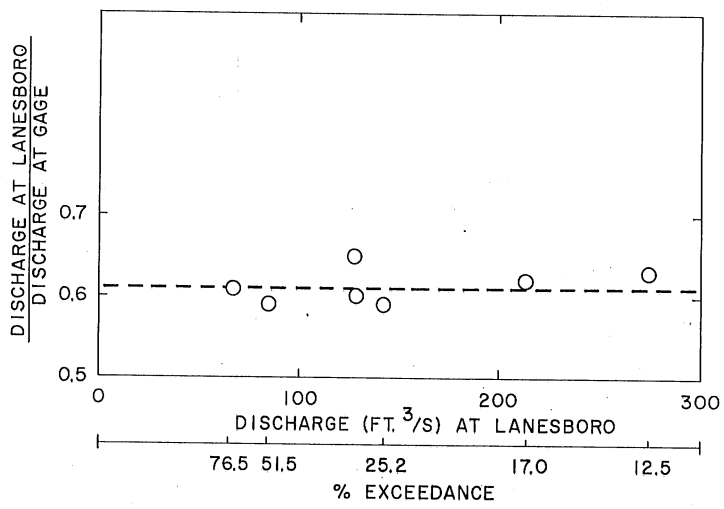

FIGURE 2.18 Ratio of discharge at the Lanesboro Dam and at a USGS gauge in an adjacent basin on 7 days of record for the partial gauging station at the dam site. (1 ft^3/s = 0.0283 m^3/s.) *(From Gulliver, Gake, and Renaud [20].)*

little scatter in the discharge ratios and because the data cover a wide range of exceedance levels on the flow-duration curve.

A flow-duration curve from the 615-mi^2 (1574-km^2) gauge was reduced by the 0.61 factor to give the curve expected at the Lanesboro Dam, shown in Fig. 2.19. When compared with the 1270-mi^2 (3251-km^2) gauge, these flow calculations in-

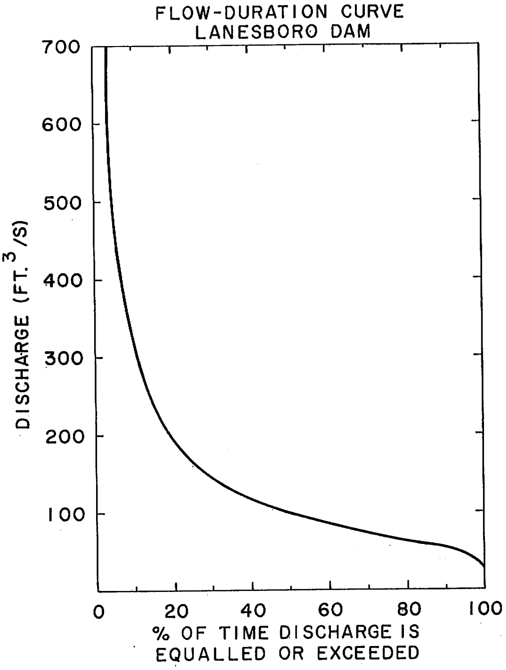

FIGURE 2.19 Flow-duration curve for the Lanesboro Dam. (1 ft^3/s = 0.0283 m^3/s.) *(From Gulliver, Gake, and Renaud [20].)*

dicate an n exponent of 0.82. It turned out that the existing equipment was sized about right for the flow at the site. This equipment was rehabilitated and brought back on line in 1984.

Rapidan Dam

The Rapidan Dam on the Blue Earth River near Mankato, Minnesota, has 60 ft (18.2 m) of head and has recently been retrofitted with a 5-MW (6702-hp) hydropower facility. Early negotiations with the power purchaser and regulatory agencies involved a rather simple breakdown of peak and off-peak energy generation:

Peak	6 h/day during weekdays
Off-peak	18 h/day during weekdays
Off-peak	24 h/day during weekends or strict run-of-river

The reservoir has 2 days of peaking capacity, and so sufficient reservoir storage for daily peaking was not a problem. The sequence outlined in Sec. 2.5 was used to adapt the flow-duration curve given in Fig. 2.20 into three flow-duration curves, representing the three operation modes given above. The Minnesota Department of Natural Resources (DNR) set a minimum stream flow of 100 ft^3/s (2.8 m^3/s), corresponding to the 70 percent exceedance level, below which no energy would be generated due to minimum turbine discharge. The preliminary analysis represented by Fig. 2.20 indicates that maximum turbine discharge would be available 43 percent of the time for peaking.

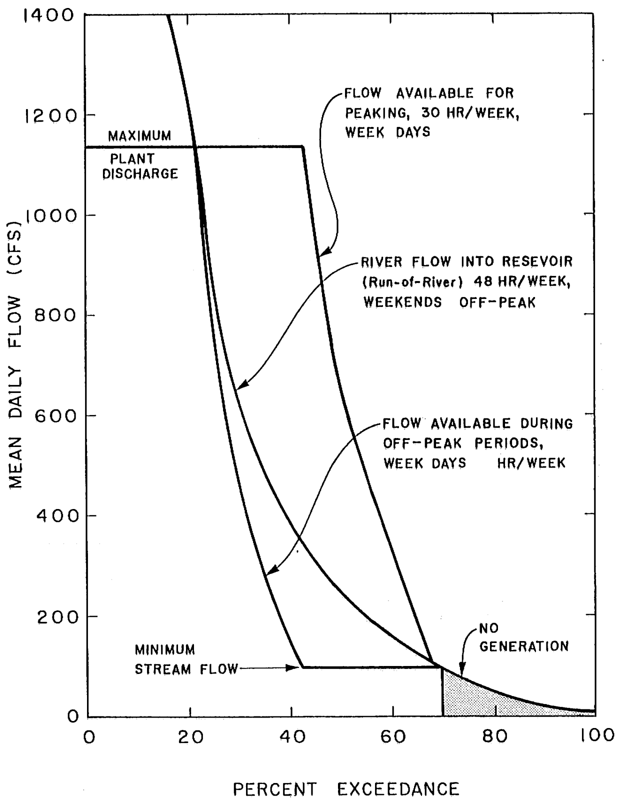

FIGURE 2.20 Flow-duration curve for various plant operations at Rapidan. (1 ft^3/s = 0.0283 m^3/s.)

In the design and licensing phase of the project the power-purchasing utility offered to buy the power and energy at its marginal rates, which vary greatly throughout any given day. The flow-duration curve method was no longer effective for estimating income, and the design firm chose a daily operational analysis similar to that of Sec. 2.6 for a single-purpose reservoir. The operational analysis was undertaken on daily flow data from an average water-year, a typical dry water-year, and a typical wet water-year.

Passamaquoddy Tidal Power Project

The extremely high tides in the Bay of Fundy have for many years been the subject of preliminary hydropower studies. One of these studies, on the Passamaquoddy Bay in the mid-1950s, was performed at the St. Anthony Falls Hydraulic Laboratory [21], with a proposed project of 345-MW (462,466-hp) potential capacity. The general region of the project is given in Fig. 2.21, with

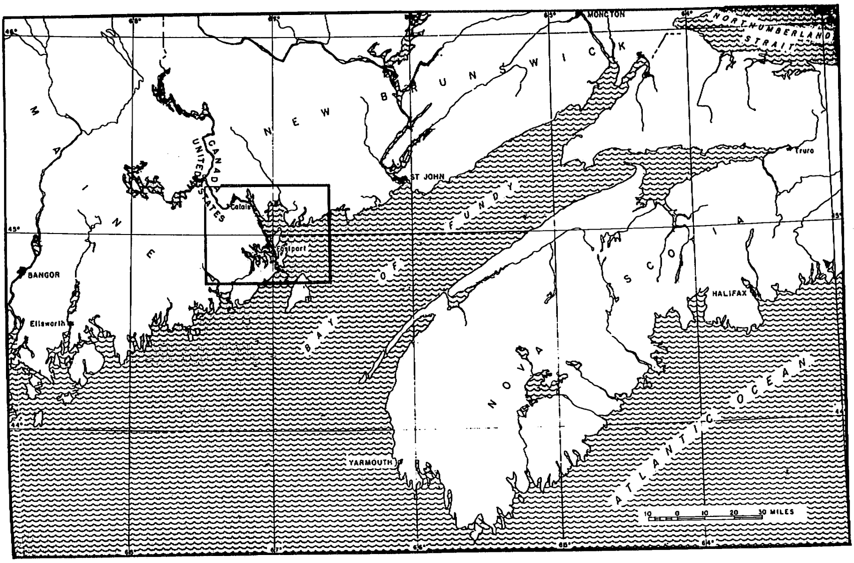

FIGURE 2.21 Area map of Bay of Fundy showing location of proposed international Passamaquoddy Tidal Power Project. *(From Straub [21].)*

an enlarged map in Fig. 2.22. The concept, illustrated in Fig. 2.23, was to build tidal barriers which would segment the Passamaquoddy Bay into two pools. During high tide, filling gates would be opened to increase the elevation of the upper pool. During low tide, emptying gates would be opened to decrease the elevation of the lower pool. The powerhouse generating units would operate continuously, passing flow from the upper to the lower pool, with gross head varying from approximately 9 ft (2.7 m) to 14 ft (4.3 m) during an intermediate tidal cycle of 18 ft (5.5 m). For the intermediate tidal cycle given in Fig. 2.23, the upper pool would be filling from 4 to 6 a.m. and from 4:00 to 6:30 p.m. The lower pool would be draining from 9 a.m. to noon and from 9:30 p.m. to 1:00 a.m. The power output would vary from 120 to 300 MW (160,858 to 402,145 hp). The gates were sited to maximize the ratio of energy production (more gates result in a greater head differential) over construction costs. For the large tidal barriers involved, the low head (and resulting high cost of power) restricted the feasibility of the project.

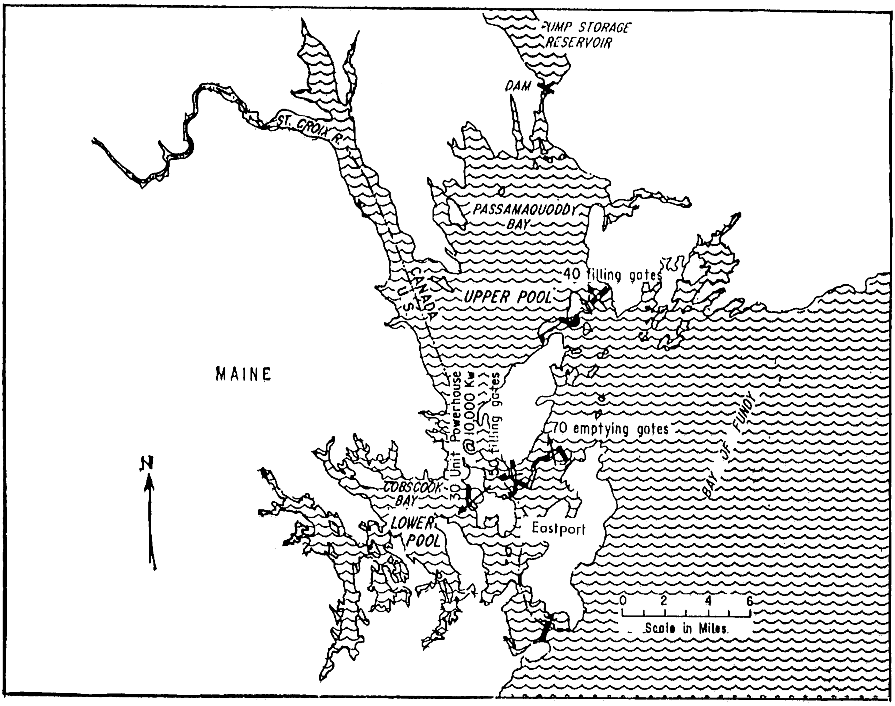

FIGURE 2.22 Enlarged map of project area corresponding to squared-off portion in Fig. 2.23. Plan involves Passamaquoddy Bay as upper pool and Cobscook Bay as lower pool, with 30-unit powerhouse operated by flow between the two pools. Projected embankment barriers are indicated by boldface bars. *(From Straub [21].)*

Hydropower Potential at Existing Minnesota Dams

The project cost-estimating methodology given in Sec. 2.7 was utilized to facilitate a survey of the hydropower potential at existing Minnesota dams [22]. The survey provided an excellent basis for testing and verifying the cost-estimating technique, and will be presented as a case study.

The survey was conducted in three steps, with the third step utilizing the cost-estimating methodology. The study began with a first-stage screening of the 853 Minnesota dam sites listed by the U.S. Army Corps of Engineers, and developed a rule-of-thumb chart from previous experience with three feasibility regions, given in Fig. 1.12 and discussed briefly in Chap. 1. The feasibility chart was designed to eliminate poor sites from consideration. Region A indicates sites with good feasibility, region B indicates marginal feasibility, and region C indicates poor feasibility. The feasibility regions are developed from the relative cost of previous projects [23], which are the numbers given next to each point in Fig. 1.12. Relative cost is the total project cost divided by the average annual energy production, and may be conceptually described as the value of energy which would pay off the project in 1 year. The feasibility regions are applicable to the energy prices in Minnesota (approximately 3.5 cents/kWh, or 2.6 cents/hp, as of

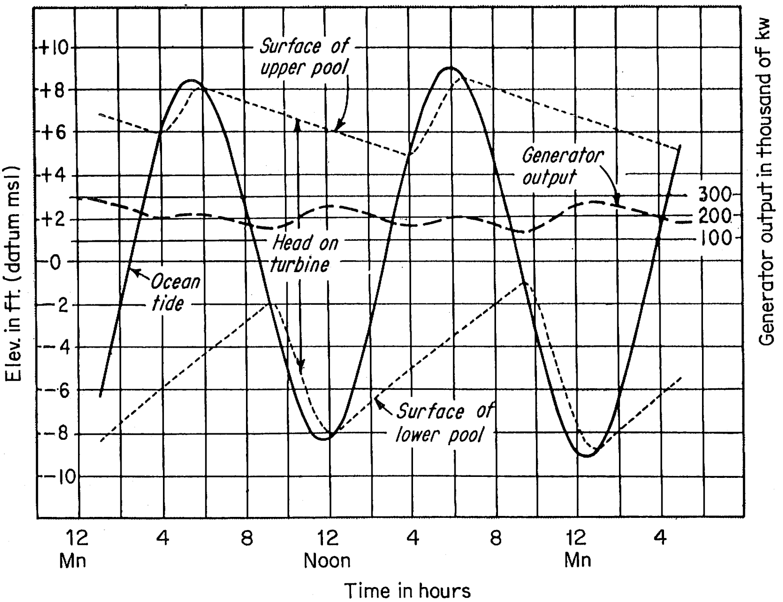

FIGURE 2.23 Typical tidal cycle at Passamaquoddy Bay for intermediate tides of 18-ft range. Chart also shows corresponding pool surface variation in the projected development, as well as generator output. *(From Straub [21].)*

1982). Existing data on heads were used at each site. The result of the first-stage screening was that 55 sites could be classified in region A, 53 sites in region B, and the remainder in region C. Hydropower facilities were already in operation at 19 of the sites. These were not eliminated, however, because estimating the potential increase in capacity at existing hydroelectric plants was also an objective.

The second portion of the survey, which is herein entitled the "second-stage screening," concentrated on improving the accuracy of the data on the 108 sites in regions A and B. Attempts to contact all the dam owners were made, and information such as plans of the dam site and headwater and tailwater elevations was requested. USGS historical records of stream discharge were used to estimate the flow at the site. If the dam site and a discharge gauge were located on the same stream, a simple drainage-area relationship was used to adjust gauge data. Forty-two of the dam sites were visited in order to compile information which was not otherwise available. The site visits proved to be crucial to the survey since in many cases the actual net head measured in the field was significantly smaller than that on record, and in extreme cases it was found that some dams no longer existed. Photographs of the dam sites were taken for use in a first assessment of proposed hydroelectric plant locations or when the site had an existing powerhouse for use in planning additional turbines. After second-stage screening, the number of sites had been reduced to 40 with good hydropower feasibility and 25 with marginal hydropower feasibility.

The third step of the survey utilized the project cost-estimating methodology described above in a prefeasibility analysis of the 65 remaining sites. A computer program was developed on an IBM PC to standardize the prefeasibility studies. Input data included a flow-duration curve; headwater and tailwater curves; values for energy and capacity; information on any existing and operating hydroelectric plants; and information on the proposed hydroelectric plant, such as overall efficiency and a weighting factor.

The program starts at 100 percent exceedance, assumes that the plant is designed to operate with a turbine design discharge equal to that exceedance flow, computes the necessary benefit/cost parameters, and then decreases the design percentage exceedance by 5 percent, repeating the computations. The project cost estimate varies with the design capacity of the plant, which is obtained by using the constant weighting factor and the envelope curves of the site factor as described previously. The program locates the design percentage exceedance with an optimum economic return, and designates the corresponding design capacity, annual energy generation, project costs, project benefits, etc. A sensitivity analysis is then performed by the program to determine the sensitivity of the benefit/cost ratio to the choice of weighting factor.

The economic feasibility criteria chosen were those typical of a municipality, with an energy-value escalation rate at 2 percent below the discount rate. Operation, maintenance, and replacement costs were taken from Ref. 4, and escalated at the energy-value escalation rate. The hydroelectric plant capacity was chosen so as to maximize net discounted benefit after 35 years of operation, corresponding to the capacity at which the incremental benefit/cost ratio is equal to 1.0. This criterion is believed to be most representative of public development.

Program output included feasibility parameters for the proposed hydroelectric plant, such as turbine discharge, plant capacity, annual energy production, equipment cost, total project cost, cost of operation and maintenance, and the benefit/cost ratio after 35 years of operation. The program then made qualitative feasibility conclusions based upon the computed benefit/cost ratio and the results of the sensitivity analysis.

One interesting observation in the survey is that, at most sites, there is a relatively wide range of power capacities over which the economic indicators did not change greatly. Thus it is difficult to determine whether a given site has 2.5-MW (3351-hp) or 5-MW (6702-hp) potential capacity, for example. Benefit/cost ratio and internal rate of return could be determined more accurately than the power potential of the site. This is because the plant factor (mean power output divided by maximum power output) goes down as the power capacity goes up, and the design point moves down in percentage exceedance. The design percentage exceedance also varies with head, typically at 70 to 80 percent for a 12-ft (3.7-m) net head and 10 to 20 percent for a net head greater than 40 ft (12 m). This assumes a strict run-of-river operation and the economic criteria of a typical municipality.

The hydropower feasibility was estimated to be positive for 25 of the sites, with 35-year benefit/cost ratios ranging from 2.43 to 1.01. The total potential capacity of these sites is estimated to be 147 MW (197,051 hp), with an annual energy generation of 521 GWh (698 million hph). The total initial cost to develop these sites is approximately $227 million (using 1982 as the base year).

Nine additional sites did not exhibit positive economic returns but could have done so, within the accuracy of the calculations as determined in the sensitivity analysis. The total potential capacity of these sites is estimated to be 19 MW (25,469 hp), with an annual energy generation of 60 GWh (80 million hph) and a total initial development cost of $28 million (using 1982 as the base year). The locations of the 35 sites are shown in Fig. 2.24.

The remaining 31 sites were found to have poor economic feasibility. This finding was primarily due to the conservative nature of the two-step screening

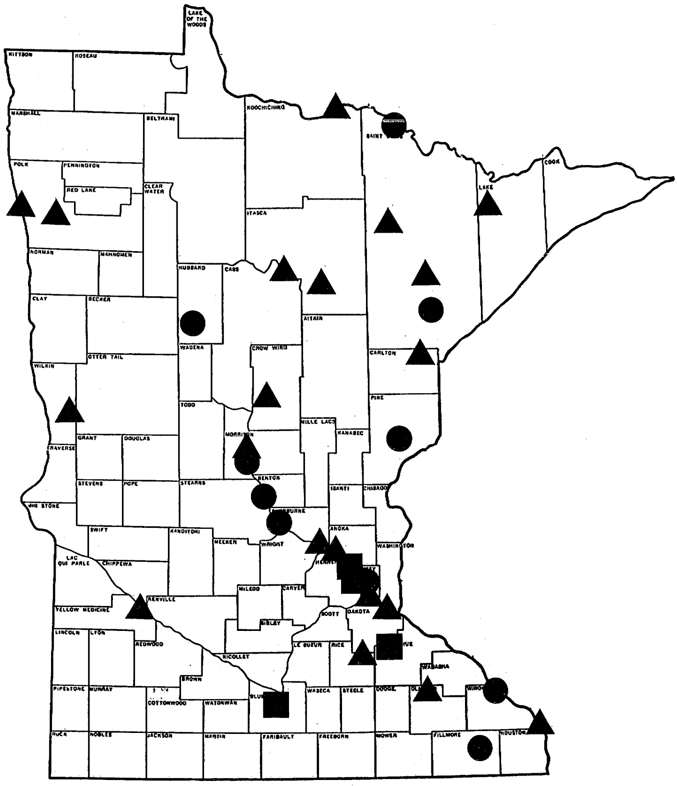

FIGURE 2.24 Location of existing Minnesota dam sites with hydropower potential, as indicated by prefeasibility studies. Circles indicate positive hydropower feasibility; triangles indicate that hydropower feasibility was not positive but could have been within the accuracy of the calculations. *(From Gulliver and Dotan [16].)*

process, designed to eliminate obviously poor sites from consideration without eliminating sites with any possible potential. In addition, 9 of the 16 sites with existing hydroelectric plants are in this group, indicating that an increase in capacity is not justified at this time. The existing capacity of these sites was not incorporated into the screening process but was in the prefeasibility analysis. Finally, 9 sites had a very poor distribution of flow throughout the year (because of long dry seasons). The duration of flow was also not incorporated into the screening process, only the average annual flow. It is thus reasonable that almost half the sites remaining after the screening process were found to have poor hydropower feasibility.

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CHAPTER 3
SMALL DAM DESIGN

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3.1 TYPES OF DAMS

In the most general sense, a dam may be defined as a barrier built across a water course for impounding water. This definition implies no restrictions upon the purpose, materials used, or size of the barrier. Thus, sills and weirs could also be covered under the general umbrella provided by the word "dam." In practice, however, dams are considered barrier structures, more complex than sills and weirs, and they require for their design, construction, operation, and maintenance the concerted effort of a number of technical disciplines, as discussed in this chapter.

Dams can be classified according to their purpose, the type of material used in their construction, and their geometry. Dams are built for power production, water supply, flood and river control, pumped storage, irrigation, recreation, and industrial waste disposal purposes. They may be conceived for permanent (long-term) life, or temporary operation.

Regarding construction materials, dams may be classified as embankment, masonry and rubble, concrete, and roller-compacted concrete (RCC) dams. Definitions for these dams have been provided by the U.S. Committee of Large Dams as follows [1].

Embankment dams: Any dam constructed of excavated materials or of industrial waste materials

Masonry and rubble dams: Any dam constructed mainly of stone, brick, or concrete blocks jointed with mortar (masonry) or unshaped and uncoursed stones (rubble)

Crib dams: A dam built up of boxes, cribs, cross timbers, or gabions filled with earth or rock

Concrete dams: Dams built of reinforced and/or unreinforced concrete

RCC dams: Dams built with a concrete of no-slump consistency that is compacted with vibratory rollers

Further variations within the above broad classifications are presented in Table 3.1, which is based on the definitions of the *Technical Dictionary on Dams* [1]. Cross sections of typical dams are presented in Figs. 3.1 through 3.10.

Classification of dams based on height has been arbitrarily proposed by the International Commission on Large Dams by defining a large dam as

1. A dam above 45 ft (15 m) high (measured from the lowest portion of the general foundation area to the crest), or
2. A dam between 30 and 45 ft (10 and 15 m) high which complies with at least one of the following:
   a. The crest length is not less than 1500 ft (500 m)
   b. The capacity of the reservoir is not less than 3.3 million ft^3 (1 million m^3)
   c. The maximum flood discharge is not less than 70,000 ft^3/s (2000 m^3/s)
   d. The dam has specially difficult foundation problems
   e. The dam is of unusual design

In this chapter, dams which are 45 ft (15 m) high or less will be considered to be small dams. However, since the design principles for both small and large dams are practically the same, examples of both types of dams will be presented throughout. Regardless of purpose or type, a dam may contain many elements, such as powerhouses, spillways, diversion features, intake and outlet works, gates and valves, fish ladders, log chutes, navigation locks, and ice booms.

This chapter deals with small dams, that is, earth, gravel, and rockfill embankment dams; concrete dams; and roller-compacted dams. Masonry, hydraulic, and tailings dams are not treated here. Discussions are primarily directed to the retaining structure itself, and not to other elements listed in the paragraph above. Readers in need of information related to subjects not treated in this chapter are referred to the extensive bibliography provided.

The information in the chapter has been organized in sections corresponding to the various types of dams, by construction material. Subjects covered in each case include foundation preparation, materials, design, construction, monitoring, inspection, and maintenance.

3.2 GEOTECHNICAL INVESTIGATIONS REQUIRED FOR DESIGN

The failure of a full-storage reservoir often results in considerable loss of life and property. It is, therefore, imperative that the retaining structure be designed in such a way that the possibility of failure is minimal. A safe dam requires that both the structure itself and the soil or rock foundation on which it is built are safe. It is because of this requirement that extensive geotechnical investigations at a proposed dam and reservoir site should precede the design of the structure.

Several factors (economic, environmental, hydrologic, hydraulic) are considered in the selection of a reservoir site from among a number of possible candidates. After the general reservoir site selection is made, attention is directed to

TABLE 3.1 Classification of Dams by Construction Materials

Embankment dams Any dam constructed of excavated natural materials or of industrial waste materials	Earth dam	Embankment dam in which more than 50 percent of the total volume is formed of compacted fine grained material obtained from a borrow area.	
	Rockfill dam	Embankment dam in which more than 50 percent of the total volume is composed of compacted or dumped pervious natural or crushed stone.	
	Hydraulic-fill dam	Embankment dam constructed of materials, often dredged, which are conveyed and placed by suspension in water.	
	Industrial waste dam	Embankment dam, usually built in stages, to create storage for the disposal of waste products from industrial processes. Embankment materials may be conventional natural soils or products from mining operation. Material placement can be either by hydraulic methods or by standard embankment compaction methods.	
Masonry dams	Masonry dam	Any dam constructed mainly of stone, brick, or concrete blocks jointed with mortar.	
	Rubble dam	A masonry dam in which the stones are unshaped or uncoursed.	
	Crib dam	A dam built up of boxes, cribs, crossed timbers, or gabions, filled with earth or rock.	
Concrete dams Any dam constructed of reinforced or unreinforced concrete	Gravity dam A dam which relies on its weight for stability	Arch gravity	An arch dam only slightly thinner than a gravity dam
		Curved gravity	Dam curved in plan view
		Cellular	Outward appearance of a gravity dam but of hollow construction

TABLE 3.1 Classification of Dams by Construction Materials *(Continued)*

	Buttress dam A dam consisting of a water-tight face supported at intervals on the downstream side by a series of buttresses	Flat slab dam or Ambursen dam or deck dam	A buttress dam in which the upstream part is a relatively thin flat slab usually made of reinforced concrete.
		Arch buttress dam or curved buttress dam	A buttress dam curved in plan.
		Multiple-arch dam	A buttress dam the upstream part of which comprises a series of arches.
		Solid head buttress dam	A buttress dam in which the upstream end of each buttress is enlarged to span the gap between buttresses.
	Arch dam A concrete or masonry dam which is curved in plan so as to transmit the major part of the water load to the abutments	Constant-angle arch dam	An arch dam in which the angle subtended by any horizontal section is constant throughout the whole height of the dam.
		Constant-radius arch dam	An arch dam in which every horizontal segment or slice of the dam has approximately the same radius of curvature.
		Double-curvature arch dam	An arch dam which is curved vertically as well as horizontally.
RCC Dams Dams built with concrete of no-slump consistency, and compacted with vibratory rollers			

Source: Definitions as provided in Ref. 1.

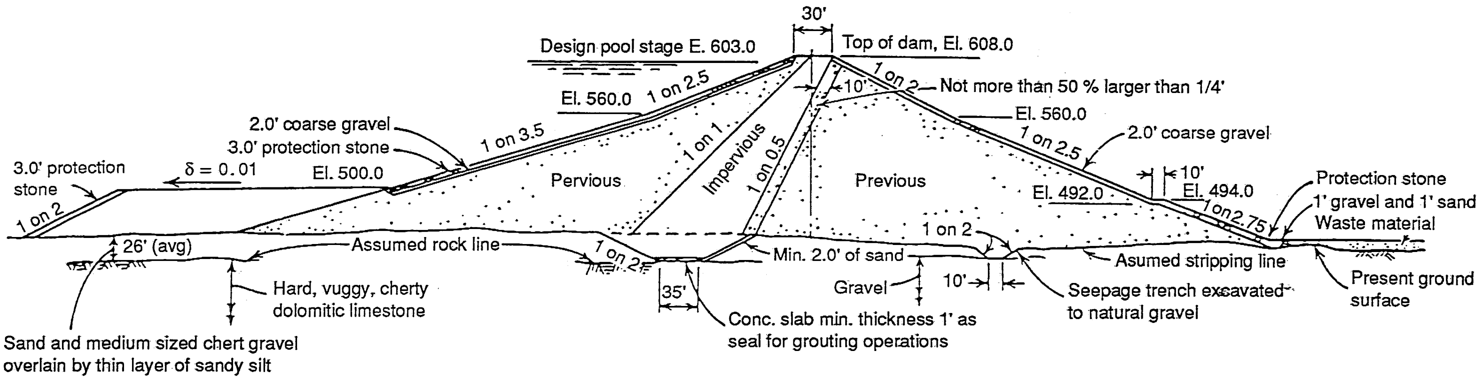


FIGURE 3.1 Clear Water Dam, Missouri. *(From Sherard et al. [2].)*

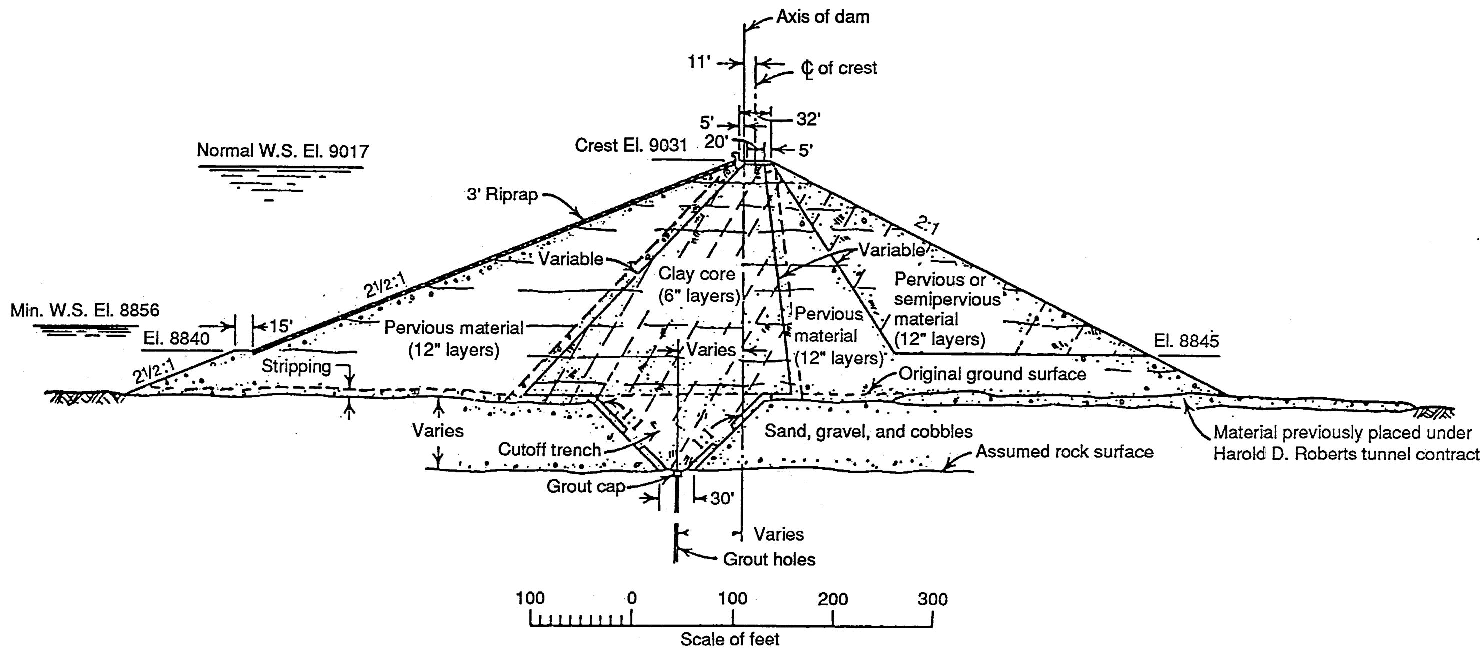


FIGURE 3.2 Dillon Dam, Colorado. *(From Sherard et al. [2].)*

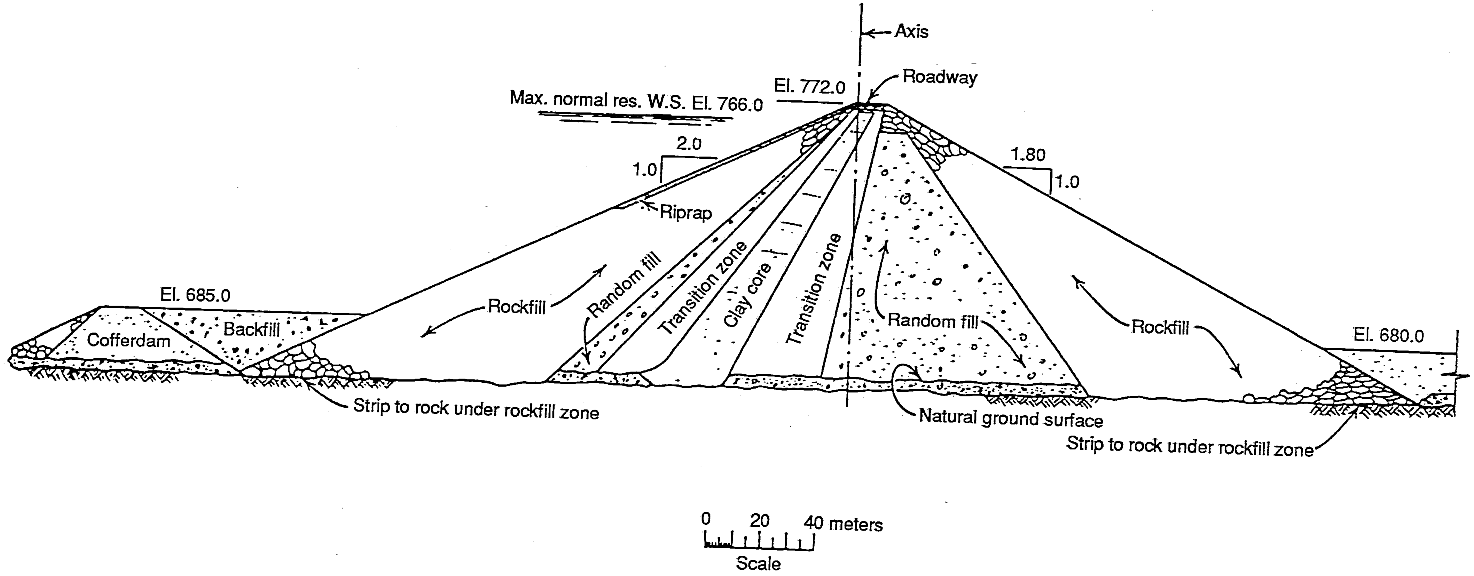


FIGURE 3.3 Furnas Dam, Brazil. *(From Sherard et al. [2].)*

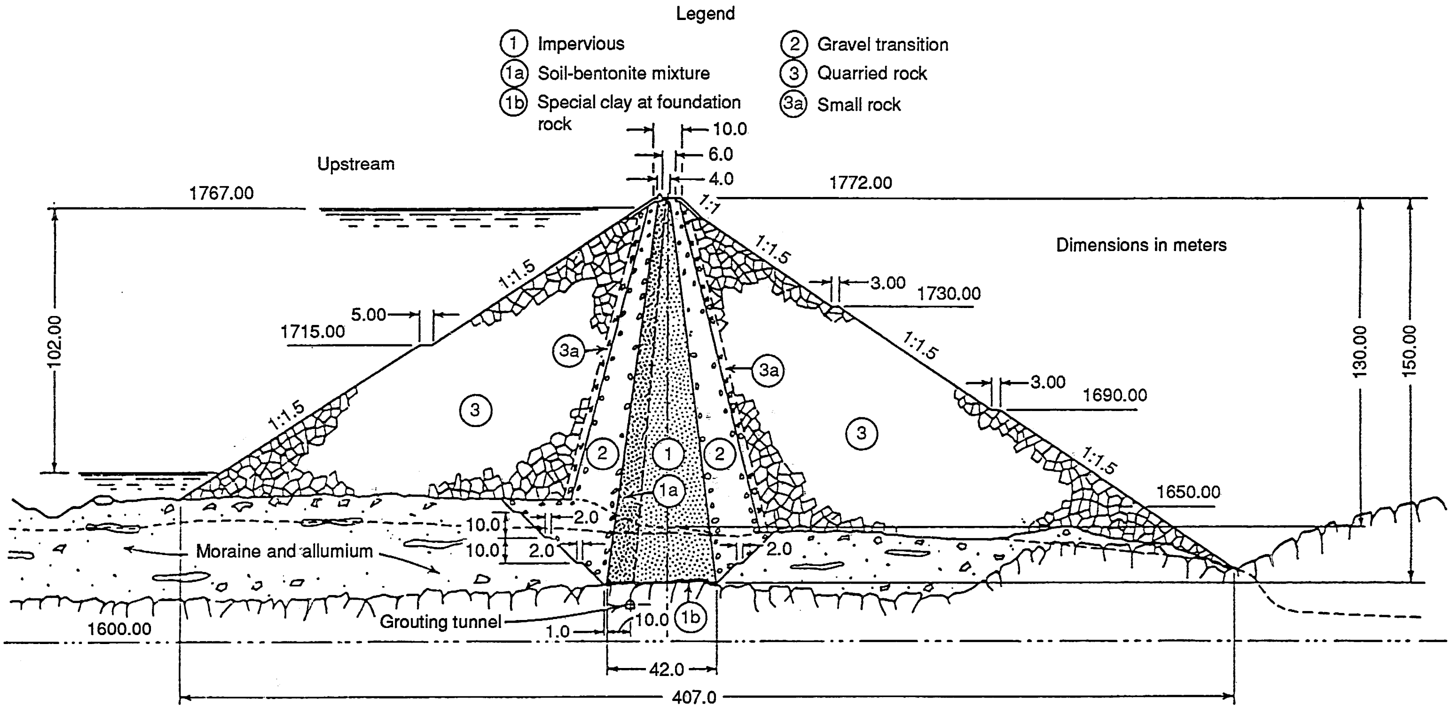
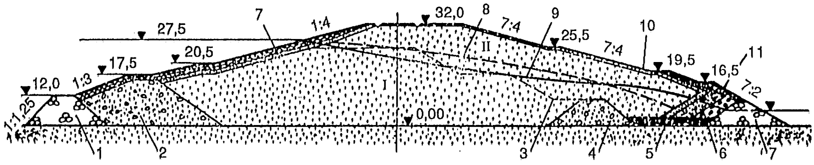


FIGURE 3.4 Gepatsch Dam, Austria. *(From Sherard et al. [2].)*



Legend

1-Cofferdams; 2-Support prism; 3-Boundary between first (I) and second (II) stages of filling; 4, 5-Drains; 6-Inclined filters; 7, 10-Slope protection; 8-Calculated line of seepage; 9-Observed line of seepage.

FIGURE 3.5 Kaira Kumskoe Hydraulic Fill Dam, Soviet Union. *(From Melentiev and Kolpashnikov [27].)*

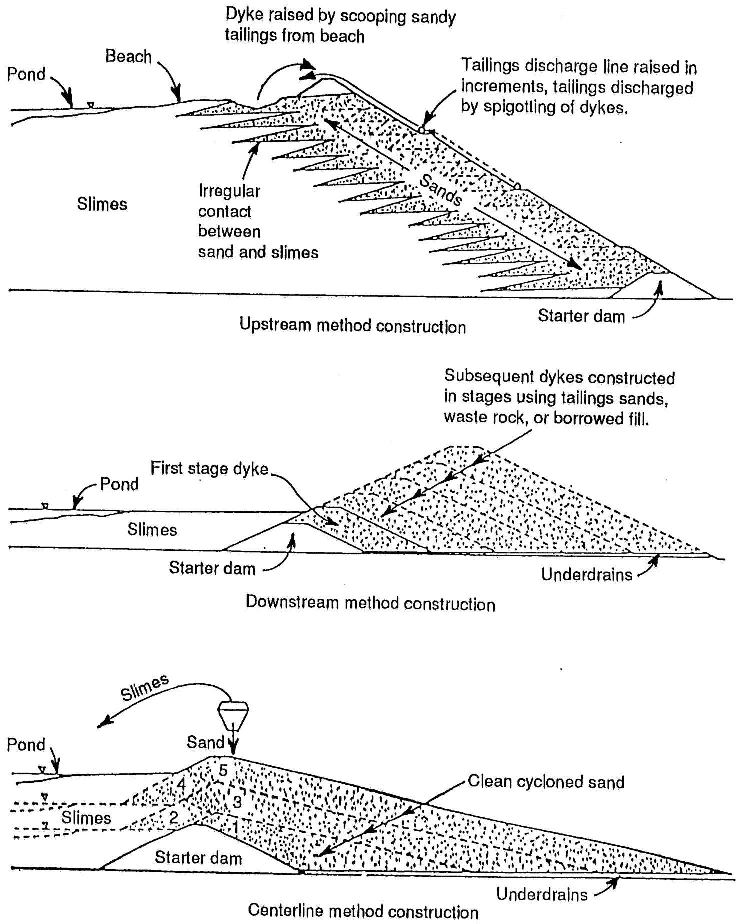


FIGURE 3.6 Upstream, downstream, and centerline tailings dams. *(From Finn [22].)*

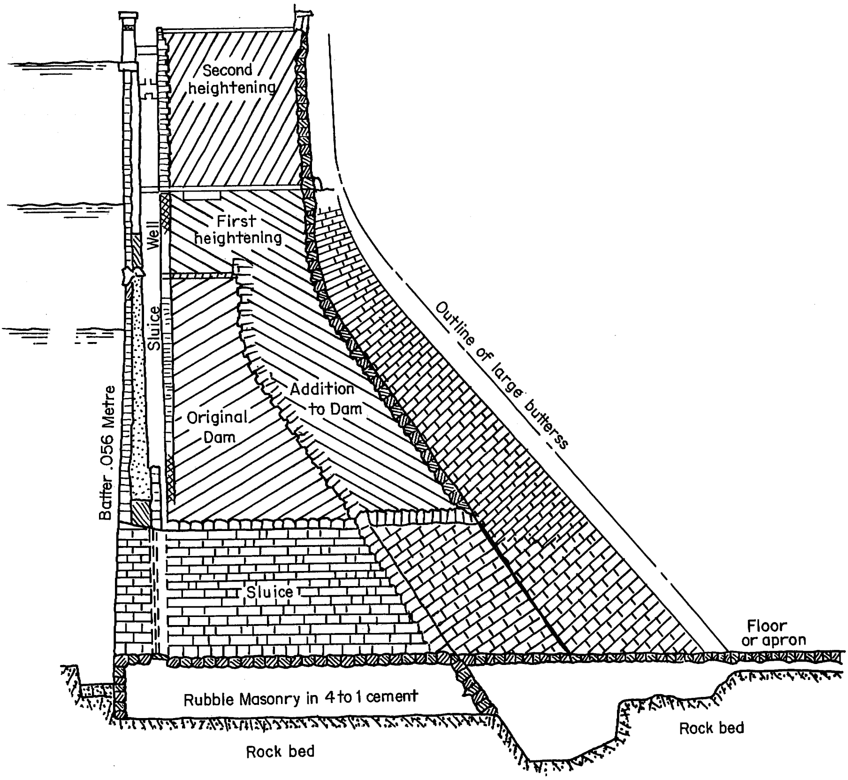


FIGURE 3.7 Old Aswan masonry dam, Egypt. *(From Zaki [23].)*

the study of the *type* of dam best suited to alternative dam site locations. Geotechnical considerations strongly influence the latter decision. Questions related to the characteristics of the foundation and its required treatment, embankment volume, practicality of construction of embankment and appurtenant structures, and availability of construction materials need to be answered. These questions cannot be resolved without adequate geotechnical information.

The objectives of the geotechnical investigations are, therefore,

- To characterize the distribution and the engineering properties (strength, compressibility, and permeability) of the soils and rocks which comprise the dam foundation and the abutments at alternative sites, and
- To study the extent and characteristics of available construction materials for embankments and for concrete

For small dams, the scope and the cost of the geotechnical investigations vary from site to site, depending on the complexity of the geology and on the variability of the soil and rock foundations. They also depend on whether the project is in the feasibility, design, or construction stage. During the feasibility stage, geotechnical investigations must be undertaken at each site to the extent that is

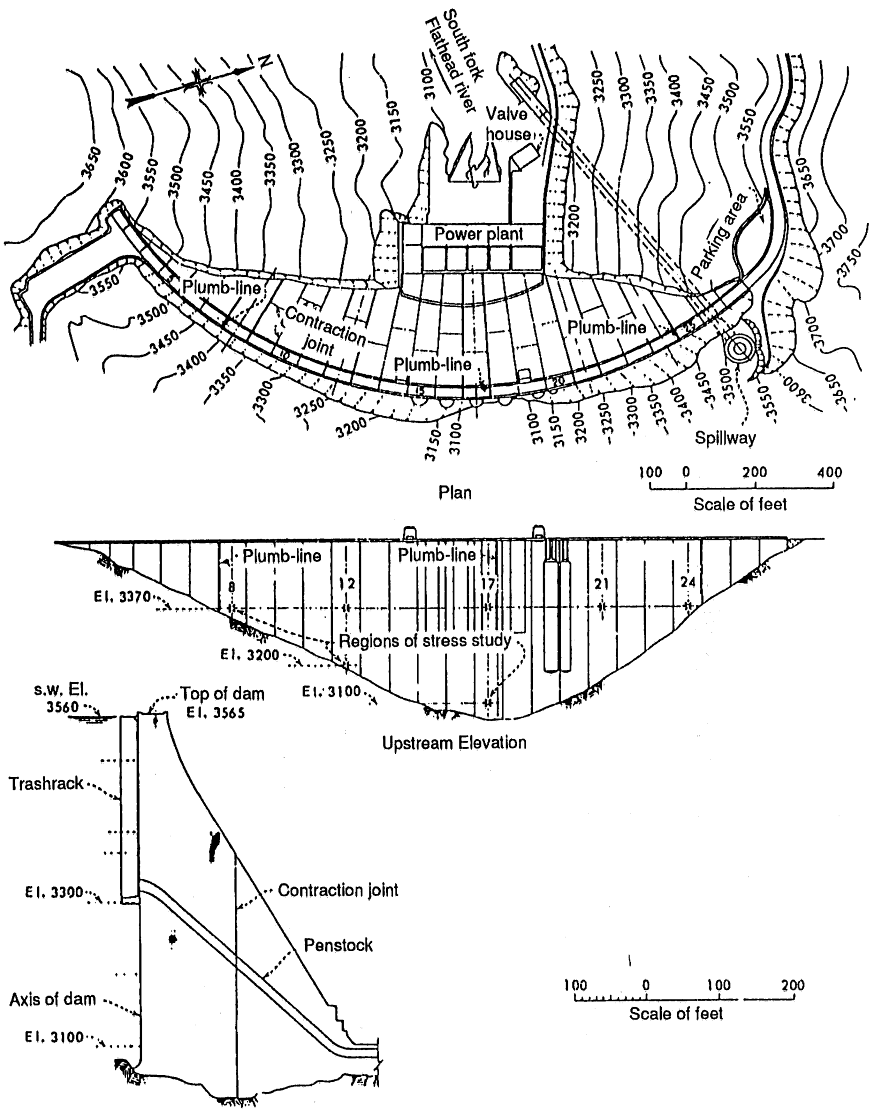


FIGURE 3.8 Hungry Horse Dam. *(From Davis and Sorensen [24].)*

necessary to permit a fair comparison of the costs of different types of dams on alternative sites. Once this phase is completed, and the type and location of the dam and of the appurtenant structures are defined, more detailed geotechnical information is obtained in order to proceed with the design of the facilities. As the project evolves from the design to the construction stages, more and more de-

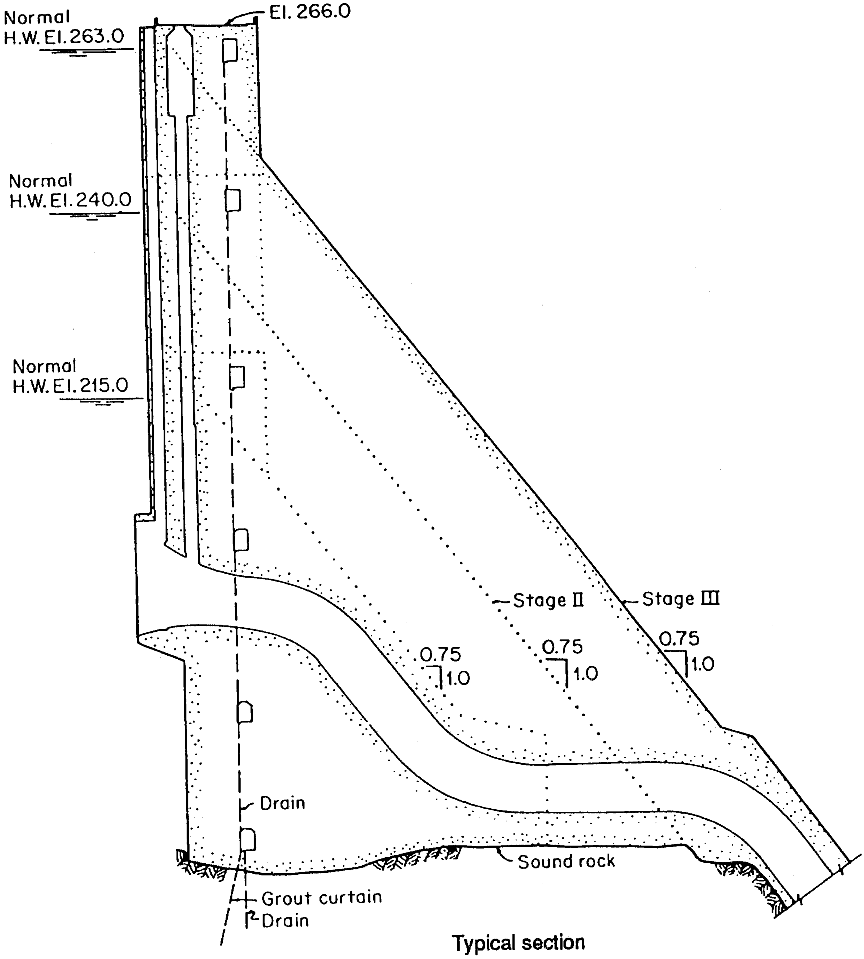


FIGURE 3.9 Guri Dam, Caroni River, Venezuela. *(From Davis and Sorensen [24].)*

tailed geotechnical information is required regarding delicate or troublesome aspects of the soils and rocks at the site.

Source of Information

As an initial step, available information concerning the site area should be collected including topographic, geologic, and soil maps and aerial and Landsat photographs.

Topographic Maps. A search should be made for those maps covering the areas of the reservoir, the dam site, and the potential borrow areas. The location and elevation of exploratory holes, trenches, and pits and significant physical fea-

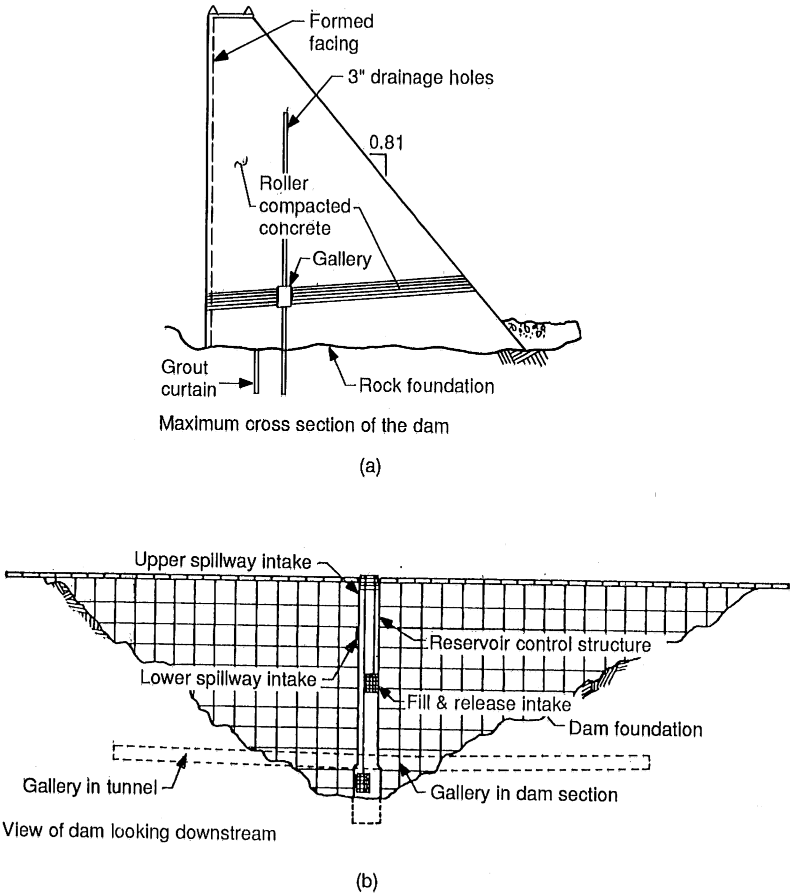


FIGURE 3.10 Middle Fork RCC Dam, Colorado, U.S.A. *(From Exxon [25].)*

tures, such as rock outcrops, landslides, or roads, trails, etc., can be placed on the map. The topography of the storage basin is used to determine the reservoir storage volume available below various levels and the presence of any saddles along the perimeter of the reservoir. The topography of the dam site can be used to estimate amount of excavation and embankment materials and to lay out the dam, appurtenant structures, and access roads.

Geologic Maps. Geologic maps are prepared for the region where the project is located and for the dam site proper. The information and the detail provided by each are different. Several types of geologic maps are available. Maps showing a plan view of the bedrock in the area is a *bedrock geologic map*. Generally such maps depict visible boundaries of rock formations and undifferentiated overburden. *Surficial geologic maps* differentiate the overburden according to its origin, such as stream alluvium and glacial or wind deposits. *Structural geologic and tectonic maps* indicate the location and characteristics of geologic faults and, generally speaking, of lineaments which can be recognized from physical features such as offset of beds and dikes; presence of gouge, or zones of badly fractured rock; or topographic features, such as linear trenches or sag valleys, offset alignment of rivers, and vegetation.

Related to structural geology is the issue of the seismic activity within the region. Catalogs of historic seismicity should be compiled for the region for later study and correlation with structural geologic and tectonic maps.

Other Maps. Useful information can also be derived from maps and documents on mineral resources and agricultural soil maps which are prepared by various public agencies.

Air Photos. These include low-altitude vertical and oblique photos; low-sun, low-altitude photos; and the sophisticated family of satellite imagery which, with appropriate filtering and data manipulation, can provide surprisingly varied and detailed information. Photos are primarily used to identify surficial features: topography, drainage and erosion patterns, vegetative cover, landslides, lineaments, joint systems, and fault zones. In some cases, however, experienced individuals are able to interpret surficial features reliably to predict deep underground conditions such as the presence of karstic formations.

Exploration Methods

Following the study of the information provided by maps and photos, a program of field exploratory work (including testing) can be prepared. This program should consist of a detailed field reconnaissance and mapping by engineers and geologists, and the execution of subsurface explorations, and soil and rock sampling, including boreholes, test pits, trenches, adits, geophysical surveys, in situ soil and rock permeability, and strength testing.

Auger borings performed by hand can successfully be performed for shallow-depth investigations where disturbed samples are satisfactory (for example, for borrow area investigations) and where the presence of groundwater or of the maximum particle size of the materials is adequately handled by the size of the equipment used. Under favorable circumstances, depths of about 18 ft (6 m) can be reached by this method, which can be extended to over 45 to 60 ft (15 to 20 m) if a tripod is available. *Machine-driven augers* vary between about 4 and 16 in (10 and 40 cm) in diameter (helical) to over 3 ft (1 m) (disk and bucket).

Augers can be used for boring the hole and for retrieving disturbed soil samples. Soil sampling (Shelby, California, and others) and testing (standard penetration tests) are done within the borehole at desired frequency. Usually boreholes are drilled without the addition of water, but if the soil formations on hand are sandy, or water is desired for testing purposes, casing of the hole is usually required in order to prevent the collapse of the walls.

Rotary drilling equipment is manufactured in a variety of types from lightweight and highly mobile, to heavy, stationary units, and with capacity and attachments capable of drilling holes in soils and rocks from less than 0.8 in (2 cm) to more than 3 ft (1 m) in diameter. Depths of several hundred meters can often be reached. Undisturbed soil samples and rock cores can be retrieved from the boreholes. At the same time, the borings can be used for permeability testing (Lugeon, Packer tests), density tests (standard penetration), modulus of deformation tests (Menard pressure test), etc.

Open test pits and *trenches* are of great use for visual observation of stratigraphy, for performing tests, and for recovery of samples. Water and soil-stability conditions permitting, these methods are highly recommended for dam foundation investigations and geologic fault studies, and are, in fact, a very expeditious method of sampling and observation in deposits containing gravels and cobbles.

Adits have two advantages: they permit the visual inspection of the subsurface soils and rock, and, if required, they allow the performance of a variety of in situ tests. The walls, floor, and roof of the adits can be mapped and photographed, and the direction of seams, discontinuities, and rock jointing observed. Adits are especially useful in the investigation of the soil and rock conditions along the abutments of a dam. Adits with minimum dimensions of about 6 ft (2 m) by 4.5 ft (1.5 m) can be excavated very economically with unsophisticated equipment.

Geophysical Surveys. These investigations are used to supplement and extend information obtained from borings, trenches, and adits. Velocities correlated with known stratigraphy can be used to delineate the thickness of overburden, zone of rock weathering, unstable slopes, and variations in general stratigraphic trends. Geophysical surveys provide an invaluable means of obtaining data from which the low-strain compression and shear elastic moduli of soils and rocks can be calculated. Geophysical programs may include surface refraction (for compression or shear-wave velocities), cross-hole, down- and uphole surveys; electric resistivity, γ–γ logging, gravity surveys, borehole logging; magnetic surveys; etc.

Field Tests. In many cases, where properly done, field tests provide the best means to obtain highly reliable information on in situ soil and rock properties, either because a larger mass of material is involved in the field determination or because of difficulties in obtaining good samples for laboratory testing. Typical examples include permeability tests (in pits or boreholes), deformation tests, large shear tests, in situ density of sandy and gravelly soils, and test fills.

Laboratory Tests. The procedures for rock, soil, and aggregate testing are well developed. Equipment and procedures, well standardized by internationally recognized associations, are available which provide means to identify and characterize the strength, compressibility, permeability, durability, and in general the adequacy of materials encountered in nature for use in the construction of embankment and concrete dams, and the study of the foundation materials at proposed dam sites.

The Role of the Geotechnical Engineer

The various tools available for the exploration of dam and reservoir sites have been summarized in the preceding section. It is important that in the work related to a particular project, these tools be judiciously used in number, location, and type, so that a complete and accurate characterization of the materials is obtained. This can only be done by geotechnical engineers experienced in the design and construction of dams who, in collaboration with knowledgeable engineering geologists, can scope and execute a satisfactory geotechnical investigation.

Because of the variety of subsurface conditions and the materials that may enter in the construction of dams, the design of every new dam calls for individualized treatment. There are, however, certain "musts" that the geotechnical investigations should address. For reference purposes, Table 3.2 provides a unified view of important issues that must be addressed by the geotechnical investigations.

3.3 SELECTION OF DAM TYPE

As far as technical feasibility is concerned, often more than one type of dam is adequate for a selected dam site location. The final selection, then, is either based on economic considerations, on preferences of the designer or owner, or on the decision of a consulting board. Following is a list of factors which the dam designer must consider in selecting the most appropriate structure for a site:

- Topography
- Dam foundation
- Availability of construction materials
- Flood hazard
- Seismic hazard
- Construction time
- Climate
- Governmental regulations
- Available resources

Topography

Narrow valleys with high rock abutments favor concrete dams.

Low rolling hills favor earth dams.

Hydraulic fill dams are frequently associated with *wide, flat alluvial plains* with minimal topographic relief.

Dam Foundation

Rock foundations, properly cleaned of weathered material and treated for water tightness, are ideal for any type of dam.

Dense sand-and-gravel foundations are adequate for all embankment dams, and for small concrete dams when proper seepage control measures are implemented.

Compressible silt and clay foundations preclude the consideration of concrete dams and require special care for rockfill dams.

Loose sand foundations in a seismic environment are subjected to potential seismic liquefaction and are inadequate for any type of dam. If the loose materials are excavated, or their physical conditions improved, then an embankment dam could be considered.

TABLE 3.2 A Checklist for Required Geotechnical Investigations

Objective of the investigation	Comments
	Rock foundations
Crushing and shearing strengths	Adequate for small dams; possible exception: shales and siltstones. Investigate weathering and microcracks and fissures; rock strength in the laboratory may differ from mass-rock strength. Investigate extent, nature, and properties of clay seams, including residual strength, effect of saturation. Investigate brecciated zones.
Rock deformation characteristics and residual stresses	Determination in the laboratory (modulus of elasticity) and in situ mass-rock deformation as affected by presence of fissures, seams, etc. Residual stresses to be determined by field testing.
Permeability	Investigate jointing, seams, and bedding in the field. Hydrologic investigations required for determining the nature of groundwater, whether normal, perched, or artesian. Perform field-permeability tests in boreholes. Investigate presence of karstic formation, limestone, other cavities.
Active tectonic faults	Studies to be done both at the dam site and within the reservoir area. Carry out trenching to observe possible fault displacement in Holocene deposits.
	Soil foundations
Strength and compressibility	Granular soils: requires determination of in situ densities and/or a field-test-correlatable densities with strength parameters such as standard penetration resistance, Becker penetration tests (for gravels), and cone penetration tests.
	Fine-grained soils: primarily based on laboratory test data.
Permeability	Proper definition requires an accurate picture of the stratigraphy, and a series of field-permeability tests (Lugeon, Packer) or well-pumping tests.

TABLE 3.2 A Checklist for Required Geotechnical Investigations *(Continued)*

Objective of the investigation	Comments
	Construction materials
Location	Identify distance to dam site, access road; thickness of overburden to be wasted, excavation difficulties, water table difficulties, in situ moisture content, blasting characteristics (in the case of rocks).
Properties	Characterization of the engineering properties for use in embankment construction or as an aggregate for concrete is done in the laboratory. Prior to, or during initial stages of, construction test fills are often carried out to verify adequacy of proposed construction procedures. Properties to be investigated include gradation, plasticity, moisture/density relationships, permeability, static and dynamic strengths, compressibility, durability, chemical makeup. Quality of materials for preparation of concrete.

Availability of Construction Materials

Materials are required for the construction of the embankment (core, shells, filters, slope protection) and manufacture of concrete. When *adequate materials are available* near a site, embankment dams can usually be built at a lower cost than concrete dams.

Availability of sands and gravels, but absence of impervious clays may favor the choice of a concrete dam. On the other hand, *if an impervious soil is readily available*, the design may favor a homogeneous embankment dam with a few internal granular filters provided for seepage control.

Flood Hazard. The possibility of *flooding* during construction favors either a concrete type of dam or a rockfill dam with or without downstream reinforcing. Associated with flooding is the *spillway* requirement. Often the cost of constructing a spillway is high. For such cases, combining spillway and dam into one structure (concrete dam) may be advantageous. In other cases, where the excavated material from a separate spillway can be used in the construction of the embankment, an earthfill embankment may be advisable.

Seismic Hazard. Potential *fault rupture* along the dam foundation precludes the consideration of any rigid structure such as a roller-compacted or a concrete-type dam. Embankment dams with large zones of sand and gravel are recommended in these cases. Potentially *strong earthquake ground motion* may rule out the consideration of rigid structures (concrete) or embankment dams built with loosely placed granular soils (hydraulic and tailings dams).

Construction Time. When construction time is limited, it is often necessary to adopt a structure which is not necessarily the most economical; for example, a dumped-rockfill rather than a smaller concrete structure, or a flatter homogeneous clay dam rather than a zoned embankment.

Climate. Construction of embankment dams during the rainy season is often limited to the pervious zones, making rockfill dams more appropriate. During freezing weather, precautions must be taken to avoid damage to freshly poured concrete in concrete dams. Rockfill dams may prove to be cheaper to construct in severe climates.

Diversion Works. Valley configuration, hydrologic, and schedule considerations can often pose serious construction difficulties which require expensive works.

Government Regulations. Federal and state governments have issued regulations relating to the construction and operation of dams. The safety, the environmental impact, and the purpose of the dam, as judged by the public agencies, will often suggest the most suitable type of structure for any given set of circumstances.

Available Resources. At some sites, neither skilled contractors for a specified construction nor adequate labor force may be available. For example, a country may have neither the experience nor the equipment necessary for the construction of a roller-compacted concrete dam or for the concrete face in a rockfill dam. In such cases, a simpler earth embankment dam may be more appropriate.

The numerous factors influencing the selection of the type of dams that have been discussed clearly indicate that such selection is far from easy and that no general rules can be advanced to aid the designer in this task. However, experience accumulated in years of practice has indicated the following comparison between the advantages and the disadvantages of embankment and concrete dams.

3.4 EMBANKMENT DAMS

Dams constructed with excavated or industrial waste materials are embankment dams (see Table 3.1). Embankment dams vary from two basic material sections, as shown in Fig. 3.1, to more complicated ones built with a variety of materials, such as that shown in Fig. 3.11. Whether it is a simple or a complex structure, the basic elements of the dam are an impervious barrier (core or facing) which opposes the flow of water, one or more zones of structural material, and shell(s) which provide support and stability to the core. For reasons to be discussed in other sections of this chapter, layers of sand-gravel material are placed in between the core and/or facings and the shells. Such granular layers are called "filters" or "transition zones," depending on their purpose.

Typical cross sections of a great variety of as-built embankment dams are presented in Refs. 2 and 3. The idealized cross sections presented in Figs. 3.12 and 3.13 imply that the embankments are placed on an impervious base. If, on the contrary, the embankment is placed on a rather pervious base and considerable water losses may occur, then to fulfill its purpose (i.e., to contain the reservoir's waters), some type of foundation treatment may be required. As shown in Fig. 3.14, this may consist of a slurry trench, fully or partially penetrating cutoff trenches (cores), upstream impervious blankets, or downstream relief wells.

Earth Dams

Elements. Figure 3.15 shows two typical embankment cross sections depicting the basic components of the structures: impervious core, up- and downstream shells, filters, drains, and transitional layers. Both dams rest on impervious foundations (rock). As implied by its name, the basic element of an earth dam is the core, or impermeable barrier. The other elements in the section of the dam are provided to ensure force and hydraulic stability.

Materials. Table 3.3 shows a soil classification chart based on the Uniform Soil Classification System. Most soils can be used for embankment construction unless they have objectionable physical or chemical properties. For example, soils with high salt content, soils with significant organic content, or silts and clays with dispersive characteristics should not be used. Organic material increases a soil's compressibility and tends to lower its shear strength. Some soils have properties which make them difficult to use. Fat clays, which may have high liquid limits, will generally be very difficult to compact. Moisture content of silts and silty soils is low for optimum compaction, requiring special care to control moisture both on the embankment and in borrow areas. They are particularly difficult to use in rainy climates. In dry climates, it will usually be necessary to add water to obtain proper compaction. In situ moisture content should be studied when a borrow area is being selected.

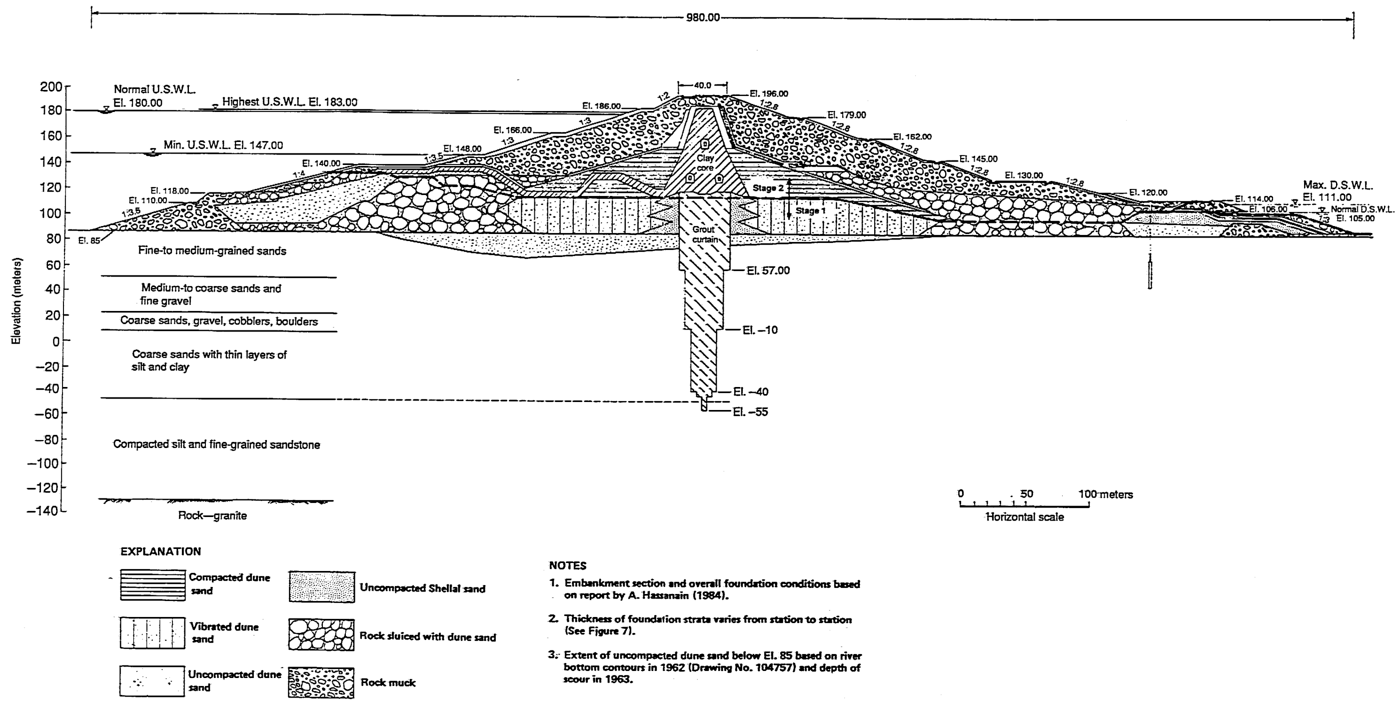


FIGURE 3.11 High Aswan Dam, Egypt. *(From Wafa and Labib [32, 26].)*

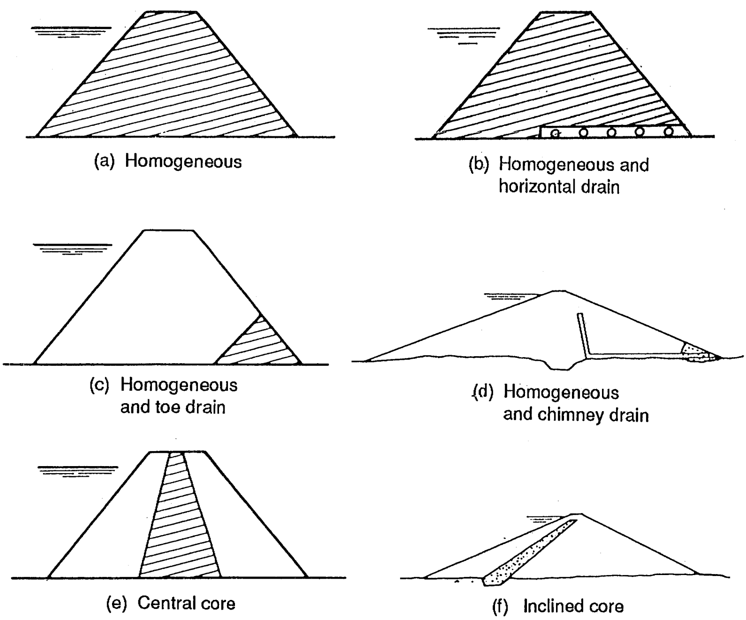


FIGURE 3.12 Earthfill dams: idealized cross sections.

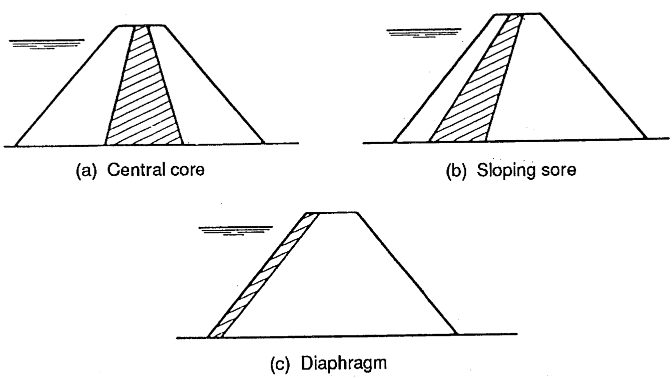


FIGURE 3.13 Rockfill dams: idealized cross sections.

Foundation and Abutments Preparation. Whether an embankment dam is located at a site with rock abutments and foundation or on a deep deposit of alluvial materials, it is necessary that, in the design of the dam, proper attention be given to the flow of water under the dam. This is to prevent water loss, but more importantly, because of the potential threat that an uncontrolled seepage may have on the stability and ultimate safety of the dam. In addition, the nature of the rock and of the overburden present may require that certain work be done in preparation for construction of the structure, as discussed below.

Rock Sites. Three aspects need to be studied: the shape and surface topography of the valley, the quality of the rock, and the permeability of the rock mass. The

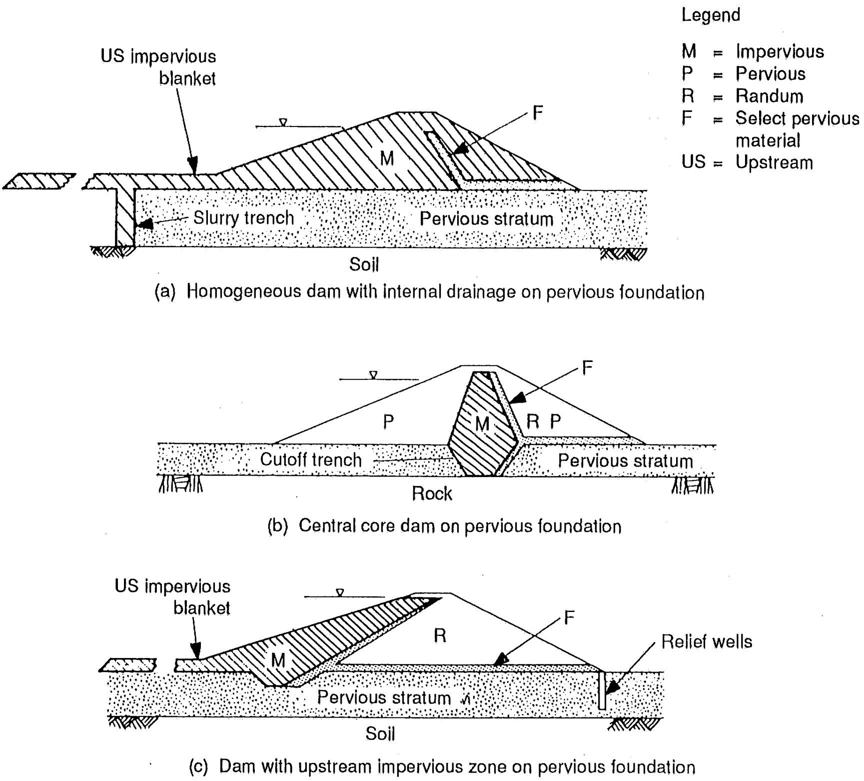


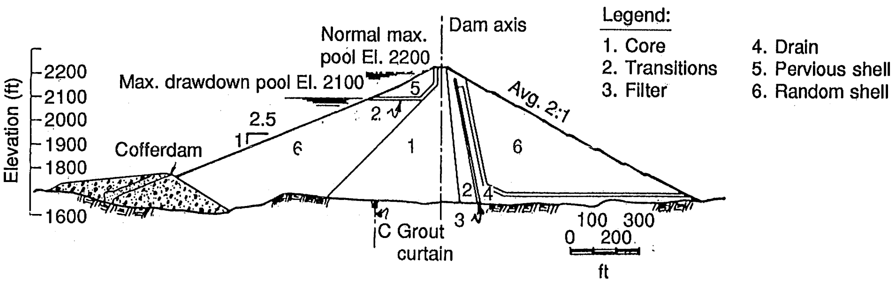
FIGURE 3.14 Embankment dams on pervious foundations. *(From bibliographic reference No. 2 on Dam Engineering.)*

shape and surface topography of the valley affect both the mobility of equipment during construction and the quality of the bond between rock and the embankment.

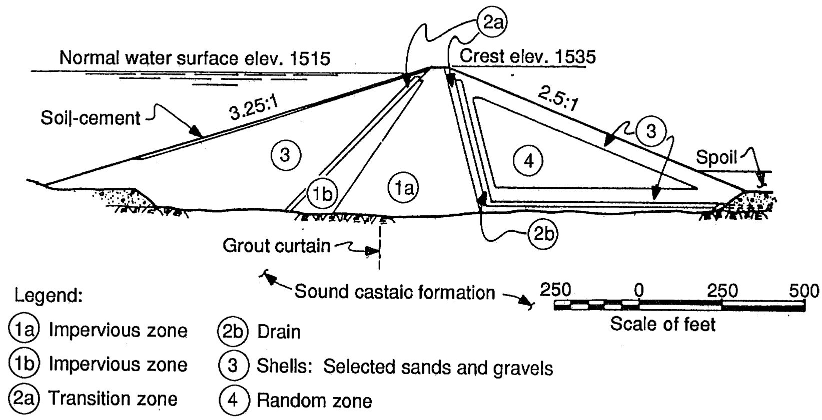
The contact surface between hard rock and the embankment usually presents compaction difficulties and is a potential path for seepage channels. In contrast, embankment materials can be compacted with less difficulty on a soft rock surface such as sandstone. The permeability of a rock foundation may range from practically zero to very large, depending on weathering, fracturing, and jointing. As a result, the applicability of different methods of seepage control to a given site must be considered in detail.

SURFACE TREATMENT: The objective of the treatment is to provide a smooth surface against which the impervious soils of the core can be compacted. If rock is soft, this operation can be accomplished without much difficulty. Hard rock surfaces are more difficult to prepare and are treated by removing protrusions and overhangs with hammers or explosives, and filling depressions with concrete, compacted soil, slush grout, gunite, and dental concrete. Generally speaking, the flatter the average abutment slope, the better the bond between the embankment and the rock.

SEEPAGE CONTROL: Several methods to control the path of possible seepage along the contact surface between the embankment and the rock foundation have been used.



(a) Cross Section of Partage Mauntain Dam (W.A.C.Bennett Dam), Canada



(b) Cross Section of Castaic Dam, California

FIGURE 3.15 Embankment cross sections depicting basic structural elements. *(From Wilson and Squier [4].)*

Concrete cutoff walls on steep abutments were popular in early times, but recently their use has been less frequent because (a) compaction of soil with heavy rollers is very difficult to achieve in areas near the wall, and (b) the frequent need to blast the excavation for the wall, which fissures the rock, making it more pervious.

Cut-off trenches backfilled with impervious soils are often used especially in situations where the rock is relatively soft, and its permeability decreases with depth. When hard rock is present, it is often preferred to treat the foundation with grout.

Grouting is used to make rock foundations and abutments more impervious, so that seepage losses are reduced, and water pressures, which may decrease the stability of the embankment, are brought under control. The determination of the need for rock grouting, and the actual technology of rock grouting, is a matter that requires the close attention of specialized individuals in the area of

TABLE 3.3 Soil Classification Chart

	Field inspection procedures	Group symbol	Typical names
	Coarse grained soils		
Gravels: Clean	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
With fines	Nonplastic fines (for identification procedures see ML below)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures
	Plastic fines (for identification procedures see CL below)	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures
Sands: Clean	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well-graded sands, gravelly sands, little or no fines
	Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Poorly graded sands, gravelly sands, little or no fines
With fines	Nonplastic fines (for identification procedures see ML below)	SM	Silty sands, poorly graded sand-silt mixtures
	Plastic fines (for identification procedures see CL below	SC	Clayey sands, poorly graded sand-clay mixtures

Fine grained soils

Soil type	Dry strength	Dilatancy	Toughness	Group symbol	Typical names
Silts and clays	None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.
	Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, clays
	Slight to medium	Slow	Slight	OL	Organic silts and organic silt clays of low plasticity
	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays
	Medium to high	None to very Slow	Slight to medium	OH	Organic clays of medium to high plasticity
Highly organic soils	Readily identified by color, odor, spongy feel and frequently by fibrous texture			Pt	Peat and other highly organic soils

Source: From Ref. 3.

rock mechanics and engineering geology. Complete references on the subject are contained in the bibliography.

Alluvial Sites. For embankments placed on a deep alluvial formation, a positive cutoff, formed in an open excavation, extended to an impervious stratum, and backfilled with compacted impervious material, is the most desirable form. When this cannot be done from a practical standpoint, other measures must be considered. Some in common usage at the present time are: (1) grout curtains, (2) concrete cutoff walls, (3) slurry trench cutoffs (earth backfilled), (4) upstream impervious blankets, (5) sheet piles, and (6) vertical drains or relief wells.

SEEPAGE CONTROL: The most satisfactory method for *grouting* deep deposits of alluvium at the present time is the *tube à manchettes* or the sleeve pipe method of grouting. The principal advantages of this procedure are that the same tube can be reentered as often as necessary to grout with different types of mixtures and to perform additional grouting, if necessary, even after a considerable lapse of time. Table 3.4 shows some of the major projects where a grout cutoff was formed through deep deposits of alluvium, consisting of a wide variety of materials, from fine-to-medium sands to coarse sands and gravels with cobbles and boulders.

Cutoff walls require specialized equipment and experience. They may be either concreted or grouted. Examples of typical concrete walls are presented in Table 3.5.

Other methods of seepage control such as the slurry trench cutoff walls and upstream impervious blankets are commonly used (Fig. 3.14*a*).

When the depth of excavation is great, or when the control of groundwater is difficult, a cutoff can be achieved through the use of a slurry trench. Excavations through pervious foundations by backhoe, dragline, or other means have extended to depths of 75 ft (25 m) and more. The width of the trench is dependent upon excavating equipment, and the sides of the trench are supported by a bentonite slurry. The location of the trench is based upon judgment. While trenches under impervious cores have been used, it is generally accepted that the best location for the trench is beyond the upstream toe of the dam so that repairs may be made if necessary. Table 3.6 contains data on various projects for which slurry trench cutoffs have been adopted.

The horizontal upstream impervious blanket, which increases the horizontal length of the average path of underseepage, is more effective in controlling seepage through a homogeneous soil foundation than the partial vertical cutoff. If the blanket is very impervious compared to the natural foundation so that relatively little seepage through the blanket occurs, then the reduction in the seepage quantities and pressures at the downstream toe are directly related to the length of the blanket. If the blanket is only slightly less pervious than the foundation material, there is a maximum length of blanket beyond which no appreciable additional value is obtained by increasing the length. The necessary thickness and length of a blanket depend on the permeability of the blanketing material, the stratification and thickness of the pervious foundation, and the reservoir depth. Thicknesses varying from 2.5 to 9 ft (0.8 to 3.0 m) are most frequently used.

Crest Width and Camber. The width of the dam crest has no appreciable influence on the embankment or foundation stability and does not have a large influ-

TABLE 3.4 Effectiveness of Grouting in Alluvium at Some Major Dams

Project (year)	Foundation materials	Max. depth, ft (m)	Hydraulic gradient across grout curtain	Postgrouting permeability, cm/s
Sylvenstein Dam, Isar River, Germany (1958)	Sand and gravel, $k=5\times10^{-1}$ cm/s	330 (100)	2 to 7	$k=1.3\times10^{-4}$
Serre Poncon Dam, Durance River, France (1959)	Sand, gravel, and cobbles, $k=3\times10^{-1}$ to 9×10^{-2} cm/sec	360 (100)	3.5 to 8	$k=2\times10^{-5}$
Terzaghi Dam (Mission Dam), Bridge River, British Columbia, Canada (1960)	Sand, gravel, and some boulders	500 (153)	3 to 4	Assumed: $E = 90\%$*
Notre-Dame de Commiers Dam, Drac River, France (1963)	Sand and gravel, $k=10$ to 3×10^{-2} cm/s	17 (50)	2.5 to 6.5	—
Mattmark Dam, Visp River, Switzerland (1967)	Sand and gravel with cobbles $k=10^{-1}$ to 10^{-3} cm/s	300 (100)	3 to 7	$k=2\times10^{-5}$
Mangla Dam (Closure Dam), Jhelum River, West Pakistan (1967)	Gravel and cobbles with sand $k=10^{-1}$ cm/s	75 (23)	3 to 4	$k=5\times10^{-5}$
Aswan Dam, Nile River, Egypt	Fine to coarse sands $k=1\times10^{-1}$ to 5×10^{-3} cm/s	835 (255)	2 to 4	$k=3\times10^{-4}$

*$E = h'H$ where h' = head loss across the curtain, and H = difference in elevation between the reservoir at maximum pool and the tailwater level.
Source: From Ref. 4.

ence on the embankment volume. It is determined only by the required working room. No dam should have a crest width of less than 9 ft (3 m), because this is the minimum needed for an access road to permit maintenance work and for the necessary construction equipment.

The 1957 Japanese Code quoted in volume 2 of Ref. 6 specifies crest width W in terms of height of dam H as

$$W = 3.6\sqrt[3]{H} - 3 \text{ (meters)}$$

TABLE 3.5 Thin Concrete Membranes Used as Vertical Cutoffs

Project	Foundation	Wall thickness, ft (cm)	Maximum depth of wall, ft (m)	Maximum head, ft (m)	Remarks
Sesquile Dam, Bogota River, Columbia, South America (1963)	Sand and gravel	1.8 (55)	255 (76)	100+ (30+)	ICOS
Cofferdam-Manicouagan 2 Dams, Quebec, Canada (1963)	Alluvium, with boulders	2.5 (76)	90 (27)	(?)	ICOS
Cofferdam-Manicouagan 5 Dam, Quebec, Canada (1964)	Sand, gravel and boulders	2.0 (60)	250 (76)	230 (70)	ICOS
Kinzua Dam (Allegheny Reservoir Dam), Pennsylvania, U.S.A. (1965)	Silts, sands and gravels	2.5 (76)	180 (55)	125 (38)	ICOS; f'_c = 3840 lb/in^2 (2740 N/cm^2); Cost: U.S. (1977) \$20.00/ft^2 [U.S. (1977) \$215/m^2]
Peneos Dam, Peneos River, Greece (1965)	Sand and gravel	2 (60)	60 (18)	160 (49)	Rodio & Co. Reinforced
Cofferdam-Arrow Dam, Columbia River, British Columbia, Canada (1967)	Sands, gravels, cobbles and boulders	2.5 (76)	165 (50)	115 (35)	ICOS* f'_c = 4000 lb/in^2 (2850 N/cm^2); Cost: U.S. (1977) \$21.00/ft^2 [U.S. (1977) \$225/m^2]
La Villita Dam, Balsas River, Mexico (1968)	Sand and gravel with cobbles	1.7 (50)	290 (88)	140 (42)	ICOS

*Process of Impressa Costruzioni Opere Specializzate, Milan, Italy.
Source: From Ref. 4.

For small dams, say 30 to 45 ft (10 to 15 m) high, this expression yields a crest width of between 15 and 18 feet (5 and 6 m).

At the end of construction, the crest should be given a sufficient camber to allow for the postconstruction settlement of the embankment and foundation without a reduction in the freeboard. The amount of settlement to be expected from an embankment dam is discussed subsequently in this chapter.

Freeboard. An earthfill or a rockfill dam (unless properly designed) can stand little or no overtopping. Overtopping during a flood, even for a very short time, can result in erosion and breaching of the dam. Thus, in determining the freeboard to provide for a dam, conservative assumptions are made.

The necessary freeboard is calculated by assuming that the maximum river flood will occur when the reservoir is full and that wind-driven waves will develop at the same time. The minimum freeboard equals the computed head on the spillway crest at maximum flood discharge, plus 1.5 times the wave height

TABLE 3.6 Slurry Trench Cutoffs

Project	Foundation material	Trench width, ft (m)	Maximum head, ft (m)	Remarks
Kennewick Levee, McNary Dam Project Columbia River, Washington State, U.S.A. Owner: Corps of Engineer	Sandy or silty gravels with zones of open gravel; $k = 0.4$ cm/s	6 (1.89) central core	15 (4.92)	Constructed in 1952; max. depth 22 ft (7 m)
Wanapum Dam, Columbia River, Washington State, U.S.A. Owner: Public Utility District No. 2 of Grant County	Sandy gravels and gravelly sands underlain by open-work gravels; k (open gravels) = 2.5 cm/s; average $k = 1$ cm/s	10 (3.28) central core	88.5 (27.8)	Preconstruction test trench, pump out and lab. piping tests. Grouting beneath trench. Construction in 1959–62. Max. depth of cutoff 190 ft (58 m)
Mangla Closure Dam, Mangla Dam Project, Jhelum River, West Pakistan. Owner: West Pakistan Water & Power Development Authority	Sandy gravel with cobbles and boulders; gap graded in range of fine gravel and coarse sand; $k = 0.4$ cm/s	10 (3.28) central core	230 (72.4) construction condition only	Constructed in 1964; max. depth 22 ft (7 m)
Duncan Lake Dam, Duncan River, British Columbia, Canada. Owner: British Columbia Hydro and Power Authority	Surface zone of sands and gravels overzone of silt to fine silty sand with some silty clay; k (surface zone) = 1 cm/s	10 (3.28) upstream berm	102 (32 m) short term	Constructed in 1965–66; max. depth 60 ft (18 m)
West Point Dam, Chattahoochee River States of Georgia and Alabama, U.S.A. Owner: Corps of Engineers	Upper stratum of alluvial soil, alternating layers of clay, silt, sand, sand and gravel; k varies from 1.8×10^{-2} to 3.5×10^{-5} cm/s. Lower stratum of residual soil, brown silty sand; $k = 0.6 \times 10^{-5}$ cm/s	5 (1.64) upstream blanket	61 ft (19.2 m)	Constructed in 1966; max. depth 60 ft (18 m). Grouting in sound rock below the trench; max. depth of cutoff 100 ft (30 m)

TABLE 3.6 Slurry Trench Cutoffs *(Continued)*

Project	Foundation material	Trench width, ft (m)	Maximum head, ft (m)	Remarks
Saylorville Dam, Des Moines River, Iowa, U.S.A. Owner: Corps of Engineers	Surface zone of impervious alluvial sandy clay. Pervious zone, medium to fine sand and gravelly coarse to fine sand; average k (gravelly sand) = 0.15 cm/s	8 (2.52) upstream berm	93 (29.2) short term	Constructed in 1976. Dam cutoff max. depth approx. 60 ft (18 m)
Brokopondo Project (Quarry A Cofferdam) Suriname River, Suriname, SA. Owner: Suriname Aluminum Company.	Uniform fine to coarse sand with some gravel; D_{10} = 0.1 mm	4 (1.26)	40 (12.6)	Constructed in 1959; max. depth 15 ft. (4.9 m).
Wells Dam, Columbia River, Washington State, U.S.A. Owner: Public Utility District No. 1 of Douglas County	Pervious gravels	8 (2.52) central core	70 (21.3)	Constructed in 1964; max. depth greater than 80 ft (24 m)
Yards Creek Lower Reservoir, New Jersey, U.S.A. Owner: Public Serv. Elect. & Gas Co. Jersey Central Power & Light, New Jersey Power & Light Company	Sands gravels, cobbles, and boulders	8 (2.52) central core	55 (16.8)	Constructed in 1964; max. depth 40 ft (12 m)
Camanche Dam-Dike 2, Mokelumne River, California, U.S.A. Owner: East Bay Municipal Utility District	Alluvial deposit with upper stratum of clayey silts, silts and clayey sands, and a lower stratum of poorly graded medium to fine sand over a thin zone of well-graded gravel; average $k = 7.5 \times 10^{-6}$ cm/s	8 (2.52)	135 (41)	Constructed in 1966; max. depth 95 ft; max. head on dam 45 ft (13.7 m); head on trench depends on groundwater level downstream

Source: From Jones [5].

(for runup on riprapped slopes), plus a safety factor. The safety factor generally varies between 2.5 and 9 ft (0.8 and 3 m). An alternative to providing extra freeboard is to construct a parapet wall (wave wall) along the crest of the embankment.

Numerous references to studies and technical papers describing experiences with the overtopping of earth and rockfill dams are presented in the bibliography.

Cores. The impervious barrier in some embankment dams is located within the embankment itself as opposed to a membrane on the upstream face of the dam. In the former case, the barrier is called a "core." The core of an embankment dam may be constructed with natural materials (pure clay, or silty, sandy, or gravelly clay), or synthetic products, such as asphalt, concrete, plastic, or other materials, alone or in combination.

The thickness of the core depends on the availability of impervious materials, on the permissible hydraulic gradients across (taking into consideration the nature of the foundation), or on scheduling factors. The core may be centrally located within the embankment or inclined and closer to the upstream face of the dam.

Thin cores may be mandatory when the supply of impervious material is limited. However, in some cases, even when there is ample supply of core material, it is more economical to design a dam with a thin core. This happens when the unit cost of placing granular soils is cheaper, because the overall embankment volume can be reduced, or because weather does not permit the compaction of a large volume of core.

Minimum safe core thicknesses are often selected on the basis of judgment, aided by experience reflected in the following criteria [2]:

1. Cores with a width of 30 to 50 percent of the water head have proved satisfactory on many dams under diverse conditions. Probably a core of this width is adequate for any soil type and dam height.
2. Cores with a width of 15 to 20 percent of the water head are considered thin, but if adequately designed and constructed filter layers are used, they are satisfactory under most circumstances.
3. Cores with widths of much less than 10 percent of the water head have not been used widely and probably should be considered only in circumstances where a large leak through the core would not lead to failure of the dam.

The maximum hydraulic head divided by the base width of the core in an embankment dam is called the "core hydraulic gradient." For embankment dams with pervious shells, the hydraulic gradient, as defined above, is approximately equal to the hydraulic gradient across the entire structure. Table 3.7 provides examples of typical hydraulic gradients in and under the core of some dams. Note that even if a thin core satisfies the gradient considerations, it is essential to protect such a core with well-designed and conservative filters on each side, especially in areas where earthquakes are a concern.

Vertical versus Sloping Cores. One advantage of the vertical core is that higher pressures will exist on the contact between the core and the foundation, thus providing more protection against the possibility of leakage along the contact.

TABLE 3.7 Hydraulic Gradients in and under Cores

Dam, country	Core of soil	Height of dam, m*	Approx. hydraulic gradient
	Vertical		
Aswan, Egypt		110	2
Blowering, Australia		112	1
Blue Mesa, U.S.A.		91	1
Copeton, Australia		109	1.2
Dartmouth, Australia (Vic.)		183	1.4
Furnas, Brazil		127	3
Gepatsch, Germany		158	4
Kajakai, Afghanistan		100	2
Llyn Brianne, U.K.		90	3
New Don Pedro, U.S.A.		180	2
Nurek, U.S.S.R.		300	2
Ord River, Australia		98	2.5
Sayansk, U.S.S.R.		225	5
Talbingo, Australia	Off-vertical	162	1.3
	Inclined		
Chivor, Colombia		238	3
Cougar, U.S.A.		158	4
Furnas, Brazil		127	2
Holjes, Norway		81	2
Infiernillo, Mexico		148	5
Jindabyne, Australia		72	3
Kenney, U.S.A.		100	3
Miboro, Japan		126	1.2
Mica, Canada		235	3
Mont Cenis, France		81	2
Nantahala, U.S.A.		80	9

*1 ft = 0.3 m
Source: From Ref. 6.

On the other hand, an upstream-sloping core has the advantage that the main downstream portion of the dam and the foundation grouting can be constructed first, and the core placed later. However, a disadvantage of upstream-sloping cores is that the location of the area of contact between the core and the foundation depends on the amount of required foundation excavation, and sometimes this cannot be predicted in advance.

Filters and Transition Zones. The core in an embankment dam is flanked on each side by drains (filters) or by shells that are constructed with larger-sized materials. Seepage forces that occur during normal reservoir operation (steady seepage), drawdown episodes (on the upstream side of the dam), or embankment cracking (due to hydraulic fracturing, differential settlements, earthquakes, or other causes) would cause migration of the finer soil particles from the core to zones with coarser materials, and eventually out of the dam. This could, in turn, cause piping, which may cause severe damage or even catastrophic erosion of the structure. Therefore, it is necessary that filters be designed so as to make soil particle migration impossible. In some cases, core and filter gradations are compatible with each other; in other cases, when the difference in the gradations is quite large, it is required that transition layers be provided between the core and the shells or between the filters and the shell, as discussed below.

Filter Design. A good filter must fulfill two requirements: it must be more pervious than the soil it is protecting, so that it indeed acts as a free-draining zone, and it must have an adequate gradation to prevent soil particles from the protected soil being carried away by seepage flows. Reference 2 lists a number of rules that were widely used in 1963 in the selection of filter materials:

1. The 15% size of the filter (i.e., the particle size which within the gradation band is coarser than the finest 15% of the soil, D_{15}) should be at least five times as large as the d_{15} size of the soil being protected by the filter.
2. The coarsest D_{15} size within the gradation band of the filter should not be larger than five times the finest d_{85} size within the gradation band of the protected soil.
3. The gradation curve of the filter should have roughly the same shape as the gradation curve of the protected soil.
4. Where the protected soil contains a large percentage of gravels, the filter should be designed on the basis of the gradation curve of the portion of the material which is finer than the 1-in sieve.
5. Filters should not contain more than about 5% of fines passing the No. 200 sieve, and the fines should be cohesionless.

More recently (1981–82), an investigation on relatively uniformly graded filters having D_{15} of about 1.0–10.0 mm was carried out by the U.S. Department of Agriculture, directed toward improving the understanding of sand and gravel filters and filters needed to protect clays and silts commonly used as cores in embankment dams. The results of such investigations were reported in [7] and [8], from which the following conclusions are summarized:

Sand and Gravel Filters

In uniform filters, or well graded filters with coefficient of uniformity at (defined as the ratio between the 60 percent size, D_{60}, and the 10 percent size, D_{10}) least up to 10, the size of the pore channels that govern permeability are determined by the D_{15} size (i.e., the diameter of the 15 percent passing). Soil particles smaller than about $0.10D_{15}$ which are carried in water suspension will generally pass through the channels. Particles larger than $0.12D_{15}$ will be retained.

The coefficient of permeability of dense filters is generally in the range between $k = 0.2D^2_{15}$ and $0.6D^2_{15}$, with an average of about $k = 0.35D^2_{15}$, where k is in centimeters per second and D_{15} is in millimeters.

The oldest and most widely used of the existing filter criteria, $D_{15}/d_{85} \leq 5$, is shown to be conservative. A main conclusion of this study was that, for filters with D_{15} larger than about 1.0 mm, the ratio $D_{15}/d_{85} \leq 5$ should be continued as the main criterion for judging filter acceptability. For finer filters used for protecting silts and clays, different criteria can be used, as discussed below.

Filter criteria that limit the D_{50}/d_{50} and D_{15}/d_{15} ratios are not founded on a sound theoretical or experimental basis and should be abandoned.

It is not necessary that the particle size distribution curve of the filter be generally similar in shape to that of the base soil.

Angular particles of crushed rock are as satisfactory as rounded alluvial particles for filters and can be designed using the same criteria. The same criteria can be used for filters which may be subjected to vibrations.

Filters for Silts and Clays. Filters placed downstream from the core of a dam should be able to control and seal concentrated leaks through the core. Such concentrated leaks develop from several causes including (1) development of open cracks from differential settlement, (2) flow at contacts with rock foundations or concrete structures, (3) construction deficiencies, (4) hydraulic fracturing, and (5) geologic fault displacement.

Design criteria for this type of filter may be summarized as follows [8]:

> For fine-grained clays a sand filter with D_{15} of 0.5 mm is conservative. For sandy clays and silts the filter criterion $D_{15}/d_{85} \leq 5$ is conservative and reasonable. The Atterberg limits of a clay have no significant influence on the needed filter. For nondispersive and dispersive clays having similar particle size distribution the needed filters are the same. For filters upstream of a clay core, quantitative filter criteria are not necessary.

Downstream Drains. Figures 3.12*b* to *d* and 3.14*a* to *c* show several embankment sections with various types of drains within the downstream shell. The drains are placed for two reasons: to reduce the pore water pressure in the downstream half of the dam, hence increasing its stability against sliding, and to control seepage at the toe of the dam. Since earth dams tend to be stratified, the horizontal permeability tends to be greater than the vertical, and a horizontal drain such as that in Fig. 3.12*b* may not be effective. To prevent trouble due to horizontal stratification, the chimney drain design (Fig. 3.12*d*) was implemented.

Chimney drains may be vertical or inclined at considerable slopes, either up- or downstream. The dimensions and the permeability of pervious drains must be chosen in such a way that the drainage system can carry away the anticipated flow without an excessive head in the drain. Drains should have a coefficient of permeability 10 to 100 times greater than that of the main embankment material.

Dimensions. Minimum thicknesses are generally controlled by construction limitations. For horizontally placed filters, the minimum recommended thickness is 6 to 8 in (15 to 20 cm) for sand, and 12 in (30 cm) for gravel. For chimney drains, or inclined filters, 9 to 15 ft (3 to 5 m) horizontal width are recommended.

Average exterior shell slopes usually vary between 2:1 and 4:1. Much flatter slopes are required where the foundation is weak. The slopes of the outer shells of the embankments are controlled by the shear strength of the fill material itself and that of the foundation. As a guide, Table 3.8 gives recommended maximum outside slopes for small dams built of different materials and having a variety of functions.

Slope Protection. Both faces of a dam must be protected against damage by natural agents. The upstream slope is subjected to forces due to water waves, ice, rain, wind, and the impact of floating objects. Protection of the upstream slope, which must extend above and below normal operating reservoir levels, may consist of riprap, precast concrete forms, soil cement, concrete slabs, or plastic or asphaltic waterproofing membranes. Riprap protection is perhaps the most widely used system of defense for the upstream slope. This is particularly so when rock is available at reasonable distances from the dam site. Riprap protection is amenable to a rational design based on reservoir wave characteristics, and it will be discussed in detail below.

A deficiency of soil-cement, concrete, or asphaltic slabs and plastic, which are alternative methods of protection of the upstream slope, is that these materials are subject to cracking and deterioration due to atmospheric agents and embankment settlement. Variations in pore water pressure in the soil behind a nearly im-

TABLE 3.8 Recommended Slopes for Small Earthfill Dams on Stable Foundations

A. Zoned earthfill dams

Type	Purpose	Subject to rapid drawdown*	Shell material classification	Core material classification†	Upstream slope	Down-stream slope
Zoned with "minimum core"‡	Any	Not critical	Rockfill, GW, GP, SW (gravelly), or SP (gravelly).	GC, GM, SC, SM, CL, ML, CH, or MH	2:1	2:1
Zoned with "maximum core"‡	Detention or storage	No	Rockfill, GW, GP, SW (gravelly), or SP (gravelly)	GC, GM SC, SM CL, ML CH, MH	2:1 2¼:1 2½:1 3:1	2:1 2¼:1 2½:1 3:1
Zoned with "maximum core"‡	Storage	Yes	Rockfill, GW, GP, SW (gravelly), or SP (gravelly)	GC, GM SC, SM CL, ML CH, MH	2½:1 2½:1 3:1 3½:1	2:1 2¼:1 2½:1 3:1

B. Homogeneous earthfill dams

Type	Purpose	Subject to rapid drawdown*	Soil classification†	Upstream slope	Downstream slope
Homogeneous or modified-homogeneous	Detention or storage	No	GW, GP, SW, SP GC, GM, SC, SM CL, ML CH, MH	‡ 2½:1 3:1 2½:1	‡ 2:1 2½:1 2½:1
Modified-homogeneous	Storage	Yes	GW, GP, SW, SP GC, GM, SC, SM CL, ML CH, MH	‡ 3:1 3½:1 4:1	‡ 2:1 2½:1 2½:1

*Drawdown rates of 6 in (150mm) or more per day following prolonged storage at high reservoir levels.

†See Table 3.3 for description of soil types. OL and OH soils are not recommended from major portions of homogeneous earthfill dams. Pt soils are suitable.

‡Pervious, not suitable.

Source: From U.S. Department of the Interior [3].

pervious protective face due to reservoir fluctuations may cause premature failure.

The downstream slope of a dam is usually provided with a well-designed surface-water collection system which minimizes erosion of the slope, particularly at the intersection of the dam and abutments, and is seeded with vegetation appropriate for the climate where the structure is located.

Wave Action. Erosion due to wave action can be a threat to the integrity of the upstream slope of a dam and even to the whole structure. To protect against wave damage, a layer of riprap may be used. The size, gradation, and thickness of the riprap layer depend upon the wave height which can be expected to develop on the reservoir. Figure 3.16 defines terms related to wave action on the slope of a dam.

Waves are generated in inland reservoirs by sustained winds. The distance the

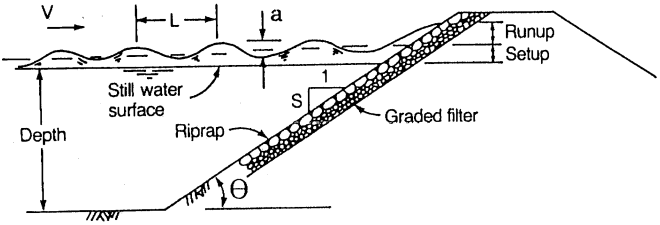
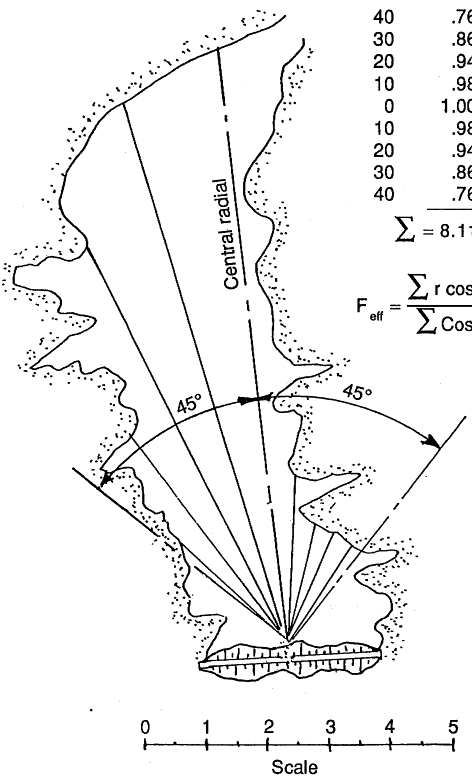


FIGURE 3.16 Schematic illustration of wave height and runup.

wind passes over water is called the "fetch." Design fetch for a reservoir is determined by the reservoir configuration, as well as the maximum overwater distance. Figure 3.17 shows a reservoir configuration, as well as the maximum overwater distance. It also shows the procedure by which effective fetch is calculated. The central ray in Fig. 3.17 is the maximum overwater distance between the dam and the far end of the reservoir. Supplementary rays are drawn as



α	Cos	r	$r\cos^2\alpha$
40	.766	1.95	1.14
30	.866	2.00	1.50
20	.940	2.00	1.77
10	.985	2.75	2.67
0	1.000	9.75	9.75
10	.985	9.20	8.93
20	.940	7.25	6.41
30	.866	4.25	3.19
40	.766	2.40	1.41
	$\sum = 8.114$		$\sum = 36.77$

$$F_{eff} = \frac{\sum r\cos^2\alpha}{\sum \operatorname{Cos}\alpha} = \frac{36.77}{8.114} = 4.53 \text{ km}$$

FIGURE 3.17 Example of calculating effective fetch.

shown, and the effective fetch is then calculated in accordance with the tabulated values.

Wind speed to be used in wave height determinations must be chosen judiciously. Wind speed in the direction of maximum fetch is required. Data on wind speed are seldom available in sufficient quantity or quality to enable the development of a wind rose which would yield wind speed, direction, and duration. If possible, 100-year and maximum wind speeds should be estimated from the nearest available records.

Wind-generated waves on an inland reservoir are not of uniform height. Instead, individual heights are part of a wave-height frequency spectrum. If the heights of 33⅓ percent of the spectrum of waves are averaged, that average height is called the "significant wave height," a_s. Only 0.4 percent of all waves will have a height exceeding 1.67 times the significant wave height. Such a wave is referred to as the "maximum wave."

The maximum wave height ($1.67a_s$) is sometimes used as the design wave for planning freeboard and riprap on large dams. However, for small dams, the significant wave height should be satisfactory. Figure 3.18 can be used to estimate the height of the significant wave once the design wind speed has been selected and the effective fetch has been calculated. Figure 3.18 also shows the minimum duration of the design wind speed required to produce the corresponding significant wave height [29].

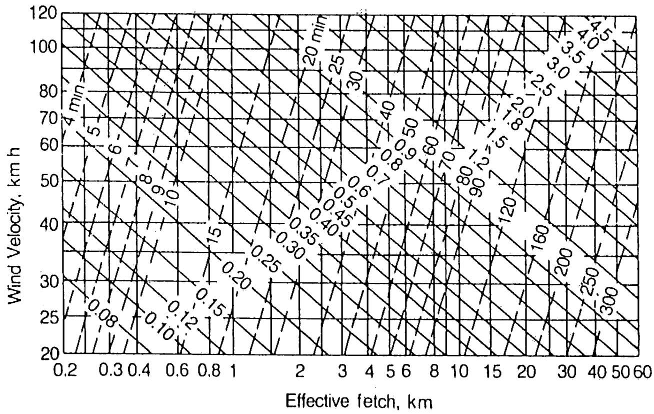


FIGURE 3.18 Significant wave height as a function of fetch-sustained wind velocity (dashed lines indicate required wind duration in minutes).

The height to which a wave will run up the face of the dam depends upon the steepness of the wave (the wave height a divided by the wave length L), the roughness of the riprap surface, and the slope of the embankment. Rougher riprap achieves greater energy dissipation and a lesser runup. Wave lengths for wind-generated waves can be computed from

$$L = 5.10T^2 \text{ (feet)}$$

$$L = 1.56T^2 \text{ (meters)}$$

where T is the wave period in seconds and L is the wavelength. Figure 3.19 can be used to obtain the wave period using the design wind speed and the calculated effective fetch.

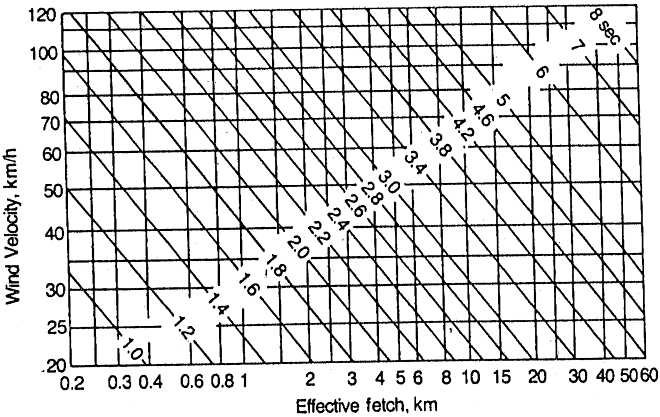


FIGURE 3.19 Period of wind-generated waves as a function of sustained wind velocity and fetch. *(From U.S. Army Corps of Engineers [2].)*

When a deep-water wave approaches the face of the dam, its height increases because of the decreasing depth. When the depth becomes less than $0.78a$, the wave will break. For an earthfill dam, the face is steep enough that breaking of the wave effectively occurs on the face of the dam, and new smaller waves do not have time to form downwind from the breaking wave. Only runup of the fully developed wave needs to be considered. Relative runup R/a can be determined through use of Fig. 3.20 for both smooth and rough dam faces as a function of wave steepness a/L and the slope of the dam. Figure 3.20 shows that runup for a wave of given height is smaller for flatter slopes. Because the wave travels farther on a flatter slope, energy dissipation reduces the wave's potential to runup.

Winds blowing over an inland reservoir produce a shear over the water surface, which in turn causes the reservoir surface to slope upward in the direction the wind is blowing. The rise in water surface produced at the dam, which is called "setup," is shown in Fig. 3.16. For very large reservoirs, setup should be calculated and used in freeboard calculations. However, for small reservoirs, setup will be insignificant and need not be considered. Freeboard above the maximum reservoir water surface should be provided at least equal to the runup distance R. Allowance must also be provided for settlement of the dam. As previously mentioned, for dams constructed on relatively incompressible foundations, a camber should be provided at least equal to 1 percent of the dam height. For dams on compressible foundations, 3 ft (1 m) or more may be required.

Riprap Design. Durable rock is desirable for use as riprap to protect the face of the dam from erosion by waves. Historical studies show that failure of riprap is very common on earthfill dams [29, 30]. Riprap normally fails by being displaced by wave action, in turn allowing erosion of the embankment. Riprap large enough and heavy enough to resist movement by wave action is necessary. A distribution of sizes is also required to prevent piping of the embankment material

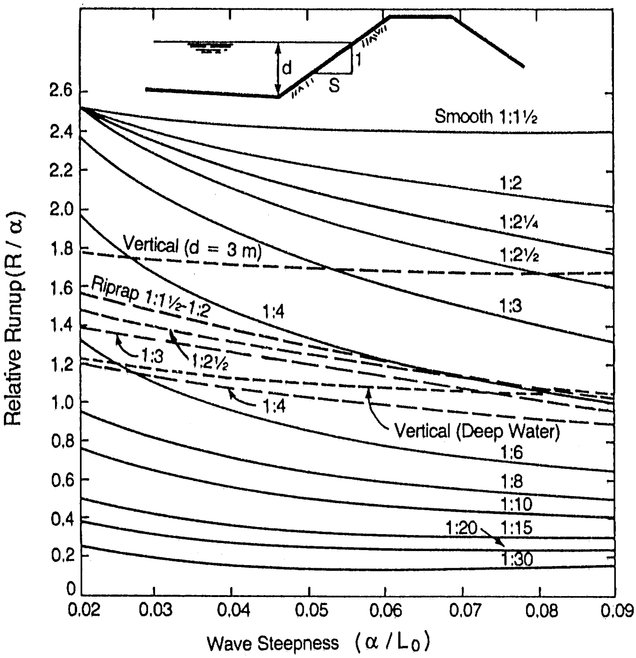


FIGURE 3.20 Relative runup of wind-generated waves striking the face of a dam. *(From U.S. Army Corps of Engineers [29].)*

through the interstices of the rocks in the riprap layer. If soft stone such as sandstone is used, waves and weather will readily break and erode the riprap to the point where it is no longer effective in damping wave action. Granite, basalt, and sound limestone are examples of rock which will be effective as riprap. Some maintenance should be expected for riprap, and it should be inspected at least annually and after any severe storm.

Weight of the required median riprap rock is calculated in terms of a stability number N_s with the equation

$$W_{50} = W_R\left[\frac{a}{N_s}(G - 1.0)\right]^3$$

where G is the specific gravity of the stone, W_R is the unit weight of the stone being used, and a is the design wave height.

Figure 3.21 can be used to select a proper stability number in terms of wave steepness a/L and the tangent of the embankment slope angle (see Fig. 3.16). A range of values of N_s can be chosen from Fig. 3.21. The upper curve in each case will result in a stone size for which the riprap will suffer a "tolerable" damage due to wave action. Tolerable damage may involve some displacement of rock during a severe storm, but no embankment erosion.

The stone diameter can be calculated from the stone weight as

$$D\ (\text{cm}) = 114.9\left(\frac{W}{W_R}\right)^{1/3}$$

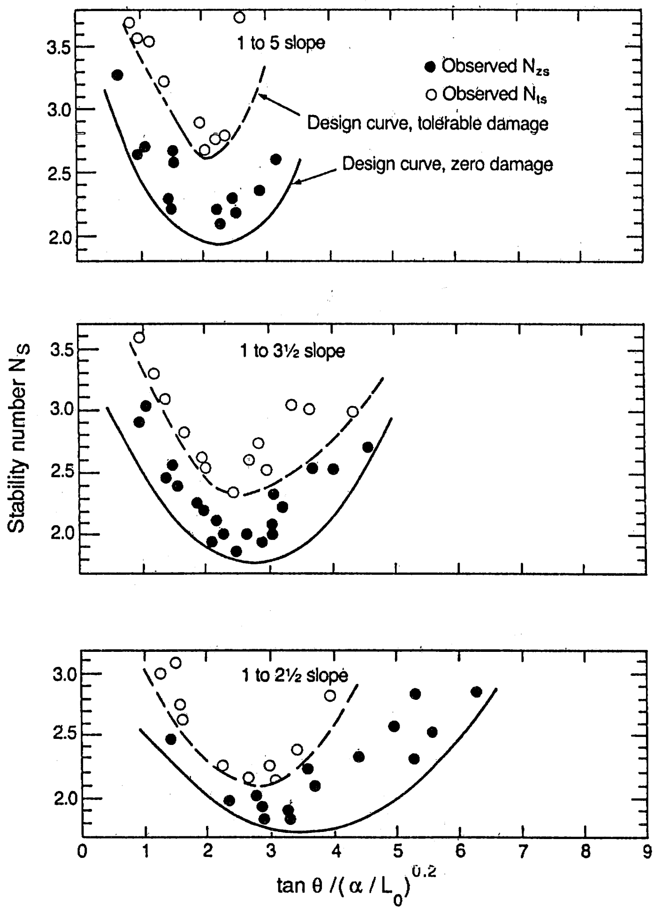


FIGURE 3.21 Riprap design stability numbers. *(From Ahrens [30].)*

$$D\text{ (in)} = 13.8\left(\frac{W}{W_R}\right)^{1/3}$$

where D is in centimeters (inches), W is in metric tons (tons), and W_R is in metric tons per cubic meter (ton per cubic foot).

Thickness of the riprap layer should be 1.5 to 2.0 times the size of the median stone. For proper riprap gradation, the weights of the maximum stone and minimum stone should be 3.6 and 0.2 times the weight of the median stone, respectively..

Normally, riprap will be obtained from a quarry where the maximum stone size is available, probably by blasting. The quarried material is run over a screen to segregate the large sizes. The finer material, which falls through the bar screen, is first placed on the face of the dam by dumping. The coarser stone is then placed on top of the finer material. This procedure forms a natural filter between the riprap and the embankment.

Stability Analysis. The safety of various embankment cross sections under various loading conditions (gravity, seepage, seismic) must be properly analyzed to provide an adequate degree of safety.

Stability analyses are usually done utilizing the principle of limiting equilibrium, which assumes that the embankment materials develop their maximum shear strength simultaneously along the entire potential rupture surface being considered. The results of stability analyses are usually expressed in terms of a factor of safety.

Methods of Analyses. Various procedures are now available to carry out the stability calculation, and most of these have conveniently been adopted for use with computers. The basic methods include circular, wedge, spiral, or any combination of these potential failure surfaces. In any of the methods of analyses, the computations may be made using the principles of either effective or total stresses.

Conditions Analyzed. Since various conditions of seepage and loading exist during the life of an embankment, it is necessary to study its stability under a variety of material strength and loading demand conditions. Critical loading conditions usually analyzed include:

- End of construction
- Steady seepage, at full reservoir
- Rapid drawdown stage
- Earthquake

Soil strength parameters must be appropriate for the type of loading and analysis to be performed.

Analyses in terms of total stresses normally utilize unconsolidated-undrained shear strengths S_u measured in triaxial, direct shear, or unconfined compression tests, in situ vane tests, or in other appropriate techniques.

Analyses in terms of effective stresses utilize a cohesion intercept c' and an angle of shearing resistance ϕ' obtained from a Mohr strength envelope of effective stresses that is determined in drained triaxial or direct shear tests, or in undrained triaxial tests with pore pressure measurements.

The correct selection of soil strength parameters is the most important element in the slope stability analysis, and the results presented by a geotechnical laboratory must be carefully examined to verify their reliability and to make certain that they are indeed representative of the materials used. For undrained strength, the testing samples should be undisturbed and the moisture content during testing should be compared to the in situ moisture content and measured independently to check that the strength has not increased because of drying. For effective strength tests, the rate of strain should be sufficiently slow to permit dissipation of pore pressures in drained tests and to permit equalization of pore pressures in undrained tests with pore pressure measurement.

In selecting strength parameters, differences in stress-strain characteristics of different soils through which the failure surface passes should be considered. For example, in an embankment on soft clay, a well-compacted fill may reach peak strength at low strain and then decrease at greater strains, while the foundation may reach peak strength at a large strain. One should determine the strength of the embankment at a strain comparable to that at which the peak strength is developed in the foundation. This should also be applied to different soil zones within an embankment of foundation.

Acceptable Factors of Safety. Different regulatory agencies prescribe different "minimum" acceptable factors of safety for the embankment dams under their jurisdiction. This is because of the large number of varying circumstances that may affect the result of the calculations. As stated in Ref. 4,

> The values of the factor of safety differ according to the shear strength selected for use in the analysis, in the way in which the pore water pressures are taken into account, and other factors influencing the method used for computing it. Every method of analysis currently in use is based on highly significant simplifying assumptions which are frequently forgotten in the discussion and the use of the results. For the most part, all current methods are semi-empirical and the justification for their use is based on experience with the performance of embankments. Thus the safety factor is actually an "experience factor" which enables a designer to compare one project with another. Frequently, the numerical value of the safety factor which a designer requires varies with the degree of his confidence in his knowledge of the shear strength of the materials and of site data, such as the influence of minor geologic details.

Typical requirements for the minimum factor of safety under static loading conditions are presented in Table 3.9

TABLE 3.9 Typical Minimum Safety Factors

Loading condition	Minimum factor of safety	Slope to be analyzed
End of construction condition	1.3	Upstream and downstream
Sudden drawdown from maximum pool	>1.0*	Upstream
Sudden drawdown from spillway crest or top of gates	1.2*	Upstream
Steady seepage with maximum storage pool	1.5	Downstream
Steady seepage with surcharge pool	1.4	Downstream

*The safety factor should not be less than 1.5 when drawdown rate and pore water pressures developed from flow nets are used in the stability analyses and where rapid drawdown is a normal operating condition as with a pumped storage reservoir.

Seismic Safety. The overall stability of an embankment subjected to earthquake ground motions has been the subject of intensive investigation in recent years, following the near failure of a dam in California [9]. A summary of the applicability of available analysis techniques to embankments of various soil types is provided in [9] as follows:

> Much progress has been made in the past 10 years in developing an improved understanding of the seismic behavior of earth and rockfill dams. The pseudo-

> static method of analysis of slope stability has been clarified and Terzaghi's foresight in warning of its potential limitations and the soil types where it is likely to provide useful results has to a large extent been validated. There does seem to be a reasonable basis to believe that the method can be extremely useful in evaluating the performance of embankments constructed of soils which do not lose significant strength during earthquakes (clays and clayey soils, dry or moist cohesionless soils or extremely dense cohesionless soils) and this is evidenced both by analyses based on Newmark's displacement-type analyses and field performance.
>
> There is also a significant body of evidence to show that this approach does not have the capability to predict potential failures in embankments constructed of loose to medium dense cohesionless soils and that more sophisticated methods of analysis, which give consideration to the pore pressure build-up during earthquake shaking, the redistribution of pore pressure with time and the potential for strength loss at damaging strain levels need to be adopted and improved for dealing with these types of problems. Highly permeable soils may be considered to represent a special case since pore water pressures induced by earthquake shaking in many of these soils may be shown to dissipate almost as rapidly as they are produced. On the other hand, in fine grained cohesionless oils, there is considerable evidence to show that the critical stability condition may not always develop during the earthquake shaking but may, in fact, be attained some minutes or hours after the shaking has stopped. Seismic analyses must therefore give consideration to post-seismic effects in these materials.

Therefore, in assessing the seismic stability of an embankment dam, an important soil behavior difference must be recognized before a decision on the types of analyses can be made.

1. *Soils that do not lose more than 15 percent of their initial strength due to earthquake shaking and have small associated displacements or build-up of pore pressures:* In this case, the pseudo-static analysis procedure provides an acceptable method, and criteria for acceptance falls more on the acceptable deformation of the dam, as given by Table 3.10. In this case:

   "The critical decision to be made by the design engineer is simply whether the soil is likely to be vulnerable to excessive strength loss or pore pressure development or not. This can be determined by tests. However, both field and laboratory experience indicates that clayey soils, dry sands and in some cases dense saturated sands will not lose substantial resistance to deformation as a result of earthquake or simulated earthquake loading and thus pseudo-static analyses will generally provide an acceptable method of ensuring adequate performance for embankments constructed of these types of soil. In cases of doubt, however, a careful laboratory study will invariably provide the information from which an appropriate engineering decision concerning the applicability of the method can be made. It should also be noted that even some soils which might be vulnerable to the development of large pore pressures and some strength loss under conditions of strong shaking may show little evidence of these effects under less intense shaking, in which case the pseudo-static principles would still be applicable." [9]
2. *Soils that lose considerable strength during earthquake shaking and develop high excess pore water pressures as a result:* For these cases, a complete dynamic analysis of the embankment is required, which is done in a series of steps as summarized below [9]:
   a. Determine the cross sections of the dam to be used in the analysis.
   b. Assess the dynamic properties of the materials comprising the dam, such

TABLE 3.10 Probable Upper-Bound Displacements for Embankment Dams Subjected to Earthquakes

	Crest acceleration	k_m	Factor of safety (FS) = 1.15 for k = 0.05 15% strength loss k_y = 0.05	FS = 1.15 for k = 0.1 15% strength loss k_y = 0.10	FS = 1.15 for k = 0.1 No strength loss k_y = 0.15
Magnitude 6½ earthquakes (little or no strength loss)					
Probable upper bound of accelerations for most earthy dams	1.0g	≈0.4	≈4.0 ft	≈1.8 ft	≈1.0 ft
	0.75g	≈0.3	≈2.7 ft	≈1.2 ft	≈6.0 in
	0.50g	≈0.2	≈1.7 ft	≈6.0 in	≈1.0 in
	0.25g	≈0.1	≈6.0 in	0	0

	Crest acceleration	k_m	Factor of safety (FS) = 1.15 for k = 0.1 15% strength loss k_y = 0.10	FS = 1.15 for k = 0.15 15% strength loss k_y = 0.15	FS = 1.15 for k = 0.15 No strength loss k_y = 0.20
Magnitude 8¼ earthquakes (little or no strength loss)					
Probable upper bound of accelerations for most earth dams	1.00g	≈0.4	≈17 ft	≈7 ft	≈3 ft*
	0.75g	≈0.3	≈10 ft	≈3 ft	≈8 in*
	0.50g	≈0.1	≈3 ft*	≈4 in*	0*
	0.25g	≈0.1	0*	0*	0*

*Acceptable performance.
Source: From Seed [9].

as shear modulus and damping characteristics. Because these material characteristics are nonlinear, it is also necessary to assess how these properties vary with strain amplitude.

c. Estimate, in conjunction with geologists and seismologists, the earthquake ground motions, and their characteristics, to which the dam and its foundation might be subjected.

d. Estimate the stresses existing in the embankment before the earthquake(s).

e. Estimate, using an appropriate dynamic finite-element analysis procedure, the dynamic shear stresses induced in the embankment by the selected base excitations.

f. Assess the resistance of the soils comprising the embankment and its foundation to the effects of cyclic loading of the type induced by earthquake shaking; this resistance should include the resistance to pore-pressure generation and the resistance to the development of strains.

g. On the basis of the evaluations of the cyclic loading resistance of the embankment materials and of the dynamic stresses obtained in *e* above, estimate the potential for excess pore-water pressure generation and redistribution within the embankment and its foundation.

h. From the knowledge of the pore pressures generated by the base motions and of the stress-strain and strength characteristics of the materials, and based on judgment and experience, evaluate the stability and potential deformations of the embankment during or after the earthquake(s).

Conduits under Reservoirs. Locating pipes and culverts within the embankment or through its foundation (if this is not rock) should be discouraged. If this is completely necessary, then procedures to estimate loads and deformations of pipes under earthbanks are available (see Bibliography), and the design should be made for the maximum loads calculated for the worst loading conditions.

Two problems must be considered: potential cracking of the conduits, which would cause leaks and potential erosion of embankment core soils, and the possibility that water may be able to travel along the outer surface of the conduit and possibly produce piping of the embankment core material. To prevent seepage along the pipe, encircling cutoff collars were often provided as a matter of standard practice. However, since it is difficult to achieve adequate compaction around these collars and the conduit, the current practice is to eliminate the collars, and instead to slope the exterior concrete–earth core interface so that the core material can be compacted against the concrete. In addition, the downstream filter is wrapped around the downstream length of the concrete conduit.

Gravel and Rockfill Dams

Rockfill dams are defined as embankment dams for which at least half of the material in the maximum cross section is composed of rock. It follows from the definition that if the shells of the earth embankment dams discussed so far in the chapter are either rock or gravel and their volume is at least half of the section, then the resulting structure would be a rockfill (or gravelfill) dam. The basic principles so far developed in connection with earthdams also apply to the rock- (gravel-) fill dams.

Rockfill dams may have impervious-faced membranes, sloping earth cores, thin central cores, or thick central cores. Due to the similarities between the rockfill and earthfill dams with impervious earth cores, only a few additional comments regarding their design and features are required here. The elements and principles of the concrete-face concrete dams are different from the principles for earth- (rock-) fill dams. Because of these differences, the emphasis in this section of the chapter will be on the concrete-face dams.

Gravel- (Rock-) fill Dams with Impervious Cores. Typical cross sections of rockfill dams were presented in Figs. 3.3 and 3.4. As with the earth dams, the basic element is the impervious core, which has the protection provided by transition (and filters), and the shell zones. Figure 3.22 presents a schematic cross section view of a rockfill dam.

The impervious core is designed and constructed as for an earthfill dam. However, a filter section is required on both the upstream and downstream side of the

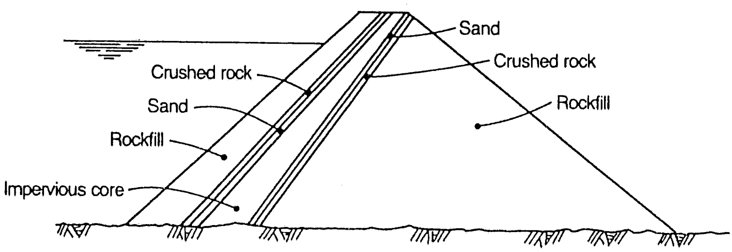


FIGURE 3.22 Rockfill dam with an impervious core (schematic cross section).

core. These filters require special transition zones since the rockfill will have much larger particle sizes than an earthfill section. The downstream filter protects against piping due to seepage as well as erosion from rainfall percolation. To provide adequate protection against piping, the transition and the filter sections should be graded outward from sand, through gravel, to rock, as shown in Fig. 3.22.

Vertical impervious cores have also been used on many dams. However, the sloping core makes construction of the various filter layers easier since the entire dam does not need to be raised simultaneously. Each layer can be placed and compacted individually, progressing in the upstream direction.

Concrete core walls have been used in the past on rockfill dams. Although most of those dams still exist, many problems have arisen including cracking and leaking core walls. The cracking is the result of stresses induced by settlement of the dam. In general, brittle materials, such as concrete, should be avoided in core walls.

Rockfill Dams with Impervious Facings. Figure 3.23 shows a typical section for a rockfill dam with an upstream impervious facing. The impervious facing is most frequently made from concrete. However, facings have been successfully constructed from plastic liners, timber, and steel. A layer of hand-placed rock or a layer of compacted small rock is constructed on the upstream side of the fill as a backing for the concrete impervious facing. This backing serves to transmit the force exerted by water in the reservoir uniformly to the rockfill.

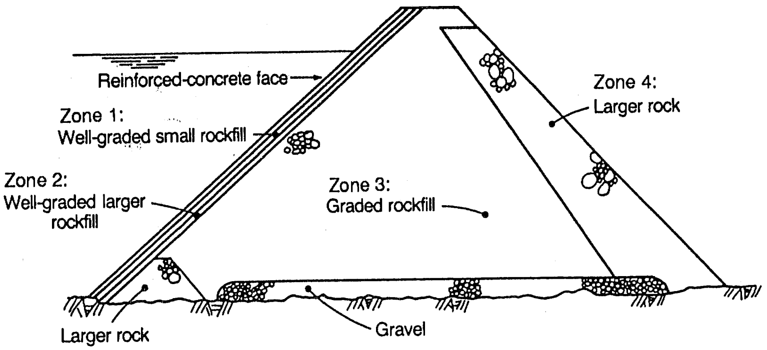


FIGURE 3.23 Typical maximum section for a rockfill dam with upstream impervious facing.

The upstream and downstream slopes of rockfill dams are quite steep when compared to earthfill dams of equal height. The slopes are constructed at approximately the angle of repose of the dumped rock, which may vary from 1.4 to 1.2 horizontal to 1.0 vertical. Careful placing of a layer of rock on the upstream side may make it possible to construct slopes which are steeper than the natural angle of repose of the rockfill.

A dam with impervious upstream facing advantageously utilizes the thrust of the water by directing the resultant force toward the foundation, thus increasing stability against sliding and helping in the closing of rock foundation fissures. In a properly designed dam, the water flow across the facing is minimal, and the section of the dam is for all practical purposes dry. Under these conditions, the absence of seepage forces allows steeper slopes than usually required in other embankment dams. The upstream facing, however, is vulnerable to damage due to the rockfill deformations especially during the first reservoir filling and subsequent to it, although at a much slower rate. In view of these considerations, the construction of modern upstream membrane-faced dams aims toward:

- Providing a support to the membrane with the smallest possible deformation
- Providing enough joints in the membrane to make it flexible enough to accommodate the rockfill deformations
- Providing for water tightness in the facing joints and along the contacts between the facing, abutments, and foundations

Elements. The basic elements of a concrete-face dam are discussed making reference to Fig. 3.23.

FOUNDATION: The foundation requirements for a rockfill dam with concrete facing are essentially the same as for a rockfill dam with an impervious core. The difference is that the special foundation preparation and treatment, which in dams with an impervious core is located under the core and the adjacent filter and transition zones (central contact area), for a rockfill dam with concrete facing is in the area of the upstream toe: a complete cleanup to sound groutable rock is usually specified within a 18- to 36-ft- (6- to 12-m) wide zone immediately downstream of the sill beam (plinth).

In the riverbed the alluvium, depending on its thickness and estimated compressibility, may or may not have to be removed. The guideline for excavation usually is that the foundation should have strengths and compressibility characteristics equal to or better than those of the rockfill.

On the abutments, overhangs are trimmed and rock surfaces smoothed to facilitate proper compaction of the rockfill. Under the sillblock, foundation preparation consists of excavation to sound, groutable rock, and construction of a grout curtain.

Sillblock (Plinth or Toe Slab). A reinforced concrete toe slab (sillblock or plinth) is provided at the upstream toe of the dam. It supplies anchoring to the main perimeter joint of the face membrane, and at the same time serves as the grout cap if a grout curtain must be constructed. Settlements of the rockfill and adjustments during the first reservoir filling tend to cause the face slab to move away from the sillblock and, as a consequence, the perimeter joint opens. The face slab and the sillblock then act independently.

To locate the sillblock during the design stage, the bedrock contours are examined and the probable elevation of the firm rock is established. The location of the sillblock is selected on this basis. If during construction firm rock is found to be locally deeper than was anticipated, the sillblock location should be locally moved upstream in the plane of the membrane in order to develop the necessary increase in depth and preserve stability for the sillblock.

The sillblock is doweled to well-cleaned rock prior to grouting. Its width varies with the quality of rock and dam height, and its thickness is at least that of the bottom section of the face slab. Typical widths vary between about 9 to 18 ft (3 to 6 m), which is much less than the width of the core and foundation contact in earth dams.

Concrete Face Slab. The trend in recent rockfill dams with concrete facing has been to use relatively thin and flexible membranes. Because the membrane is supported on a zone of relatively fine-grained material, it is fully supported and not designed for bending moments. Under the action of normal water load, the membrane is constrained to follow the strains in the rockfill in the plane of the face by development of a shear force between the membrane and the rockfill. The strain in the membrane is independent of its thickness. The membrane will be in compression over most of its extent. Consequently, watertightness and durability in the long term, and not structural strengths, are the main considerations for selecting thickness of the membrane.

The reinforcement required in the far slab is arbitrary, and its main purpose is to prevent cracking due to thermal expansion and shrinkage prior to filling the reservoir. Steel reinforcing equal to 0.5 percent of the slab area is commonly used in each direction, based on the selected thickness of concrete membrane plus an about 4-in (10-cm) allowance for unevenness in the face surface. The excess thickness depends on gradation of the supporting zone, being least for a relatively fine base. This reinforcing steel ratio has been reduced in recent dams (0.3 percent in some low dams).

Joints in the membrane are required to permit deformations to occur without rupturing the slab and to divide the membrane into suitable sizes for construction. In modern dams, excessive deformations and tendency for damage to the membrane as a result of fill settlement have been minimized by placement of the membrane only after the dam has been constructed to its full height and the entire face has been compacted. Slip-forming the entire height of the membrane permits elimination of horizontal joints, except near the foundation contact where the slip form cannot be used because of the irregular elevation along the sillblock. Triangular panels are formed in these locations and can include an additional vertical joint to better distribute any differential movements due to changes in geometry and dam height. In modern practice, based on previous performance, no horizontal joints are used in the face slab, and vertical joints are cold joints which often use water stops. In sizing the face slab, its thickness is commonly calculated as $t(\text{m}) = 0.3 + 0.003H(\text{m})$ or $t(\text{ft}) = 1 + 0.003H(\text{ft})$.

Care must be taken in the construction of the joints in face slabs to decrease the potential for leakage. Typically, the joint design includes:

- Water stops in vertical joints
- Carrying reinforcing continuous through the construction joints
- Double water stops in the perimeter joint between the face and the plinth

Typical joint designs are presented in Fig. 3.24.

Rockfill. In a modern concrete-face rockfill dam, the embankment typically consists of zones, each of which performs a different function (see Fig. 3.23). Zone 1 is the face-slab supporting base; zones 2 and 3 are the main body of the embankment designed to transfer water loads to the foundation, and zone 4 is a downstream zone consisting of oversized rock.

Zone 1 is intended to provide a dense, semipervious and erosion-resistant base for the concrete face slab. It usually consists of a well-graded sandy gravel or processed crushed rock with 2.8- to 3.2-in (7- to 8-cm) maximum size, not less than 10 percent passing No. 200 sieve, and a coefficient of permeability not greater than about 3×10^{-4} in/s (10^{-3} cm/s). Zones 2 and 3 materials consist of hard rockfill. For ease in compaction, maximum size in zone 3 is limited to 28 to 32 in (70 to 80 cm). These gradation requirements are not strict, and in fact are adjusted to local material availability.

In general, rockfill for a concrete-face rockfill dam has the same requirements and characteristics as for a rockfill dam an with an impervious core. Special

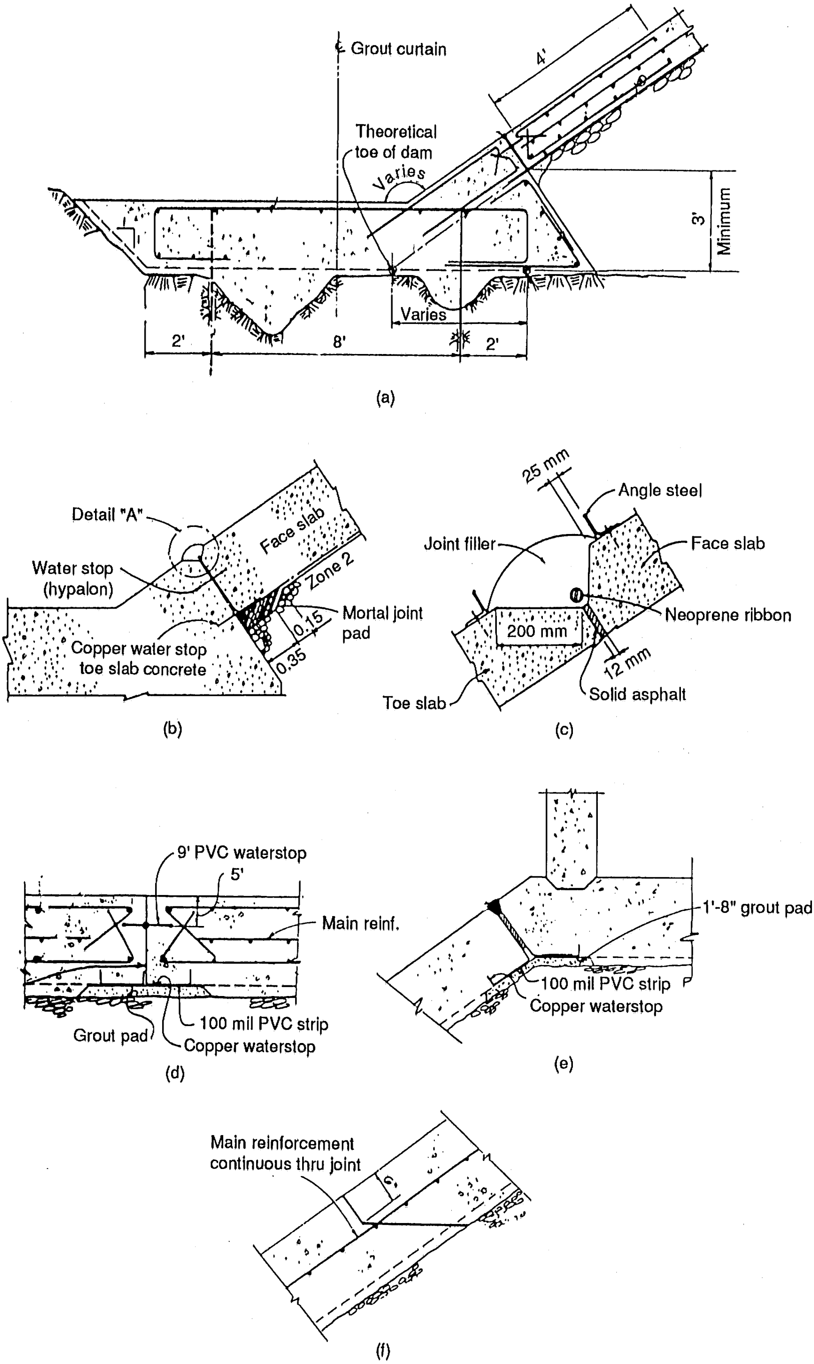


FIGURE 3.24 Typical joint designs for an upstream concrete face.

requirements are necessary only immediately behind the concrete membrane, as described below.

The width of zone 1 immediately behind the concrete face is usually 9 to 18 ft (3 to 6 m) (measured horizontally). The compaction of this zone is specified as that required to achieve a density typically at least 95 percent of the maximum in the 20,000 ft-lb/ft^3 (1 million N-m/m^3 compaction test). Zone 1 is compacted in layers. The lift thickness in areas accessible for roller compaction is as required to achieve the specified degree of compaction. In areas not accessible to a roller, where vibratory hand compactors are used, the maximum thickness of layers is typically restricted to less than 12 in (30 cm).

Because the gradation of zone 1 is fine compared to that of the rockfill, a transition zone (zone 2) is provided with a minimum width of about 12 ft (4.0 m). The transition zone may not be required in cases where the gradation of the zone supporting the concrete facing can be selected during the processing and where the rockfill is expected to be of relatively small size.

To minimize settlements and to obtain high fill strengths, high compactive effort of the rockfill (zone 3) is required. Rockfill placement criteria are based on use of heavy vibratory rollers and relatively thin uncompacted fill layers.

It is also essential that the fill immediately behind the sillbeam and its contact with the concrete membrane be compacted to a high density. In order to assure that it is as incompressible as possible, the fill is usually overbuilt above the sillbeam. This permits the roller compacting the inclined face of the fill to override the top of the sillbeam, thus ensuring proper compaction in this area. The excess fill is excavated immediately prior to placement of the concrete membrane.

A free-draining blanket extending from the sillblock to the downstream toe of the dam is often provided to assure that water pressures cannot build up in the rockfill. The gradation of the blanket is selected to create a filter with respect to the foundation. On rock foundations, the main concern may be to prevent erosion of any soft zones or loose material that may be contained in joints and fissures in the bedrock. Typical minimum thickness of the blanket is about 3 ft (1.0 m) measured on high points on rock.

Design Considerations

ROCKFILL STRENGTH: The strength characteristics of the rockfill material are usually estimated on the basis of comparisons with similar granular materials, similarly compacted, as reported in the technical literature. Comprehensive investigations (see Bibliography) of a variety of rockfill materials provide guidance on their frictional characteristics under different values of confining pressures and stress boundary conditions (triaxial or plane strain). The values presented in Table 3.11 provide guidance for the selection of the friction angle for use in design.

Near the surface of a rockfill, where pressures are very low, the angle of friction may attain values as high as 55°, whereas the angle of friction of the same material under the high pressures near the base of the embankment may be only about 40°. Residual angles of friction may be substantially lower than these values, possibly as low as 35° to 36° [11].

STABILITY (STATIC AND DYNAMIC): Under static forces, for a properly constructed rockfill with concrete facing founded on rock, the only stability concern is the potential for unraveling of the rockfill slope, which can be analyzed simply by such techniques as the infinite slope.

The overall stability of the dam to resist the water force is ample for normal rock foundation and rockfill structures. However, the stability of the sillblock un-

TABLE 3.11 Angle of Friction for Average Rockfill

Normal pressure, $lb/in^2 (N/cm^2)$	Friction angle ϕ'_t*, degrees	Friction angle ϕ'_{ps}†, degrees
2 (1.4)	53	57
5 (3.6)	50.5	54
10 (7.1)	43.5	52
20 (14.3)	46.5	50
50 (35.6)	44	47.5
100 (71.3)	42	45.5
200 (143)	39.5	43
500 (356)	37.5	41

*After Leps.
†Estimated by authors based on Leps' values.
Note: ϕ'_t = friction angle under triaxial deformation condition; ϕ'_{ps} = friction angle under plane strain conditions.
Source: From Seed et al. [11].

der the hydrostatic and gravity forces should be checked to ensure that its location and dimensions are appropriate. The sillblock must be stable under the most unfavorable combination of forces in order to prevent any rotation which could reduce the effectiveness of the consolidation grout and damage the joint with the upstream concrete facing.

Tables 3.12 and 3.13 show height and upstream and downstream slopes of concrete-faced dams built in areas of relatively low and moderate seismicity. The information is summarized in terms of earthquake parameters in Table 3.14. It should be noted that most of the data in Table 3.13 come from dams built in areas of relatively low seismicity, and in fact none of the dams have been subjected to strong shaking. In spite of this, it is a widespread belief among many engineers that the concrete-faced rockfill dam is by its own nature quite resistant to earthquake forces. This is because of the following arguments [11]:

- The water pressure acts on the upstream face, and the entire rockfill embankment mass acts to provide stability.
- There is no water inside the embankment, and there can be no buildup of pore pressures due to earthquake shaking and thus no tendency for strength reductions on this account; as a result deformations should be small.
- Modern concrete-faced rockfill dams are designed with an upstream zone of small rock and soil, thus having a permeability much smaller than that of the main rockfill embankment. Thus, even if the concrete facing were cracked during a strong earthquake causing leakage, the embankment would still be stable because the amount of water which could pass through the less pervious upstream zone is much less than the quantity that can flow safely through the main rockfill embankment.

To investigate the seismic response of concrete-faced rockfill dams in highly seismic environments, analyses were made [11] which resulted in the recommended slopes presented in Table 3.15. The values shown can be used as a guide for designing the downstream slopes. The upstream slopes could be slightly steeper since their movement is restrained by the water pressure, but generally similar slopes may be considered desirable in some cases because of smaller allowable displacements and other factors.

TABLE 3.12 Slopes of CFR Dams over 150 ft (50 m) High Built since 1950 in Areas Believed to Have Low Seismicity

Location	Dam	Upstream slope	Downstream slope
Northern California	Lower Bear No. 1	1.3	1.4
	Lower Bear No. 2	1.0	1.4
	Courtright	1.3	1.3
	New Exchequer	1.4	1.4
	Wishon	1.3	1.4
West Virginia	Bailey	2.0	2.0
Colorado	Cabin Creek	1.3	1.3
Australia	Kangaroo Creek	1.3	1.4
	Pindari	1.3	1.3
	Cethana	1.3	1.3
	Pindari Raised	1.3	1.3
	Little Para	1.3	1.4
	Mackintosh	1.3	1.3
	Mangrove Creek	1.5	1.6
	Mangrove Creek	1.5	1.6
	Raised	1.3	1.3
	Murchison	1.3	1.3
	Awonga	1.3	1.3
	Glennies Creek	1.3	1.3
	Bastayan	1.3	1.4
	Lower Pieman	1.3	1.3
	Boondooma	1.3	1.3
Canada	La Joie	1.1	1.5
	Outardes 2	1.4	1.4
France	Fades	1.3	1.3
	Le Rouchain	1.4	1.4
Brazil	Areia	1.4	1.4
	Ita	1.3	1.3
	Segredo	1.3	1.3
	Machadinho	1.3	1.3
Colombia	Alto Anchicaya	1.4	1.4
	Golillas	1.6	1.6
	Salvajina	1.5	1.5
	La Miel	1.5	1.5
Nigeria	Shiroro	1.3	1.3
Thailand	Khao Laem	1.4	1.4
Sri Lanka	Kotmale	1.4	1.4
Sarawak	Batang AI (Main)	1.4	1.4
	Batang AI (Saddle)	1.4	1.4
	Batang AI	1.4	1.4
	Average	1.36	1.40

Source: From Seed et al. [11]:

TABLE 3.13 Slopes of Concrete-Faced Rockfill Dams Built after 1950 in Moderately Seismic Areas

Location	Upstream slope	Downstream slope	Height, ft (m)
Mexico (Pinzanes, 1956)	1.2	1.3	201 (67)
Mexico (San Idelfonso, 1959)	1.4	1.4	186 (62)
Yugoslavia (Rama, 1967)	1.3	1.3	330 (110)
Venezuela (Neveri, 1981)	1.4	1.5	345 (115)
Venezuela (Yacambu, 1982)	1.5	1.5	450 (150)
Panama (Fortuna, 1982)	1.3	1.4	315 (105)
Alaska (Terror Lake, 1985)*	1.5	1.4	174 (58)
Indonesia (Cirata Dam, 1984)†	1.5	1.5	375 (125)
Kenya (Kansuru, 1974)	1.7	1.5	168 (56)
Average	1.42	1.42	

*No downstream inhabitants.
†Designers say downstream slope raveling is predicted.

TABLE 3.14 Summary of Typical Current Practice in the Selection of Slopes for Concrete-Faced Rockfill Dams

	Upstream slope	Downstream slope
Areas of low seismicity ($< 0.15g$)	1 on 1.36	1 on 1.40
Areas of moderate seismicity ($\approx 0.35g$)	1 on 1.42	1 on 1.42

Source: From Seed et al. [11].

TABLE 3.15 Conclusions from Analysis of Concrete-Faced Rockfill Dams

	Earthquake magnitude	Peak base acceleration	Peak crest acceleration	Average downstream slope for displacements of 2 ft (0.6 m) or more	Average downstream slope for displacements of 1 ft (0.3 m) or less
Areas of low to moderate seismicity	6½	$<0.1g$	$<0.25g$	1.35	1.4
	6½	$\approx 0.15g$	$\approx 0.45g$	1.4	1.40
	7½	$\approx 0.15g$	$\approx 0.45g$	1.4	1.40
	8½	$\approx 0.15g$	$\approx 0.45g$	1.45	1.45
	6½	$\approx 0.3g$	$\approx 0.75g$	1.5	1.5
Areas of high seismicity	7½	$\approx 0.3g$	$\approx 0.75g$	1.55	1.60
	8½	$\approx 0.3g$	$\approx 0.75g$	1.65	1.7
	6½	$\approx 0.5g$	$\approx 1.0g$	1.55	1.55
Areas of very high seismicity	7½	$\approx 0.5g$	$\approx 1.0g$	1.6	1.65
	8½	$\approx 0.5g$	$\approx 1.0g$	1.8	1.8

Source: From Seed et al. [11].

OTHER COMMENTS: Observation of the performance of a number of concrete-faced dams supports the following general conclusions:

1. To be successful, a high degree of compaction of the rockfill must be achieved. Good compaction minimizes movement under load, which often leads to large leakage and extensive maintenance.
2. The rockfill should be essentially complete before the face is constructed.
3. Movements of the concrete face are independent of the slab thickness, i.e., the slab floats on the rockfill.
4. A point of weakness in many dams is the joint between the face slab and the toe block. It is essential that a high degree of compaction in the rockfill at the junction with the perimeter toe block be obtained.
5. Joints and water stops are to be constructed following strictly designed details based on previous experience.
6. Rockfill placement during winter months must be avoided, or if this is necessary, carefully controlled.

Reinforced Rockfill Dams. Reinforced rockfill dams can be designed to resist the flow of water through or over the embankment itself. Motivations for reinforcement may be because:

- The dam will be overtopped frequently by normal or flood flows.
- A significant uncertainty exists in estimating the maximum flood for which diversion works are designed.

Procedures to estimate flow capacity and stability, and to design the reinforcement are abundant in the literature. The reader is referred to the bibliographic references.

Embankment Dam Deformations. A survey of the crest settlements and the deflections of 68 rockfill dams was recently reported [12]. Envelopes of both of these deformation parameters are presented in Figs. 3.25 and 3.26. The zero-time point in these figures has been taken to be the time at which the initial measurements were made, that is, when the first readings were taken after construction was completed.

The long-term settlement of well-constructed earth dams ranges between 0.2 and 0.4 percent of the embankment height, depending on soil type. For dams on relatively incompressible foundations, 1 percent of the height is recommended [3] as the compression for which the camber of the embankment should be designed.

Embankment Dam Performance. A number of studies have been made of dam failures and accidents. These studies have included data from dams built in the United States and in other nations after 1900. The data clearly indicate that properly designed, constructed, and maintained dams are safe structures. The percentage of dams constructed in any decade which have failed has declined as time goes on and has remained below 0.3 percent since 1930. An overall summary of the causes of failure is presented in Fig. 3.27, and is expanded in Table 3.16, where preventive or corrective measures are also shown.

Regarding earthquake effects, major concrete or masonry gravity dams have

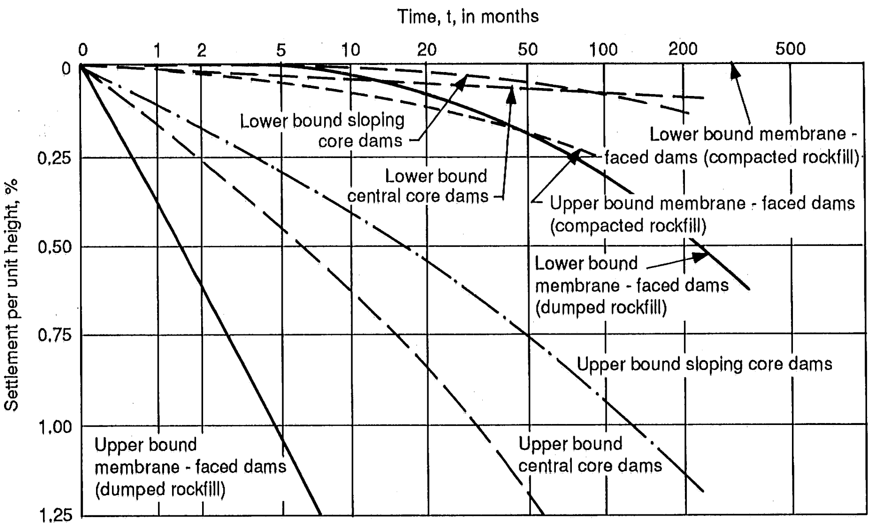


FIGURE 3.25 Envelopes of settlement curves. *(From Clements [12].)*

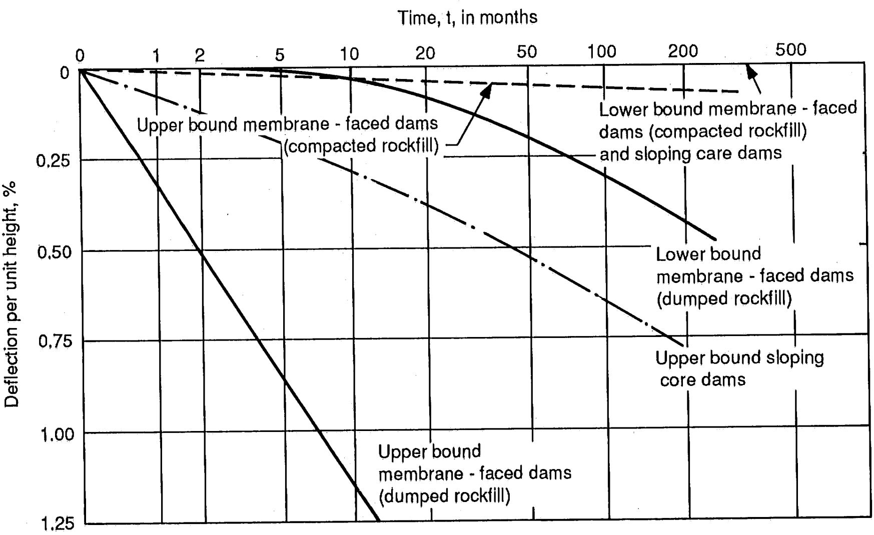


FIGURE 3.26 Envelopes of deflection curves. *(From Clements [12].)*

rarely been subjected to significant ground shaking, so there is little experience on the seismic performance of these structures. One of the most significant such experiences was provided by the Koyna (India) earthquake of December 11, 1967. Koyna Dam, a 306-ft- (102-m-) high concrete gravity dam, suffered struc-

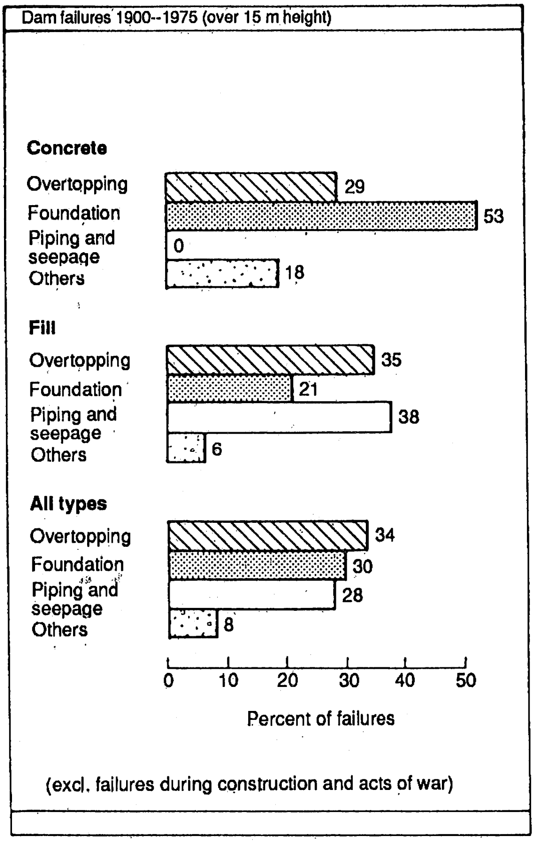


FIGURE 3.27 Cause of dam failure. *(From National Research Council [13].)*

tural damage due to the resulting ground accelerations, which exceeded 0.5*g*. Another important case history is provided by Pacoima Dam (California), a 335-ft-(111-m-) high concrete arch dam, which survived very intense ground shaking during the 1971 San Fernando earthquake with no structural damage. These two case histories have had a profound influence on the understanding of the earthquake performance of concrete dams. Comparable experience is, unfortunately, unavailable for masonry dams.

In embankment dams, possible ways in which an earthquake may cause failure include:

1. Disruption of dam by major fault movement in the foundation
2. Loss of freeboard due to differential tectonic ground movements
3. Slope failures induced by ground motions
4. Loss of freeboard due to slope failures or soil compaction
5. Sliding of dam on weak foundation materials

TABLE 3.16 Earth Dam Failures

Form	General characteristics	Causes	Preventive or corrective measures
Overtopping	Flow over embankment, washing out dam.	*Hydraulic Failures (30% of all failures)* Inadequate spillway capacity. Clogging of spillway with debris. Insufficient freeboard due to settlement, skimpy design.	Spillway designed for maximum flood. Maintenance: trash booms, clean design. Allowance for freeboard and settlement in design: increase crest height or add flood parapet.
Wave erosion	Notching of upstream face by waves, currents.	Lack of riprap, too small riprap.	Properly designed riprap.
Toe erosion	Erosion of toe by outlet discharge.	Spillway too close to dam. Inadequate riprap.	Training walls. Properly designed riprap.
Gullying	Rainfall erosion of dam face.	Lack of sod or poor surface drainage. *Seepage Failures (40% of all failures)*	Sod, fine riprap: surface drains.
Loss of water	Excessive loss of water from reservoir and/or occasionally increased seepage or increased groundwater levels near reservoir.	Pervious reservoir rim or bottom. Pervious dam foundation. Pervious dam. Leaking conduits.	Banket reservoir with compacted clay or chemical admix: grout seams, cavities. Use foundation cutoff: grout upstream blanket. Impervious core. Watertight joints: waterstops: grouting.
	Settlement cracks in dam.	Remove compressible foundation, avoid sharp changes in abutment slope, compact soils at high moisture.	
	Shrinkage cracks in dam.	Use low-plasticity clays for core, adequate compaction.	
Seepage erosion or piping	Progressive internal erosion of soil from downstream side of dam or foundation backward toward the upstream side to form an open conduit or "pipe." Often leads to a washout of a section of the dam.	Settlement cracks in dam.	Remove compressible foundation, avoid sharp changes, internal drainage with protective filters.

TABLE 3.16 Earth Dam Failures *(Continued)*

Form	General characteristics	Causes	Preventive or corrective measures
	Shrinkage cracks in dam.	Low-plasticity soil: adequate compaction; internal drainage with protective filters.	
	Pervious seams in foundation.	Foundation relief drain with filter: cutoff.	
	Pervious seams, roots, etc., in dam.	Construction control: core; internal drainage with protective filter.	
	Concentration of seepage at face.	Toe drain: internal drainage with filter.	
	Boundary seepage along conduits, walls.	Stub cutoff walls, collars: good soil compaction.	
	Leaking conduits.	Watertight joints: water stops; materials.	
	Animal burrows.	Riprap, wire mesh.	
Foundation slide	Sliding of entire dam, one face, or both faces in opposite directions, with bulging of foundation in the direction of movement.	*Structural Failures (30% of all failures)* Soft or weak foundation. Excess water pressure in confined sand or silt seams.	Flatten slope: employ broad berms; remove weak material; stabilize soil. Drainage by deep drain trenches with protective filters: relief wells.
Upstream slope	Slide in upstream face with little or no bulging in foundation below toe.	Steep slope. Weak embankment soil. Sudden drawdown of pond.	Increased compaction: better soil. Flatten slope, rock berms: operating rules.
Downstream slope	Slide in downstream face.	Steep slope. Weak soil. Loss of soil strength by seepage pressure or saturation by seepage or rainfall.	Flatten slope or employ berm at toe. Increased compaction: better soil. Core: internal drainage with protective filters: surface drainage.
Flow side	Collapse and flow of soil in either upstream or downstream direction.	Loose embankment soil at low cohesion, triggered by shock, vibration, seepage, or foundation movements.	Adequate compaction.

Source: From National Research Council [13].

6. Piping failure through cracks induced by ground motions
7. Overtopping of dam due to seiches in the reservoir
8. Overtopping of dam due to slides or rockfalls into the reservoir
9. Failure of spillway or outlet works

A careful review [14] of the available case histories of embankment dam performance has been carried out, with the following conclusions:

1. Hydraulic fill dams have been found to be vulnerable to failures under unfavorable conditions and one of the particular unfavorable conditions is the shaking produced by strong earthquakes.
2. Many hydraulic fill dams, however, have performed well for many years and when they are built with reasonable slopes on good foundations they can survive moderately strong shaking—up to about 0.2 g from Magnitude 6½ to 7 earthquakes with no harmful effects.
3. Virtually any well-built dam can withstand moderate earthquake shaking, say with peak accelerations of about 0.2 g and more, with no detrimental effects.
4. Dams constructed of clay soils on clay or rock foundations have withstood extremely strong shaking ranging from 0.35 to 0.8 g from a Magnitude 8¼ earthquake with no apparent damage.
5. Two rockfill dams have withstood strong shaking with no significant damage and if the rockfill is kept dry by means of a concrete facing they should be able to withstand extremely strong shaking with only small deformations.
6. For dams constructed of saturated cohesionless soils and subjected to strong shaking, a primary cause of damage or failure is the build-up of pore water pressures in the embankment and the possible loss of strength which may accrue as a result of these pore pressures. Great caution is required in attempting to predict this type of failure by pseudo-static analyses, and dynamic analysis techniques seem to provide a more reliable basis for evaluating field performance.

Inspection, Maintenance, and Instrumentation

Because it is difficult to completely identify all geotechnical problems which can arise in the foundation and the abutments, it is necessary to inspect the completed dams on a regular basis. During the first year of operation, the abutments, the downstream slope of the embankment, and the valley floor should be inspected at least weekly (daily during filling if the reservoir fills quickly). All seepage should be noted carefully and the rate measured.

Comparison of each measurement with preceding values will call attention to any increase in seepage rate. If seepage is normal and all drains and filters working properly, seepage water will be clear. A qualitative assessment of seepage clarity should be recorded during each inspection. If the seepage becomes muddy, there will be cause for concern, since muddy water indicates that piping is occurring within the abutment, the embankment, or the foundation. If muddy discharge continues, the reservoir should be drawn down as quickly as possible and every effort should be made to determine the reason for the seepage and to develop means to control the piping.

A postconstruction survey of the embankment configuration should be made and recorded. Concrete benchmarks should be established on the dam crest, near the downstream toe, and on the valley floor near the dam's toe. Annual measurements should be made of the elevation and location of these benchmarks to detect any bulging which might herald a slide of the embankment.

Riprap should also be inspected annually to document any movement or deterioration. Replacement of riprap in damaged locations should be made as soon as possible to prevent possible erosion of the embankment. Erosion often occurs along the groin, the line of contact between the embankment and the abutment. A gravel layer can be placed along the groin to reduce the potential for erosion. Monthly inspections should also include observation of the groins.

Any emergency equipment, such as guard gates at the penstock entrance or guard valves within the penstock, should be operated monthly to make certain that excessive rust or other damage does not prevent operation during emergencies.

The Committee on Safety of Existing Dams of the National Research Council [13] recommends that an effective dam inspection program should include three types of inspections as follows:

1. Periodic technical inspections
2. Periodic maintenance inspections
3. Informal observations by project personnel

Technical inspections are those involving specialists familiar with the design and construction of dams and include an assessment of the safety of project structures. Maintenance inspections are those performed at a greater frequency than technical inspections in order to detect at an early stage any significant developments in project conditions, and they should consider operational capability as well as structural stability. The third type of inspection is actually the continuing effort performed by onsite project personnel (dam tenders, powerhouse operators, maintenance personnel) in the course of performing their normal duties.

Maintenance is an ongoing process that should never be neglected and that should continue throughout the operating life of the dam. To provide proper maintenance services, all material regarding the design, construction, and operation of the dam should be available in a location where it is readily accessible for the inspection and maintenance program.

The safety of an existing dam can be improved and its life lengthened by a carefully planned and implemented surveillance program. A key part of such a program is a visual examination of the structure, the reservoir, and the appurtenant works (as discussed in Chap. 2). However, surveillance must be more than visual observations. Settlements may go undetected without proper regular measurements of monuments. Comparison of seepage qualities from one inspection to another and over the years is difficult by visual observation and estimation. There are also conditions within a dam that cannot be seen but that can be measured by instrumentation. Thus, even for a simple structure, some type of instrumentation may be needed to improve and supplement the visual examination.

The purpose of instrumentation in an existing dam is to furnish data to determine if the completed structure is functioning as intended and to provide a continuing surveillance of the structure to warn of any developments which endanger its safety.

A variety of instruments are now available to monitor the several performance indicators of a dam, as listed below [13]:

Phenomena measured	Instrument
Pore water and groundwater measurements	Piezometer, closed system and open system, observation wells
Earth pressure measurements	Earth pressure cells, wholly embedded in soil, at contact plane between soil and structure
Deformation measurements; horizontal and vertical	Survey equipment transits, theodolites, electronic distance measurement equipment (EDME), and levels
Internal deformation, rotational and tilting	Extensometers, inclinometers, tiltmeters
Load and strain measurements, surface installation or embedded	Strain meters, load cells, concrete stress cells
Temperature measurements	Temperature sensors
Seepage	Weirs, flow meters, and flumes
Earthquake shaking	Strong ground motion instruments

Guidelines for selection of instrument, location within the dam or foundation, and installation procedures can be found in Ref. 13. Guidelines for frequency of reading are provided in Tables 3.17 and 3.18.

3.5 CONCRETE DAMS

Concrete dams are most often one of three types: gravity, arch, or buttress (see Sec. 3.1). Each type of dam lends itself to a particular situation (see Sec. 3.3). The arch dam is most likely used in a narrow site with steep walls of sound rock. Gravity dams depend upon their own weight to be stable and are generally used where the foundation is rock and earthfill in proper quality and quantity is not available. Low concrete dams having a height of less than 30 ft (10 m) can also be built on earth foundations, but particular precautions must be taken to ensure that allowable foundation stresses are not exceeded and that the possibility of piping due to seepage under the dam is minimized. When a concrete dam is constructed on an earth foundation, uplift forces created by reservoir water pressures must be controlled through the use of cutoffs or aprons.

Prior to final design of a concrete dam, it is necessary to determine the characteristics of the material which will be used in its construction. Compressive, tensile, and shear strengths must be determined from the design mix. In addition, the instantaneous and sustained moduli of elasticity, Poisson's ratio, thermal conductivity, specific heat, and diffusivity must be determined for the mix. For the foundation material, it will be necessary to know the shear strength, permeability, compressive strength, and Poisson's ratio. When the foundation is not homogeneous, it will be necessary to determine the preceding quantities for each material present. In-depth structural analysis of arch and buttress dams is not presented because of the detail required and because such dams will very seldom be economical.

TABLE 3.17 Frequency of Readings for Earth Dam Instrumentation

	Progress report during construction		Periodic report operation	
	Frequency of readings			
	Construction	Shutdown	First year	Regular
Piezometer readings (separate gages)	Twice monthly	Monthly	Monthly	Monthly
Piezometer readings (master gage)	Monthly	Alternate months	Approximately 6 months after completion of dam	Annually on same date as a set of separate gage reading
Porous tube piezometer readings	Twice monthly	Monthly	Monthly	Monthly
Internal vertical and horizontal movement readings (crossarm or HMD)	Complete set of readings each time a unit is installed	Monthly	Complete set approximately 6 months after dam is completed	Every 2 years
Foundation settlement readings (baseplates)	Complete set of readings each time an extension is added	Monthly	Approximately 6 months after dam is completed	Every 2 years
Measurement points—cumulative settlement and deflection readings	Monthly, if required, or when dam is completed	Monthly, if required	Approximately 6 months after dam is completed	Every 2 years
Measurement points—cumulative settlement and deflection readings spillway and outlet works	Monthly as portions of structures are completed	Monthly	Approximately 6 months after structure is completed	Every 2 years
Measurement points—cumulative settlement readings-spillway floor slabs	Monthly as slabs on structure are completed	Monthly, if required	Approximately 6 months after structure is completed	Every 2 years

Source: From National Research Council [13]

TABLE 3.18 Frequency of Readings for Concrete Dam Instrumentation

Type reading	Frequency
Embedded instrument	7 to 10 days during construction; semimonthly or monthly afterward. More frequently during periods of reservoir filling or rapid drawdown.
Deflection-measuring devices	Weekly; closer intervals during periods of special interest.
Uplift pressure measurement	Monthly, except for initial filling, which should be a 7- to 10-day interval.
Target deflection and pier net triangulation measurements	Semiannually during period of minimum and maximum air temperature. Additional measurements during early stages of reservoir filling.
Leveling across top of dam and vicinity	Periodically. More frequently in the early stages of operation and less frequently at later stages, depends on conditions encountered.

Source: From National Research Council [13].

Design Loads

Each concrete dam must be analyzed considering the following structural loads (see Fig. 3.28):

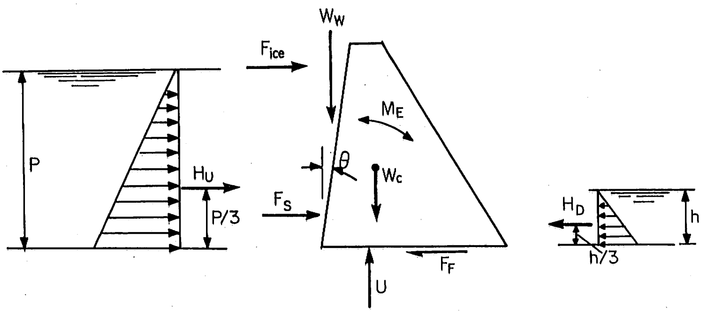


FIGURE 3.28 Forces acting on a concrete gravity dam.

1. The weight W_c (dead load) of the dam itself. All weight is assumed to be transferred directly to the foundation without shear stress between adjoining blocks. The force due to weight acts at the centroid of the section.
2. Hydrostatic forces produced by water in the reservoir and the tailwater. These may be separated into horizontal component H_u and vertical components W_w, as shown in Fig. 3.28.
3. Hydrostatic uplift acting under the base of the dam. The uplift pressure is assumed to vary linearly from that created by the full reservoir at the upstream

face to the pressure created by tailwater depth at the downstream toe. The integrated pressure over the base U should reflect the presence of any drains.

4. Forces due to silt deposited in the reservoir and in contact with the dam Fs. Normally 85 lb/ft^3 (1300 kg/m^3) is used as specific weight of silt.
5. Ice loads F_{ice} should be considered in cold climates where thick ice will form and can be expected to remain frozen for weeks or months. A horizontal load of 10,000 lb/ft (14,000 kg/m) is assumed to act on the dam at the level where ice will form if ice depths of 2 ft (0.6 m) or greater can be expected. For dams approximately 90 ft (30 m) or more in height, systems can be installed (buffers) to add air near the bottom of the dam. The rising air will entrain warmer water from below the surface and prevent freezing at the face of the dam.
6. Earthquake forces must be assumed to act both horizontally and vertically through the center of gravity of the dam if the site is in a seismically active area. The forces may be applied statically unless horizontal accelerations exceed $0.10g$. Dynamic analysis of the structure must be done for larger seismically produced accelerations. Local authorities must be contacted to obtain maximum expected horizontal and vertical accelerations for earthquakes. The forces applied are actually inertia forces equal to the product of the earthquake acceleration and the mass of the dam. Their direction should be assumed so as to produce critical stress.
7. Dynamic forces are also induced by the relative movement of the dam and reservoir during an earthquake. The horizontal pressure force for a unit width of dam due to earthquake is given by

$$F_e \doteq 0.726 C \alpha \gamma h^2$$

where F_e = the force per unit width of dam
α = coefficient which when multiplied by the acceleration due to gravity, 32.2 ft/s^2 (9.8 m/s^2), gives the horizontal acceleration in an earthquake
γ = unit weight of water, 62.4 lb/ft^3 (1000 kg/m^3)
h = depth of water in the reservoir
C = a constant depending on the upstream slope of the dam

The constant C is given approximately by

$$C \doteq 0.7(1 - \theta/90)$$

where θ = the angle between the upstream face of the dam and a vertical line (in degrees)

The moment about the base of the dam is given by

$$M_e = 0.299 C \alpha \gamma h^3$$

where M_e = the moment per unit width of dam

A normal combination of forces to be considered during design includes hydrostatic forces produced by a reservoir at maximum normal elevation, temperature stresses produced by temperatures normal for that time of year, weight of the dam, and ice and silt load. An unusual load combination could include loads produced by the reservoir at maximum elevation coupled with stresses produced by minimum temperatures and appropriate dead load, hydrostatic forces, and silt loadings. A load combination including earthquake forces would be classified as a severe loading. Factors of safety for unusual and severe loadings should be smaller than those acceptable for usual loads.

For a small concrete dam designed for overflow, full consideration should be given to hydrostatic forces in both upstream and downstream directions, but the weight of water flowing over the structure should not be considered except when analyzing the dam forces produced by a flood for a dam with an uncontrolled spillway.

Stability

The dam must first of all be safe from overturning for all loading conditions. The contact stress between the foundation and the dam must be greater than zero at all points or the dam will be unsafe against overturning. Contact stress is calculated by considering all overturning moments and the weight of the dam, and assuming a straight-line distribution of stress on the base. To assure that the stress does not go to zero, the resultant of all horizontal and vertical forces must pass through the middle one-third of the base.

The maximum compressive forces in the concrete and on the foundation must be less than the specified concrete compressive strength divided by 1.0, 2.0, and 3.0 for the severe, unusual, and normal loading cases, respectively. Shear stresses on any plane within the dam must be less than the specified concrete shear strength divided by 1.0, 2.0, and 3.0 for the severe, unusual, and normal loading cases, respectively.

The dam must be safe against sliding. The static coefficient of friction between the foundation and the dam can be calculated from the expression:

$$f = \frac{\Sigma F_h}{\Sigma F_u}$$

where f = static coefficient of friction
ΣF_h = sum of all horizontal forces acting on dam
ΣF_u = sum of all vertical forces acting on dam

The coefficient of static friction may vary in the range from 0.6 to 0.75. However, a value should be determined by laboratory test for the actual foundation material. The acceptable coefficient of friction should be not more than the specified value divided by 2.0, 1.5, and 1.25 for the severe, unusual, and normal loading combinations, respectively.

Configuration

The concrete gravity dam will frequently have a vertical upstream face to concentrate the weight of the dam upstream. The downstream face usually has a constant slope. The dam is usually laid out and then an analysis of all loads and stresses is conducted. The section is then increased or decreased as needed to meet the stability requirements discussed under the previous heading. Each successive layout should be an improvement on the previous section.

Width of the top of the dam is usually determined by the width required to move any necessary equipment over the dam.

For an overflow dam, the layout is similar, except that the downstream surface should correspond to that of the spillway. The width of the base of the overflow section is increased or decreased as required for stability. If gates are incorporated, forces produced by water in the reservoir should be applied with the gates closed.

Freeboard for a concrete dam can be economically provided with a thin concrete or timber parapet wall constructed along the upstream side of the top of the dam.

Foundation

The foundation for a concrete dam must be carefully prepared. All alluvial overburden must be excavated to expose the rock. The rock must be thoroughly cleaned and all loose or cracked rock removed. All cracks should be grouted. All weak or badly fractured rock must be removed and the hole backfilled with concrete. In general, all seams should be excavated to a depth of at least 0.1 times the dam height. However, final judgment on depth of excavation should be made at the site. If the crack is large, or if the material filling it is quite soft, greater excavation may be required. Rock in the foundation should be excavated to eliminate any sharp changes in elevation where stress concentrations can occur.

If the foundation is shale, chalk, mudstone, or siltstone, it will be desirable to cover the exposed rock with a 4- to 6-inch (10- to 15-cm) thickness of mortar to prevent rapid deterioration of the foundation by weather.

The foundation must be carefully examined to determine its permeability. If joints, crevices, or strata of permeable material exist, excessive uplift and/or excessive leakage can occur, and it may be necessary to grout the foundation. In grouting, holes are drilled into the foundation and cement grout is pumped into the holes under pressure. The decision of whether and how deep to grout can only be made by an experienced geologist after studying the detailed geology of the site. Because every foundation is unique, it is always advisable to have an experienced geologist or foundation engineer inspect the foundation prior to design, but particularly prior to construction.

Spillways and Outlet Works

One basic advantage of a concrete dam is that it can be designed as an overflow structure. The overflow structure is constructed of reinforced concrete and must be designed in accordance with the criteria for any concrete dam. Spillway sections are usually located near the old streambed.

Bottom outlet works are usually incorporated in the concrete dam section. They are required to drain the reservoir for maintenance or to furnish required flow in the stream. For a small dam, such as considered herein, the outlet works is usually a steel conduit cast into the dam. A bulkhead or guard gate is frequently provided on the upstream face.

Power outlets may be cased into the dam if the powerhouse is immediately downstream from the dam. However, most small dams provide a means of diverting water into a power canal, a flume, or a power tunnel. The location of such structures is usually on one bank, immediately upstream from the dam.

3.6 ROLLER-COMPACTED CONCRETE DAMS

Definition

Roller-compacted concrete, RCC, or rollcrete, is defined as a lean concrete of no-slump consistency that can be compacted by means of vibratory rollers. The stiff consistency makes it different from conventional concrete mixtures.

History of Development

The idea of a new material for the construction of dams was first proposed in 1971 [15]. It was then postulated that an increase in shear strength of the cement-enriched granular fill materials would result in significant reduction of dam cross section as compared with a typical embankment dam. Use of continuous-placement methods similar to those utilized in an embankment dam would accomplish savings in time and money as compared with the construction of the usual gravity concrete dam.

First trials of rollcrete were carried out by the Corps of Engineers in the early seventies. These were followed in 1975 by the construction of portions of the embankment section of Tarbela Dam in Pakistan. Some of the early applications of RCC to hydroelectric projects are listed in Table 3.19. Recent RCC dams constructed in the United States during this decade are listed in Table 3.20.

TABLE 3.19 Roller-Compacted Concrete Applications in Hydroelectric Projects

Project and country	Year completed	Quantity, yd^3 (m^3)	Type of application
Itaipu, Brazil	1978	34,000(26,000)	Backfill to replace poor rock in channel downstream of diversion structure
Bonneville II, U.S.A.	1978	17,000(13,000)	Cover for rock excavation to prevent air slaking
Revelstoke, Canada	1979	9,940 (7,600)	12-ft (3.6-m-) thick cap on 150-ft- (46-m-) high cofferdam
Guri, Venezuela	1981	20,360(15,570)	Solid cofferdam to allow construction of second tailrace channel
Tucurui, Brazil	1982	16,000(12,230)	Interior of walls of navigation lock structure

Source: From Dinchak and Hansen [16].

TABLE 3.20 Roller-Compacted Concrete Gravity Dams in the United States

Project	Max. height, ft (m)	RCC quantity, yd^3 (m^3)	Description	Cement, yd^3	Cost/yd^3 ($)	Constr. Period
Willow Creek Dam—Heppner, Oregon Owner/Engr: ColE, Walla Walla, Washington Contr: Eucon Corp. Tri Cities, Washington	169 (52)	433,000 (331,000)	Concrete gravity dam with precast concrete panel vertical upstream face	118 average	19	1982
North Loop Detention dams (2)—Austin, Texas Owner: Trammell Crows Co., Austin, Texas Engr: Freese & Nichols, Inc. Ft. Worth, Texas Contr: H. B. Zachry Co., San Antonio, Texas	32 (10)	20,670 (15,800)	Two concrete gravity dams on downstream side of earth embankment	200 80 FA	26.2	1984
Winchester Dam, Winchester, Kentucky Owner: Winchester Municipal Utilities Engr: Parrott, Ely, & Hurt, Lexington, Kentucky & Palmer Engrg. Co., Winchester, Kentucky Contr: McCormick Contractors, Winchester, Kentucky	70 (21)	32,000 (24,500)	Concrete gravity dam with membrane lined precast concrete panels upstream	175	32.5	1984
Middle Fork Dam—Parachute, Colorado Owner: Exxon Co. USA, Houston, Texas Engr: Morrison-Knudsen Engrs., Denver, Colorado (was International Engineering Co.) Contr: Avery Structures, Inc., Buena Vista, Colorado	124 (38)	55,000 (42,000)	Concrete gravity dam with conventional concrete both faces	120	less than $25*	1984
Great Hills Dam—Austin, Texas Owner: Trammell Crow Co., Austin, Texas Engr: Camp, Dresser, & McKee, Austin, Texas Contr: H. B. Zachry Co., San Antonio, Texas	37 (11)	13,000 (10,000)	Concrete gravity dam on both sides of roadway embankment	246 98 FA	—	1984–85
Galesville dam—Azalea, Oregon Owner: Douglas County, Roseburg, Oregon Engr: Morrison-Knudsen Engrs., San Francisco, California Contr: Venture Const. Inc., Auburn, Washington	167 (51)	223,000 (170,500)	Concrete gravity dam with conventional concrete upstream, unformed downstream	91 87 FA average	$21.42† average	1985

Upper Stillwater dam—Duchesne, Utah Owner/Engr: USBR, Denver, Colorado Contr: Tyger Constr. Co., Spartanberg, South Carolina	290 (88)	1,400,000 (1,070,000)	Concrete gravity dam with slipformed concrete both faces	129 289 FA	$23.81	1985–87
Monksville Dam, Ringwood, New Jersey Engr: O'Brien & Gere, Blue Bell, Pennsylvania Owner: No. Jersey Dist. Water Supply Comm. & Hackensack Water Co., Hackensack, New Jersey Contr: S. J. Groves, Co., Minneapolis, Minnesota	150 (46)	289,000 (221,000)	Concrete gravity dam with conventional concrete both faces	108	Greater than £16.52 orig. bid	1986
Grindstone Canyon Dam, Ruidoso, New Mexico Engr: Boyle Engrg Corp, Albuquerque, New Mexico Owner: Village of Ruidoso, NM Contr: ARCO Materials, Farmington, New Mexico	139 (42)	114,500 (87,500)	Concrete gravity dam with conventional concrete both faces	125 + 50 FA	$25.58 + mob.	1986

*Aggregate supplied by owner.
†Plus $6.75/yd^3 for 6-in aggregate by previous road relocation contractor.
Source: From Hansen [17].

Materials

RCC normally utilizes available sand and gravel materials unprocessed or with minimal processing, although aggregates for conventional concrete or quarried rock crushed to an acceptable gradation can also be utilized. Gradation control can be more relaxed than for ordinary concrete, and, in fact, in some cases the fines (material passing No. 200 sieve) do not have to be washed from the aggregate. Fines may or may not be beneficial, and this must be determined by trial mixes. Typical grading and plasticity characteristics of fine aggregate used by the Corps of Engineers in some of their RCC projects are shown in Tables 3.21 and 3.22. Coarse aggregate grading and its proportion relative to the total aggregate are shown in Tables 3.23 and 3.24. Sufficient water content must be added to dampen all fines, including the cementicious materials, so that they will be able to coat and adhere to the larger particles. It has been stated that the best compaction produces the best product, and the best compaction occurs with the wettest mix compatible with the weight of the vibratory rollers. Approximate water requirements and entrapped air contents for various maximum sizes of the coarse aggregate are shown in Table 3.25.

As with all other concretes, the water/cement ratio is determined from strength requirements. Typical water/cement (w/c) ratios versus compressive strength curves are shown in Fig. 3.29.

Pozzolans (flyash) are used in RCC to reduce the hydration heat and to minimize cement costs. Experience has shown [19] that pozzolans in the northwestern United States are most effectively used in concrete at about 35 percent volume. These materials could produce considerable increase in strength of the product.

TABLE 3.21 Typical Grading of Fine Aggregate Used for RCC

Sieve size		Size separation					
in, No.	mm	Percent by weight passing each size*					
⅜ in	9.5	100	100	100	100	100	100
No. 4	4.75	100	100	98	100	99	98
No. 8	2.36	89	84	87	77	84	71
No. 16	1.18	85	71	80	63	69	44
No. 30	0.60	82	62	73	51	53	32
No. 50	0.30	61	55	49	43	25	26
No. 100	0.15	17	45	15	20	8	21
No. 200	0.075	11	34	6	11	3	15
Fineness modulus		1.66	1.83	1.98	2.46	2.62	3.08
Type†		N	N	B	M	M	B

*These fine aggregate gradations were used for laboratory mix designs that produced satisfactory concrete.

†N = natural, M = manufactured, B = blend of natural and manufactured sand.

Source: From Schradek [18].

TABLE 3.22 Liquid Limits and Plasticity Index, Fine Aggregates

Liquid limit	Plastic index	Maximum % passing no. 200 (0.075 mm)
0–25	0–5	7.0
0–25	5–10	6.5
0–25	10–15	3.0
0–25	15–20	2.0
0–25	20–25	1.0
25–35	0–5	6.5
25–35	5–10	4.0
25–35	10–15	3.0
25–35	15–20	1.5
25–35	20–25	1.0
35–45	0–5	6.0
35–45	5–10	5.5
35–45	10–15	4.5
35–45	15–20	3.5
35–45	20–25	1.0
45–55	0–5	4.0
45–55	5–10	3.5
45–55	10–15	2.5
45–55	15–20	2.0
45–55	20–25	1.0

Source: From Schradek [18].

TABLE 3.23 Range of Coarse Aggregate Gradings Used for RCC

Sieve size		Size separation					
		Percent by weight passing each size*					
in, No.	mm	3-in	NMSA	1½-in	NMSA	¾ in	NMSA†
3	75	100	100				
2½	62.5	95	98				
2	50	86	78				
1½	37.5	72	60	100	100		
1	25.0	49	40	92	84	100	100
¾	19.0	35	31	53	70	100	98
½	12.5	25	17	27	45	96	70
⅜	9.5	14	14	14	28	32	49
No. 4	4.75	1	2	1	2	2	4

*These coarse aggregate gradings used for laboratory mix designs that produced satisfactory concrete. Gradations based on compromise of (*a*) idealized maximum density curves; (*b*) pit run proportions; and (*c*) minimal processing.

†Coarse aggregate fraction of ¾ in. to zero stockpiles used for RCC structural and bedding mixes.

Source: From Schradek [18].

TABLE 3.24 Coarse Aggregate as Percent of Total (By Volume)

Nominal maximum size and type coarse aggregate	Fineness modulus			
	2.4	2.6	2.8	3.0
3 in (75 mm) crushed	68	67	66	65
3 in (75 mm) rounded	70	69	68	67
1½ in (37.5 mm) crushed	62	61	60	59
1½ in (37.5 mm) rounded	64	64	62	62
¾ in (19.0 mm) crushed	56	55	54	53

Note: For RCC without entrained air and with manufactured fine aggregates. The coarse aggregate content may be increased 2 percent if natural sand is used.
Source: From Schradek [18].

TABLE 3.25 Approximate Mixing Water and Air Contents

	Water, lb/yd³ for indicated nominal maximum size of aggregate†						
	¾-in		1½-in		3-in		6-in,
Modified vebe, s*	Average	Range	Average	Range	Average	Range	average
12 to 16‡	260	245–270	—	—	—	—	—
20 to 24§	—	—	234	200–265	184	160–200	154
Approximate air content, %	1.0	—	1.0	—	1.0	—	1.0

Note: 1 inch = 2.54 cm; 1 lb/yd³ = 5.82 N/m³. For adjustment in mixing water, assume ± 3.0 lb/yd³ change in mixing water = ± 10-s modified Vebe change.
*Design values, standard deviation for 45 tests = ± 6 s. These quantities are for use in computing cement factors for trial batches. Lower range should be used for natural aggregates or lower sand contents, higher range for crushed aggregates or higher fine aggregate contents.
‡Water contents for bedding mixes used on hardened lift joints.
§Data for 6-in extrapolated from other MSA mixes.
Source: From U.S. Army Corps of Engineers [18].

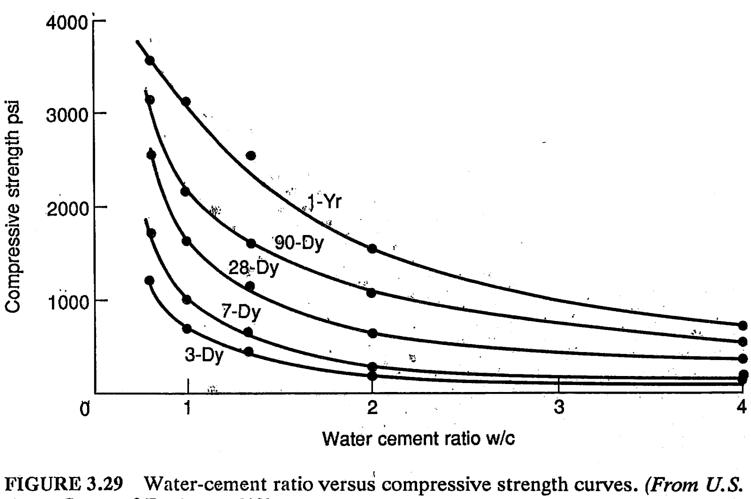


FIGURE 3.29 Water-cement ratio versus compressive strength curves. *(From U.S. Army Corps of Engineers [18].)*

Construction Methods

Mixing of RCC can be done by the same conventional methods utilized for concrete, or by continuous processors. Continuous mixing may be in a tilt drum mixer or a pubmill similar to that used to mix soil-cement. Continuous mixing is preferred where high production rates are needed.

RCC can be transported in bottom-dump trailer trucks or scrapers commonly used for gravel and earthfill placements. Rear-dump trucks have been used successfully, but they require more careful control due to potential segregation of the materials when dumping.

After dumping, the materials are spread with a standard dozer or grader. Segregation from dumping can be minimized by blending during the spreading operation. Where it is necessary to place materials against rock abutments, forms or blockouts and the use of smaller compaction equipment may be necessary. The spreading equipment provides initial compaction and deposits the material in even layers.

After spreading, the materials are compacted with vibratory rolling equipment. The compactive effort required is a function of layer depth and aggregate gradation but will basically require work similar to that used for select earthfill. Field control can be accomplished by determining in-place density or by specifying a minimum number of passes for different types and sizes of vibratory rollers.

Design

Figures 3.30 and 3.31 present cross sections of two recently completed RCC dams. Basically, there is no essential difference between these cross sections and that of a conventional concrete dam, such as that presented in Figs. 3.8 and 3.9. Considerations about site requirements, foundation preparations, performance monitoring, and instrumentation, as presented for the embankment and concrete dams in Sec. 3.4 and 3.5 of this chapter, are equally applicable to RCC dams.

A comparison between various design and construction parameters for RCC and conventional mass concrete dams is presented in Table 3.26.

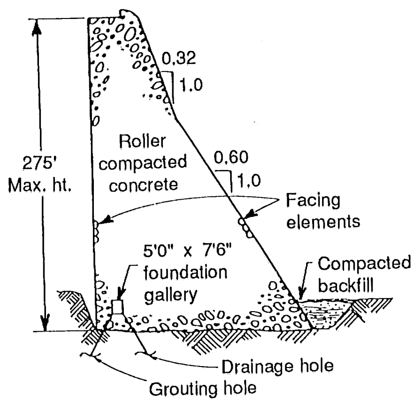


FIGURE 3.30 Final design—Upper Stillwater Dam section. *(From Dinchak and Hansen [16].)*

3.7 MASONRY DAMS

Masonry dams are extremely labor intensive and, thus, are economical only where labor is relatively inexpensive and fractured stone (rubble) of durable quality is readily available. Cement mortar is used to embed each stone. Many notable masonry dams have been built in the past century and have provided excellent service. The theory used in designing a masonry dam is the same as

TABLE 3.26 Comparison of Roller-Compacted Concrete and Conventional Mass Concrete Construction in Dams

Aspect	Roller-compacted concrete	Conventional mass concrete in dams
Cement content	2.5 to 6.0%	6.0 to 10.0%
Aggregate grading control	Not restricted to normal concrete aggregate grading limits—may be pit or crusher run material	Restricted to normal concrete aggregate grading limits
Aggregate quality	Less restrictive than ASTM, particularly in fines content	Restricted to ASTM C33
Batching & mixing	Weigh batching with batch-type mixer or volumetric batching with continuous mixing	Weigh batching with batch type mixer
Consistency	Moist—no slump	Plastic—2.5- to 6.0-cm slump
Transporting & placing	Large end or bottom-dump trucks or earthmoving scrapers	4- to 6-m^3 concrete hoppers and buckets
Placement sequence	Lift over the entire dam surface	Layers within monolith blocks
Lift thickness	30 to 45 cm*	1 to 3 m*
Time between lifts	No restriction	1 to 3 days
Consolidation	Heavy, smooth drum, self-propelled vibratory roller	Immersion-type concrete vibrators with vibrator crew
Monolith joints	None	12- to 21-m intervals*
Horizontal construction joint cleanup	None to minor removal of tracked-in mud	Green-cutting, sandblasting, or high-pressure water blasting
Dam design section	Unlimited—can be constructed without formwork to natural slope of 0.8V:1H or up to vertical with precast or slipformed concrete facing elements	Unlimited—can be formed and constructed to any design section
Production rate	Generally controlled by the batching and mixing plant capacity	Generally controlled by transporting and placing equipment
Variation in compressive strength	20 to 40% (Relates to aggregate grading control and concrete density)	10 to 20%
Durability	Dependent on quality of exposed concrete	Dependent on quality of exposed concrete
Temperature control measures	Dependent on ambient temperature and lifts/day	Precooling and postcooling
Estimate cost per mM^3 for 100,000- to 1,000,000-m^3 dam†	USA \$20 to \$40†	USA \$55 to \$85†

*1 m = 3.28 ft; 1 cm = 0.4 in; 1 m^3 = 35.3 ft.
†Estimated costs for the continental United States, approximately 1982.
Source: From Mass [20].

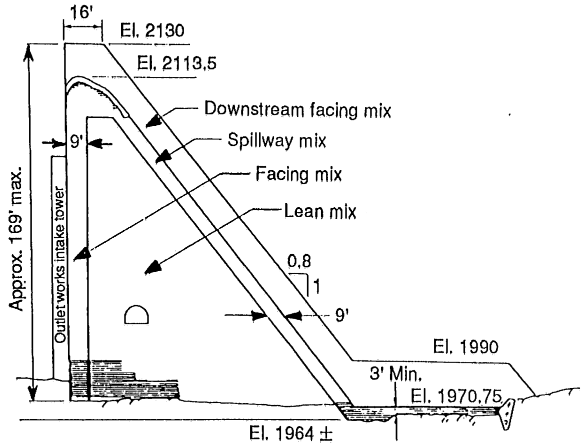


FIGURE 3.31 Typical section of Willow Creek Dam. *(From Dinchak and Hansen [16].)*

described for a concrete dam. However, allowable stresses for masonry are much more uncertain than for concrete. In 1910, Baker (cited in Ref. 21) recommended the data in Table 3.27 as allowable stresses for rock and mortar of excellent quality.

TABLE 3.27 Allowable Stress Levels for Masonry Dam Materials

Material	Allowable stress lb/ft^2	Allowable stress kg/m^2
Rubble	20,000–30,000	97,650–146,470
Squared stone	30,000–40,000	146,470–195,300
Limestone	40,000–50,000	195,300–244,120
Granite	50,000–60,000	244,120–292,950

Source: From Hinds et al. [21].

All forces considered in concrete dam design should be considered in the design of a masonry dam. Since allowable stresses for masonry are lower than for concrete, the final structure will usually have a wider base than a concrete dam. Unit weight of the rock to be used must be determined by test prior to design.

Foundation preparation for a masonry dam is as equally important as in a concrete dam, and the same precautions should be followed. In addition, a 6- to 8-in (15- to 20-cm) layer of mortar should be placed over the foundation prior to laying of the masonry. Particular care must be taken to obtain clean sand, free of earth and organic material, for use in mixing the mortar. Clean water is also vital. In addition, rocks should be well cleaned of mud or other adherents prior to being placed in the dam.

3.8 SPILLWAY DESIGN

The spillway is designed to pass flows larger than can be used for hydroelectric generation. The importance of the spillway cannot be overemphasized. Most dam failures have occurred because the spillway was incapable of passing a particular flood, with the result that the dam was overtopped and breached. Many spillways with adequate capacity have failed because severe erosion occurred at the base of the spillway, resulting in damage to the spillway, the dam, or both.

Overtopping of earthfill dams will usually result in their breaching, unless the overtopping is of very short duration, such as that produced by runup of waves. Concrete dams may, however, withstand moderate overtopping provided the foundation is adequate. Rockfill dams may withstand minimal overtopping.

The flood for which a spillway should be designed has not been uniformly accepted by the profession. Criteria specified by the U.S. National Dam Safety Act (1972) provides the guidelines shown in Table 3.28, where PMF indicates probable maximum flood. Thus, a small dam located in a drainage area not far above an inhabited village might require a spillway designed for a PMF. However, the designer must also consider that, in the event of a PMF, the failure of the small dam may have only an insignificant effect upon the depths and velocity of flood flow downstream. On the other hand, the incremental cost required to increase a spillway size is usually not feasible because

TABLE 3.28 Criteria Specified by the U.S. National Dam Safety Act

Size classification				
	Reservoir storage, s		Dam height, H	
Category	Acre-feet	Cubic meters	Feet	Meters
Small	50–1000	0.06×10^6 to 1.23×10^6	25–40	7.6–12.2
Intermediate	1000–5000	1.23×10^6 to 61.7×10^6	40–100	12.2–30.5

Hazard potential		
Category	Loss of life	Economic loss
Low	None expected	Minimal
Significant	Few	Appreciable
High	More than a few	Excessive

Recommended design flood		
Hazard	Size	Design flood
Low	Small	50–100 year
	Intermediate	100—½ PMF*
Significant	Small	100-year—½ PMF
	Intermediate	½ PMF—PMF
Large	Small	½ PMF—PMF
	Intermediate	PMF

*Probable maximum flood.
Source: From Army Corps of Engineers [31].

of difficulty in assigning a value to human life where at least significant hazard is involved.

Frequently, two spillway structures will be provided. The first, a service spillway, may be a small overflow concrete structure. This spillway will be designed to pass all small floods which produce no danger of overtopping the dam. A second spillway, an emergency spillway, may be located off the dam and will be designed to supplement the service spillway during the large design flood. Some damage should be expected if the emergency spillway operates.

The simplest and most dependable form of spillway is an uncontrolled crest. If the design discharge for the spillway is large, the required length of the uncontrolled crest may be very long. If the required crest length cannot be developed at the site, it may be necessary to provide gates on the crest which control the flow, except when large flood flows must be passed. Flashboards, stop logs, rectangular gates, and radial gates are commonly used on small dams.

From the standpoint of operation, radial gates are easiest to operate. The resultant of pressure forces acting on the radial gate is normal to the circular surface, thus causing no moment about the trunion. Only the weight of the gate itself must be lifted when the gate is opened. Figure 3.32*a* shows a typical radial gate installation.

Rectangular vertical gates can be constructed as roller gates, which makes it possible to raise them under full hydrostatic pressure. However, slide gates without rollers can be very difficult to operate under pressure because of the large friction forces developed. Figure 3.32*b* shows a typical vertical gate installation.

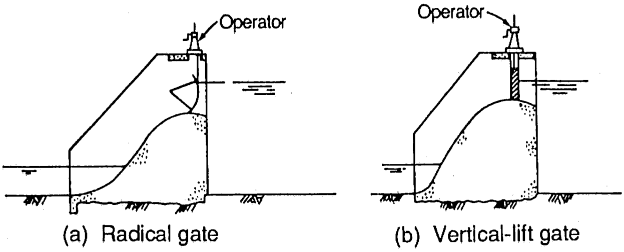


FIGURE 3.32 Gate installations on a concrete spillway. *(From Cassidy [28].)*

Stop logs are narrow rectangular beams which can be placed in slots on the spillway crest to raise the reservoir surface. Because of friction in the slots, they are very difficult to install or remove under overflow conditions. They should not be used in situations where unexpected floods can occur, since advance warning is required to remove the flashboards prior to a flood. Figure 3.33 shows a typical stop-log installation.

Flashboards consist of individual boards which are held on the spillway crest by vertical pipes or columns anchored to the crest. The flashboards can be designed to fail if the level in the reservoir reaches a given level, thus providing an automatic operation. If they are not designed to fail automatically, sufficient warning time must be possible to allow removal of the flashboards before a flood arises.

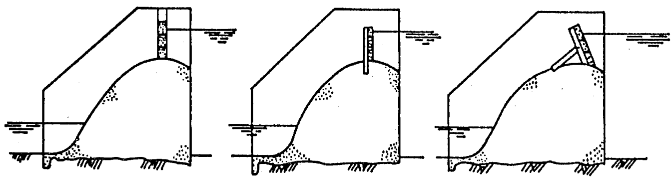


FIGURE 3.33 Flashboard installations on a concrete spillway. *(From Cassidy [28].)*

Riprap will be required in the channel downstream from the spillway to control erosion unless the channel is rock. To prevent erosion of the dam, the spillway should be extended downstream from the toe of the dam a sufficient distance to ensure that velocities near the embankment are below the magnitude which will produce scour. In some cases, riprap protection at the toe of the dam may be required.

3.9 DIVERSION DURING CONSTRUCTION

Except for unusual situations, it will be necessary to divert the natural stream during construction. Diversion through a tunnel driven in one abutment provides the most convenient diversion, since the entire dam site can be excavated and the foundation cleaned in one operation. A cofferdam is constructed on the upstream and downstream side of the site. Once construction on the dam has advanced far enough, the tunnel can be plugged. A concrete bulkhead is usually constructed to make the initial closure on the tunnel. The tunnel can in some cases be used as an outlet works if a concrete plug is constructed in the tunnel with a conduit and valves installed in the plug.

More commonly on small dams, a diversion channel is excavated along one abutment. A cofferdam is constructed upstream and downstream of the site. The upstream cofferdam diverts the natural flow of the stream into the diversion channel. An outlet works is constructed in the first portion of the dam to be completed. Once the dam construction has progressed sufficiently, a new cofferdam is constructed around the remaining site and the dam is completed. Figure 3.34 illustrates this type of diversion in stages. Normally, diversion structures and cofferdams are designed for approximately a 10-year flood. In some cases, the cofferdams are designed to be incorporated into the final section of the dam. Pumping will usually be required to keep the dam site dry, since seepage through and around the cofferdams will be inevitable. The cofferdams, although low, should be considered to be earthfill dams and designed accordingly.

Closure of the final diversion is the most critical operation and should be planned for that time of year when the lowest flow is assured. Closures are frequently accomplished by dumping rock and earth into the channel rapidly. However, it may be easier and more economical to construct a closure structure into which stop logs or a gate can be lowered to close the section.

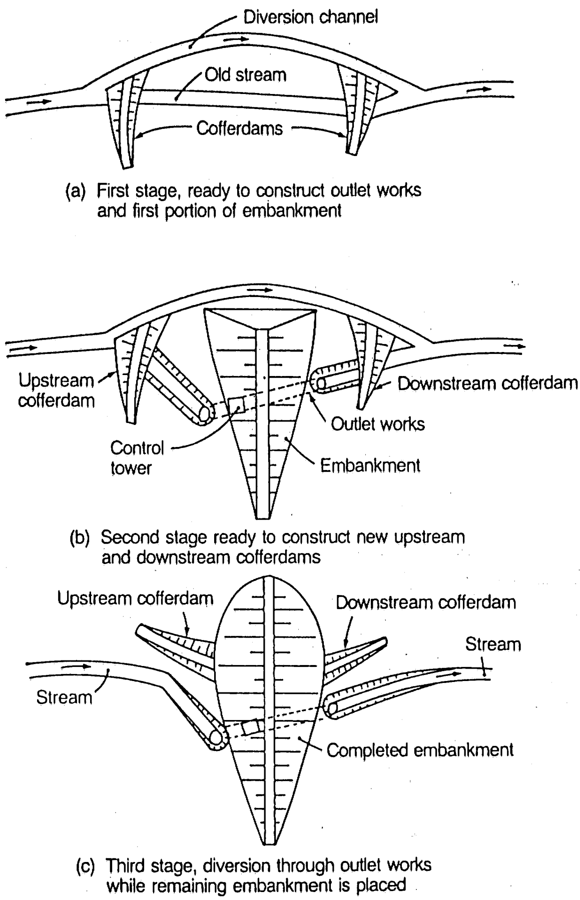


FIGURE 3.34 Typical diversion plans. *(From Cassidy [28].)*

3.10 ACKNOWLEDGMENTS

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CHAPTER 4
HYDRAULIC TURBINES

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4.1 INTRODUCTION

Modern hydraulic turbines are the result of many years of gradual development. Because of economic incentives, large units have been developed which can achieve efficiencies in excess of 96 percent. Based on such achievements, one would assume that hydroturbine technology is in a mature state. However, recent advances in numerical analysis, utilizing large mainframe computers, have resulted in better efficiencies for a variety of turbine types, improved cavitation performance, and improved manufacturing techniques. Unfortunately, smaller units have not benefited to any great degree from the research and development of larger units. Smaller turbines have always been relatively more expensive than large units. To offset the higher cost per installed capacity, manufacturers are now offering a line of standardized machines, with many components being interchangeable among units. The problem is that selection of a turbine for a given application becomes a compromise, since there are a fixed number of sizes to choose from. Recently, computer-aided design (CAD) techniques have matured to the point where standardized designs can be scaled up or down to custom-tailor a given turbine design to the prescribed conditions of head and flow at a given site. This improvement, combined with modern computer-aided manufacturing (CAM), allows for delivery of standardized design, but custom-tailored turbines in about the same time as standardized turbines manufactured by older methods.

The emphasis on the design and manufacture of very large turbines is shifting toward the production of medium to small turbines, especially in developed nations, where much of the potential for large base-load plants has been realized. A new market area is also developing for refurbishing older turbines with replacement runners having greater capacity and improved efficiency.

The recent rapid escalation in the cost of energy has made many smaller sites economically feasible and has greatly expanded the market for small turbines. The increased value of energy also justifies the added cost of increasing the capacity of existing plants. Added to this is the rapidly increasing need for turbines in the less-developed nations where hydropower represents an attractive source of energy that can be easily developed. Because there are differences in the design objective for refurbished units and new units of various sizes, it is anticipated that new developments will occur relatively rapidly as the marketplace creates new competitive pressures and opportunities for manufacturers.

Since we are currently in transition from a relatively stagnant period of design and development to a renaissance of turbine development, this chapter is structured in such a way that the reader is provided with the basics of turbine performance as well as with a summary of turbine characteristics which affect the operation of a hydropower site. Cookbook procedures for turbine selection are avoided. Instead, a firm grounding in the basic principles of turbine hydrodynamics is given, which should provide the reader with the necessary tools to not only select a turbine from the current offerings but also to provide the ability to evaluate new turbine types which may appear in the future.

4.2 HISTORICAL PERSPECTIVE

Evolution of the Modern Turbine from the Water Wheel

The water turbine has a rich and varied history [1, 2]. It has been developed as a result of a natural evolutionary process from the waterwheel. Originally used for direct drive of machinery, the use of the water turbine for the generation of electricity is a comparatively recent activity. Much of its development occurred in France which, unlike England, did not have the cheap and plentiful sources of coal which sparked the industrial revolution in the eighteenth century. Nineteenth-century France found itself with its most abundant energy resource being water. To this day, *houille blanche* (literally "white coal") is the French term for water power.

In 1826 the Société d'Encouragement pour l'Industrie Nationale offered a prize of 6000 francs to anyone who "would succeed in applying at large scale, in a satisfactory manner, in mills and factories, the hydraulic turbines or wheels with curved blades of Bélidor" [2]. Bélidor was an eighteenth-century hydraulic and military engineer who, in the period 1737 to 1753, authored a monumental four-volume work, *Architecture Hydraulique*, a descriptive compilation of hydraulic engineering information of every sort. The waterwheels described by Bélidor departed from convention by having a vertical axis of rotation and being enclosed in a long cylindrical chamber approximately one meter in diameter. Large quantities of water were supplied from a tapered sluice at a tangent to the chamber. The water entered with considerable rotational velocity. This preswirl, combined with the weight of water above the wheel, was the driving force. The original tub wheel had an efficiency of only 15 to 20 percent.

Water turbine development proceeded on several fronts during the period 1750 to 1850. The classical horizontal-axis waterwheel was improved by such engineers as John Smeaton (1724–1792) of England and the French engineer J. V. Poncelet (1788–1867). This resulted in waterwheels having efficiencies in the range of 60 to 70 percent. At the same time, reaction turbines (somewhat akin to the modern lawn sprinkler) were being considered by several workers. The great Swiss mathematician Leonhard Euler (1707–1783) investigated the theory of operation of these devices. A practical application of the concept was introduced in France in 1807 by Mannoury de Ectot (1777–1822). His machines were, in effect, radially outward-flow machines. The theoretical analyses of Burdin (1790–1893), a French professor of mining engineering who introduced the word "turbine" in engineering terminology, contributed much to our understanding of the principles of turbine operation and underscored the principal requirements of shock-free entry and exit with minimum velocity as the basic requirements for high efficiency. A student of Burdin, Benoit Fourneyron (1802–1867), was responsible for putting his teacher's theory to practical use. His work led to the development of high-speed outward-flow turbines with efficiencies of the order of 80 percent. The early work of Fourneyron resulted in several practical applications and the winning of the coveted 6000-franc prize in 1833. After nearly a century of development, Bélidor's tub wheel had been officially improved.

Fourneyron spent the remaining years of his life developing some 100 turbines in France and Europe. Some turbines even found their way to the United States, the first in about 1843. The Fourneyron centrifugal turbines were designed for a wide range of conditions, with heads as high as 114 m and speeds as high as 2300 r/min. Very low head turbines were also designed and built.

As successful as the Fourneyron turbines were, they lacked flexibility and were only efficient over a narrow range of operating conditions. This problem was addressed by S. Howd and U. A. Boyden (1804–1879). Their work evolved into the concept of an inward-flow motor as a result of the work of James B. Francis (1815–1892). The modern Francis turbine is the result of this line of development. At the same time, European engineers addressed the idea of axial-flow machines, which today are represented by "propeller" turbines of both fixed-pitch and Kaplan types.

Just as the vertical-axis tub wheels of Bélidor evolved into modern reaction turbines of the Francis and Kaplan types, development of the classical, horizontal-axis waterwheel reached its peak with the introduction of the impulse turbine. The seeds of development were sown by Poncelet in 1826 with his description of the criteria for an efficient waterwheel. These ideas were cultivated by a group of California engineers in the late nineteenth century, one of whom was Lester A. Pelton (1829–1908), whose name is given to the Pelton wheel, which consists of a jet or jets of water impinging on an array of specially shaped buckets closely spaced around the periphery of a wheel. Thus, it can be said that the relatively high speed reaction turbines trace their roots to the vertical-axis tub wheels of Bélidor, whereas the Pelton wheel can be considered as a direct development of the more familiar horizontal-axis waterwheel. Turbine configurations as we know them today are generally in the form as originally developed. For example, the overwhelming majority of Pelton wheels have horizontal axes. Vertical-axis Pelton wheels are a relatively recent development. In over 250 years of development, many ideas were tried. Some were rejected and others were retained and incorporated in the design of the hydraulic turbine as we know it today. This development has resulted in highly efficient devices, with efficiencies as high as 96 percent in the larger sizes. In terms of design concept, these fall into roughly three categories: reaction turbines of the Francis or propeller design and impulse wheels of the Pelton type. The rest of this chapter is devoted to a review of the principles of operation, the classification and selection of turbines for given operating conditions, and a review of performance characteristics and operational limitations. Most of the development efforts to date have been placed on large turbines, with small-turbine technology consisting chiefly of scaling down larger turbines. The validity of this concept is reviewed, and areas where improvements can be made are addressed.

4.3 BASIC PRINCIPLES

Euler's Equation

The torque on the runner of a turbine can be found through conservation of radial momentum. In other words, the torque on a runner is the difference between the rate of angular momentum entering the runner and that exiting. Referring to Fig. 4.1, this can be written as

$$T = \rho Q(r_1V_1 \cos \alpha_1 - r_2V_2 \cos \alpha_2) \tag{4.1}$$

where ρ is the fluid density and Q is the volumetric rate of flow. Since the power produced is proportional to the product of mass flow rate and head, we can write the following:

$$T\Omega = \rho gQH_u \tag{4.2}$$

$$\Omega r_1 = u_1 \tag{4.3}$$

$$\Omega r_2 = u_2 \tag{4.4}$$

Thus,

$$H_u = \frac{u_1V_1 \cos \alpha_1 - u_2V_2 \cos \alpha_2}{g} \tag{4.5}$$

where Ω is the rotational speed in radians per second, T is the torque, and H_u is the head utilized. We must be careful to keep in mind that V_1 and V_2 are absolute quantities, whereas u_1 and u_2 are the peripheral speeds at entrance and exit, respectively. In a fixed frame of reference, the absolute velocity **V** is related to the vector sum of the relative velocity **v** and the velocity of a body moving with velocity **u**. In vector notation:

$$\mathbf{V} = \mathbf{u} + \mathbf{v} \tag{4.6}$$

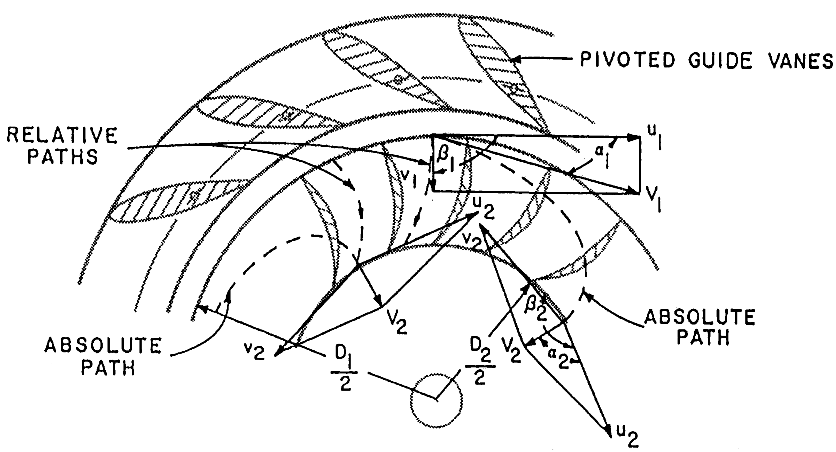


FIGURE 4.1 Definition sketch for radial flow turbine runner.

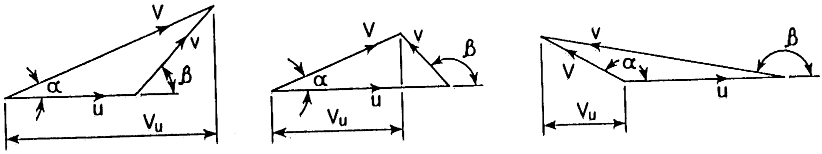


FIGURE 4.2 Typical velocity triangles.

We will define the angles α and β as, respectively, the angles made by the absolute and relative velocities of a fluid with the linear velocity u of some body. This is illustrated in Fig. 4.2. It is obvious from inspection of the figures that

$$V \sin \alpha = v \sin \beta \tag{4.7}$$

$$V \cos \alpha = u + v \cos \beta \tag{4.8}$$

The energy equation written between two points is given by

$$\left[\frac{p_1}{\gamma} + z_1 + \frac{V_1^2}{2g}\right] - \left[\frac{p_2}{\gamma} + z_2 + \frac{V_2^2}{2g}\right] = H_L + \frac{u_1 V_1 \cos \alpha_1 - u_2 V_2 \cos \alpha_2}{g} \tag{4.9}$$

where γ is the specific weight, ρg, H_L represents frictional losses and the last term on the right-hand side of the equation represents the head absorbed by the turbine. It follows from Eqs. (4.7) and (4.8) that

$$V^2 = v^2 + u^2 + 2uv \cos \beta \qquad (4.10)$$

$$uV \cos \alpha = u(u + v \cos \beta) \qquad (4.10)$$

Combining Eqs. (4.9), (4.10), and (4.11) results in the so-called energy equation in a rotating frame of reference.

$$\left[\frac{p_1}{\gamma} + z_1 + \frac{v_1^2 - u_1^2}{2g}\right] - \left[\frac{p_2}{\gamma} + z_2 + \frac{v_2^2 - u_2^2}{2g}\right] = H_L \tag{4.11}$$

Note that if there is no flow, $v_1 = v_2 = 0$ and the equation reduces to that for a vortex. If there is no rotation, the equation reduces to the familiar form of the energy equation.

Turbine Efficiency and Losses

Definitions. The hydraulic efficiency of a turbine is defined by

$$\eta_h = \frac{H_u}{H} \tag{4.12}$$

where H_u is the head utilized by the runner and H is the net head on the turbine, defined as the difference between the total head at the entrance to the turbine proper (entrance to the spiral casing) and the total head at the tailrace. The definition of net head is illustrated in Fig. 4.3.

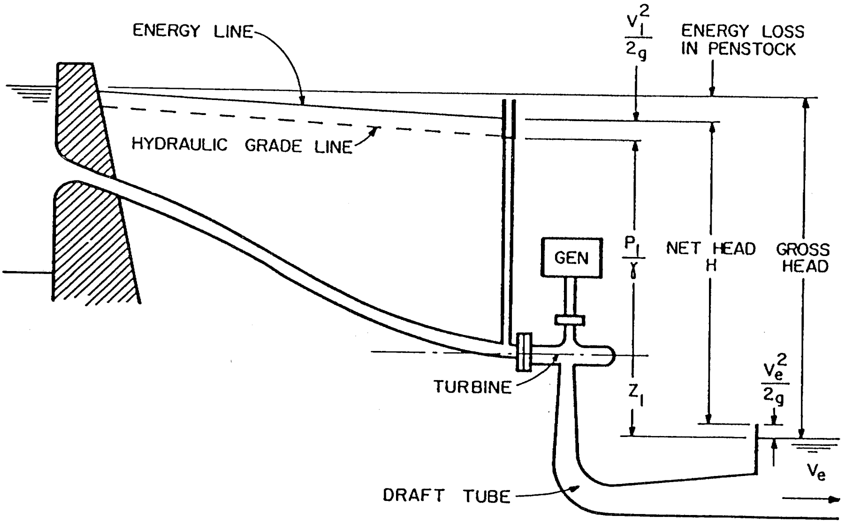


FIGURE 4.3 Definition sketch for net head.

It should be mentioned that different definitions exist for net head depending on the value of $V_e^2/2g$ that is used. In the International Electrotechnical Test Code [3], $V_e^2/2g$ is defined as the velocity head at the draft tube exit, whereas the ASME Power Test Code [4] uses the velocity head of the flow in the tailrace. The differences can be substantial, with calculated efficiencies varying by as much as 3 percent using different definitions of net head. This will be discussed further under draft tubes. The hydraulic efficiency expresses the effectiveness of the transfer to the runner of the available power in the fluid that flows through it. Also illustrated in Fig. 4.3 is the definition of gross head, the difference between headwater and tailwater elevations.

The volumetric efficiency η_v of a turbine is defined as the ratio of the rate of flow that effectively acts on the rotor Q_e to the total rate of flow through the turbine. That is,

$$\eta_v = \frac{Q_e}{Q} = \frac{Q - Q_L}{Q} \tag{4.14}$$

where Q_L represents leakage around the runner, which produces no work. The efficiency η_v is a measure of the effectiveness of the turbine seals.

Finally, the mechanical efficiency is defined as the ratio of the power available at the shaft of the machine to the power transmitted from the water to the runner. That is,

$$\eta_m = \frac{\text{bp}}{\text{bp} + \text{fp}} \tag{4.15}$$

where bp is the brake power (available at the shaft) and fp represents the power consumed by mechanical friction and by viscous losses other than losses in the turbine passages. These losses include friction in the bearings and stuffing boxes and the so-called disk friction, the viscous losses in the fluid that fills the space between the runner and the adjacent casing.

The overall efficiency of the turbine is the product of the three efficiencies

$$\eta = \eta_h \eta_v \eta_m \tag{4.16}$$

so that, by definition, the power developed by the turbine is given by

$$P = \eta\gamma QH \tag{4.16a}$$

Energy Losses. The differences between the head H_u utilized by the runner, as given by Eq. (4.5), and the net head H on the turbine, defined in Fig. 4.3, is made up of viscous losses in various parts of the installation. These include losses in the spiral casing, in the guide vanes and runner passages of the turbine, and in the draft tube; shock losses at the entrance to the runner if the relative velocity of the fluid leaving the guide vanes is changed abruptly in magnitude and/or direction as it enters the runner; and the losses at submerged discharge from the draft tube into the tailrace.[1] Draft tubes are considered integral parts of the turbine and their performance is a contributing factor to the hydraulic efficiency η_h defined by Eq. (4.13).

Viscous losses in fluid machinery are complex. Typically, viscous effects are concentrated in a relatively thin region (a boundary layer) adjacent to surfaces of flow passages, stay vanes, wicket gates, etc. Skin friction is important where the boundary layer remains attached to the bounding surface. In regions of adverse pressure gradient (pressure increasing in the direction of flow) boundary layer separation can occur, producing relatively large regions of highly turbulent flow which results in form losses. Boundary layer theory is discussed in detail by Schlichting [5]. The combination of skin friction and form losses account for the so-called energy losses in the turbine. When a turbine is operating near its design point, boundary layer separation is negligible, and the dominant flow characteristics in the flow passages can be calculated, neglecting viscous effects. Modern numerical schemes are used on mainframe computers. These calculations have led to new runner designs having significantly higher efficiencies than older designs. Unfortunately, off-design performance cannot be calculated with confidence since boundary layer separation occurs, which leads to highly complex flow patterns that defy accurate description with existing numerical schemes. Recently, many accounts of numerical flow calculations have appeared in the literature [6]. Model testing is still an important factor in the development of new designs. This will be discussed later in this section.

An important element of a turbine installation is the draft tube, which is used to recover as much of the velocity head of the water as possible as it leaves the runner. The recovery of the velocity head is very important at high specific speeds because the velocity head at the exit from the runner may be as much as 50 percent of the net head H of the installation. (For low specific speeds, the velocity head is of the order of 3 to 4 percent of H.) It can be shown that the fraction of net head remaining as velocity head in the runner discharge scales with the square of specific speed [7, 8]. An efficient recovery within the draft tube allows a decrease of the exit diameter D_2 of the turbine, and thus has a direct bearing on the cost of the turbine. A description of typical draft tube geometries is presented later. Diffuser performance is briefly discussed here. The efficiency of a diffuser η_D is defined in terms of the piezometric head difference between inlet (subscript

[1]This last item may or may not be included in the computed efficiency of the turbine depending on the power test code utilized, as discussed above.

1) and exit (subscript 2) normalized with respect to the pressure recovery in an inviscid flow:

$$\eta_D = \frac{p_2 - p_1}{\frac{1}{2}\rho(V_1^2 - V_2^2)} \tag{4.17}$$

which implies

$$\gamma H_{L_{1-2}} = (1 - \eta_D)\ \tfrac{1}{2}\rho(V_1^2 - V_2^2) \tag{4.18}$$

For straight diffusers with circular cross sections there exists an optimum included angle of divergence, 2α, for given area ratio A_2/A_1 and Reynolds number at entry, that yields a maximum η_D. Obviously, for large angles of divergence, separation of the boundary layer fluid produces large losses. For very small included angles, the length of the diffuser becomes considerable and the losses increase again. The optimum angle lies between $2\alpha = 4°$ and $8°$, and decreases as Re increases.

The efficiency η_D depends strongly on the boundary layer thickness at entry. For $2\alpha = 8°$, $\eta_D \sim 0.9$ for boundary layer thicknesses of the order of 0.5 percent of $D_1/2$, and decreases to $\eta_D \sim 0.7$ for thicknesses on the order of 5 percent of $D_1/2$. For draft tubes, specifically, it has been found that for angles of divergence 2α that are somewhat larger than the optimum, an improvement in efficiency for Kaplan and propeller turbines is obtained by designing for a small swirl component at the entrance to the diffuser in the same direction as the runner rotation. This permits somewhat large divergence angles and reduces the excavation depth. For curved diffusers, the efficiency decreases markedly with deflection angle. Generally, it is best to turn the flow without an area increase and obtain the pressure recovery in straight diffuser sections.

Similarity Considerations

Similitude Theory. The grouping of parameters brought about by dimensional or inspectional analysis permits the writing of any physical relationship in terms of fewer dimensionless quantities representing ratios of significant forces for the problem. This provides a method to extrapolate model test data to prototype situations by equating corresponding dimensionless numbers. In addition, similarity considerations applied to hydraulic machinery provide an answer to the following important question: Given test data on the performance characteristics of a certain type of machine under certain operating conditions, what can be said about the performance characteristics of the same machine, or of a geometrically similar machine, under different operating conditions? Similarity considerations provide, in addition, a means of cataloging machine types and thus aid in the selection of the type suitable for a particular set of conditions.

The problem of similarity of flow conditions can be summarized as follows: Under what conditions will geometrically similar flow patterns with proportional velocities and accelerations occur around or within geometrically similar bodies? Obviously, the forces acting on corresponding fluid masses must be proportionally related, as are the kinematic quantities, so as to ensure that the fluid will follow geometrically similar paths. An answer to this question can be obtained by examining the fundamental laws of motion and identifying the relevant forces. While these laws cannot yet be used to predict theoretically the flow conditions in a machine with unknown performance characteristics, the information they provide on forces and boundary conditions enables the determination of an answer to the similitude problem.

Similarity of the velocity diagrams at the entrance to the runner is a necessary requirement. Referring to Fig. 4.1, the ratio V_1/u_1 must be held constant, assuming equal angles α_1 in model and prototype. If V_n denotes the radial components of the velocity (normal to the flow passages), we have

$$Q_e = f_b \pi \frac{B}{D_1} V_n D_1^2 \tag{4.19}$$

where f_b represents the fraction of free space in the inlet passages of the runner ($f_b \sim 0.95$) and B is the width of the passages. With

$$V_n = V_1 \sin \alpha_1 = v_1 \sin \beta_1 \tag{4.19a}$$

Eq. (4.19) becomes

$$Q_e = \left(f_b \pi \frac{B}{D_1} \sin \alpha_1\right) V_1 D_1^2 \tag{4.20}$$

Since

$$u = \Omega r = \frac{\pi n}{60} D$$

we have

$$\frac{V_1}{u_1} = \frac{1}{f_b \pi \frac{B}{D_1} \sin \alpha_1} \frac{Q_e}{u_1 D_1^2} = \frac{1}{f_b \pi \frac{B}{D_1} \sin \alpha_1} \frac{60}{\pi} \frac{Q_e}{n D_1^3} \tag{4.21}$$

Constancy of the ratio Q_e/nD^3 or $Q_e/\Omega D^3$ is then a necessary condition for similarity.

If the assumption is made that viscous forces are small relative to inertia forces and thus can be neglected in first approximation, and furthermore, that the fluid does not change its physical properties as it passes through the machine (which excludes compressibility effects and cavitation, to be dealt with later), the only other forces that appear in the fundamental equations of motion are the pressure forces. Their ratio to inertia forces is proportional to $\Delta p/\rho V^2$ or, if the head H_u utilized by the runner is introduced, to gH_u/V^2. Under the assumptions made, the condition

$$\frac{Q_e}{\Omega D^3} = \text{constant} \tag{4.22a}$$

is sufficient for similarity. The condition

$$\frac{gH_u}{V^2} = \text{constant or } \frac{gH_u}{\Omega^2 D^2} = \text{constant} \tag{4.22b}$$

follows from the basic laws and permits calculation of the head H_u for similar operating conditions.

Velocity coefficients, Φ, C_1, and C_2 are customarily introduced as

$$u_1 = \Phi\sqrt{2gH} \qquad V_1 = C_1\sqrt{2gH} \qquad V_2 = C_2\sqrt{2gH} \tag{4.23}$$

In terms of these coefficients, the hydraulic efficiency η_h defined by Eq. (4.13) can be written with the use of Eq. (4.5) as

$$\begin{aligned}\eta_h &= \frac{u_1 V_1 \cos\alpha_1 - u_2 V_2 \cos\alpha_2}{gH} \\ &= 2\Phi\left[C_1 \cos\alpha_1 - \frac{D_2}{D_1} C_2 \cos\alpha_2\right]\end{aligned} \tag{4.24}$$

If the viscous losses embodied in η_h can be assumed to occur under hydrodynamically rough conditions, in the sense that the losses are independent of Reynolds number Re and depend only on geometric ratios and relative roughness (Re must be high enough for the losses to be purely turbulence-controlled), then η_h must be the same in model and prototype, provided that the relative roughnesses are the same and the geometry is faithfully reproduced. Under these conditions, Eqs. (4.22*a* and *b*) become

$$\frac{gH}{\Omega^2 D^2} = \text{constant} \tag{4.25a}$$

Analogous considerations can be made about the volumetric efficiency η_v. Here Reynolds number effects may be more significant due to the smallness of the leakage-flow passages. But if one can assume Re independence and under strict geometric similarity (including surface roughness and running clearances), η_v must be the same in model and prototype and Eq. (4.22*a*) becomes

$$\frac{Q}{\Omega D^3} = \text{constant} \tag{4.25b}$$

Equations (4.25*a* and *b*) permit calculation of the net head H and total flow rate Q for similar operating conditions.

Specific Speed. Similar flow conditions are ensured by the constancy of the ratio $Q/\Omega D^3$, which implies constancy of the ratio $gH/\Omega^2 D^2$. In other words,

$$\frac{gH}{\Omega^2 D^2} = f\left(\frac{Q}{\Omega D^3}\right) \tag{4.26}$$

This relationship can also be written in terms of a third dimensionless number which does not involve the representative dimension D of the machine and which can replace either of the two arguments in Eq. (4.26). Such a number can be obtained by appropriate multiplication of powers of the dimensionless numbers in Eq. (4.26).

$$N_{sQ} = \left(\frac{Q}{\Omega D^3}\right)^{1/2}\left(\frac{\Omega^2 D^2}{gH}\right)^{3/4} = \frac{\Omega Q^{1/2}}{(gH)^{3/4}} \tag{4.27}$$

This dimensionless number is called the "specific speed." For hydraulic turbines, however, the definition of the specific speed is often based on the power P delivered by the turbine as a variable, instead of the flow rate Q. The corresponding dimensionless number for P is $P/\rho\Omega^3 D^5$, which is a function of $gH/\Omega^2 D^2$.

Eliminating D between these two numbers, one gets

$$N_s = \frac{\Omega(P/\rho)^{1/2}}{(gH)^{5/4}} \tag{4.28}$$

The two specific speeds are related by

$$N_s = \sqrt{\eta}\, N_{s_Q} \tag{4.29}$$

If we choose N_s as the independent variable in these relationships, then all other dimensionless combinations can be expressed as functions of N_s. These include also the dimensionless torque, $T/\rho\Omega^2 D^5$, and the efficiencies, under the assumption of negligible scale effects.

The specific speed describes a specific combination of operating conditions that ensures similar flows in geometrically similar machines. Thus, it has attached to it a specific value of the efficiency η (assumed approximately constant for similar flow conditions regardless of size). It is then customary to label each series of geometrically similar turbines by the value of N_s which gives maximum η for the series. Unless otherwise stated, this is the N_s value referred to when the term "specific speed" is used. The value of N_s thus defined permits the classification of turbines according to efficiency. Each geometric design has a range of N_s values where it can be used with only the value of N_s corresponding to peak efficiency being specified. In subsequent sections this idea will be used to classify turbine designs.

The parameter N_s as defined is dimensionless. It is common in practice to drop g and ρ from the definition and define n_s as

$$n_s = \frac{n\sqrt{P}}{H^{5/4}} \tag{4.30}$$

or equivalently

$$n_{s_Q} = \frac{n\sqrt{Q}}{H^{3/4}} \tag{4.30a}$$

with n in revolutions per minute. In USCS units, P is in horsepower, and H is in feet. In metric units, P is either in metric horsepower or kilowatts, and H is in meters. The relationship between various definitions of dimensional n_s and dimensionless N_s are

$$\begin{aligned} n_s &= 43.5N_s \text{ (USCS units)} \\ n_s &= 193N_s \text{ (metric units using metric horsepower)} \\ n_s &= 166N_s \text{ (metric units using kilowatts)} \\ n_{s_Q} &= 52.9N_{s_Q} \text{ (metric units)} \\ n_{s_Q} &= 129N_{s_Q} \text{ (USCS units)} \end{aligned} \tag{4.31}$$

Scaling Formulas. The similarity arguments used to arrive at the concept of specific speed indicate that a given machine of diameter D operating under a head H will discharge a flow Q and produce torque T and power P at a rotational speed Ω, all given by

$$Q = Q_{11}D^2\sqrt{2gH} \tag{4.32}$$

$$T = T_{11}\gamma D^3 H \tag{4.33}$$

$$P = P_{11}\gamma D^2\sqrt{2gH^3} \tag{4.34}$$

$$\Omega = \Omega_{11}\frac{\sqrt{2gH}}{D} \tag{4.35}$$

In theory, the coefficients Q_{11}, T_{11}, D_{11}, and Ω_{11} are fixed for a given machine operating at a fixed specific speed, independent of the size of the machine. Thus, in principle, the measured operating characteristics of a small machine or model can be scaled up to predict the performance of a large machine, using Eqs. (4.32) to (4.35). The subject of scale effects or deviations from these similarity laws is discussed in the section on model tests.

Cavitation

"Cavitation" can be defined as the formation of the vapor phase in a liquid flow when the hydrodynamic pressure falls below the vapor pressure of the liquid. It is distinguished from boiling, which is due to the vapor pressure being raised above the hydrodynamic pressure by heating. Cavitation will occur at any point where the pressure falls below vapor pressure. The critical pressure for incipient cavitation is usually lower than the vapor pressure, the actual values depending on the size and number of small gas bubbles (nuclei) present in the liquid. This factor is especially important for cavitation tests at model scale [9, 10] but it generally is not significant in prototype machines. The degree of cavitation will depend on the setting of the machine, i.e., the elevation of the machine relative to tailwater elevation (see Fig. 4.3). As will be discussed subsequently, a model turbine test stand can simulate, under controlled conditions, the performance of a turbine at varying settings ranging from cavitation-free operation to well-developed cavitation within the unit. To fix ideas, consider the total energy at the minimum pressure point in a machine, given by

$$\frac{V^2}{2} + \frac{p_m}{\rho} + gz = \text{total energy per unit mass}$$

For convenience, z is measured relative to the tailwater elevation. At constant head, the total energy at any point in the machine will remain fixed. Thus, increasing z results in decreasing p_m. When $p_m < p_v$, cavitation occurs. When cavitation is incipient, or limited, individual bubbles form which are swept away and collapse in regions of higher pressure. Calculations, as well as sophisticated laboratory experimentation, indicate that collapsing bubbles create very high impulsive pressures. This results in substantial noise (a cavitating turbine sounds like gravel is passing through it). More important, the repetitive application of the shock loading due to bubble collapse at liquid-solid boundaries results in pitting of the surface. As the process continues, cracks form between the pits and solid material is spalled out from the surface. The mechanical effects of cavitation are enhanced by the high temperatures created by collapsing bubbles and the presence of oxygen-rich gases which come out of solution. The details of the erosion process are complex, but the results are of practical significance. Many compo-

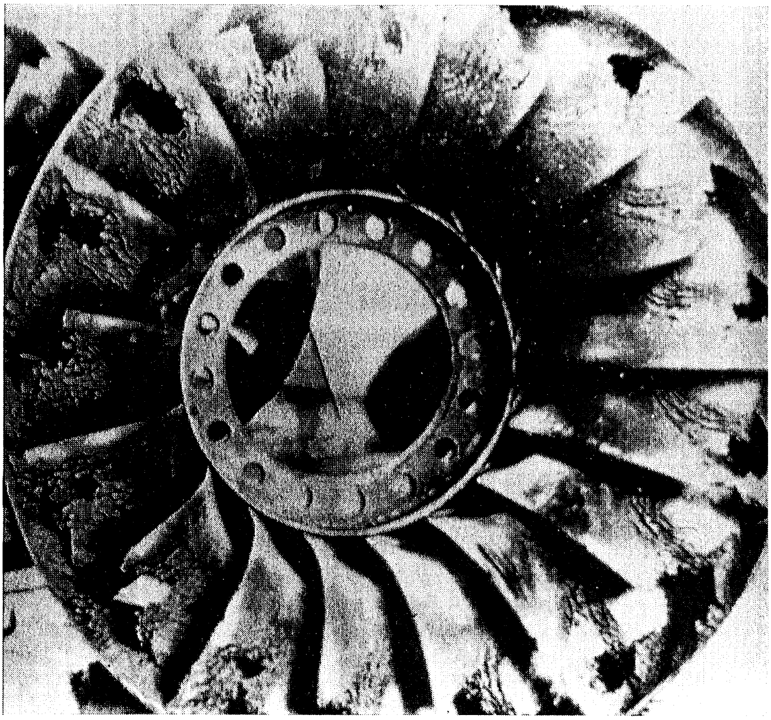


FIGURE 4.4 Cavitation erosion on a turbine runner.

nents of a turbine are susceptible to extensive damage, as illustrated in Fig. 4.4. As the setting z is increased further, more developed forms of cavitation occur, since the pressure is below critical in a larger portion of the turbine. Large vapor-filled cavities remain attached to the surface of the runner, and in extreme cases to the surfaces of wicket gates, stay vanes, etc. Each cavity or pocket is formed by the liquid flow detaching from the rigid boundary of an immersed body or flow passage. The maximum length of a fixed cavity depends on the shape of the flow passage. Termination of the cavities may occur by reattachment of the liquid stream at a downstream position on a solid surface or the cavity may extend well beyond the body. The latter case is known as "supercavitation." Under these circumstances, the pressure distribution on the boundary can be substantially altered. If developed cavitation occurs on the runner or wicket gates of the turbine, the performance of the unit is substantially reduced. Cyclical growth and collapse of the cavities can also occur, producing vibration. Thus cavitation can degrade performance and produce vibration, as well as reduce the operational lifetime of the machine through erosion.

The Cavitation Index. The fundamental parameter in the description of cavitation is the cavitation index:

$$\sigma = \frac{p_o - p_v}{\frac{1}{2}\,\rho V_o^2} \tag{4.36}$$

A given state of cavitation is assumed to be a unique function of σ for geometrically similar bodies. If σ is greater than a critical value, say σ_c, there is no cavitation and the various hydrodynamic parameters are independent of σ. When σ is less than σ_c, various hydrodynamic parameters such as the lift and drag of various components and the power and efficiency of a turbine are functions of σ. Noise, vibration, and erosion also scale with σ. It should be emphasized that the value of σ where there is a measurable change in performance is *not* the value of σ where cavitation can first be determined visually or acoustically. The critical value of σ_c can be thought of as a performance boundary such that

$\sigma > \sigma_c$	no cavitation effects
$\sigma < \sigma_c$	cavitation effects: performance degradation, noise, and vibration

There are two ways of defining the critical value of sigma. The incipient cavitation number σ_i is normally determined in a test facility by lowering the static pressure at constant velocity until cavitation first occurs. A more repeatable parameter is the desinent cavitation index σ_d, which is based on the static pressure necessary to extinguish cavitation after the pressure has been lowered sufficiently for cavitation to occur. The precise value of σ_c defined by inception or desinence is normally only determined in the laboratory. To avoid confusion, σ_c will be used to define the boundary between cavitation and noncavitating flow. It should not be confused with more pragmatic definitions such as the value of the cavitation index at a measurable change in hydraulic performance expressed by power, capacity, or efficiency, or at a measurable change in vibration level.

Thoma's Sigma. The flow in a turbine is obviously complex and not easily quantified. There still is, however, a definite need to define operating conditions with respect to cavitation. For example, it is sometimes necessary to specify under what conditions the degree of cavitation will be the same for the same machine operating under different heads and speeds, or for two machines of similar design but different size, e.g., a model and a prototype. The accepted parameter for this purpose is the Thoma sigma, σ_T. A definition for σ_T follows naturally from the definition for σ, since $\frac{1}{2}\rho U^2 \sim \gamma H$:

$$\sigma_T = \frac{H_{sv}}{H} \tag{4.37}$$

where H_{sv} is the net positive suction head. Referring to Fig. 4.3, this is defined as

$$H_{sv} = H_a - z - H_v + \frac{V_e^2}{2g} + H_l \tag{4.38}$$

where H_a is the atmospheric pressure head, z is the elevation of a turbine reference plane above the tailwater level, V_e the average velocity in the tailrace, and H_l the headloss in the draft tube. If we neglect the draft tube losses and the exit velocity head, Thoma's sigma is

$$\sigma_T = \frac{H_a - H_v - z}{H} \tag{4.39}$$

Cavitation Inception. Cavitation inception is assumed to occur at any given position when the pressure at that point falls below the vapor pressure of the liquid. This is not quite true. Actually, the critical pressure for cavitation to occur is a function of the amount of free and dissolved gas in a liquid. This is an important consideration for model testing, but generally speaking, the vapor pressure of the liquid is an acceptable measure of the critical pressure for most prototype situations.

It can be shown that the lowest pressure in any flow is at a boundary surface, if viscous effects (i.e., turbulence and vorticity) can be neglected. For steady flow, this implies

$$\sigma_c = -C_{pm} \tag{4.40}$$

where C_{pm} is the minimum value of the pressure coefficient defined by $C_p \equiv (p - p_o)/\frac{1}{2}\rho U_o^2)$, where p is the pressure at a given position on the surface of the body. In the case of a blade section or a strut, the value of C_{pm} depends on both the shape and angle of attack, α. Inception on hydrofoils has been studied by numerous investigators [11–13]. Typical data are shown in Fig. 4.5. Similar trends in cavitation characteristics are evident in the cavitation data for a turbine runner when properly interpreted. For a fixed wicket gate setting, the relative angle of attack of the runner blades of a turbine is a function of the flow coefficient, $Q/\Omega D^3$. Cavitation inception for a propeller turbine at a fixed wicket gate setting is shown schematically in Fig. 4.6. It is no accident that Figs. 4.5 and 4.6 are qualitatively similar.

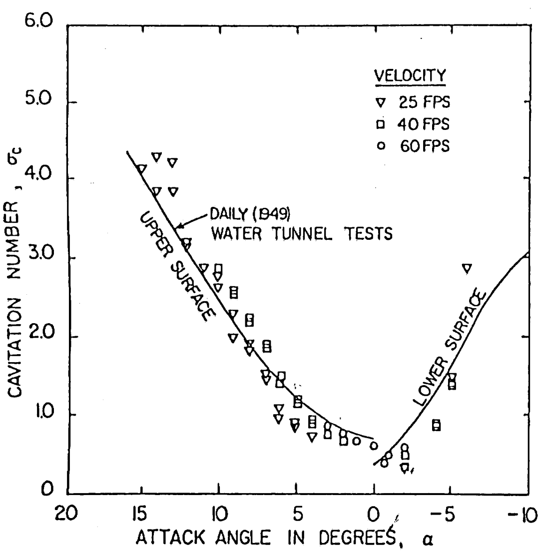


FIGURE 4.5 Cavitation inception characteristics of a NACA 4412 hydrofoil. *(Adapted from Kermeen [11].)*

Great strides have been made in numerical analysis of the flow in turbines. This permits the calculation of C_{pm} and an evaluation of the inception point for a turbine under various operating conditions. Results are encouraging when computed and measured values of σ_i are compared for the machine at its best oper-

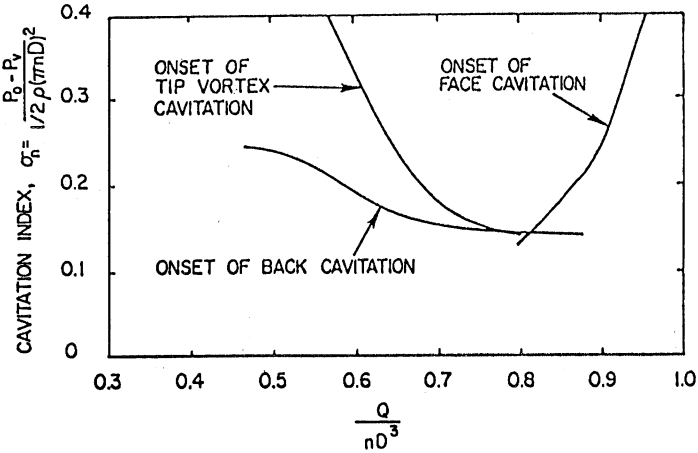


FIGURE 4.6 Cavitation inception characteristics of a typical propeller turbine.

ating point [14]. Results are not as good for off-design operation. This is due to flow separation and vortex formation.

Turbulence, surface roughness, and vortex formation are factors that complicate the inception problem, because minimum pressure is not necessarily as it is under idealized conditions at a flow boundary. Thus σ_i is usually greater than $-C_{pm}$ measured at the surface. Some idealized examples from laboratory studies are presented in Fig. 4.7. Figure 4.7*a* is a photo of jet cavitation. In this example, the mean pressure is well above vapor pressure. Cavitation occurs due to the *instantaneous* fluctuations in pressure below vapor pressure because of turbulence. Figure 4.7*b* shows cavitation in the vortex formed at the tip of a hydrofoil. Note that the surface of the hydrofoil is free of cavitation. The strong, swirling flow in the core of the vortex produces very low pressures. Vortex cavitation is an important phenomenon. Cavitation can occur in the vortices formed in the clearance passages of propeller turbines, in the flow passages of Francis turbines in off-design operation, or at the hub or in the draft tube of both propel-

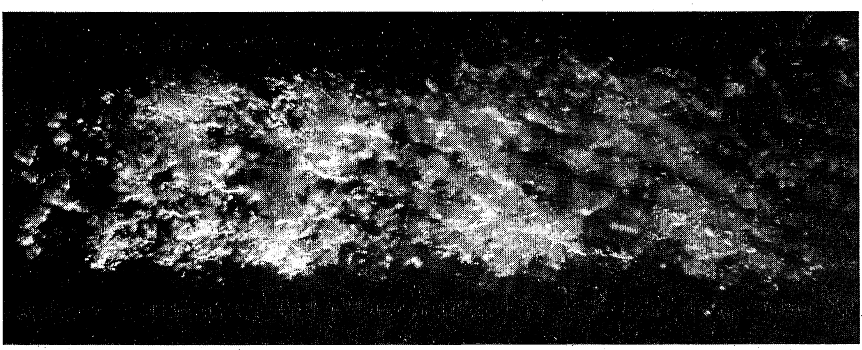


FIGURE 4.7(*a*) Some examples of cavitation in laboratory-scale flows. Cavitation in a turbulent jet.

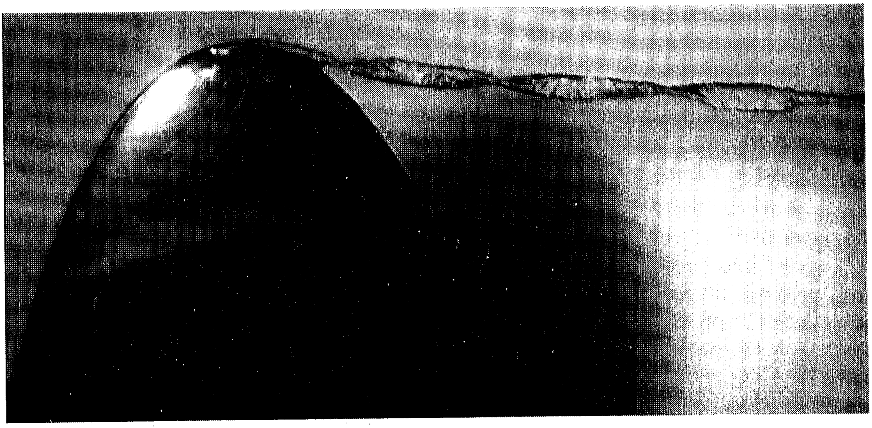


FIGURE 4.7(*b*) Some examples of cavitation in laboratory-scale flows. Cavitation in a tip vortex.

ler and Francis turbines. The minimum pressure is governed by the circulation Γ and core radius r_c:

$$C_{pm} = -2\left[\frac{\Gamma}{2\pi r_c V_o}\right]^2 \tag{4.41}$$

where V_o is a reference velocity. Unfortunately, it is difficult to quantify the circulation and the core radius. In the case of a hub vortex, Γ is controlled by the operation of the turbine and is proportional to the amount of swirl in the discharge. In the case of a tip vortex, Γ is proportional to blade loading. The core radius is related to the boundary layer thickness at the point of separation. Because of this factor, the pressure field in a vortex is very sensitive to variations in Reynolds number. This is illustrated in Fig. 4.8. The experimental situation under which these data were collected is an idealized hub vortex, the amount of swirl being controlled by the angle of attack of a series of fixed blades, as shown in the illustration. This is representative of the vortex formed in the draft tube of a Francis/propeller unit, usually during off-design operation. This diagram clearly shows that observations of vortex cavitation in a model will not extrapolate to the prototype because of the differences in Reynolds number.

The examples of cavitation in separated flows and vortex flows already illustrate that σ is not a unique parameter for the correlation of cavitation phenomena. In fact, there are a myriad of "scale effects" which preclude using cavitation index alone as the scaling parameter for cavitation phenomena, just as the affinity laws for turbine performance are modified by viscous effects. The problems of scaling cavitation are more complex since the basis of the cavitation index is not only that pressure scales exactly with velocity squared but also that cavitation occurs when the hydrodynamic pressure at a given point is equal to the vapor pressure. As already discussed, cavitation inception is also influenced by the level of dissolved gas in the flow, as well as by the number density and size distribution of microbubbles or nuclei in the flow [9, 10]. In addition, surface roughness will increase the susceptibility to cavitate [15]. All of these details are of practical importance but are beyond the scope of this text.

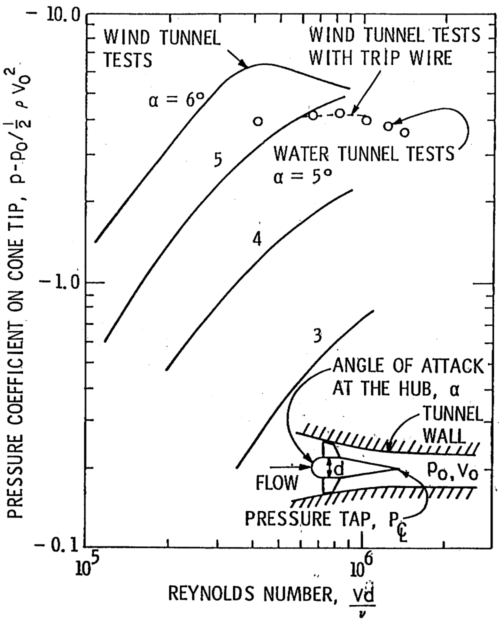


FIGURE 4.8 Minimum pressure in a core of a hub vortex. *(From Arndt [9].)*

Developed Cavitation. When the value of σ is less than σ_c, fixed or attached cavities can form on the suction side of a lifting surface. The minimum pressure is the vapor pressure, independent of upstream velocity and pressure; hence

$$C_{pm} = -\sigma \qquad \sigma < \sigma_c \tag{4.42}$$

Assuming the pressure distribution on the pressure side to be uninfluenced by cavitation on the suction side, it is easily seen that the lift coefficient, C_L, should be roughly proportional to C_{pm}. This is shown in Fig. 4.9. At each value of the angle of attack α there is a value of σ above which C_L is independent of this parameter. At lower values of σ, C_L decreases with decreasing σ. Note that as angle of attack increases, C_L increases and "cavitation stall" occurs at increasingly higher values of σ. In a turbomachine, the picture is qualitatively the same. As previously mentioned, the angle of attack is proportional to flow coefficient at a fixed wicket gate setting. Obviously the flow is more complicated, but the analogy between a hydrofoil and the blade section of a propeller turbine is easily drawn.

Cavitation Erosion. A detailed explanation of cavitation erosion is also beyond the scope of this text. Most of the research in this area has been directed toward the mechanics of bubble collapse and the associated impulsive pressures, as well as a quantification of those material properties that are of importance in resistance to cavitation. Little has been done to correlate cavitation erosion with the properties of a given flow field. However, it is important to have in mind that cavitation erosion will scale with a high power of velocity at a given cavitation number and that cavitation erosion does not nec-

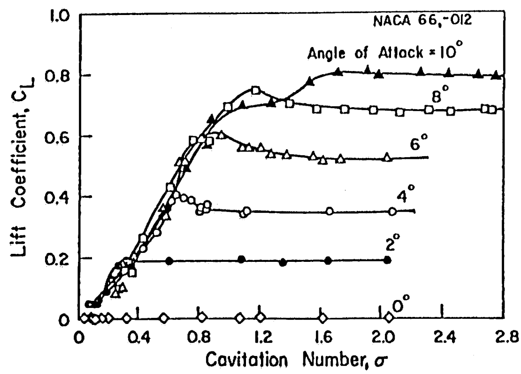


FIGURE 4.9 Variation in lift coefficient with cavitation number. *(From Kermeen [12].)*

essarily increase with a decrease in the cavitation index. Figure 4.10 is a compilation of data obtained in various laboratories for cavitation erosion due to flow over various axisymmetric bodies as well as two-dimensional flow over a backward-facing step. For the purposes of this illustration, erosion rate is de-

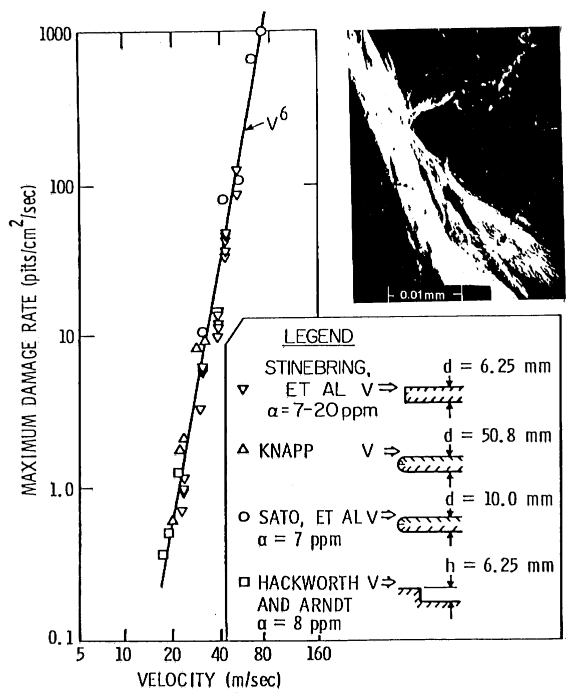


FIGURE 4.10 Cavitation erosion data. Constant cavitation number. A microphotograph of a typical pit is shown in the insert. *(From Arndt [10].)*

fined as the pitting rate of soft aluminum in number of pits per unit area per unit time, and corrections have been made for variations in dissolved gas. Very little quantitative information is available, but it has been observed that the pitting rate is measurably reduced with an increased concentration of gas [16]. A typical microphotograph of a pit is shown in the inset to Fig. 4.10. The important feature is that at constant σ the pitting rate scales with the sixth power of velocity. The pitting rate is *not* quantitatively equivalent to the measured weight loss observed in a specially designed erosion test apparatus. However, Stinebring et al. [16] have been able to measure the energy absorbed per pit. This is directly proportional to pit volume, which is found to scale with velocity to the fifth power. Thus, energy absorbed per unit area is

$$\frac{\text{Energy}}{\text{Unit area time}} = \frac{\text{pits}}{\text{area time}} \times \frac{\text{energy}}{\text{pit}} = \sim V^6 \times V^5 = \sim V^{11} \tag{4.43}$$

Obviously, the erosion rate is very sensitive to velocity in the initial stages of cavitation. Since the velocity in a turbine passage is proportional to the square root of head, this also implies that the magnitude of the erosion problem is more severe in high-head installations.

In many cases, cavitation erosion can be traced to the region at the trailing edge of an attached cavity. The number of bubbles that collapse in this region[2] in a given period of time will be a function of the cavity geometry, which in turn is a function of the cavitation index. This is illustrated in Fig. 4.11, which is admittedly for an idealized situation, namely cavitation on an axisymmetric body under carefully controlled laboratory conditions. Note that the maximum pitting rate does not occur at the lowest possible value of the cavitation index. This is an important concept that should be recognized by every hydraulic engineer.

Advanced stages of cavitation erosion can be simulated in the laboratory by a variety of different devices [10]. Detailed results cannot be discussed here, but some observations that are pertinent to engineering design can be mentioned. Thiruvengadam [19] has analyzed a great deal of erosion data and has concluded that for engineering purposes, the erosive intensity of a given flow field can be quantified in terms of depth of penetration per unit time $\dot{y}$ and the strength of the material being eroded, S_e,

$$I = \dot{y}\, S_e \text{ (power per unit area)} \tag{4.44}$$

The intensity of I is a function of a given flow field. The rate of penetration can be calculated from the weight loss per unit time $\dot{W}$ and the surface area of the eroded material, A_c

$$\dot{y} = \frac{\dot{W}}{\gamma_m A_c} \tag{4.45}$$

where γ_m is the specific weight of the eroded material. Many different forms of S_e have been tried. The most-used value appears to be ultimate strength, basically a

[2]Recent research indicates that the cavitation process in this region is very complex, and the actual erosion mechanism might be due to the collapse of microvortices formed in the wake of the attached cavity [17, 18].

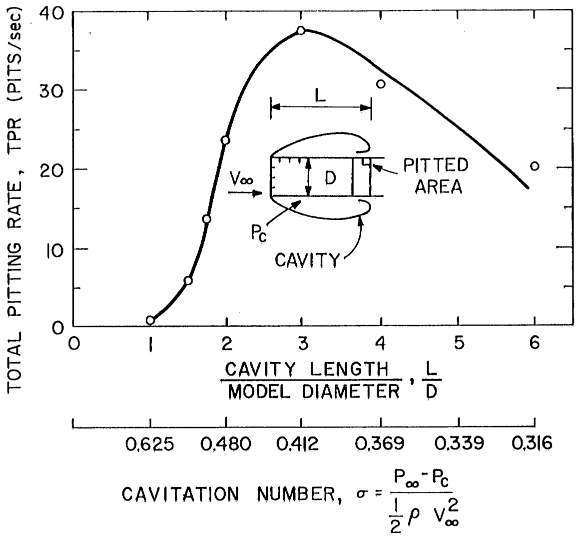


FIGURE 4.11 Variation of cavitation erosion with cavity length and cavitation number. *(From Stinebring [16].)*

weighted value of the area under a stress-strain curve. Two forms have been proposed [19–21]:

$$S_e = \tfrac{1}{3}e\,(2S_u + S_y) \tag{4.46}$$

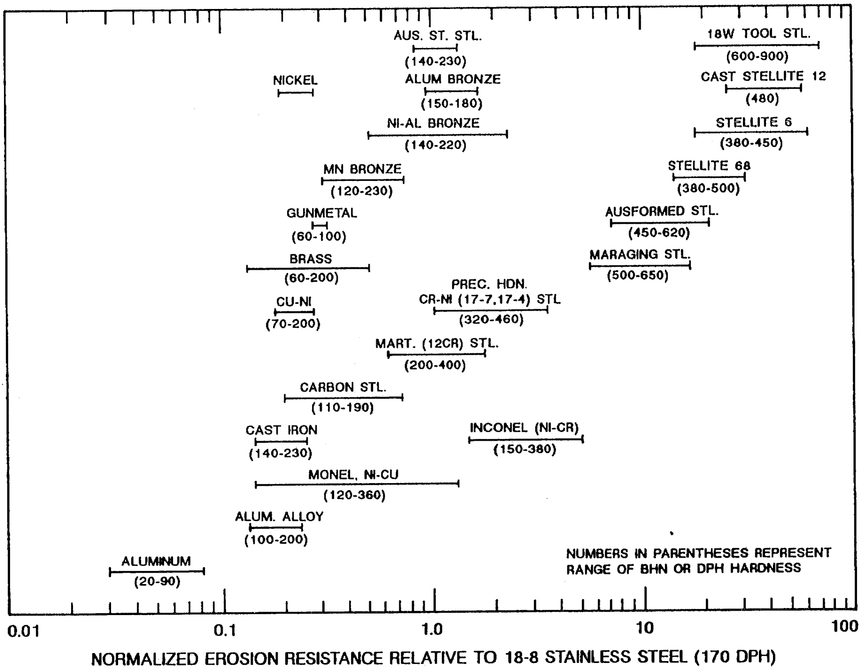
$$S_{e_1} = \tfrac{1}{2}\,\frac{S_u^{\,2}}{E} \tag{4.47}$$

where e = percent elongation
E = Young's modulus
S_u = ultimate strength
S_y = yield strength

A ranking of various engineering materials relative to cast carbon steel is given in Table 4.1. Thiruvengadam [19] was also able to show that although various materials have different rates of weight loss when subjected to the same cavitating flow and that the rate of weight loss varies with time, a normalized erosion rate versus time characteristic is similar for a wide range of materials; this is shown in Fig. 4.12. Here, the rate of weight loss is normalized with respect to maximum rate of weight loss and time is normalized to the time at which maximum weight loss occurs.

Thus, Thiruvengadam's simplified theory allows for rapid determination of I for a given flow by measuring $\dot{y}$ for a soft material. Service life for a harder material can then be predicted from the ratio of the strengths of the hard and soft material.

Turbine Cavitation. The introductory material on cavitation was based on laboratory research on simple geometric shapes. The flow in a turbine is much more complex, but the principles of cavitation inception, erosion, and performance breakdown are still applicable. Each type of turbine will cavitate when operated at a value of σ_T that is less than a certain critical value of Thoma's sigma, σ_{TC}.



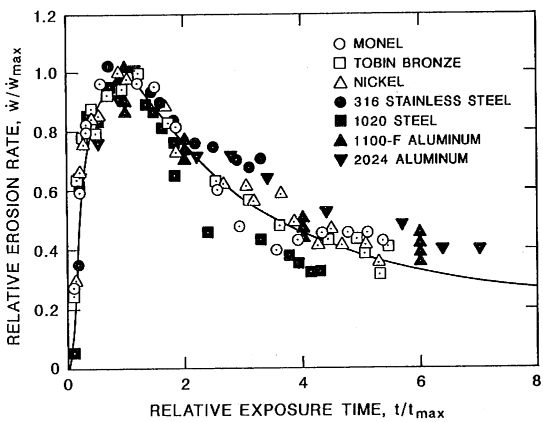


FIGURE 4.12 Normalized erosion rate. *(Adapted from Thiruvengadam [19].)*

Clearly, cavitation can be totally avoided only if the value σ_T at a given installation is greater than σ_{TC}. The value of σ_T for a given installation is known as the plant sigma σ_p. For a given turbine operating under a given head, the only variable is the turbine setting z. Normal practice is to set the turbine such that σ_p is actually less than σ_{TC} but greater than the value of σ_T corresponding to performance breakdown σ_{TB}. The value of σ_p then controls the allowable setting above tailwater:

$$z_{\text{allow}} = H_a - H_v - \sigma_p H \tag{4.48}$$

It must be borne in mind that H_a varies with elevation. As a rule of thumb, H_a decreases from the sea-level value of 10.3 m (33.8 ft) by 1.1 m (3.6 ft) for every 1000 m (3280 ft) above sea level. Thus a turbine sited at Leadville, Colorado (elevation 3000 m or 9840 ft above sea level), for example, would have an allowable turbine setting that is more than 3 m (9.8 ft) less than that at sea level. In fact, z_{allow} could easily be negative, implying a required turbine setting below the tailwater elevation.

The determination of σ_{TB} is usually done by a model test. A schematic of the variation performance breakdown with σ_T is shown in Fig. 4.13. This figure is a composite of turbine tests (courtesy of Voith Company) and acoustic data from Deeprose et al. [22]. Note the similarity between the trend of performance with σ_T and the correlation of lift coefficient with σ as shown in Fig. 4.9. The photographs of cavitation in the runner at various values of σ_T indicate that a measurable change in performance is only noted after there is fully developed cavitation in the runner passages. Note also that the maximum noise level, and presumably the maximum rate of cavitation erosion, occurs at a value of σ_T intermediate be-

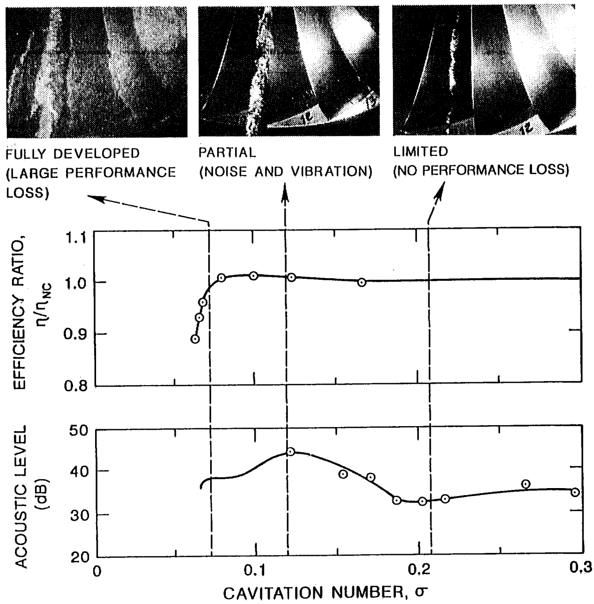


FIGURE 4.13 Schematic of the relationship between performance breakdown, noise, and vibration and various stages of cavitation.

tween σ_{TC} and σ_{TB}. A slight rise in efficiency is often noted at intermediate values of σ_T.

Suction Specific Speed. σ_{TB} is a function of the type of turbine involved, i.e., the specific speed of the machine. A cavitation scaling parameter often used in pump application is the suction specific speed

$$S = \frac{\Omega\sqrt{Q}}{(gH_{sv})^{3/4}} \tag{4.49}$$

The suction specific speed is a natural consequence of considering dynamic similarity in the low-pressure region of a turbomachine. The dynamic relations are

$$\frac{gH_{sv}D_e^4}{Q^2} = \text{constant} \qquad \frac{gH_{sv}}{\Omega^2 D_e^2} = \text{constant} \tag{4.50}$$

where D_e is the eye or throat diameter. These relations hold when the kinematic condition for similarity of flow in the low-pressure region of the machine is satisfied

$$\frac{Q}{\Omega D_e^3} = \text{constant} \tag{4.51}$$

Elimination of D_e in Eq. (4.50) yields the suction specific speed. Using Eq. (4.16*a*) for the power developed by a turbine, the relationship between σ_T, N_s, and S is given by

$$\sigma_T = \frac{1}{\eta^{2/3}}\left[\frac{N_S}{S}\right]^{4/3} = \left[\frac{N_{sQ}}{S}\right]^{4/3} \tag{4.52}$$

If S can be assumed to be constant, Eq. (4.52) produces a relationship between σ_T and N_s. Allowable values of S do vary, but an acceptably conservative value in nondimensional units is 3. A comparison between Eq. (4.52), assuming $S = 3$, and actual turbine experience σ_p as quoted in the literature [23, 24], is shown in Fig. 4.14. Note that the curve for σ_p falls below the equivalent curve for pumps. This implies an equivalent allowable S for turbines higher than for equivalent pumps. Note also that the trend of σ_{TB} for turbines has a steeper slope than the constant S lines. This could imply that different specific-speed designs are not equally close to the optimum with regard to cavitation or that the factor S cannot be considered a constant. It should be kept in mind that since the flow direction for a pump and a turbine are in opposite directions, only the inception point should be similar. Under conditions of developed cavitation, the flow situation could be quite different, with cavity closure occurring on the runner in the case of a pump, whereas it would occur downstream of the runner in the case of the equivalent turbine.

Figure 4.14 is a useful chart for estimating the turbine setting for various types of turbines in conjunction with Eq. (4.48). This is a useful procedure for preliminary design and comparison between different types of turbines for the same installation. However, the manufacturer's recommendation should be followed in the final design.

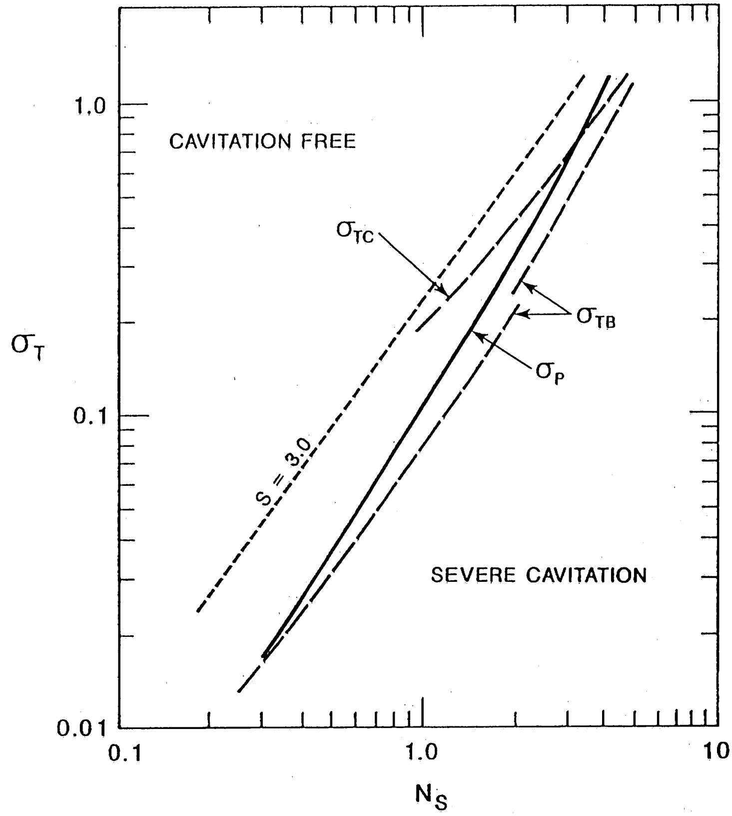


FIGURE 4.14 Comparison of turbine model test data σ_{TB} average of plant settings σ_P and theoretical estimate of inception σ_{TC}. Typical pump limits are shown with dotted lines for a suction specific speed of 3.

Cavitation Erosion in Turbines. As shown in Fig. 4.15, cavitation pitting can occur at several components of a hydraulic turbine. The dominant type of pitting is usually on the surface of the runner or on the walls of the discharge ring. Cavitation can also occur on other components, such as wicketgates. Arndt et al. [24] have recently reported the results of a study of the operational and design characteristics of 729 hydroturbines installed since 1950, having a capacity greater than 20 MW or a discharge diameter greater than 3 m. Although design parameters such as unit speed and specific speed for a given head have little variation from manufacturer to manufacturer, cavitation pitting rate varies widely even when relative comparisons are made. These variations are attributable to variations in setting, variations in manufacturing tolerances, and variations in the operational history of a given unit. As will be discussed, the results from current theory and laboratory experiments are in agreement with certain trends noted in the field, but certain aspects of cavitation erosion in turbines are not well understood and require further fundamental research.

Using Eqs. (4.44) and (4.45), a nondimensional erosion parameter can be de-

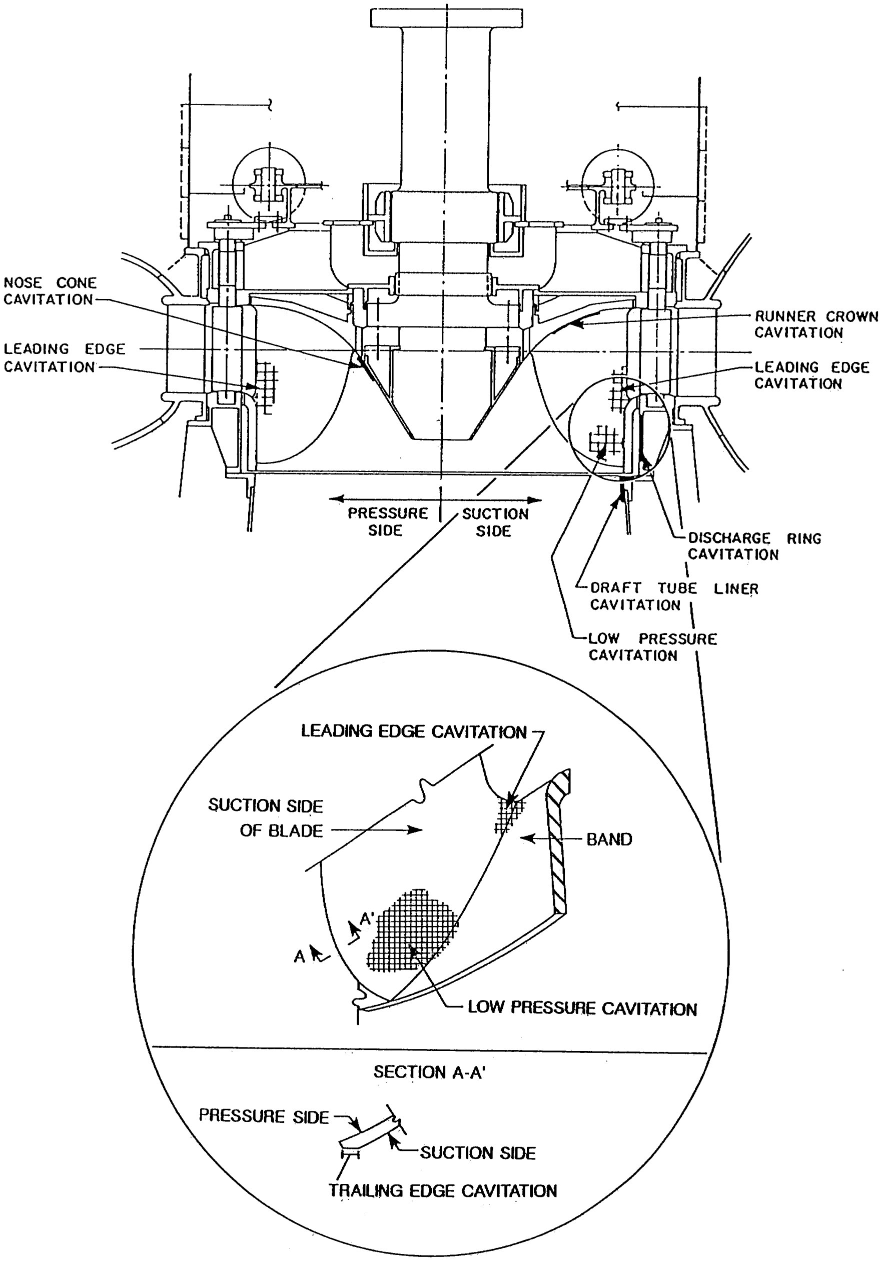


FIGURE 4.15 Schematic of cavitation damage on various components of a Francis turbine. *(Adapted from Sinclair and Rodrigue [26].)*

veloped by normalizing I with P/d^2, and assuming A_c is proportional to d^2, where P is the power and d is the diameter of the machine:

$$I_n = \frac{\dot{W}\, S_e}{\gamma_m\, P} \tag{4.53}$$

As shown in Fig. 4.14, the setting is normally a compromise between cavitation-free operation and performance breakdown. If this compromise is consistently the same for all types of machines, I_n would not vary with specific speed. Arndt et al. [24] found that I_n varied considerably, but an upper limit of $I_n = 0.5$ was independent of specific speed. A lower limit of about 0.001 was observed. However, the lower limit does appear to be sensitive to specific speed. Arndt et al. suggest that this can be explained by taking the dynamics of bubble collapse into account.

In establishing guarantees, manufacturers normally assume that the rate of weight loss is proportional to the square of diameter, which is consistent with Eq. (4.53) since $P \sim d^2$ at constant head (Eq. 4.34). The normal cavitation guarantee duration is 8000 h, with 3000 h suggested for peaking units and pump turbines. Reference 25 proposes that weight loss of guarantees range between $1.9d^2$ and $0.49d^2$ (weight loss in kilograms and diameter in meters; 1 kg = 2.2 lb, 1 m = 3.28 ft). Arndt et al. [24] found that erosion rate was more sensitive to the size of the machine than the simple square law assumed by the industry; again, this was qualitatively correlated with the results of bubble dynamics, but a definitive scaling procedure is not at hand. It would be prudent to use more conservative settings for larger machines to avoid unexpected problems when scaling up smaller machines of the same type.

It was also found that there was a significant advantage in using stainless steel in the manufacture of turbines [24]. The use of stainless steel overlays in regions prone to cavitation pitting is not as effective in resisting erosion, since galvanic corrosion can accelerate the damage due to cavitation. New manufacturing techniques and the reduction in the cost of stainless steel make the use of stainless steel more attractive than carbon steel. It should be emphasized that the choice of material does not affect the cavitation itself, but the use of cavitation-resistant materials such as stainless steel will normally provide protection against damage. Further details can be found in Ref. 26.

4.4 TURBINE TECHNOLOGY

Overall Description of a Hydropower Installation

The hydraulic components of a hydropower installation consist of an intake, penstock, guide vanes or distributor, turbine, and draft tube. The intake is designed to withdraw flow from the forebay as efficiently as possible, with no or minimal vorticity. Trash racks are commonly provided to prevent ingestion of debris into the turbine. Intakes usually require some type of shape transition to match the passageway to the turbine and also incorporate a gate or some other means of stopping the flow in case of an emergency or turbine maintenance. Some types of turbines are set in an open flume; others are attached to a closed-conduit penstock. In all cases, efforts should be made to provide uniformity of the flow, as the degree of uniformity has an effect on the efficiency of the turbine. For low-head installations, the diameter of a closed penstock must be quite large to accommodate the large discharges necessary for a given power output. Its size is a compromise between head loss and cost. The selection of the actual penstock configuration is dependent on the location of the powerhouse with respect to the dam.

For some types of reaction turbines, the water is introduced to the turbine through casings or flumes which vary widely in design. The particular type of casing is dependent on the turbine size and head. For small heads and power output, open flumes are commonly employed. Steel spiral casings are used for higher heads, and the casing is designed so that the tangential velocity is essentially constant at consecutive sections around the circumference. This requirement necessitates a variable cross-sectional area of the casing. Some examples of intakes and casings are shown in Fig. 4.16, where dimensions are given in terms of the runner diameter. As the inflow has an effect on the turbine efficiency, the design of the spiral casing is carried out by the turbine manufacturer.

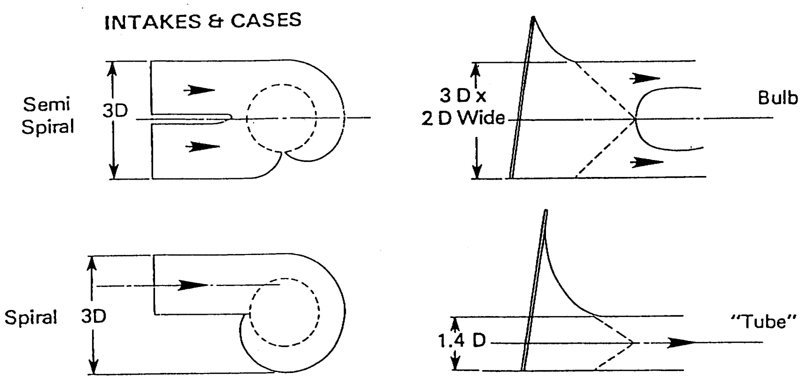


FIGURE 4.16 Typical intake and case dimensions. *(From Mayo [37]. Reprinted with permission.)*

Discharge control for some types of reaction turbines is provided by means of adjustable guide vanes or wicket gates around the outer edge of the turbine runner. The vanes are tied together with linkages and their positioning is regulated by a governor. The wicket gates are shown schematically in Fig. 4.1. The flow areas can be readily varied from zero to a maximum by rotation of the vanes. In addition, the velocity diagrams at the entrance and exit are a function of the guide vane position and, therefore, the efficiency of the turbine also changes. Wicket gates can also be used to shut off the flow to the turbine in emergency situations. Various types of valves are installed upstream of the turbine for this purpose for turbines without wicket gates.

One purpose of the draft tube is to reduce the kinetic energy of the water exiting the turbine runner. Within limits, a well-designed draft tube will permit installation of the turbine above the tailwater elevation without losing any head. Different designs of a draft tube are common, ranging from a straight conical diffuser to configurations with bends and bifurcations. Some typical shapes and rel-

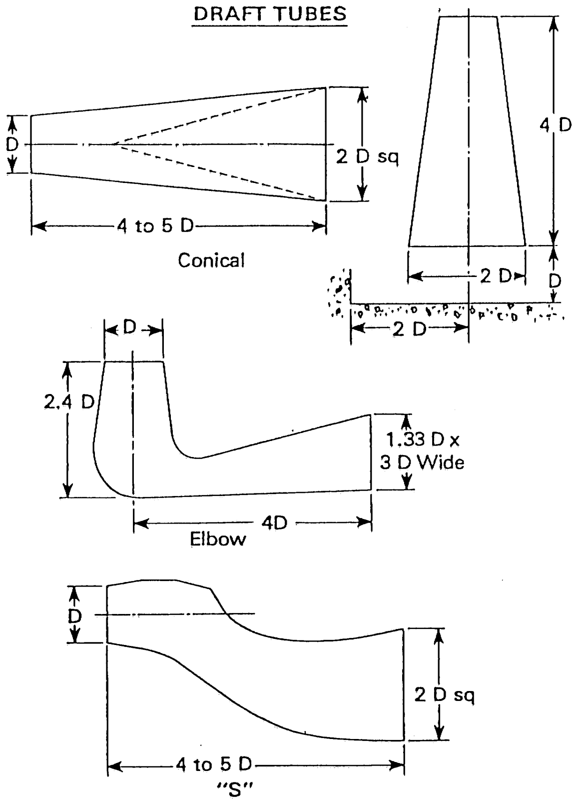


FIGURE 4.17 Typical draft tube dimensions. *(From Mayo [37]. Reprinted with permission.)*

ative dimensions are shown in Fig. 4.17, where the dimensions again are given in terms of the runner diameter.

The simplest form of draft tube is the straight conical diffuser. Efficiency of energy conversion is dependent on the angle of the diverging walls, as discussed in Sec. 4.3. Small divergence angles require long diffusers to achieve the area necessary to reduce the exit velocity. Long diffusers increase the required construction effort and cost; therefore, the angle in some cases may be increased up to about 15° from the typical optimum value of about 7°. In addition to the increased loss through a large-angle diffuser, flow separation can lead to unstable flow. Flow instability is to be avoided, as it has an adverse effect on turbine performance.

For some types of turbine installations, such as a vertical-axis turbine, the flow must be turned through a 90° angle after leaving the turbine. This is accomplished by adding an elbow between the turbine and draft tube, which has an influence on the draft tube performance, as discussed briefly in Sec. 4.3, and requires careful design. Experimental data on diffusers are available in the literature. However, the flow leaving the turbine runner can have a swirl component of velocity which has an effect on the draft tube efficiency. The magnitude of the swirl is dependent on the type of turbine and operating conditions. Excessive swirl can result in surging in the draft tube, as well as load fluctuations and pressure fluctuations that can cause mechanical vibrations of severe magnitude. However, a small swirl component has been found to be beneficial, as pointed out in Sec. 4.3. A draft tube design adequate for one type of runner may not be satisfactory for another. Therefore, the draft tube is considered an essential part of the turbine, and its design is carried out by the turbine manufacturer. There are very few data in the literature on draft tube design. A recent example is Ref. 8.

An example of some typical losses and their sources are shown in Fig. 4.18 for a Kaplan turbine. For small discharges, major losses occur in the runner and distributor. This is typical for a turbine operating at off-design conditions and is associated with the shock losses in the runner. As the discharge increases, the runner and distributor losses decrease to relatively small values. The draft tube losses increase, but the largest increase is in the losses at the draft tube exit. It becomes obvious that efforts should be made to reduce this loss, which can be accomplished by enlarging the draft tube exit area. However, because such enlargement increases the construction cost, compromises must again be made.

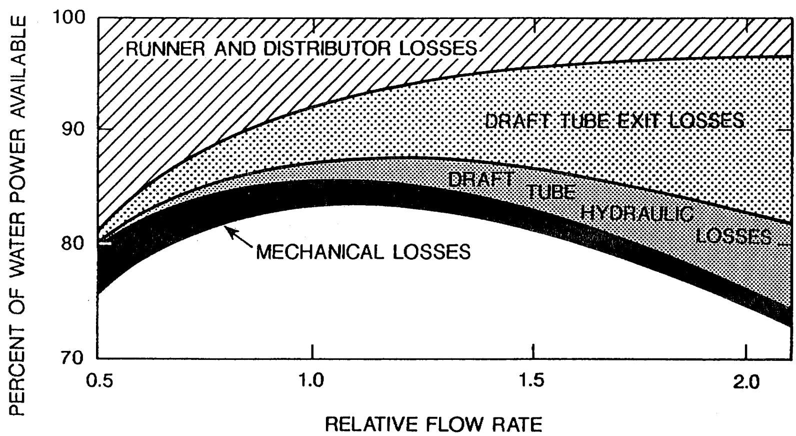


FIGURE 4.18 Energy balance for a medium-speed Kaplan turbine model as a function of unit discharge and 1-m (3.28-ft) head. *(Adapted from Kovalev [43].)*

Turbine Classification and Description

There are two basic types of turbines, denoted as impulse and reaction. In an impulse turbine, the available head is converted to kinetic energy before entering the runner, the power available being extracted from the flow at atmospheric pressure. In a reaction turbine, the runner is completely submerged and both the pressure and the velocity decrease from inlet to outlet. The velocity head at the inlet to the turbine runner is typically less than 50 percent of the total head available. In either machine the torque is equal to the rate of change of angular momentum through the machine as expressed by the Euler equation (Eq. 4.1).

Impulse Turbines. Modern impulse units are generally of the Pelton type and are restricted to relatively high head applications (Fig. 4.19). One or more jets of water impinge on a wheel containing many curved buckets. The jet stream is directed inwardly, sideways, and outwardly, thereby producing a force on the

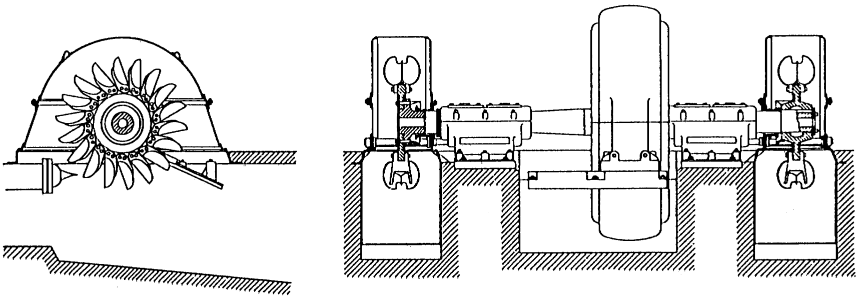


FIGURE 4.19 Double-overhung impulse wheel. *(From Daugherty and Franzini [42]. Reprinted with permission.)*

bucket, which in turn results in a torque on the shaft. All of the available head is converted to kinetic energy at the nozzle. Any kinetic energy leaving the runner is "lost." It is essential that the buckets be designed in such a manner that exit velocities are a minimum. No draft tube is used since the runner operates under approximately atmospheric pressure and the head represented by the elevation of the unit above tailwater cannot be utilized.[3] Since this is a high-head device, this loss in available head is relatively unimportant. As will be shown later, the Pelton wheel is a low-specific-speed device. Specific speed can be increased by the addition of extra nozzles, the specific speed increasing by the square root of the number of nozzles. Specific speed can also be increased by a change in the manner of inflow and outflow. As shown in Fig. 4.20, a Turgo turbine can handle relatively larger quantities of flow at a given speed and runner diameter by passing the jet obliquely through the runner in a manner similar to a steam turbine. The jet impinges on several buckets continuously, whereas only a single bucket per jet is effective at any instant in a Pelton wheel. The Banki-Mitchell turbine illus-

[3]In principle, a draft tube could be used, which requires the runner to operate in air under reduced pressure. Attempts at operating an impulse turbine with a draft tube have not met with much success.

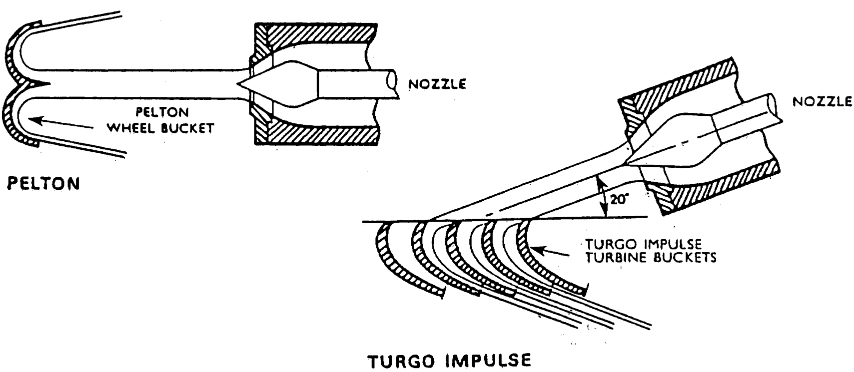


FIGURE 4.20 Turgo and Pelton wheels contrasted. The jet on the Turgo strikes three buckets continuously, whereas on the Pelton it strikes only one. A similar speed-increasing effect can be had on the Pelton by adding another jet or two. Turgo turbines are only available in small sizes.

trated in Fig. 4.21 is a variation on this theme. The flow passes through the blade row twice, first at the upper portion of the wheel and again at the lower portion. The flow exits the blade in the opposite direction from the first pass and hence this configuration tends to be self-cleaning, since debris impinging on the periphery of the runner at the top dead center is removed by the flow on the second pass at essentially bottom dead center.

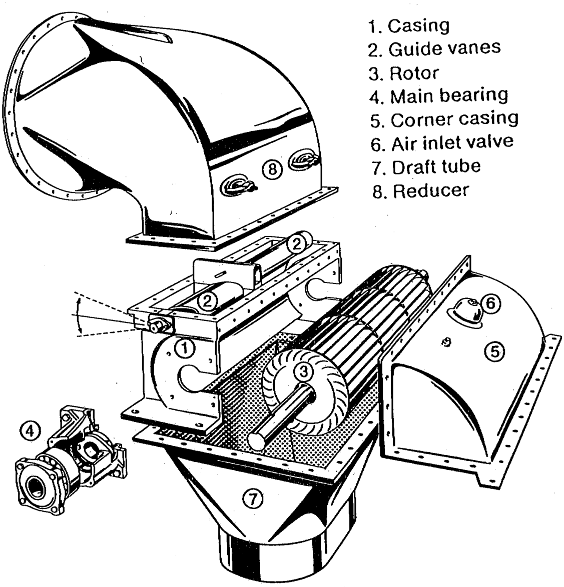


FIGURE 4.21 Ossberger cross-flow turbine.

Most Pelton wheels are mounted on a horizontal axis, although newer vertical-axis units have been developed. Because of physical constraints on orderly outflow from the unit, the maximum number of nozzles is generally limited to six or less. While the power of a reaction turbine is controlled by the wicket gates, the power of the Pelton wheel is controlled by varying the nozzle discharge by means of an automatically adjusted needle, as illustrated in Fig. 4.22. Jet deflectors, Fig. 4.22*a*, or auxiliary nozzles, arranged as in Fig. 4.22*b*, are provided for emergency unloading of the wheel. Additional power can be obtained by connecting two wheels to a single generator or by using multiple nozzles. Since the needle valve can throttle the flow while maintaining essentially constant jet velocity, the relative velocities at entrance and exit remain unchanged, producing nearly constant efficiency over a wide range of power output. This is a desirable feature of Pelton and Turgo wheels. Throttling of the Banki-Mitchell turbine is accomplished differently, as illustrated in Fig. 4.21. An adjustable guide vane is used which functions in a manner similar to the wicket gates in a reaction turbine. If operating conditions require, the guide vanes can be divided into two separately controlled sections. For most installations, the lengths of the two guide vane sections are in the ratio of 1:2, allowing for utilization of one-third, two-thirds, or the entire runner, depending on the flow conditions. This combination provides a relatively flat efficiency curve in the range of 15 to 100 percent of maximum power.

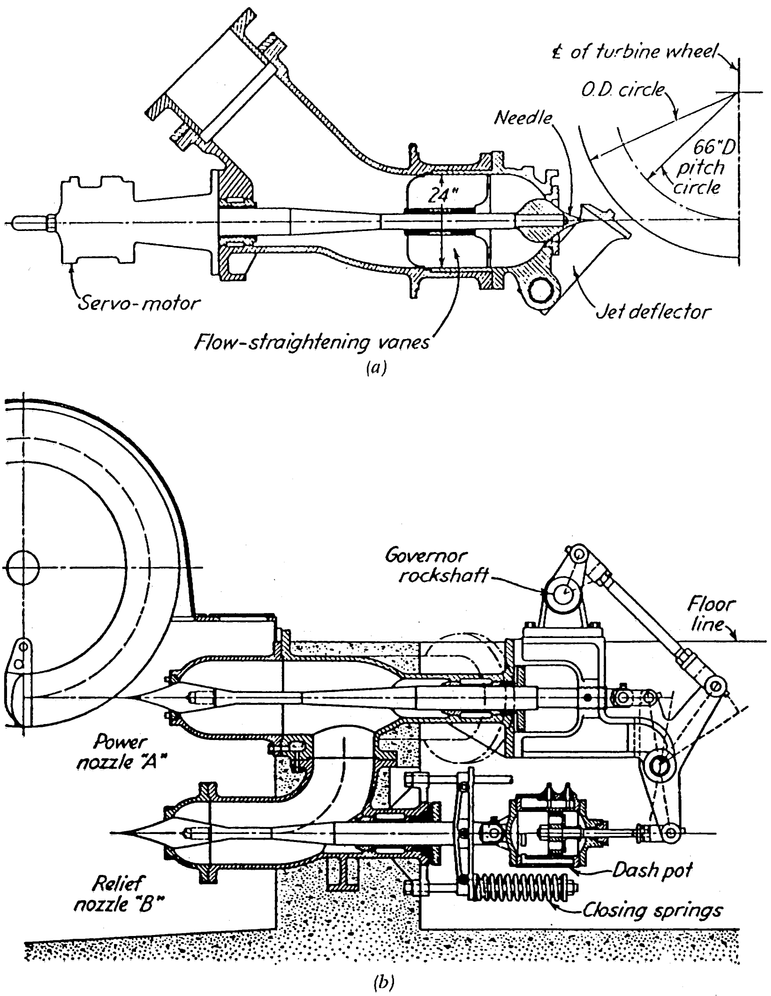


FIGURE 4.22 (*a*) Pelton 45° elbow-type needle nozzle with jet deflector. (*From J. W. Daily [38]. Reprinted with permission.*) (*b*) Pelton nozzle with auxiliary relief nozzle. (*From Daily [38]. Reprinted with permission.*)

Reaction Turbines. Reaction turbines are classified according to the variation in flow direction through the runner. In radial- and mixed-flow runners, the flow exits at a radius different (in modern designs the inlet flow is always inward) than the radius at the inlet. If the flow enters the runner with only radial and tangential components, it is a radial-flow machine. The flow enters a mixed-flow runner with both radial and axial components. Francis turbines are of the radial- and mixed-flow type, depending on the design specific speed. Two Francis turbines are illustrated in Fig. 4.23. The radial-flow runner (Fig. 4.23*a*) is a low-specific-speed design, whereas the mixed-flow runner (Fig. 4.23*b*) achieves peak efficiency at considerably higher specific speed.

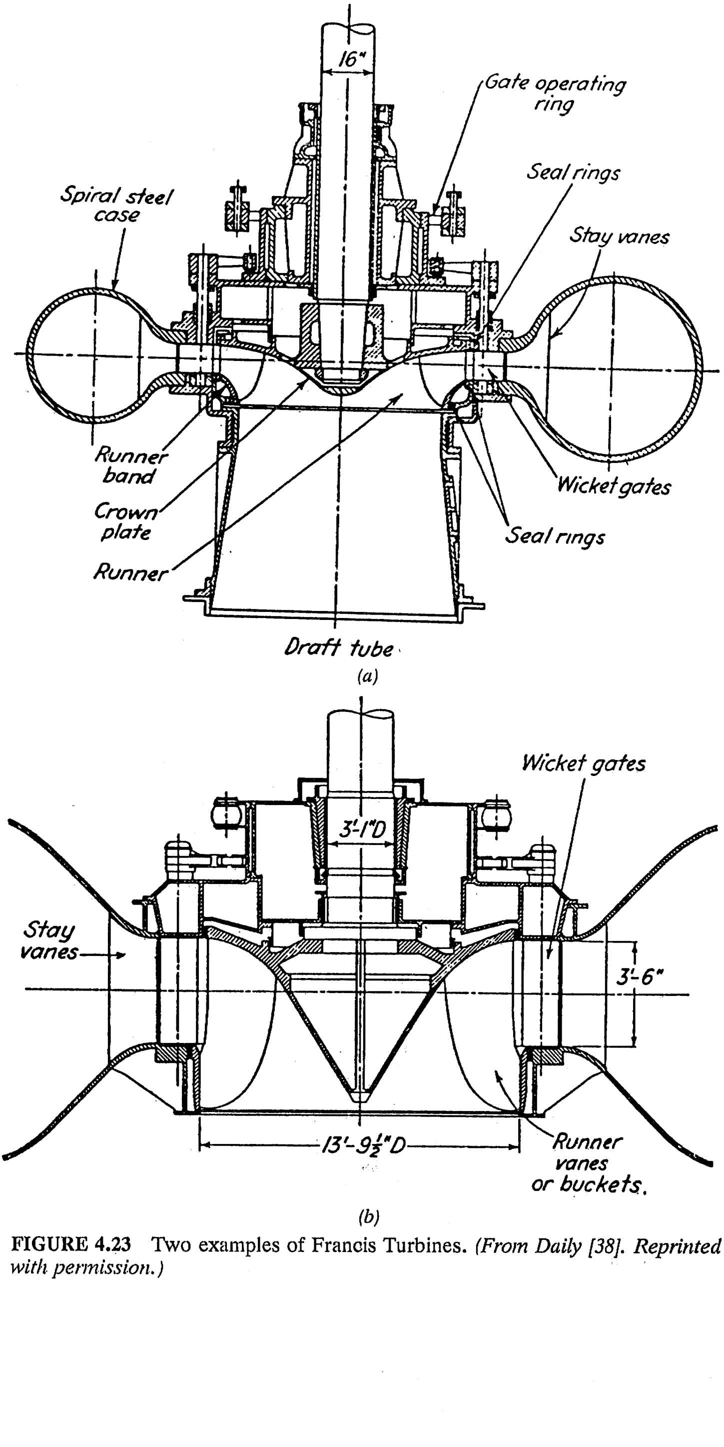


FIGURE 4.23 Two examples of Francis Turbines. *(From Daily [38]. Reprinted with permission.)*

Axial-flow propeller turbines are generally either of the fixed-blade or Kaplan (adjustable-blade) variety. The "classical" propeller turbine, illustrated in Fig. 4.24, is a vertical-axis machine with a scroll case and a radial wicket gate configuration that is very similar to the flow inlet for a Francis turbine. The flow enters radially inward and makes a right angle turn before entering the runner in an axial direction. The Kaplan turbine has both adjustable runner blades as well as adjustable wicket gates. The control system is designed in such a manner that the variation in blade angle is coupled with the wicket gate setting in a manner which achieves best overall efficiency over a wide range of flow rates. The classical vertical design does not take full advantage of the geometric properties of an axial-flow runner. The flow enters

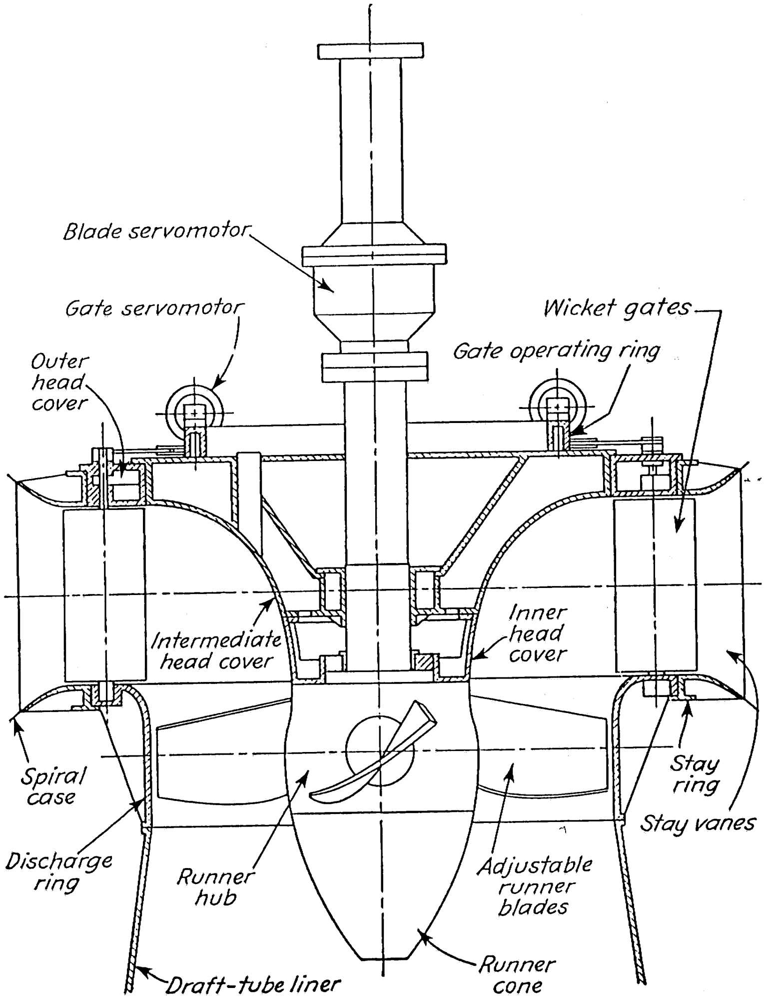


FIGURE 4.24 Smith-Kaplan axial-flow turbine with adjustable-pitch runner blades, $N_s \sim 3.4$. *(From Daily [38]. Reprinted with permission.)*

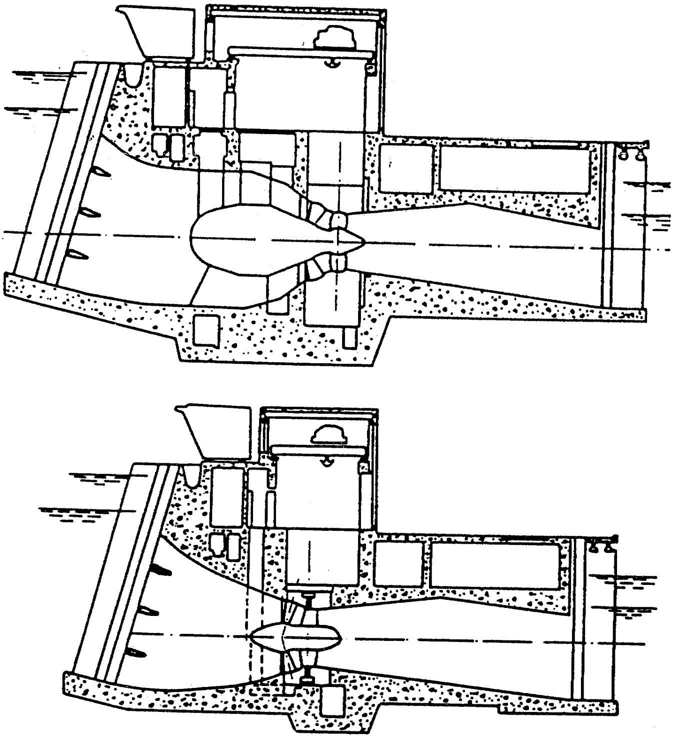


FIGURE 4.25 (*a*) Comparison of structures required for Straflow vs bulb turbine with same output and head. *(From U.S. Department of Energy [39].)*

the scroll case in a horizontal direction, issues radially inward from the spiral case where it forms a vortex, and discharges into the draft tube in a vertical direction with very little whirl component remaining. The flow must then again be turned through 90° to discharge into the tailwater in a horizontal direction. From a design point of view, this is less than desirable for many reasons. The flow field entering the runner is highly complex, and it is difficult to design the proper pitch distribution from hub to tip for minimal shock losses. There are additional losses in the elbow, and the tortuous flow path required from inlet to outlet requires additional civil works.

More modern designs take full advantage of the axial-flow runner; these include the tube, bulb, and Straflo types illustrated in Fig. 4.25. The flow enters and exits the turbine with minor changes in direction. A wide variation in civil works design is also permissible. The tubular type can be fixed-propeller, semi-Kaplan, or fully adjustable. An externally mounted generator is driven by a shaft which extends through the flow passage either upstream or downstream of the runner. The bulb turbine was originally designed as a high-output, low-head unit. In large units, the generator is housed within the bulb and is driven by a variable-pitch propeller at the trailing end of the bulb. Pit turbines are similar in principle to bulb turbines, except that the generator is not enclosed in a fully submerged compartment (the bulb). Instead, the generator is in a compartment that extends above water level. This improves access to the generator for maintenance. An example is shown in Fig. 4.26. Smaller bulb and

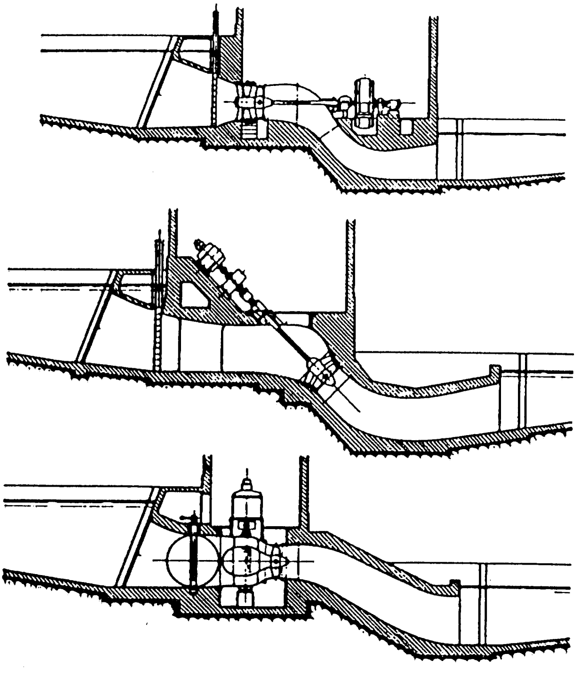


FIGURE 4.25 *(Continued)* (*b*) Various tube turbine arrangements. *(From U.S. Department of Energy [39].)*

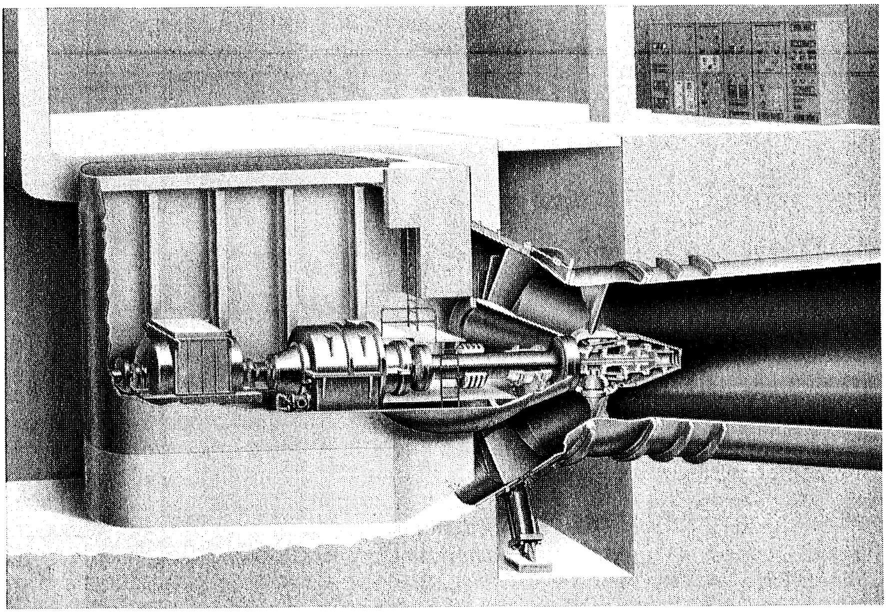


FIGURE 4.26 Schematic of pit turbine. *(Courtesy of Voith GmbH).*

pit units are available in which an externally mounted generator is driven by a right-angle drive housed within the bulb (Fig. 4.27). Because of the simplicity of installation, modern axial-flow machines are of considerable interest for low-head applications.

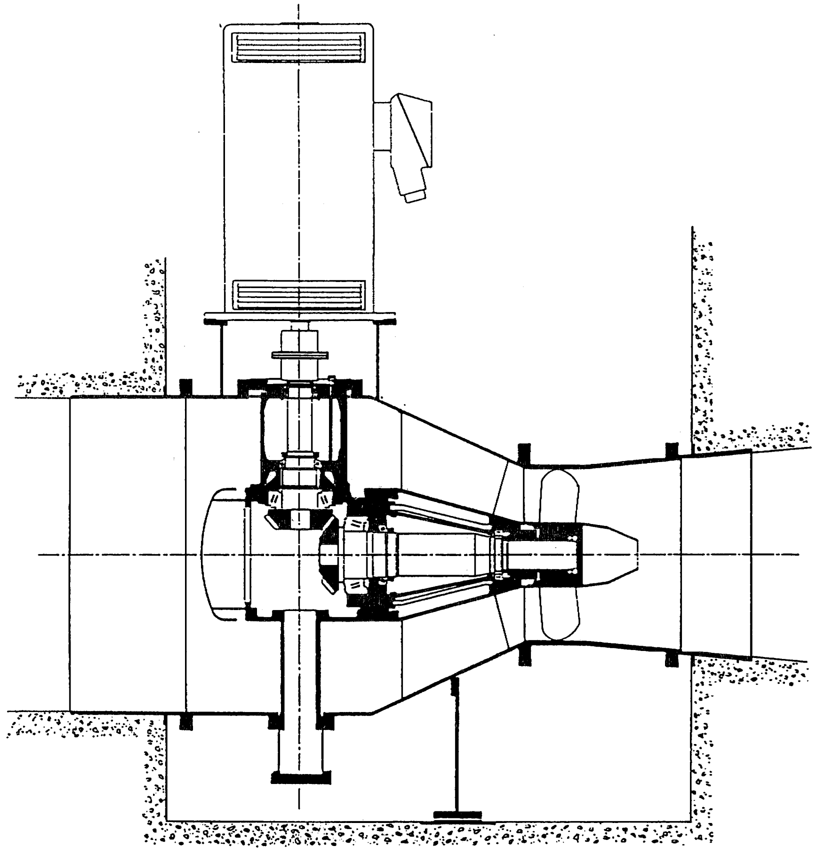


FIGURE 4.27 Right-angle drive bulb turbine. *(Courtesy of Neyrpic.)*

In addition to the radial-flow and mixed-flow Francis and axial-flow propeller units, there is the Deriaz turbine, which is a mixed-flow propeller unit of the Kaplan type. This turbine was originally developed for pumped storage applications, but shows great promise for applications in the medium-head range. The turbine consists of a series of controllable-pitch blades mounted on a conical hub. The turbine can have either a conventional scroll case and gate apparatus or a more specialized flap system for controlling the inlet flow. This unit provides a flat efficiency curve over a wide range of power similar to standard Kaplan propeller units but, because of the mixed-flow design, is applicable to higher-head applications.

Performance Characteristics

Comparative Performance of Impulse and Reaction Turbines. The two basic types of turbines tend to operate at peak efficiency over different ranges of specific speed. This is due to geometric and operational differences. In order to give the reader a perspective of the operational characteristics of each type, a brief discussion of operational principles is presented below. This is followed by a summary of the performance of commercially available equipment in subsequent sections.

Impulse Wheels. Typical types of impulse wheels are illustrated in Figs. 4.19 to 4.22. For a given pipeline, there is a unique jet diameter that will deliver maximum power to a jet. Denoting the jet diameter by d_j, the power is given by

$$P_j = \gamma Q \frac{V_j^2}{2g} = \gamma \frac{\pi}{8g} d_j^2 V_j^2 \tag{4.54}$$

Let Δh denote the difference between the reservoir surface elevation and the nozzle elevation. Neglecting losses at the entrance to the pipe and in the nozzle, the energy equation is given by

$$\frac{V_j^2}{2g} + f \frac{L}{d_p} \frac{V_p^2}{2g} = \Delta h \tag{4.55}$$

where V_j is the jet velocity, V_p is the velocity in the pipe, and d_p and L denote, respectively, the pipe diameter and length. As the size of the nozzle opening is increased, the flow rate Q gets larger while the jet velocity V_j gets smaller, since the losses in the pipeline increase with Q. Using $V_p = V_j(d_j/d_p)^2$ and Eq. (4.55), it can be shown that maximum power is obtained when

$$\Delta h = 3f \frac{L}{d_p} \frac{V_p^2}{2g} \tag{4.56}$$

$$H = \frac{V_j^2}{2g} = \frac{2}{3} \Delta h \tag{4.57a}$$

and

$$d_j = \left[\frac{d_p^5}{2fL}\right]^{1/4} \tag{4.57b}$$

Thus, for a given penstock geometry and Δh, the maximum power available to the turbine can be calculated. It should be noted that the maximum possible plant efficiency is two-thirds for this case. (Recall that power is proportional to flow and head). For the usual design problem in which Q and Δh are given, the maximum possible plant efficiency could theoretically be unity if d_p is so large that the loss term in Eq. (4.55) is negligible. The jet velocity V_j would then be given by $V_j = \sqrt{2g\Delta h}$ and the jet diameter by $\pi d_j^2/4 = Q/\sqrt{2g\Delta h}$. This is a minimum jet diameter. Note that selection of d_p and d_j in specific cases requires an economic analysis and depends strongly on the characteristics of the site and also on the turbines that are available.

Of the head available at the nozzle inlet, a small portion is lost to friction in the nozzle and to friction on the buckets. The rest is available to drive the wheel. The actual utilization of this head depends on the velocity head of the flow leaving the turbine and the setting above tailwater. Optimum conditions, corresponding to maximum utilization of the head available, dictate that the flow leaves at essentially zero velocity. Under ideal conditions, this occurs when the peripheral speed of the wheel is one-half the jet velocity. In practice, optimum power occurs at a speed coefficient, $\phi = u_1/\sqrt{2gH}$, somewhat less than 0.5. In fact, it can be shown that best efficiency will occur when $\phi = \frac{1}{2}C_v \cos \alpha_1$, where C_v is the velocity coefficient for the nozzle,

$$C_v = \frac{V_j}{\sqrt{2gH}} \tag{4.58}$$

and α_1 represents the effective angle between the jet velocity and the peripheral velocity of the runner at the entrance to the bucket. This is illustrated in Fig. 4.28. Since maximum efficiency occurs at fixed speed for fixed H, V_j must remain constant under varying flow conditions. Thus the flow rate Q is regulated with an adjustable nozzle. There is some variation in C_v and α_1 with regulation and maximum efficiency occurs at slightly lower values of ϕ under partial power settings. Present nozzle technology is such that the discharge can be regulated over a wide range at high efficiency.

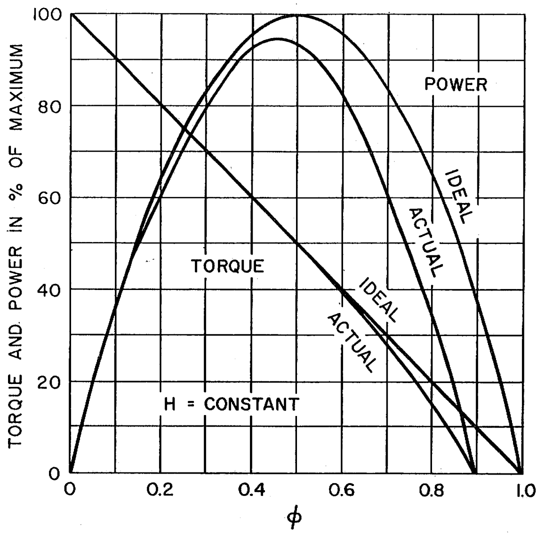


FIGURE 4.28 Ideal and actual variable-speed performance for an impulse turbine. *(Adapted from Daily [38], with permission.)*

A given head and penstock configuration establishes the optimum jet velocity and diameter. The size of the wheel determines the speed of the machine. For a wheel of diameter D, the speed in radians per second is

$$\Omega = 2\frac{u_1}{D} = \frac{2\phi}{D}\sqrt{2gH} \tag{4.59}$$

Using Eq. (4.16*a*) for the power P and the equation

$$Q = V_j\frac{\pi d_j^2}{4} = C_v\sqrt{2gH}\,\frac{\pi d_j^2}{4} \tag{4.60}$$

for the flow rate Q, one obtains for the specific speed of the machine the relationship

$$N_s = 2^{1/4}\sqrt{2\pi\eta C_v}\,\phi\frac{d_j}{D} \tag{4.61}$$

or approximately

$$N_s = 1.3\frac{d_j}{D} \tag{4.62}$$

Practical values of d_j/D for Pelton wheels to ensure good efficiency are in the range 0.04 to 0.1, corresponding to N_s values in the range 0.05 to 0.13 (10 to 25 in metric units using metric horsepower). In Turgo turbines, the relative wheel diameter can be half that of a Pelton wheel resulting in specific speeds approximately twice that of the conventional design. Higher specific speeds are possible with multiple nozzle designs. The increase is proportional to the square root of the number of nozzles. Crossflow turbines can operate at even higher specific speed (N_s = 0.6) because the width of the runner can be much larger than the diameter, which permits large values of flow through a relatively small diameter runner. This is possibly one of the reasons why the crossflow turbine has seen application over such a wide range of head and power. However, in considering an impulse unit, one must remember that efficiency is based on net head and the net head for an impulse unit is generally less than the net head for a reaction turbine at the same gross head because of the lack of a draft tube.

Reaction Turbines. The main difference between impulse wheels and reaction turbines is the fact that a pressure drop takes place in the rotating passages of the reaction turbine. This implies that the entire flow passage from the turbine inlet to the discharge at the tailwater must be completely filled. A major factor in the overall design of modern reaction turbines is the draft tube. This was not always the case. In earlier days, when low-speed, large-diameter Francis turbines were installed under low heads, the lack of a draft tube or the use of a very short conical draft tube resulted in relatively minor losses. This was not particularly critical for installations which are underdeveloped based on today's standards since water was spilled over the dam much of the year. Currently, it is desirable to reduce the overall equipment and civil construction costs by using high-specific-speed propeller runners, and the draft tube is extremely critical from both a flow stability and an efficiency viewpoint. Since the runner diameter is relatively small, a substantial percentage of the total energy is in the form of kinetic energy leaving the runner. To recover this efficiently, considerable emphasis should be placed on the draft tube design.

The practical specific speed range for reaction turbines is much broader than for impulse wheels. This is due to the wider range of variables which control the basic operation of the turbine. As an illustration of the design and operation of reaction turbines for constant speed, refer again to Fig. 4.1. The pivoted guide vanes allow for control of the magnitude and direction of V_1, i.e., V_1 and α_1. The relationship between blade angle, inlet velocity, and peripheral speed for shock-free entry can be obtained from Eqs. (4.7) and (4.8), as

$$\cot \beta_1 = \frac{C_1 \cos \alpha_1 - \phi}{C_1 \sin \alpha_1} \tag{4.63}$$

Without the ability to vary the blade angle, it is obvious that the requirement for shock-free entry cannot be completely satisfied at partial flow. This is the distinction between the efficiency of fixed-propeller and Francis types at partial loads and the fully adjustable Kaplan design.

Referring to Eq. (4.24), optimum hydraulic efficiency would occur when α_2 is equal to 90°. However, overall efficiency of the turbine is dependent on the optimum performance of the draft tube which occurs with a little swirl in the flow. Thus, best overall efficiency occurs with $\alpha_2 \cong 75°$ for high-specific-speed turbines. The hydraulic efficiency [Eq. (4.24)] is approximately

$$\eta_h = 2\phi C_1 \cos \alpha_1 \tag{4.64}$$

With α_1 in the range of 10° to 25° and $C_1 \cong 0.6$, the speed coefficient ϕ is approximately 0.8, compared to a little less than 0.5 for an impulse turbine. Note also that $C_1 = 0.6$ implies that only 36 percent of the available head is converted to velocity head at the turbine inlet, compared to 100 percent for the impulse wheel.

The determination of optimum specific speed in a reaction turbine is more complex since there are more variables. For a radial-flow machine (refer to Fig. 4.1), a relatively simple expression can be derived. Combining Eqs. (4.19), (4.19*a*), (4.23), (4.16*a*), and (4.59), the expression for specific speed, Eq. (4.28), is

$$N_s = 2^{5/4}\left[2\pi\eta_l f_b C_1 \sin \alpha_1 \frac{B}{D_1}\right]^{1/2} \phi \tag{4.65}$$

or approximately

$$N_s = 5.5\left[C_1 \sin \alpha_1 \frac{B}{D_1}\right]^{1/2} \phi \tag{4.66}$$

In the smaller sizes of axial-flow machines, there are standardized units. These units are made up of standard components, such as shafts and blades. For such cases, it is easy to show that

$$N_s \sim \frac{\sqrt{\tan \beta}}{n_B^{3/4}} \tag{4.67}$$

where β is the blade pitch angle and n_B is the number of blades. The advantage of controllable pitch is also obvious from this formula, the best specific speed simply being a function of pitch angle.

Using standardized design charts for Francis turbines (Fig. 4.29), N_s is normally found to be in the range 0.3 to 2.5 (58 to 480 in metric units using metric horsepower). Note that the actual values of the reference diameters used in Fig.

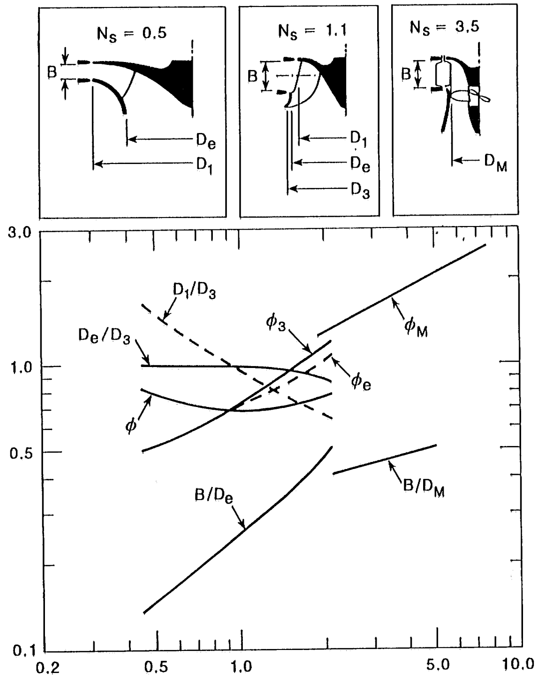


FIGURE 4.29 Empirical design constants for reaction turbines. *(Data from Davis [40] and de Sierro and de Lava [41].)*

4.29 are defined in the figure itself and do not necessarily correspond to D_1 and D_2 in Fig. 4.1.

It should be further noted that for Francis units, ϕ is approximately constant and N_s is proportional to $(B/D_1)^{1/2}$. It can be shown that velocity component based on the peripheral speed at the throat, ϕ_e, is proportional to N_s. For axial-flow machinery, B/D_M is approximately constant and ϕ_M is also proportional to N_s. For minimum cost, peripheral speed should be as high as possible, consistent with cavitation-free performance. Under these circumstances, N_s for a given head would vary inversely with the square root of head. This is precisely the trend noted in a survey of installed equipment, as shown in Fig. 4.30.

Performance Comparison. The physical characteristics of various runner configurations are summarized in Fig. 4.31. It is obvious that the configuration changes with speed and head. This can be expressed in terms of peak efficiency versus specific speed, as illustrated in Fig. 4.32. As already discussed, impulse turbines are efficient over a relatively narrow range of specific speed, whereas Francis and propeller turbines have a wider useful range. Variable geometry is an important consideration when a turbine is required to operate over a wide range of load. Pelton wheels and Turgo wheels tend to operate efficiently over a wide range of power loading because the needle valve is capable of metering flow at a constant discharge velocity. Thus, the relative velocities through the runner remain fixed in magnitude and direction, which allows for maximum runner efficiency independent of flow rate.

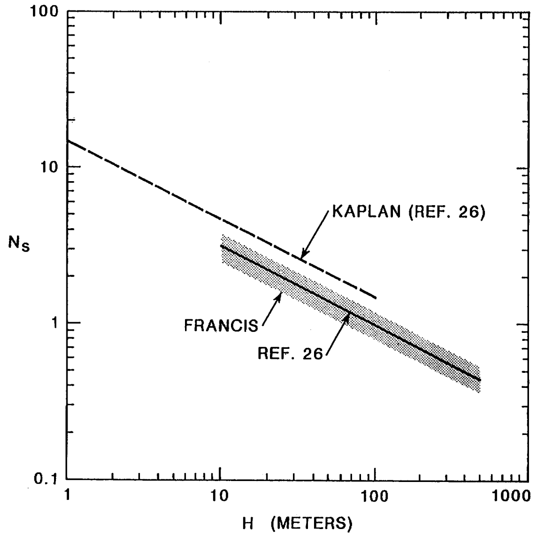


FIGURE 4.30 Specific speed versus head. *(Adapted from Sinclair and Rodrigue [26].)*

A comparison of efficiency variation with load as a function of the level of sophistication of a tubular turbine is illustrated in Fig. 4.33. As shown, fixed gates and blade settings result in peak efficiency at 100 percent load, and the efficiency drops off rapidly with changes in load. On the other hand, a Kaplan-type tubular turbine can maintain efficiency over a relatively broad range of conditions. Also illustrated in the same diagram is the variation of efficiency for an impulse wheel. Although its peak efficiency is less than the high-speed tube turbine, the impulse unit is able to maintain a relatively high efficiency over a wide range of conditions. Both Francis and Deriaz units are designed to operate at medium specific speed. The efficiency of these two turbines is compared in Fig. 4.34. Again, the advantages of controllable-pitch blades are evident in this comparison. The decision of whether to select a simple configuration with a relatively "peaky" efficiency curve or to go to the additional expense of installing a more complex machine with a broad efficiency curve will depend on the expected operation of the plant. If the head and flow are relatively constant, then the less expensive choice is justified. On the other hand, many run-of-the-river plants may be more economical with the installation of Kaplan or Deriaz units.

Detailed performance maps that are typical of Francis- and Kaplan-type turbines are shown in Figs. 4.35 and 4.36. Figure 4.35 clearly illustrates the narrower operating range of a fixed-geometry Francis turbine when compared to the fully adjustable Kaplan type. Note that maximum efficiency does not occur at maximum gate opening but at an opening of about 75 percent, where 85 percent power is produced under design head. Figure 4.36 makes quite clear the operating advantages which accrue with the increased complexity of the Kaplan design.

Because various types of turbines tend to operate best over different specific speed ranges, the head and flow available at a given site dictate what

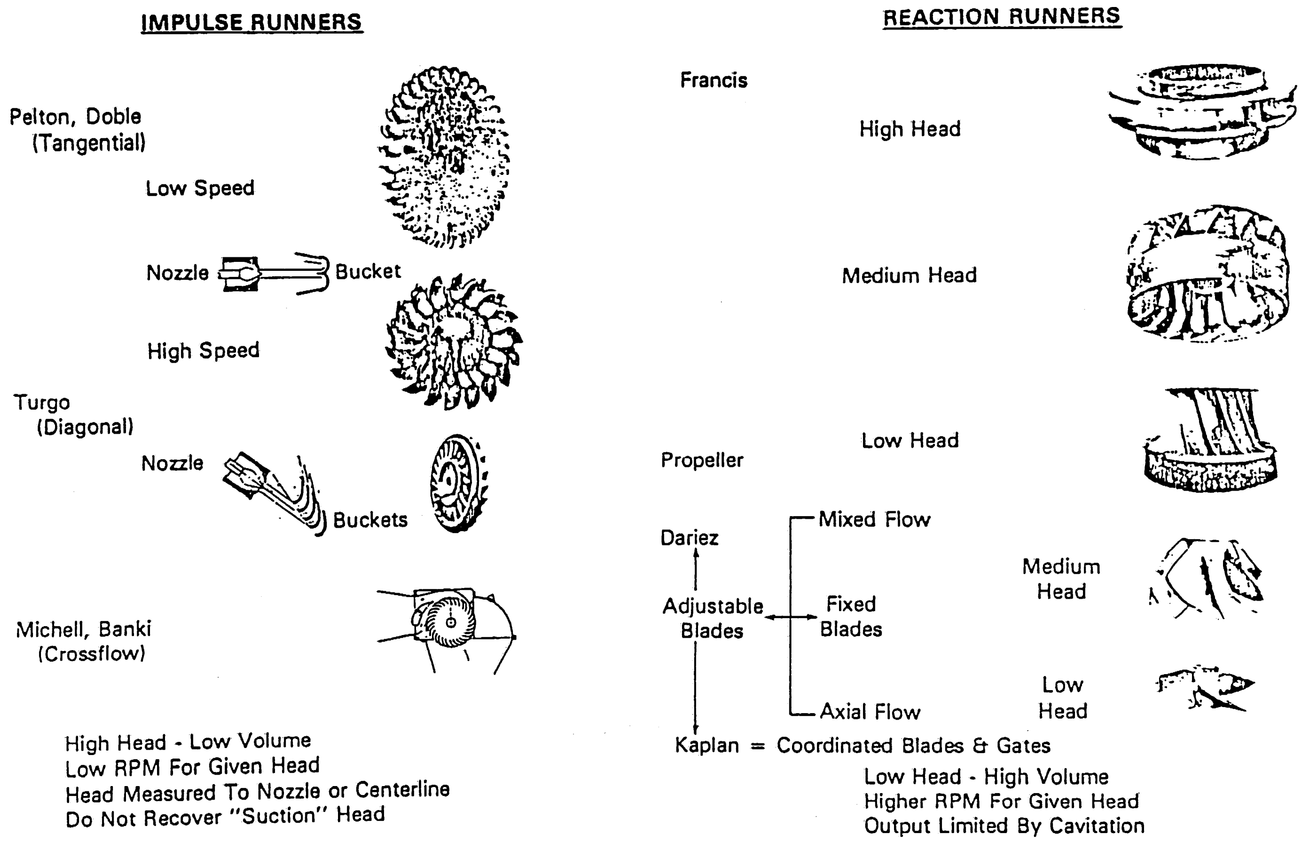


FIGURE 4.31 Physical characteristics of various turbine runners compared. *(Adapted from Mayo [37], with permission.)*

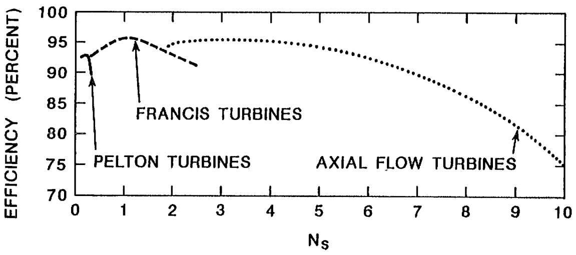


FIGURE 4.32 Efficiency of various types of turbines as a function of specific speed.

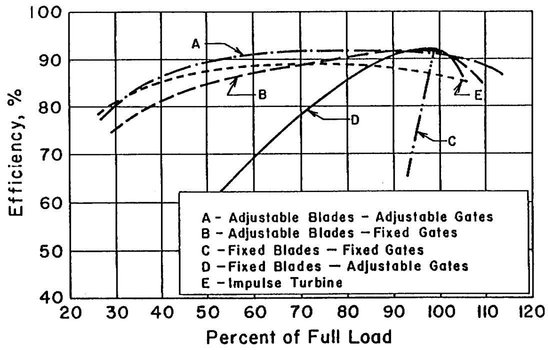


FIGURE 4.33 Turbine efficiency as a function of load. Comparison between an impulse turbine and various configurations of a high-speed propeller unit.

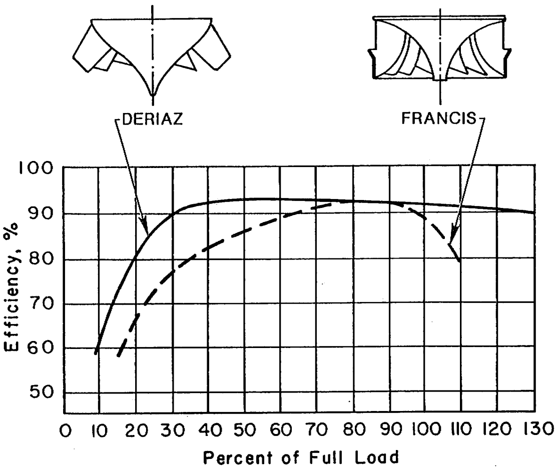


FIGURE 4.34 Efficiency versus load of Deriaz and Francis turbines of equivalent specific speed and size.

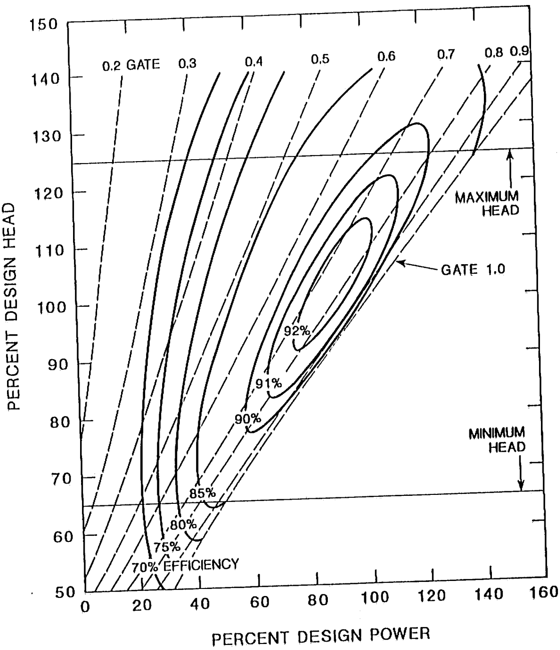


FIGURE 4.35 Francis turbine performance—23 to 46 m (75 to 150 ft) head range. *(From Bureau of Reclamation [36].)*

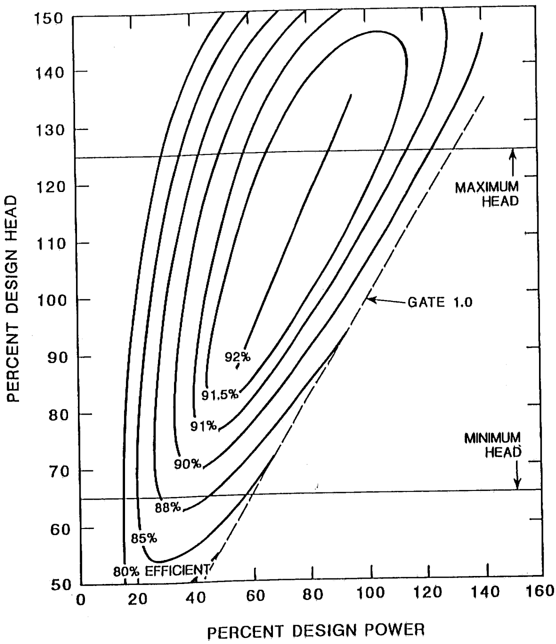


FIGURE 4.36 Performance diagram for an adjustable blade turbine, N_s = 3.3. *(From Bureau of Reclamation [36].)*

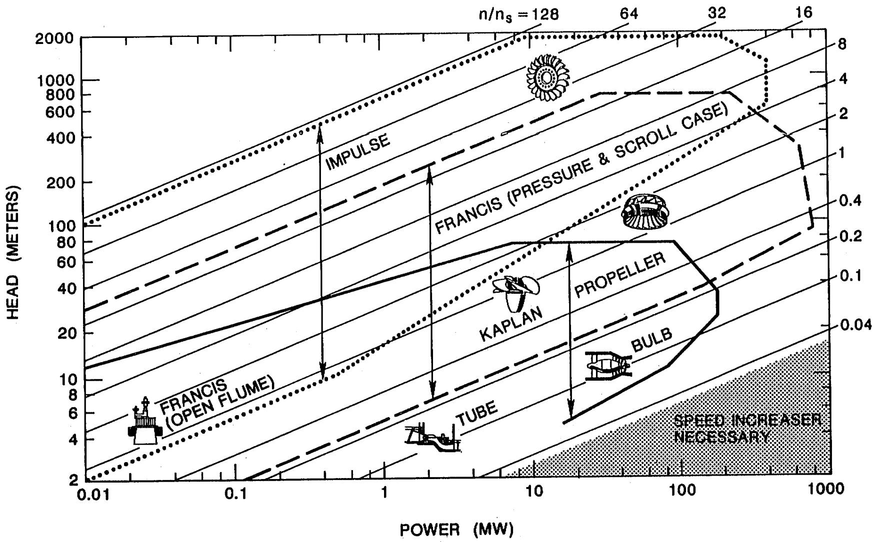


FIGURE 4.37 Application chart for various turbine types. (Specific speed is in metric units of head (meters) and power (kilowatts).)

options are practical. This is illustrated in Fig. 4.37, where the various types of turbines that would be useful at various combinations of head and desired power output are plotted. The range of application of various designs is based on several criteria. For example, each type of turbine is optimum over a given range of specific speed. The rotational speed of the turbine is dictated by the generator when it is directly driven. Sixty-cycle generators are generally limited to the range 60 to 900 r/min. At constant n and n_s, head and flow are related by

$$H \sim \left[\frac{n}{n_s}\right]^{4/5} P^{2/5} \tag{4.68}$$

Thus, the upper limits for each turbine type represent maximum r/min (if possible without cavitation) and minimum n_s. The lower boundary is determined from the lowest r/min and maximum n_s without cavitation. An upper limit on head for each turbine type is dictated by cavitation (limits on setting) and mechanical considerations. This is shown in Fig. 4.37, where lines of constant n/n_s are plotted. Note that each turbine type is bracketed by a range of n/n_s and limits on head and power. In small units, generator speed is often in the range 600 to 900 r/min. This means that there is a range of head and flow (low head, high flow rate) in which a speed increaser is necessary to match turbine speed to generator speed.

Survey of Equipment

Most large-scale turbine equipment is custom-designed and manufactured for a given application; on the other hand, the recent interest in small-scale hydropower has stimulated the turbine manufacturers to produce turbines suitable for this application. In many cases, larger units have been scaled down to match the lower head and power requirements. As cost of equipment has a significant impact on the economic feasibility of a small-scale installation, a major thrust has been made to develop standardized units to reduce cost. Many of these standardized units are supplied complete with the generator and auxiliary equipment. The larger and well-established equipment manufacturers are adding such equipment as a line item, and the number of smaller manufacturers is rapidly increasing in response to the anticipated demand. The small companies are in general developing equipment for the lower power outputs in the range of less than 20 kW.

Table 4.2 is based on commercially available equipment. Included in this summary is the range of application of both custom-designed and standardized design equipment. This table should be compared with Fig. 4.37. This summary should be used only as a guide to available turbines, as different units are rapidly being offered by the various manufacturers. It can readily be seen that several types of turbines are available for a given head and power output. The cross-flow turbine covers a wide range of conditions in the low power range. The propeller turbine classification includes the vertical Kaplan, bulb, and tube turbine units. The standard configuration Kaplan turbine is commonly used for the higher heads and power output, and the bulb for essentially the same output at lower heads. The tube turbine range includes the lower heads and power outputs. Standardized tube turbines are available in this range and may be economically attractive for mini hydro projects and also for upgrading or rehabilitation of old hydropower stations.

TABLE 4.2 Turbine Performance Characteristics

Turbine type	Head, m* min	Head, m* max	Min/max head as % of rated head	Capacity,† kW min	Capacity,† kW max	Min/max capacity as % of rated power
Pelton	<2	2000	—	0.1	500,000	—
Francis	<2	1000	50–125	0.1	1,000,000	35–115
Kaplan	<2	200	45–150	0.5	500,000	10–115
Bulb	<2	40	45–140 and over	2	100,000	10–115
Rim	<2	>30	45–140	100	8,000	10–115
Tube	<2	25	55–140	0.5	60,000	35–115
Crossflow	<2	>30	50–125	1	2,000	10–115

*1 m = 3.28 ft.
†Based on the manufacturing capabilities of different turbine manufacturers, and not on data of existing installations.

Some standardized propeller turbines are also available for micro-hydropower sites, which are low-head and have power outputs of less than 100 kW. These small units have been developed specifically for this application, and attempts have been made to simplify the machine and thus lower initial equipment costs. The simplification may result in reduction of efficiency of the unit, and this should be considered in the assessment of economic feasibility.

Accurate performance data are usually not available for smaller turbines. In fact, model tests are often not performed for turbines smaller than about 5 to 10 MW. As an example, consider a 500-r/min, 15-MW turbine operating at its design point with an efficiency of 92 percent under a 100-m (328-ft) head. The same design could be scaled down to 500 kW at 1200 r/min and 50-m (164-ft) head. Since the specific speed is constant, the size, speed, and head will vary according to

$$\frac{H_1}{H_2} = \frac{n_1^2 D_1^2}{n_2^2 D_2^2} \tag{4.69}$$

Thus, $D_1/D_2 = 3.39$. The change in efficiency could be estimated by using a standard formula (see next section) in reverse (a step-down equation, if you will). This yields $\eta_2 = 89.9$ percent. Scaling down to even smaller sizes would bring about even more dramatic reductions in efficiency. This is true only for reaction turbines in which leakage and frictional losses are disproportionately higher in the smaller sizes.

Present Trends in Turbine Development

As previously mentioned, attention is being directed toward the development of standardized turbines to cover a wide range of applications. Some turbine manufacturers are exploring the possibility of using pumps operated as turbines. It is expected that continued efforts will be made in this area. For many remote and relatively inaccessible sites, lightweight turbines of small size would be attractive. The use of plastics, etc., for various elements of smaller turbines could perhaps reduce cost through mass production techniques as well as reduce weight. These elements may require more maintenance, but the lower cost of the parts may offset the increased maintenance cost.

4.5 PERFORMANCE TESTING

Model Tests

Model testing is an important element in the design and development phases of turbine manufacture. Most laboratories equipped with model turbine test stands are owned by manufacturers. However, there are independent laboratories available where relative performance evaluations between competing manufacturers can be carried out. Major hydro projects have traditionally had proof-of-performance tests in model scale as part of the contract (at either an independent laboratory or the manufacturer's laboratory). In addition, it has been shown that competitive model testing at an independent laboratory can lead to large savings at a major project because of improved efficiency [27].

Recently, turbine design procedures have been dramatically improved through the use of sophisticated numerical analysis of the flow characteristics. These analysis techniques, linked with design programs, provide the turbine designer with powerful tools for achieving highly efficient turbine designs. In spite of this progress, computational methods require fine-tuning with model tests. In addition, model testing is necessary for determining performance over a range of operating conditions and for determining quasi-transitory characteristics. Model testing can also be used to eliminate or mitigate problems associated with vibration, cavitation, hydraulic thrust, and pressure pulsation [28].

A typical turbine test loop is shown in Fig. 4.38. All test loops perform basically the same function. A model turbine is driven by high-pressure water from a head tank and discharges into a tail tank, as shown. The flow is recirculated by a pump, usually positioned well below the elevation of the model to ensure cavitation-free performance of the pump while performing cavitation testing with the turbine model. One important advantage of a recirculating turbine test loop is that cavitation testing can be done over a wide range of cavitation indices at constant head and flow. The experimental data shown in Fig. 4.13 were obtained in a test rig similar to the one shown in Fig. 4.38.

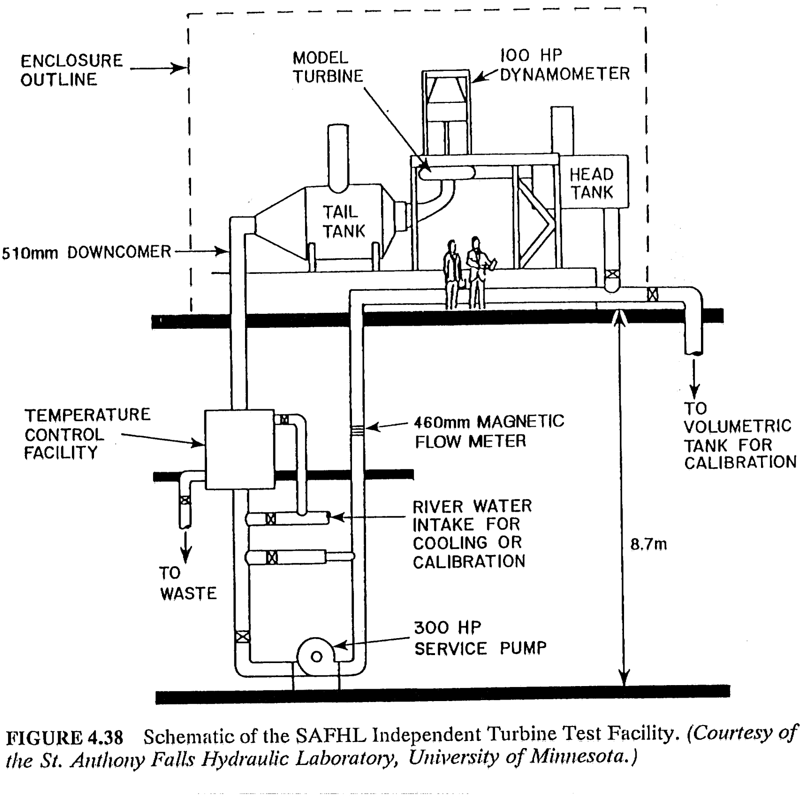


FIGURE 4.38 Schematic of the SAFHL Independent Turbine Test Facility. *(Courtesy of the St. Anthony Falls Hydraulic Laboratory, University of Minnesota.)*

Figure 4.39 is a plot of the range of head, flow, and specific speed of various turbine models that can be tested in the test loop of Fig. 4.38. The major limitations to the range of testing in a test loop are the service pump characteristics and the useful range of torque and speed of the dynamometer. Many facilities have provision for more than one pump to operate in series or in parallel to expand the useful operating envelope of the test loop.

The extrapolation of model test data to prototype values has been a subject of considerable debate for many years. In principle, Eqs. (4.32) to (4.35) can be used to predict prototype values of flow, speed, power, etc., from model tests.

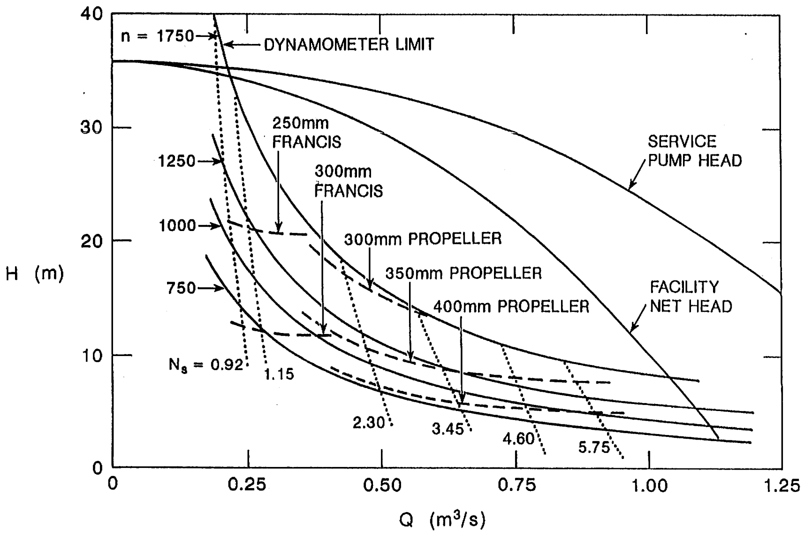


FIGURE 4.39 Typical range of operating conditions for the turbine test stand shown in Fig. 4.38. *(Courtesy of the St. Anthony Falls Hydraulic Laboratory.)*

This assumes that the flow patterns in model and prototype are identical. Unfortunately, there are many factors that lead to scale effects, i.e., the prototype efficiency and model efficiency are not identical at a fixed value of specific speed. The cited scale-up formulae are based on inviscid flow. As discussed in Sec. 4.3, there are several sources of energy loss which lead to an efficiency that is less than ideal. All of these losses follow different scaling laws and, in principle, perfect similitude can only be achieved with a full-size model, i.e., the prototype. There have been several attempts at rationalizing the process of scaling up model test data [29]. The new International Electrotechnical Test Code [30] proposes to place losses into two categories: (a) those dependent on Reynolds number, and (b) those that are not. The difference in efficiency between model and prototype is then

$$\Delta\eta = \delta(1 - \eta_m)\left[1 - \left[\frac{Re_m}{Re_p}\right]^{0.16}\right] \tag{4.70}$$

where Re = $2\Omega R^2/\nu$
R = the radius of the runner at the outlet
ν = kinematic viscosity

and where δ is the percentage of losses that are Reynolds number–dependent at peak efficiency, and the subscripts m and p denote model and prototype, respectively. δ is dependent on both design specific speed and Reynolds number. The new IEC test code proposes to standardize δ for different machines at a reference Reynolds number of 7×10^6. It is also proposed to apply $\Delta\eta$ over the full operating range of the machine, which is not correct since the relative importance of the different losses varies. It should also be pointed out that other losses such as in the draft tube and "shock losses" at the runner inlet may not be independent of Reynolds number. However, a simplified scaling procedure is necessary for standardization of laboratory practice. Further details can be found in Refs. 28 to 30.

Field Tests

A model test will verify the performance of a given turbine design. It should be emphasized that model test results are only valid when geometric similitude is adhered to, i.e., there is no guarantee that the prototype machine is an accurate reproduction of the design. In addition, approach flow conditions, intake head losses, the effect of operating other adjacent units, etc., are not simulated in model tests. For these reasons, field performance tests are often performed.

There are several different types of field tests which serve different purposes. The *absolute* efficiency is measured for acceptance or performance tests. *Relative* efficiency is often measured when operating information or fine-tuning of turbine performance is desired. Field tests are also carried out for commissioning a site and for various problem-solving activities. Basic procedures are covered by codes of the American Society of Mechanical Engineering and the International Electrotechnical Commission [4, 31]. The major difference between an "absolute" and a relative or index test is in the measurement of flow rate. Net head is evaluated in the same manner for each procedure. There are a variety of methods for measuring flow that are code-accepted. These include the pressure-time technique, tracer methods (salt velocity, dye dilution), area-velocity (Pitot tubes or current meters), volumetric (usually on captive pumped storage sites), venturi meters, and weirs. The thermodynamic method is actually a direct measure of efficiency. Flow is not measured. In addition to the code-accepted methods, it has been demonstrated that acoustic meters can measure flow in the field with comparable accuracy [32].

The *pressure-time technique* relies on measuring the change in pressure necessary to decelerate a given mass of fluid in a closed conduit. The method requires the measurement of the piezometric head at two cross sections spaced a distance L apart. For a constant cross section, the relationship between head and flow is obtained from the integration of Newton's second law:

$$Q = \frac{gA}{L} \int_0^{t_0} \Delta\left[\frac{p}{\gamma} + z\right] dt \qquad (4.71)$$

where A is the cross-sectional area of the conduit and $\Delta[(p/\gamma) + z]$ is the difference in the piezometric head. t_0 is the time necessary to decelerate the flow. A downstream valve or gate is necessary for this procedure. Leakage past this valve must be minimal and must be measured accurately for inclusion in the overall flow rate. Equation (4.71) is valid only for an inviscid fluid. In practice, frictional effects have to be taken into account. This is illustrated in Fig. 4.40. This is a typical plot, as might be recorded using a personal computer– (PC-) based data acquisition system [33]. Initially, the head difference is negative due to frictional resistance. As the valve is closed, the head difference becomes positive. When the fluid comes to rest, the head difference drops to zero. The shaded area in the diagram corresponds to the integral in Eq. (4.71). An iterative procedure is necessary to determine the base of the area, shown as a dotted line, which corresponds to the viscous head loss that is a function of the instantaneous flow rate. This procedure is relatively simple using a computer and the measured initial fric-

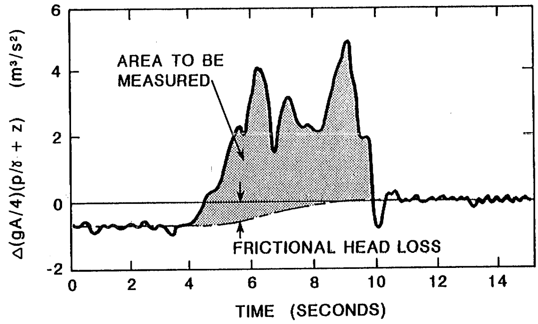


FIGURE 4.40 Schematic of pressure-time method using a computer-based data acquisition system. *(Adapted from Hecker and Nystrom [33].)*

tional loss. The limitations to this technique include the requirement to reject the load for each test point, and the need to estimate or measure any leakage. An adequate length of conduit is required and the conduit geometry must be accurately measured.

As presently written, the test codes refer to a device for recording the piezometric head difference photographically, using manometers (the original Gibson apparatus). However, it is anticipated that revised versions of the codes will include the use of modern pressure-sensing and data acquisition equipment. Modern instrumentation considerably simplifies the procedure and results in more repeatable results [34].

The *salt velocity method* is based on measuring the transit time, between two sensors, of an injected cloud of concentrated salt solution. Given the volume of the conduit between sensors, the flow rate may be calculated from the average transit time. The passage of the salt cloud at a given location is detected by electrodes that measure the change in conductivity of the liquid. A typical set of data is shown in Fig. 4.41. The time of transit is determined from the time between centroids of the conductivity-time curves for each of a pair of locations.

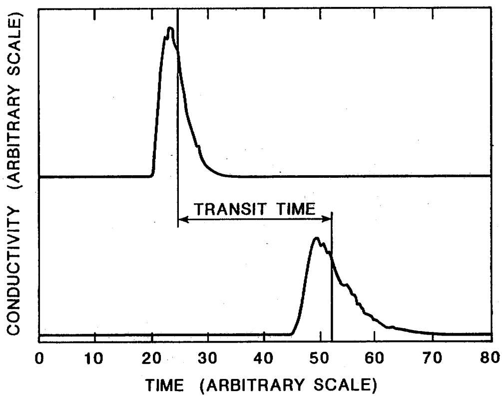


FIGURE 4.41 Schematic of data obtained with the salt velocity method. *(Adapted from Hecker and Nystrom [33].)*

The *dye-dilution method* is based on conservation of a tracer continuously injected into the flow. A sufficient length for complete mixing is necessary for accurate results. The data required are the initial concentration and injection flow rate of the tracer and the measured concentration of the fully mixed tracer at a downstream location. The flow rate is determined from conservation of mass:

$$Q = \frac{Q_1 C_1}{C_2 - C_0} \tag{4.72}$$

where C_0 is the background concentration of the tracer, C_1 is the concentration of the injected tracer solution at a flow rate Q_1, and C_2 is the concentration of the fully mixed tracer. Suitable tracers include radioisotopes, salts, and fluorescent dyes. Fluorescent dyes such as rhodamine WT are quite popular because of commercially available fluorometers that can make accurate measurements of concentration. The method is quite simple, but care is necessary to achieve precise results. Further details on both tracer methods can be found in Refs. 33 and 34.

In principle, *area-velocity measurements* are also quite simple. Either Pitot tubes or propeller-type current meters are used to measure point velocities that are integrated over the flow cross section. The method is applicable to either closed conduits or open channels. A relatively uniform velocity distribution is necessary for accurate results. A single unit can be traversed across the conduit or a fixed or movable array of instruments can be used to reduce the time for data collection. Figure 4.42 shows a rack of current meters that can be moved vertically up and down in the stop-log slots of an intake channel. Further details can be found in Refs. 33 and 34. Suitable texts on hydraulics can be referred to for discussion of measurement of flow with weirs or venturi meters.

The *thermodynamic method* is a direct indication of turbine efficiency. Flow rate is not measured. In its simplest form, the method assumes adiabatic conditions, i.e., no heat transfer from the flow to its surroundings. Under these conditions, that portion of the available energy not utilized in the machine to produce useful work results in increased internal energy of the fluid, which is sensed as an increase in temperature. By measuring the temperature upstream and downstream of the turbine, the efficiency can be determined from the energy equation in the following form:

$$\eta = 1 - \frac{C_p J(\theta_2 - \theta_1)}{H} \tag{4.73}$$

where C_p is the specific heat at constant pressure, H is the net head, J is the mechanical equivalent of heat, and θ_1 and θ_2 are the water temperatures upstream and downstream, respectively. Although the method is very simple in principle, there are many factors that need to be considered in practice to achieve accurate results such as the effects of heat transfer and variation of fluid properties with temperature and pressure [35].

Acoustic flow meters have been developed which produce results with a precision equal to or greater than the code-accepted methods [32]. Flow velocity is determined by comparing acoustic travel times for paths diagonally upstream and downstream between pairs of transducers. The speed of sound is assumed constant. The difference in travel time is related to the component of flow velocity along the acoustic path (increased travel time upstream, decreased travel time downstream). An extensive evaluation and comparison of this method has been reported [34].

Index tests circumvent the problem of accurate flow measurement by measur-

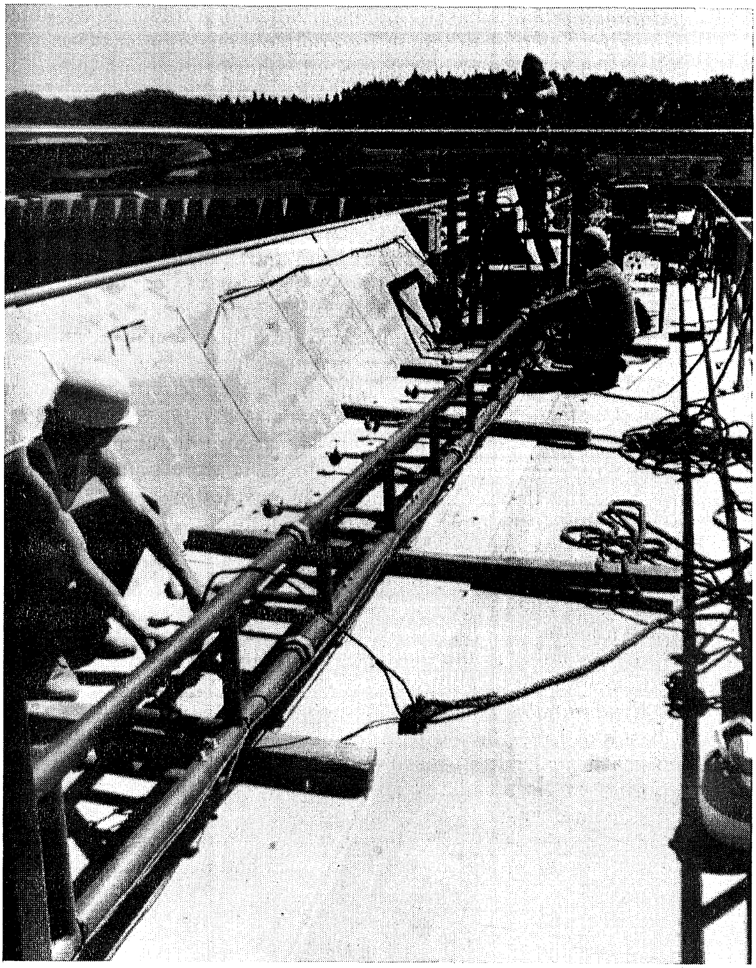


FIGURE 4.42 View of a rack of current meters used to measure the flow into a 4-MW pit turbine. *(Courtesy of the St. Anthony Falls Hydraulic Laboratory.)*

ing relative flow detected by the differential pressure between two points in the water passages leading to the runner. Often the differential pressure is measured with Winter-Kennedy taps which are positioned at the inner and outer radii of the spiral case of a turbine. Calibration of properly placed Winter-Kennedy taps show that flow rate is very closely proportional to the square root of the pressure difference. Index testing is useful for calibration of relative power output versus gate opening and for optimizing the various combinations of gate opening and blade setting in Kaplan units. The use of index testing to optimize cam settings in Kaplan turbines has resulted in substantial increases in weighted efficiency (i.e., a flatter efficiency curve over the full range of operation). A typical example of turbine performance determined by index testing is shown in Fig. 4.43. Further details on index testing is given in Ref. 4.

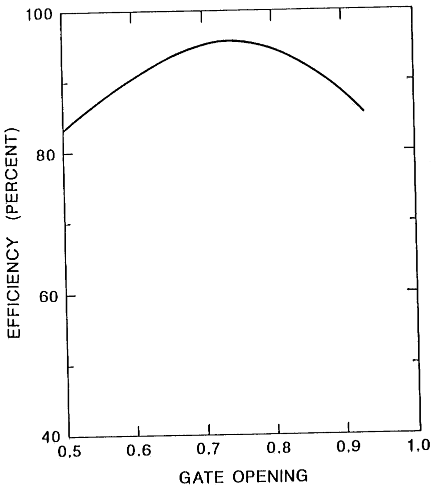


FIGURE 4.43 Typical calibration of a turbine using the index method. *(Courtesy of the St. Anthony Falls Hydraulic Laboratory.)*

4.6 HYDRAULIC STRUCTURES AND OPERATIONAL CONSIDERATIONS

Integration of Turbine with Inlet and Outlet Works

From the discussion in Sec. 4.4, it is apparent that several types of turbines may be appropriate for given flow conditions. It is therefore necessary to make a decision as to which particular unit is most economical. In addition to the cost of the turbine and generator, the cost of the associated civil works must be considered, as this cost can represent a large portion of the total cost of a hydropower installation. Some types of turbines require larger civil works than others. Several alternate preliminary layouts should be evaluated, each of which may have different inlet and draft tube requirements. The necessity of this evaluation has become increasingly evident with the recent interest in retrofitting existing sites for power production. The original turbines either may not exist or may not be capable of repair, or it may be desirable to replace the unit with a modern turbine of different capacity. The condition and the extent of the existing civil works may influence the type of turbine to be finally selected. For example, if an open-flume Francis turbine was originally installed but is no longer usable, it may prove to be more economical to install a new turbine of the same type or a different design with roughly the same overall dimensions as the original equipment if the rest of the structure is still in good condition. In other cases, it may be most cost-effective to abandon the existing structure and select a different type of turbine. It is therefore necessary to analyze each site on an individual basis. In so doing, considerable cost savings may be realized.

Cavitation and Turbine Setting

Another factor that must be considered prior to equipment selection is the evaluation of the turbine with respect to tailwater elevations. As previously discussed, hydraulic turbines are subject to pitting due to cavitation. For a given head, a smaller, lower-cost, high-speed runner must be set lower (i.e., closer to tailwater or even below tailwater) than a larger, higher-cost, low-speed turbine runner. Also, atmospheric pressure or plant elevation above sea level is a factor, as are tailwater elevation variations and operating requirements. This is a complex subject which can only be accurately resolved by model tests. Every runner design will have different cavitation characteristics; therefore, the anticipated turbine location or setting with respect to tailwater elevations is an important consideration in turbine selection.

Cavitation is not normally a problem with impulse wheels. However, by the very nature of their operation, cavitation is an important factor in reaction turbine installations. The susceptibility for cavitation to occur is a function of the installation and the turbine design. As already discussed, this can be expressed conveniently in terms of Thoma's sigma. The critical value of σ_T is a function of specific speed, as illustrated in Fig. 4.14. As the specific speed increases, the critical value of σ_T increases dramatically. For minimization of cavitation problems, σ_p must be in excess of the value for performance breakdown σ_{TB} denoted on the chart. This can have important implications for turbine settings and the amount of excavation necessary. The criteria for establishing the turbine setting have already been discussed in Sec. 4.3 under "Cavitation." As an example, consider a 500-kW machine operating at 500 r/min under a head of 10 m (32.8 ft). The specific speed is 3.8. This will be an axial-flow turbine having a critical σ_p of about 1.0. At sea level, the maximum turbine setting would be

$$z_{B_m} = 10 - 0.09 - (1.0 \times 10) \approx 0.0 \text{ m}$$

If the same turbine was installed at Leadville, Colorado, elevation ~ 3000 m (9840 ft), the maximum turbine setting would be

$$z_B = 6.7 - 0.09 - (1.0 \times 10) = -3.39 \text{ m } (-11.1 \text{ ft})$$

Considerable excavation would be necessary. Thus, cavitation can be an important consideration.

Speed Regulation

The speed regulation of a turbine is an important and complicated problem. The magnitude of the problem varies with size, type of machine and installation, type of electrical load, and whether or not the plant is tied into an electrical grid. It should also be kept in mind that runaway or no-load speed can be higher than the design speed by factors as high as 2.6. This is an important design consideration for all rotating parts, including the generator.

It is beyond the scope of this section to discuss the question of speed regulation in detail. Unfortunately, the cost of standard governors is disproportionately high in the smaller sizes. Regulation of speed is normally accomplished through flow control. Adequate control requires sufficient rotational inertia of the rotating parts. When load is rejected, power is absorbed, accelerating the flywheel; and when load is applied, some additional power is available from deceleration of the flywheel. Response time of the governor must be carefully selected, since rapid closing time can lead to excessive pressures in the penstock.

A Francis turbine is controlled by opening and closing the guide vanes which vary the flow of water according to the load. The actuator components of a governor are required to overcome the hydraulic and frictional forces and to maintain the guide vanes in fixed position under steady load. For this reason, most governors have hydraulic actuators. On the other hand, impulse turbines are more easily controlled. This is due to the fact that the jet can be deflected or an auxiliary jet can bypass flow from the power-producing jet without changing the flow rate in the penstock. This permits long delay times for adjusting the flow rate to the new power conditions. The spear or needle valve controlling the flow rate can close quite slowly, say 30 to 60 s, thereby minimizing any pressure rise in the penstock.

Several types of governors are available which vary with the work capacity desired and/or the degree of sophistication of control. These vary from pure mechanical to mechanical-hydraulic and electrohydraulic. Electrohydraulic units are sophisticated pieces of equipment and would not be suitable for remote regions. The precision of governing necessary will depend on whether the electrical generator is synchronous or asynchronous (induction type). There are advantages to the induction type of generator. It is less complex and therefore less expensive, but has typically slightly lower efficiency. Its frequency is controlled by the frequency of the grid it is feeding into, thereby eliminating the need of an expensive conventional governor. It cannot operate independently but can only feed into a network and does so with lagging power factor which may or may not be a disadvantage, depending on the nature of the load. Long transmission lines, for example, have a high capacitance and in this case the lagging power factor may be an advantage.

Some general features of the overall regulation problem can be demonstrated by examination of the basic equation for a rotating system

$$J\frac{d\Omega}{dt} = T_t - T_L \tag{4.74}$$

where J = moment of inertia of rotating components
Ω = angular velocity
T_t = torque of turbine
T_L = torque due to load

Three cases may be considered in which T_t is equal to, less than, or greater than T_L.

For the first case, the operation is steady. The other two cases imply unsteady operation, since $d\Omega/dt$ is not constant, and usually a governor is provided so that the turbine output matches the generator load.

Speed regulation is a function of the flywheel effect of the rotating components and the inertia of the water column of the system. The start-up time of the rotating system [36] is given by

$$t_s = \frac{J\Omega^2}{P} = \frac{Jn_0^2}{6818HP_r} \tag{4.75}$$

where J = flywheel effect of generator and turbine, kg · m/s² (1 kg · m/s = 7.22 lb · ft/s)
n_0 = normal turbine speed, r/min
HP_r = rated metric horsepower (1 metric hp = 0.735 k W = 1.01 hp)

The starting-up time of the water column is given by

$$t_p = \frac{\Sigma LV}{gH_r} \tag{4.76}$$

where L = length of water column
V = velocity in each component of the water column
H_r = rated head

For good speed regulation, it is desirable to keep $t_s/t_p > 4$. Lower values can also be used, although special precautions are necessary in the control equipment. It can readily be seen that higher ratios of t_s/t_p can be obtained by increasing J or decreasing t_p. Increasing J implies a larger generator, which also results in higher costs. The start-up time of the water column can be reduced by reducing the length of the flow system, by using lower velocities, or by addition of surge tanks, which essentially reduce the effective length of the conduit. A detailed analysis should be made for each installation, as for a given length, head, and discharge the flow area must be increased to reduce t_p, which leads to associated higher construction costs.

A method for determining the speed rise as a result of load rejection is incorporated in Fig. 4.44 for several specific-speed machines. The abscissa is the ratio of t_g/t_s, where t_g is the full closing time of the governor

$$t_g = 0.25 + t_c \tag{4.77}$$

and t_c is the rated governor time in seconds, which generally varies from 3 to 5 s. With the ratio determined, the percent speed rise S_R for no water hammer and a

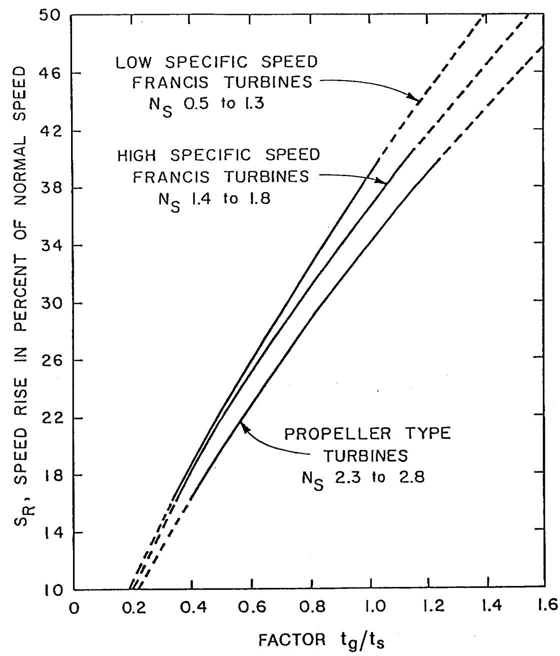


FIGURE 4.44 Speed rise for full gate load rejected with no water hammer. *(From Bureau of Reclamation [36].)*

given specific speed can be found in Fig. 4.44. This value should be modified to include the start-up time of the pipeline, or

$$S_R' = S_R \left[1 + \frac{t_p}{t_c}\right] \tag{4.78}$$

It is desired to keep the speed rise for full-load rejection to less than 45 percent, although in some cases higher percentages can be permitted if regulating ability is sacrificed. If the speed rise is excessive, consideration should be given to providing surge tanks. Further discussion is beyond the scope of this section, and the reader is referred to the previously mentioned reference for more detail.

Emergency and Abnormal Conditions

Emergency conditions can arise if the system experiences a sudden drop in load and the guide vanes remain open as a result of failure in the regulating system. The speed will rise rapidly until a maximum is reached, which is called the *runaway speed*. The runaway speed is dependent on the type of turbine, distributor opening, head, and in the case of a Kaplan turbine, the runner blade angle.

Based on field tests, an equation has been developed for use in predicting the runaway speed n_r for various types of turbines [36]. This formulation is

$$\frac{n_r}{n_d} = K_n \left[\frac{H_{max}}{H_d}\right]^{1/2} (0.28N_s + 1.45) \tag{4.79}$$

where n_d = design speed, r/min
N_s = specific speed
H_{max} = maximum head
H_d = design head

Thus, the ratio of runaway to normal speed is higher for propeller turbines than for Francis turbines and may attain values up to about 2.6. This factor must be considered in the design of the turbine and generating equipment as the increase in centrifugal force can be substantial.

As previously mentioned, the runaway speed for an adjustable-blade runner is a function of the blade angle. If the blade angles are increased from their optimum value, the runaway speed is decreased. At the larger blade angles, the shock losses are higher and equilibrium conditions are reached at lower speeds. However, an increase in blade angle can also result in serious vibration, which can cause damage to the turbine unit.

If the blade angle is decreased, the velocity vectors are changed so that the losses are reduced and the runaway speed is increased. In fact, the theoretical runaway speed with a closed blade runner approaches infinity. This is not actually realized, however, due to frictional windage losses in the generator.

4.7 EXAMPLES

Two simplified examples of application of the concepts developed in this chapter are given below. The purpose of the examples is to illustrate the use of the charts and similarity laws. Use of multiple turbines has not been considered, and possible load and/or discharge variations have not been included in these simplified cases.

EXAMPLE 1 Select a turbine for $H = 100$ m (328 ft) and $Q = 2$ m³/s (70.6 ft³/s). Assuming $\eta = 0.85$, the power is

$$P = \gamma Q H \eta = \frac{1000 \times 2 \times 100 \times 0.85 \times .735}{75} = 1667 \text{ kW}$$

From Fig. 4.37, it is seen that either an impulse or Francis turbine may be suitable. The specific speed is related to the rotational speed by

$$n_s = \frac{n\sqrt{P}}{H^{5/4}} = \frac{n\sqrt{1667}}{(100)^{5/4}} \qquad \text{or} \qquad n = 7.75 n_s$$

Several values of n_s will be used to determine n. Velocity coefficients ϕ are taken from Figs. 4.28 and 4.29 for each turbine, and diameters of the runners are calculated from

$$\phi\sqrt{2gH} = \frac{\pi D_1 n}{60}$$

and

$$\phi_e\sqrt{2gH} = \frac{\pi D_e n}{60} \qquad \text{(only for Francis turbine)}$$

The width of the Francis runner B can also be obtained from the ratio B/D_e in Fig. 4.29, and σ_{TB} taken from Fig. 4.14 for the Francis turbine. The computations are summarized in Table 4.3.

TABLE 4.3 Summary of Computations

	n_s	N_s	n, r/min	ϕ	ϕ_e	D_1, m*	D_e, m*	B/D_e	B, mm†	σ_p
Single	5	0.03	38.8	0.45		9.82				
Jet	10	0.06	77.5	0.45		4.92				
Pelton	20	0.12	155	0.45		2.46				
	30	0.18	232.5	0.45		1.65				
	75	0.45	581	0.83	0.50	1.21	0.73	0.135	98.0	0.03
Francis	100	0.60	775	0.74	0.56	0.81	0.61	0.17	103.9	0.05
	150	0.90	1163	0.71	0.69	0.52	0.50	0.235	117.7	0.09
	200	1.20	1550	0.70	0.78	0.38	0.43	0.285	121.4	0.15

* 1 m = 3.28 ft.
† 1 mm = 0.04 in.

If the practical upper limit for d_j/D_1 is about 0.1, then $D_1 \sim 2.4$ m (7.9 ft), which corresponds to the unit with a specific speed of about 0.12. For generation of 60-Hz power, the rotational speed n is $7200/p$, where p is the number of poles, preferably an even number divisible by 4. With $p = 44$, $n = 164$ r/min, $D_1 = 2.32$ m (7.61 ft), and

$n_s = 21$ or $N_s = 0.13$; these conditions may be reasonable. Use of a speed increaser would probably be desirable to reduce the size of the generator.

From the tabulated values of the Francis turbine, the physical size of the turbine is quite small, and the r/min high. If a four-pole generator is considered, $n = 1800$ r/min, corresponding to $n_s = 232$ or $N_s = 1.4$. A preliminary check can be made of the cavitation limits, and the final turbine setting is based on recommendations of the manufacturer. The allowable turbine setting is given by Eq. 4.48.

$$Z = H_a - H_v - \sigma_p H$$

Assuming $H_a = 8.6$ m (28.2 ft) at an elevation 1500 m (4920 ft) above sea level and neglecting H_v, the setting for a turbine with $n_s = 150$ and $\sigma_p = 0.09$ is 0.4 m (1.3 ft) below tailwater. For $n_s = 100$ and $\sigma_p = 0.05$, the setting z is 3.6 m (11.8 ft) above tailwater, a more reasonable value. An eight-pole generator requires $n = 800$ r/min, and $n_s = 103$ or $N_s = 0.62$. This turbine probably is the best selection for this example, as the generator costs would be lower. In any case, the total costs of the turbine, generator, and other civil works must be evaluated in the final selection.

EXAMPLE 2 A retired dam site with $H = 10$ m (33 ft) is being rehabilitated. A machinery broker has a used Francis turbine to be sold at low cost which is rated at $H = 15$ m (49 ft), $P = 300$ kW, $n = 450$ r/min, and $\eta = 0.9$. Calculate the discharge Q_1, power P_1, and speed n_1 at which the turbine must operate at the proposed site.

If the specific speed is kept constant, the efficiency should be the same for both cases. The specific speed is

$$n_s = \frac{n\sqrt{P}}{H^{5/4}} = \frac{450\sqrt{300}}{15^{5/4}} = 264$$

and

$$Q = \frac{P}{\gamma H \eta} = \frac{300 \times 75}{0.735 \times 1000 \times 15 \times 0.9} = 2.27 \text{ m}^3/\text{s } (80.1 \text{ ft}^3/\text{s})$$

From the similarity relationships for constant diameter,

$$\frac{H}{n^2} = \frac{H_1}{n_1^2} \quad , \quad \frac{Q}{n} = \frac{Q_1}{n_1} \quad \text{and} \quad \frac{P}{n^3} = \frac{P_1}{n_1^3}$$

Therefore

$$n_1 = n\sqrt{\frac{H_1}{H}} = 450\sqrt{\frac{10}{15}} = 367 \text{ r/min}$$

$$Q_1 = Q\frac{n_1}{n} = 2.27 \times \frac{367}{450} = 1.85 \text{ m}^3/\text{s}$$

$$P_1 = P\left[\frac{n_1}{n}\right]^3 = 300\left[\frac{367}{450}\right]^3 = 163 \text{ kW}$$

With a 20-pole generator and 60-Hz current, $n = 7200/20 = 360$ r/min. As this n is slightly less than that required for constant specific speed, the efficiency would be slightly lower for the same head of 10 m. In a situation of this type, one must also be certain that the machine used in the new application is not overstressed, which is not the case here.

4.8 SUMMARY

It has been shown that the head utilized by a turbine runner to produce power can be derived from a suitable form of Euler's equation of motion. The head utilized, and consequently the power developed, is dependent on the velocity vectors of the inlet and exit flow of the runner. The velocity vectors are determined by the operational conditions and the turbine design. Overall efficiency of the turbine is the product of the hydraulic, volumetric, and mechanical efficiencies. Each of these is dependent on various energy losses in the turbine unit, and the origin of these losses has been briefly discussed.

Similarity considerations permit the formulation of dimensionless numbers. These numbers are useful in the extrapolation of test data taken with a model turbine to full-scale conditions and therefore predict performance. One of the most significant dimensionless numbers is the specific speed, which consists of a combination of operating conditions that ensures similar flows in geometrically similar machines. Each type of machine has a value of specific speed that gives maximum efficiency, and it is therefore convenient to classify the various turbine designs by the specific speed at best efficiency.

Cavitation must be avoided in turbines, as it results in loss of performance and can cause erosion damage to the runner and possibly other parts of the structure. Each particular turbine type has its own cavitation limits which are determined from model tests. High-specific-speed turbines are more susceptible to cavitation than low-specific-speed units. The setting of the turbine with respect to the tailwater elevation must be carefully considered to ensure cavitation-free operation and is based on the manufacturer's recommendations.

Turbines can be classified in two broad groups: impulse and reaction turbines. An impulse turbine is driven by a high-velocity jet impinging on buckets around the periphery of the wheel, whereas the reaction turbine requires that the flow passages be completely filled. Reaction turbines can be subdivided further into radial-, mixed-, or axial-flow types. The radial- and mixed-flow types have fixed runner blades, except for the Deriaz turbine, and the axial-flow machine may have either fixed or adjustable blades. In addition to differences in blade geometry, each type of reaction turbine has different requirements for a draft tube. The draft tube is considered part of the turbine, and its energy losses are charged to the turbine performance. Some draft tube configurations may require a large amount of excavation to achieve the desired turbine setting.

The overall efficiency of an impulse turbine is quite constant over a broad range of operating conditions, which is achieved by throttling of the flow at the nozzle. Fixed-blade reaction turbines have a more peaky efficiency curve, whereas the efficiency curve for adjustable-blade units is relatively flat. The latter unit is particularly suited for installations subject to a wide variation in flow conditions. However, an economic analysis must be made to justify the higher cost of the fully adjustable turbine.

A wide variety of turbines are becoming commercially available for application to small-scale hydropower sites. Standardized units are offered by manufacturers in a range of sizes to reduce equipment costs. With increased demand for lower-cost units to make marginal sites feasible, it is expected that further developments in standardization will be made. The manufacturers should be contacted for their recommendations.

Several operational conditions are also significant. Most turbines are designed to operate at a constant rotational speed which is controlled by a governor. Some general guidelines are given concerning speed regulation. With a sudden loss in electrical load, the turbine will reach a runaway speed that is considerably greater than the normal speed. The turbine and generator must be designed to tolerate the additional centrifugal forces. Provisions should also be made for emergency shutdown of the flow to the turbine, either by wicket gates or appropriate valves.

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CHAPTER 5
HYDRAULIC CONVEYANCE DESIGN

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5.1 BACKGROUND

Hydraulic conveyance facilities are the required canals, intakes, penstocks, etc., to convey the water from a reservoir to the turbine, and from the turbine back to the river downstream. Their design is normally the responsibility of civil engineers working on the hydroplant design. They are an important component of this design task, since the problems encountered at hydropower facilities are often related to an inadequate hydraulic design.

The generator output of a hydropower facility depends upon the *net* head available to the turbine, by the equation

$$P(\text{kW}) = \frac{e\ \gamma\ (\text{lb/ft}^3)\ Q\ (\text{ft}^3/\text{s})\ H_n\ (\text{ft})}{737}$$

$$P(\text{kW}) = \frac{e\ \gamma\ (\text{N/m}^3)\ Q\ (\text{m}^3/\text{s})\ H_n\ (\text{m})}{1000} \tag{5.1}$$

where P = generator power output, kW
e = turbine or generator efficiency as a fraction
γ = specific weight of the water
Q = turbine discharge
H_n = net head on the turbine or generator unit

The net head on the turbine unit is equal to the gross available head (the difference between headwater and tailwater elevations) minus the sum of head losses in the hydraulic conveyance system. The basic idea is to get the flow to and from the turbine without losing too much head and with no structural or operational problems. At the same time, the costs of these components and their operation must be considered. Thus, the final decision is economic: to minimize the sum of construction, operation, and maintenance costs of hydraulic conveyance facilities, the value of energy that is not generated and lost to head losses, and the operation, maintenance, and outage costs caused by inadequacies in the hydraulic conveyance facility design.

The hydraulic conveyance facilities typically encountered in these projects are (from upstream to downstream):

- *Head-race or power canal:* The head race is an open channel which carries the flow from the reservoir to the powerhouse intake.
- *Intake:* An intake is any transition from an open channel to a closed conduit flow.
- *Tunnel:* The power tunnel usually does not experience extreme pressures because a surge tank or other device to control pressure surges is often located downstream. The tunnels are normally lined.
- *Surge tanks, air chambers, pressure-relief valves:* These are devices to limit the pressure surges created by transient hydroplant operation. They are generally located downstream of the conveyance facility that they protect. Frequently more than one device is a cost-effective protection of a hydroplant.
- *Penstock:* The penstock is the conduit carrying the water from the tunnel (or intake) to the turbine. The penstock may contain a number of valves, junctions, etc., which require special consideration.
- *Turbine and draft tube:* The turbine and draft tube are part of the turbine or generator unit, and not considered to be hydraulic conveyance facilities. These are discussed in Chap. 4.
- *Tailrace:* The tailrace is an open channel or a tunnel designed to carry the flow back into the main river stem.

5.2 HEAD-RACE AND TAILRACE CANALS

The optimal location for the powerhouse or penstock intake is not always at the dam structure. A cost-effective facility design frequently uses a head race to carry the water to a preferable intake location. The head-race length is highly site-specific. When the head race takes the form of a long channel, it is often called a "power canal" or a "diversion channel." A short head race may be formed by concrete walls in front of the intake, designed to guide the flow. Finally, some hydroplants have no head race. An example of a medium-length head race is given in Fig. 5.1.

The tailrace is an open channel or tunnel used to return the flow into the stream. In many cases, the tailrace is short or nonexistent and does not need to be included in the head-loss calculations. A long tailrace is often used, however, to bypass a rapids section and gain additional head or for other reasons specific to the site.

The design of a head race or tailrace channel is a trade-off between excavation and lining costs versus the lost power cost of channel head losses. Sediment, ice, surges, and wind waves also need to be considered. Manning's equation is normally used to calculate channel head loss:

$$Q = \frac{C_m}{n} A R^{2/3} S^{1/2} \tag{5.2}$$

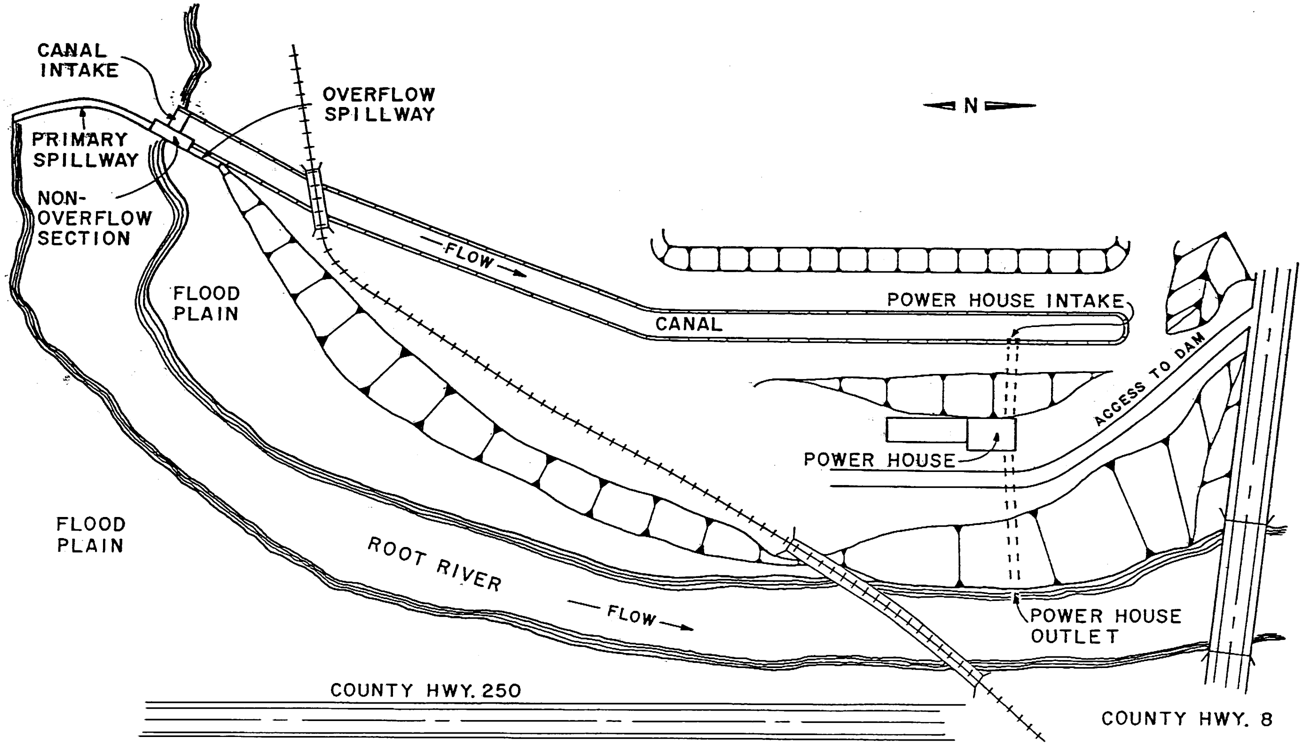


FIGURE 5.1 The Lanesboro, Minnesota, dam and hydropower facilities. Flow is diverted from the primary spillway, through a power canal, to the powerhouse intake.

where Q = channel discharge
A = channel cross-sectional area
R = channel hydraulic radius = A/P
P = wetted perimeter
S = slope of the energy gradeline = head loss per unit length of channel
C_m = a unit conversion coefficient; C_m = 1.0 for units of meters and seconds, and C_m = 1.49 for units of feet and seconds
n = Manning's roughness coefficient.

The head loss in a given channel, then, is

$$h_L = L\left[\frac{nQ}{C_m AR^{2/3}}\right]^2 \tag{5.3}$$

where h_L = head loss
L = length of channel

Table 5.1 lists some typical values of Manning's n for the material commonly used to line head-race and tailrace channels. Chow [1] gives more information on estimating Manning's n. Other losses in channels, such as at expansions, contractions, entrances, and bends, are also discussed by Chow [1].

TABLE 5.1 Values of Manning's Roughness Coefficient for the Channel-Lining Material Most Common in Head Race and Tailrace Channels

Channel lining and condition	Manning's n		
	Minimum	Normal	Maximum
Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035

TABLE 5.1 Values of Manning's Roughness Coefficient for the Channel-Lining Material Most Common in Head Race and Tailrace Channels *(Continued)*

Channel lining and condition	Manning's n		
	Minimum	Normal	Maximum
Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
Dressed ashlar	0.013	0.015	0.017
Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
Vegetal lining	0.030	...	0.500
Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

Source: From Chow [1].

Best Hydraulic Section

The *best* hydraulic cross section minimizes the wetted perimeter and the cross-sectional area for a given channel geometry and depth. The bottom width for a trapezoidal channel with a best hydraulic section is given by the equation

$$b = 2y(\sec\theta - \operatorname{ctn}\theta) \tag{5.4}$$

where b = bottom width
y = channel depth
θ = angle of side slope with horizontal ($\leq 90°$)

When any side slope (θ) is allowable, e.g., concrete lining, $b = 2y/\sqrt{3}$ gives the best hydraulic section.

The best hydraulic section is often a good first approximation for the most economical design. The amount of overburden that must be used, the cost of lining relative to excavation, ease of access to the site, and disposal of the excavated material must be considered in determining the channel geometry [2].

Transitions

Changes in channel geometry to accommodate bridges, headworks, piers, obstructions, etc., are transitions where the head loss is handled separately from Manning's equation.

The head loss at abrupt rectangular channel expansions with subcritical flow ($Fr_1 < 1$) may be determined from Bernoulli's equation [3] to be

$$h_L = \frac{V_1^2}{2g}\left[\left[1 - \frac{b_1}{b_2}\right]^2 + \frac{2Fr_1^2\, b_1^3\,(b_2 - b_1)}{b_2^4}\right] \tag{5.5}$$

where V_1 = cross-sectional mean velocity before the expansion
b_1, b_2 = channel width upstream and downstream of the expansion, respectively
g = acceleration of gravity
y_1 = channel depth upstream

When $Fr_1 < 0.5$, and $b_2/b_1 < 1.5$, Eq. (5.5) reduces to

$$h_L = \frac{(V_1 - V_2)^2}{2g} \tag{5.6}$$

which will also be used for abrupt expansions in a conduit. Equation (5.6) may be used for trapezoidal channels as long as the limits are considered.

The head loss is considerably reduced by tapering the transition. A taper of 1:4, for example, will give

$$h_L = 0.3\,\frac{(V_1 - V_2)^2}{2g} \tag{5.7}$$

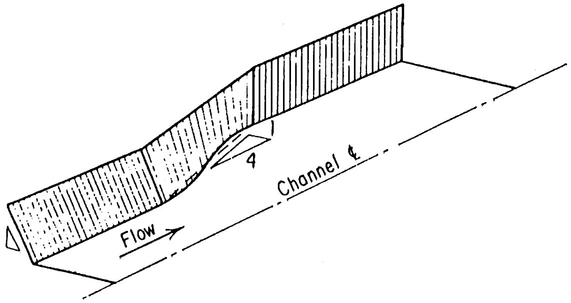


FIGURE 5.2 Warped transition from trapezoidal to rectangular section. *(From Henderson [3].)*

A typical channel transition with taper is given in Fig. 5.2. A warped transition is required for converting a trapezoidal section to a rectangular and vice versa.

For channel contractions, the head-loss equation is

$$h_L = K_c \frac{V_2^2}{2g} \tag{5.8}$$

where K_c = a loss coefficient
V_2 = cross-sectional mean velocity downstream of the contraction

Values of K_c have been experimentally measured to be between 0.23 and 0.35 for sharp-edged contractions, and between 0.11 and 0.18 for cylinder-quadrant contractions, shown in Fig. 5.3. Chow [1] and French [2] give more details on determining head losses at transitions in open channels.

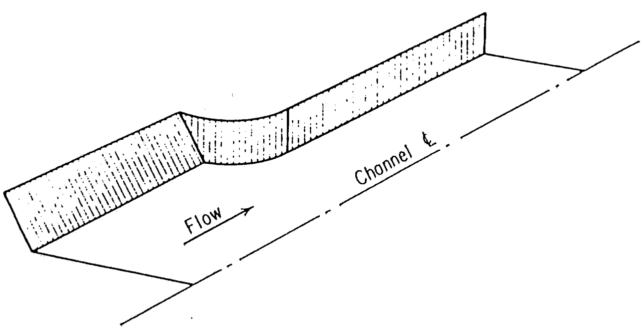


FIGURE 5.3 Cylinder-quadrant contractions for subcritical flow. *(From Henderson [3].)*

Permissible Velocities, Side Slopes, and Freeboard

Tailrace and head-race channels must also be designed to avoid operational problems such as deposition of sediment. Velocities of between 2 and 3 ft/s (0.6 and

0.9 m/s) will normally prevent deposition, and 2.5 ft/s (0.75 m/s) will prevent the growth of vegetation sufficient to affect channel conveyance [2].

Ice is also a consideration during winter in cold climates. An ice sheet normally forms on the water surface over the winter, increasing the wetted perimeter by between 30 and 60 percent. If the underside ice roughness is assumed to be equal to that of the channel, application of Eq. (5.2) indicates that the discharge (or conveyance) will be reduced by between 18 and 24 percent for the same cross-sectional area. This, however, is preferable to the other possibilities, anchor ice and frazil ice, which are formed within a fast-flowing water, carried to the trash racks, or some location in the channel, and form a dam which can close the intake or canal. The formation of an ice sheet precludes the possibility of anchor ice or frazil ice formation. Tranquil flow in the canal without upwelling is required to guarantee formation of an ice sheet. This can be achieved with velocities of approximately 1.5 ft/s (0.5 m/s).

The maximum permissible velocity in the channel is determined by either the cost of an excessive head loss or erosion of the lining or channel material. For initial estimates, a maximum permissible velocity may be determined for various types of channel material from Table 5.2. A more detailed tractive force analysis is required [1, 2] for channel design. The limiting tractive force (bottom shear stress) is also given in Table 5.2. Chow [1] has suggested suitable side slopes for trapezoidal channels which are listed in Table 5.3. Many channels have been built with steeper side slopes than those listed, especially when the material is cohesive.

Freeboard is the vertical distance between the water surface and the lining or the top of the channel. For hydropower canals, the primary reasons to have this

TABLE 5.2 Maximum Permissible Velocities and the Corresponding Unit-tractive-force Values

Material	n	Clear water		Water transporting colloidal silts	
		V, ft/s*	τ_0, lb/ft^2*	V, ft/s	τ_0, lb/ft^2
Fine sand, colloidal	0.020	1.50	0.027	2.50	0.075
Sandy loam, noncolloidal	0.020	1.75	0.037	2.50	0.075
Silt loam, noncolloidal	0.020	2.00	0.048	3.00	0.11
Alluvial silts, noncolloidal	0.020	2.00	0.048	3.50	0.15
Ordinary firm loam	0.020	2.50	0.075	3.50	0.15
Volcanic ash	0.020	2.50	0.075	3.50	0.15
Stiff clay, very colloidal	0.025	3.75	0.26	5.00	0.46
Alluvial silts, colloidal	0.025	3.75	0.26	5.00	0.46
Shales and hardpans	0.025	6.00	0.67	6.00	0.67
Fine gravel	0.020	2.50	0.075	5.00	0.32
Graded loam to cobbles when noncolloidal	0.030	3.75	0.38	5.00	0.66
Graded silts to cobbles when colloidal	0.030	4.00	0.43	5.50	0.80
Coarse gravel, noncolloidal	0.025	4.00	0.30	6.00	0.67
Cobbles and shingles	0.035	5.00	0.91	5.50	1.10

*1 ft/s = 0.305 m/s; 1 lb/ft$_2$ = 0.415 N/m$_2$.
Source: From Chow [1].

TABLE 5.3 Suitable Side Slopes for Channels Built in Various Types of Materials

Material	Side slope
Rock	Nearly vertical
Muck and peat soils	¼:1
Stiff clay or earth with concrete lining	½:1to1:1
Earth with stone lining or earth for large channels	1:1
Firm clay or earth for small ditches	1½:1
Loose, sandy earth	2:1
Sandy loam or porous clay	3:1

Source: From Chow [1].

freeboard are wind waves, increases in channel depth caused by sedimentation, and surges caused by hydroplant operation. Guidelines for freeboard are illustrated in Fig. 5.4. Surges, however, may require special consideration. A reduction in power output will cause a positive, upstream traveling surge in the head race and a negative, downstream traveling surge in the tailrace. Freeboard needs to be allowed for the positive surge; negative surge in the tailrace can begin an oscillation with the turbine and governor that can reach damaging extremes. An increase in power output will cause a negative, upstream traveling surge in the head race which can reduce the available head and thus the plant power output for a certain period.

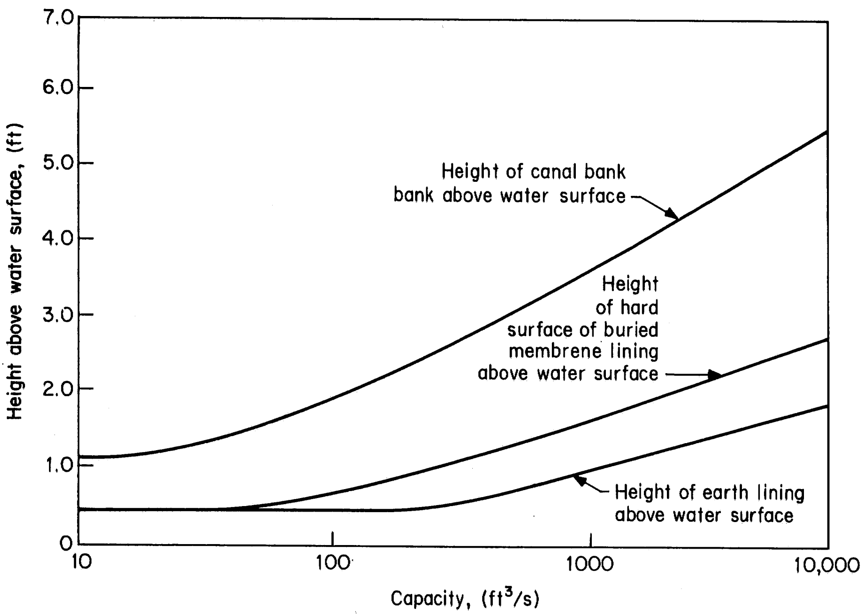


FIGURE 5.4 Freeboard for canal banks and freeboard for hard-surface buried membrane and earth linings. *(From U.S. Bureau of Reclamation [4].)*

Linings

Head-race and tailrace canals are often provided with a lining. The lining can potentially eliminate leakage losses, allow a higher velocity, reduce Manning's n, allow steeper side slopes, and have less maintenance than an unlined canal. Thus, the channel with lining will be significantly smaller than one without and have the same conveyance (carry the same discharge). The cost of the lining can therefore be offset by a reduction in excavation, land costs, and maintenance costs.

Concrete linings poured in place are the most common because they fulfill all of the potential advantages listed above. Precast concrete slabs have also been used for smaller channels. Another common lining is riprap or stone paving, which protects the canal against erosion but does not reduce friction or prevent seepage. The maximum velocity for a riprapped channel is approximately 7 ft/s (2 m/s). Finally, stone and brick linings with cement mortar are common in many older hydroplants. The primary reason these linings are not often used today is the high labor cost associated with laying the stone or bricks. In countries with low labor costs and a ready supply of materials, stone and brick linings should be considered.

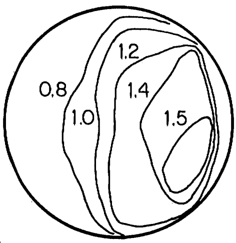
5.3 INTAKE STRUCTURES

An intake is the transition between a free surface and closed conduit flow. Intake structures at hydroelectric facilities consist of sluicing facilities for trash, ice, and sediment; fish protection facilities; trash racks; and a flow constriction to bring the water from the reservoir into the penstock. The intake structures are a matter of concern for design engineers because the depth of the intake, the horizontal location, the location of the various structures relative to each other, and the approach flow angle are site-specific. In addition, the hydraulics of intake structures are rather complex, being three-dimensional and sometimes unsteady, and not easily packaged into design handbook form. Case studies are often the best way to illustrate and identify the potential hydraulic problems of an intake structure. Because the design is highly site-specific, most designers commission a hydraulic model study to test, verify, and improve their design when possible. The trash racks are usually slanted at 15° off vertical so that trash will move to the water surface where it can be removed by manual raking or an automatic raking system. Some trash racks are also designed to keep any fish larger than fingerlings out of the penstock-turbine system, although the primary spacing criterion is the size of an object the turbine will pass without damage.

Velocity Profiles

The hydroturbine is designed to operate with a "relatively uniform" approach velocity profile. If the nonuniformities in the velocity profile are too great, the turbine will not perform as anticipated. In addition, most turbine manufacturers specify a given quality of the velocity profile that must be attained for their performance warranty to be valid. Thus, *having a poor quality (nonuniform) velocity profile is essentially the same as purchasing a turbine without a performance warranty*. This is especially important for low-head hydroelectric projects, where there is very little distance between the turbine and the intake, and a uniform velocity profile cannot be attained in the tunnel-penstock system.

A recent study of the impact of approach flow characteristics on turbine performance by Fisher and Franke [5] described the influence of a highly non-uniform velocity profile on turbine performance. The profile studied and the results are given in Fig. 5.5, indicating that approximately a 2 percent reduction in turbine-generator efficiency occurred over the range of operating heads. The effect of swirl was also studied, although the swirl component was not reported, and shown to reduce turbine performance by 1 percent at the best efficiency discharge and 3 percent at the discharge that occurred at full output.



H/H_{opt}	Δ Efficiency (%)
0.77	−2.4
0.89	−2.1
1.07	−1.9
1.12	−1.7

FIGURE 5.5 Axial inlet velocity contours and the resulting change in turbine efficiency over a range of net head; H/H_{opt} = the head at optimum performance (peak efficiency). *(From Fisher and Frankee [5].)*

Fisher and Franke [5] also surveyed three manufacturers to determine the requirements for velocity profiles. The results are the fairly subjective set of requirements given in Table 5.4. They then reported on model tests where the ve-

TABLE 5.4 Requirements of Three Manufacturers for the Quality of Approach Flow to the Turbine

Manufacturer #1:

1. Flow free from separations and air entraining vortices
2. Velocity distribution within ±10% of average velocity
3. Maximum angle deviation from axial of 5°
4. Deviation of discharge for each of four quadrants shall not vary more than ±5% of the total discharge

Manufacturer #2

1. No vortex formation entering the turbine inlet
2. No cross-flow velocities greater than 5% of the average velocity at the upstream measuring section of the turbine performance model test (approximately the location of the upstream gate)
3. No axial velocity deviating from the average velocity by more than ±5%, at the same location described in 2 above

TABLE 5.4 Requirements of Three Manufacturers for the Quality of Approach Flow to the Turbine *(Continued)*

Manufacturer #3

1. Flow free of separations
2. No air entraining vortices, only short duration non-entraining vortices
3. Uniform as possible velocity profile over the complete flow cross section
4. Deviations of the local flow velocity from the mean flow velocity not greater than approximately ±5%
5. A deviation of the local flow direction from axial should not exceed 5°. Possibly existing deviations up to this value must, related to the total flow cross section, not result in a rotation of the flow
6. Discharge through the left or right intake half should not differ more than ±5% of the total discharge

Source: From Fischer and Frankee [5].

locity profile did and did not affect turbine performance and established recommendations for a more definitive set of guidelines:

1. The velocity distribution should fall inside of the lines drawn in Fig. 5.6.
2. The flow should be free of air-entraining vortices.
3. All cross-flow velocities should be less than 5 percent of the average axial velocity at the trash rack.

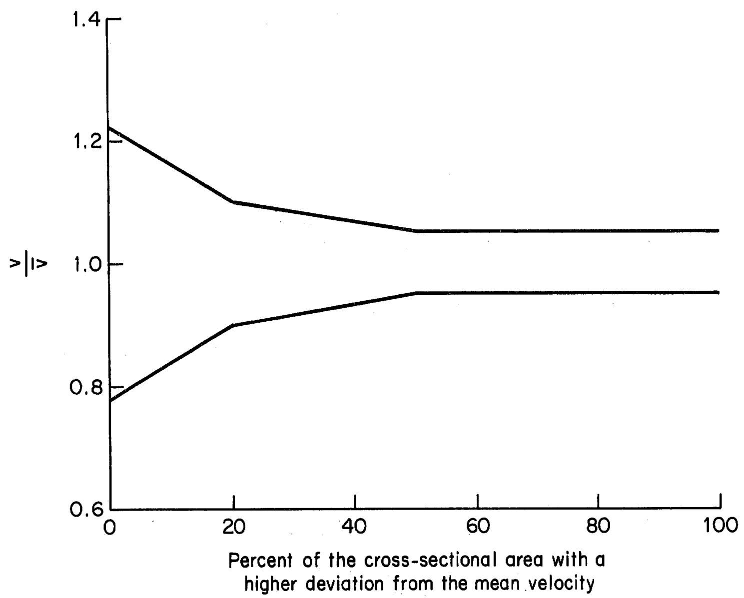


FIGURE 5.6 Recommended guidelines for velocity profiles upstream of the turbine. $\bar{V}$ = mean velocity. Most of the velocities should be inside of these curves. *(From Fisher and Frankee [5].)*

4. Deviations of average axial velocity for the four intake quadrants should not vary more than 10 percent from the average axial velocity.

The third criterion is very difficult to obtain. The author recommends that

5. A hydraulic model study be performed, especially on low-head projects, to determine the velocity profile entering the intake at an appropriate location.
6. The above criteria should be adjusted according to the results of the hydraulic model study and included in the solicitation for bids from equipment manufacturers. The manufacturer's performance guarantees should be based upon this intake velocity distribution.
7. If a hydraulic model study is not performed, recommendation no. 3 above should be changed to "the mean cross-flow velocity will be less than 10 percent of the mean axial velocity" at a location in the intake where measurements are possible, if a performance dispute arises.

Head Losses

The head loss through trash racks and intakes is a function of the velocity head

$$h_L = K_t \frac{V_t^2}{2g} + K_i \frac{V^2}{2g} \tag{5.9}$$

where h_L = head loss (ft of water)
K_t = head-loss coefficient for trash racks
K_i = head-loss coefficient for intake
V_t = velocity approaching trash racks
V = velocity of flow in penstock or tunnel downstream from inlet

Equation 5.9 is valid as long as the flow is fully developed or uniform, which never occurs at an intake. For a converging flow (accelerating) at hydroplant intakes, the trash rack head loss should be multiplied by a velocity correction factor of between 1 and 3 [6]. A velocity correction factor of 2 is usually a conservative choice for accelerating flow. Miller [7] and Idel'chik [8] have compiled an extensive amount of information on head-loss coefficients at intakes and trash racks which is given in Figs. 5.7, to 5.9.

Intake Vortices

Free-surface vortices are a highly organized flow phenomenon that occurs due to the residual angular momentum in the flow at a closed-conduit intake. They occur commonly at free-surface flows into a closed conduit such as a sink or a bathtub drain. In large closed-conduit intakes, however, free-surface vortices are a severe problem and are to be avoided. Free-surface vortices have been found to cause flow reductions, vibrations, structural damage, surging due to the formation and dissipation of vortices, and a loss of efficiency in turbines. In certain instances, they have also been a safety hazard. An example of a severe intake vortex is given in Fig. 5.10.

Hydraulic turbines are designed with the requirement that the flow in the penstock be straight and uniform. When a vortex is present at an intake to the pen-

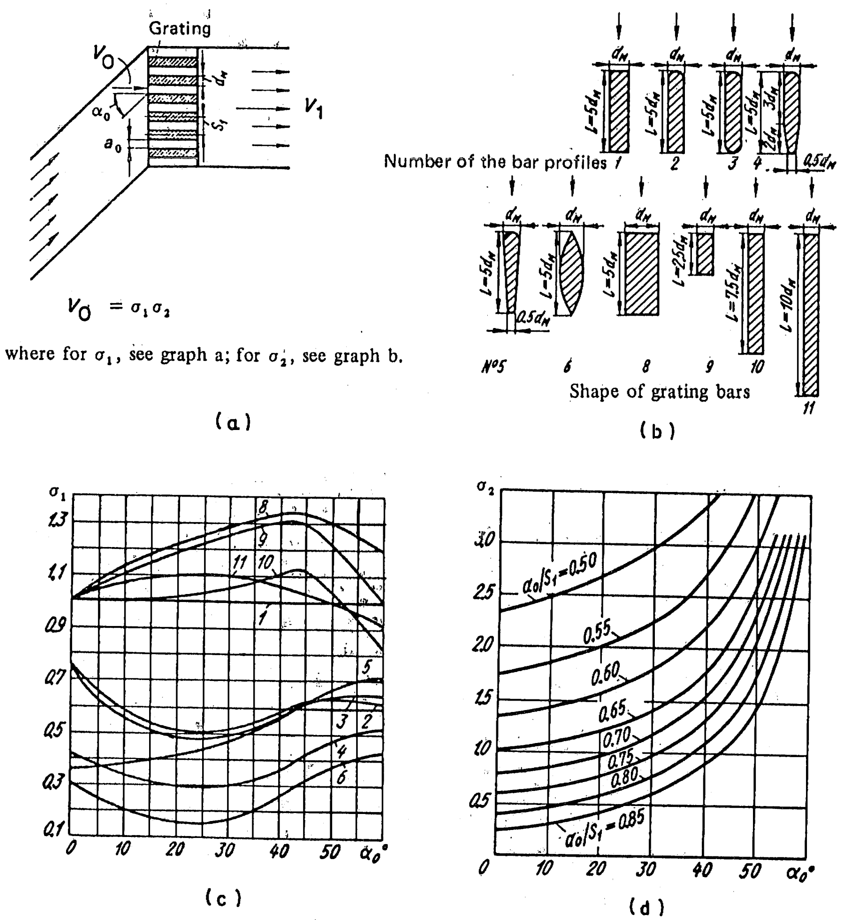


FIGURE 5.7 Head-loss coefficients K_t for trash racks with approach flow at an oblique angle, α_0. *(From Idel'chik [8].)*

stock, the intake is exposed to a highly swirling and unsteady flow which is carried into the penstock. If the penstock is not sufficiently long to eliminate this swirl, the turbine will run off of design, corresponding to a loss of efficiency and possibly cavitation, in addition to the other problems mentioned above. If the free-surface vortex entrains air, the air replaces the water in the intake and the discharge is reduced. Thus, a severe intake vortex can keep a plant out of operation.

One of the major problems encountered during intake design is the specification of submergence and other design parameters in order to avoid strong free surface vortex formation. Most of the research work on free surface vortices to date has been performed on a site-specific basis, where a hydraulic model study is performed for a given hydroelectric plant intake to determine whether vortices will be present. Unfortunately, there are no definitive design guidelines available to avoid intake vortices. Intakes are currently designed using general rule-of-thumb guidelines and hydraulic model studies when necessary.

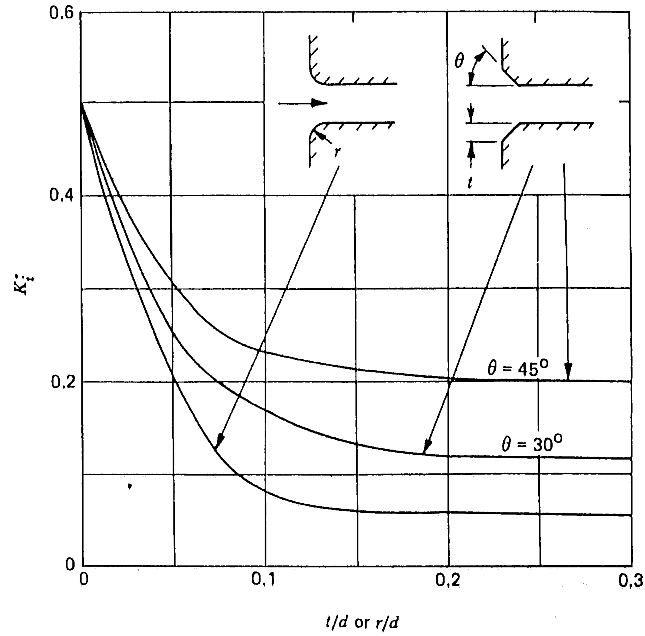


FIGURE 5.8 Loss coefficients for flush mounted intakes. *(From Miller [7].)*

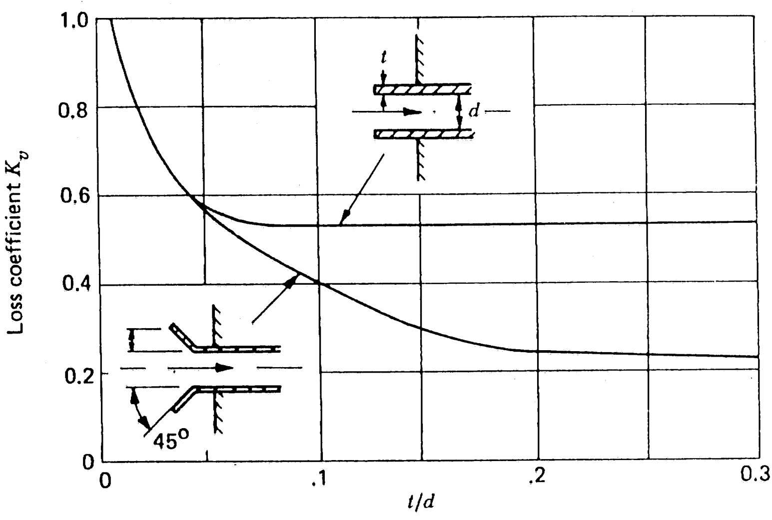


FIGURE 5.9 Loss coefficients for reentrant intakes. *(From Miller [7].)*

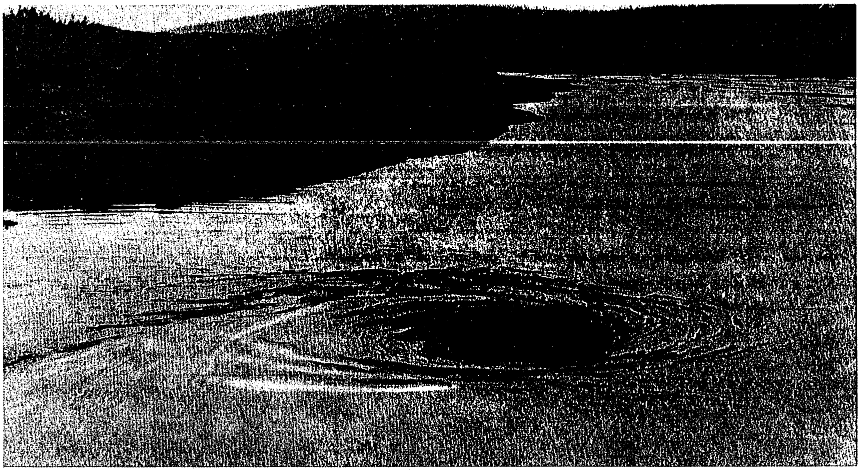


FIGURE 5.10 Vortex which formed at Horspranget, Sweden, hydropower intake on August 15, 1949. *(From Rahm [9].)*

The parameters influencing the occurrence of intake vortices are illustrated in Fig. 5.11. These are

1. *Arrangement.* Experience has shown that a vertically arranged intake has a much greater tendency for vortex problems than one which is horizontally arranged. A vertically inverted intake, such as on a pump or siphon penstock, has a vortex formation tendency similar to the horizontal arrangement.
2. *Submergence.* More specifically S/D, a greater submergence will reduce the tendency for vortex formation due to increased friction in the water column.
3. *Intake Froude number.* The number $V/\sqrt{gS}$ or $V/\sqrt{gD}$ where g is the acceleration of gravity; $V/\sqrt{gS}$ is more fundamentally sound in that it is a ratio of inertial to viscous forces in the water column, but $V/\sqrt{gD}$ is easier to use in design and is more common.
4. *Approach circulation.* If the flow enters the intake at an angle which gives it an overall swirl, that swirl will tighten up and form a vortex as it enters the intake.

Sweeney et al. [11] state that at pump intakes, no organized or subsurface vortices equal to or greater than that visually represented by a coherent swirl into the intake (dye-core vortices) can be allowed. Trash-pulling and air-core vortices, therefore, should also be avoided. A similar criterion is appropriate for hydroturbine intakes, since the flow through the hydromachine is similar. One difference from a pump, however, is that a turbine has guide vanes upstream of the runner that may eliminate a small swirl. Another difference is that wall friction in a long penstock may eliminate swirl before it reaches the turbine [12]. Baker and Sayer [13] found that the swirl in high Reynolds number pipe flow decayed exponentially such that

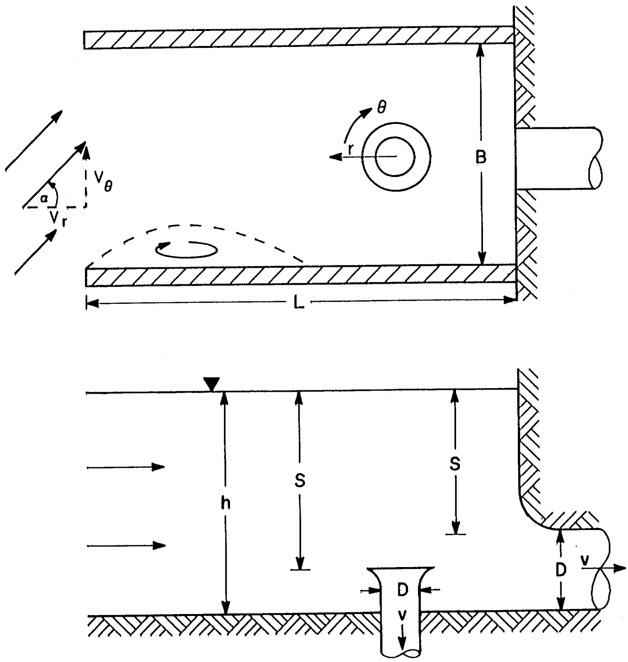


FIGURE 5.11 Schematic representation of typical vertical and horizontal intake configurations (both arrangements incorporated into each sketch), and the parameters influencing intake vortices. *(From Gulliver et al. [10].)*

$$\frac{\text{Swirl at } x = L}{\text{Swirl at } x = 0} = e^{-0.02L/D_p} \tag{5.10}$$

where x is the distance along the pipe and D_p is the pipe diameter. Thus, penstock lengths of 150 and 200 diameters will reduce swirl to 5 and 2 percent of that entering the penstock, respectively. The amount of swirl allowed before impacting turbine performance, however, has not been documented. The conservative approach is to allow no coherent subsurface swirl into hydroplant intakes.

Some general rule-of-thumb guidelines which are applicable for horizontal intakes are given in Fig. 5.12 [10]. For a horizontal intake with $S/D > 0.7$ and $v/\sqrt{gD} < 0.5$, Fig. 5.12 indicates that vortex problems are unlikely. Otherwise, a model study is required. No similar guidelines are currently possible for vertical intakes, which require more submergence.

It is sometimes unclear which submergence, velocity, and intake diameter to choose for use in Fig. 5.12. These parameters should be measured at the location where their influence on vortex formation is greatest. Small hydraulic structures generally have a simple penstock arrangement such as that shown in Fig. 5.11, which may then be used to compute S/D and Fr. A larger structure, however, may have a very long constriction length and/or a submergence which is not well defined.

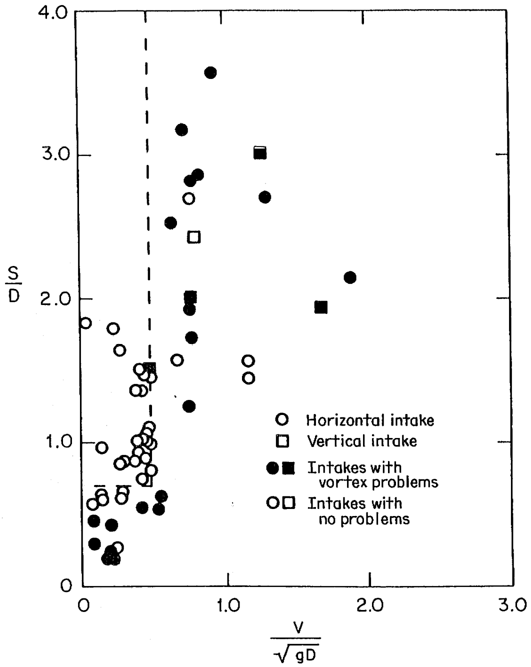


FIGURE 5.12 Dimensionless plot of data obtained from intakes at existing installations and model studies of proposed installations. Dashed line encloses a region where prior experience with horizontal intakes indicates that vortices are unlikely. *(From Gulliver et al. [10].)*

Some engineering judgment will be needed to identify the location which gives the most representative velocity and diameter (or vertical distance of the opening) for vortex formation. Often, the stop-log slots are a good location for measuring V and D, as in the sample intake shown in Fig. 5.13. If there is doubt, more than one location may be chosen for computing V and D to give an indication of the tendency for vortex formation.

One interesting aspect of Fig. 5.12 is that there is no "safe" dimensionless submergence for intakes with an intake Froude number greater than 0.5. There are, however, intakes in this region with no vortex problems. The reason, of course, is approach circulation, the ignored parameter in this type of analysis. Intakes with a very good approach flow, such that there is little large-scale circulation entering the intake, can operate without free-surface vortices at a high intake Froude number. On the other hand, intakes with a poor approach flow can have free-surface vortices at intake Froude numbers as low as 0.5, and possibly lower. In this region, a model study must currently be recommended under all circumstances.

Hydraulic model studies of intakes are usually undertaken to determine whether vortices will develop and to suggest design alterations to eliminate any vortex problems.

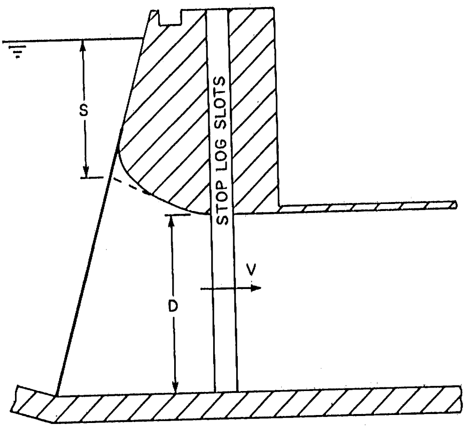


FIGURE 5.13 Sample intake structure showing the choice of the *V*, *S*, and *D* parameters. *(From Gulliver et al. [10].)*

There is a wide variety of antivortex devices which may be used. They fall into the following general categories:

1. *Farfield flow guiding:* Long approach channel walls or walls which are shaped to eliminate separation zones can sufficiently reduce circulation before the intake is approached [14, 15]. Guide vanes may also be used to reduce circulation [16]. It is often expensive to eliminate vortices with guide vanes, because it is difficult to rectify a flow that is both nonuniform and approaches the intake at an angle; sometimes guide vanes are used in conjunction with other antivortex devices.
2. *Swirl reduction:* A number of antivortex devices which reduce swirl at or near the intake have been used successfully. One of the more successful is a submerged raft [15], shown in Fig. 5.14. Steel grating near the intake which can also serve as a trash rack has proven moderately successful [14, 15, 17]. A wedge-type vortex suppressor over the intake given in Fig. 5.15 will reduce moderate amounts of swirl [16]. For small intakes with high velocities, the hooded inlet illustrated in Fig. 5.16 that was developed by Blaisdell and Donnelly [18] has proven successful and relatively inexpensive.
3. *Intake velocity reduction:* Another method of eliminating vortices is to decrease intake velocity by increasing intake cross-sectional area. A cover, such as that shown in Fig. 5.17, for example, will increase area and convert a vertical intake into one which is horizontal [18–20].
4. *Head-loss devices:* A significant head loss will straighten the approach flow regardless of whether it is near to or far from the intake. Baffles, plates, etc., may be oriented so that the head difference across the device will force the flow in a given direction. One problem is that the success of head-loss devices is directly proportional to the amount of head loss. The reduction in power output is sometimes significant [21].

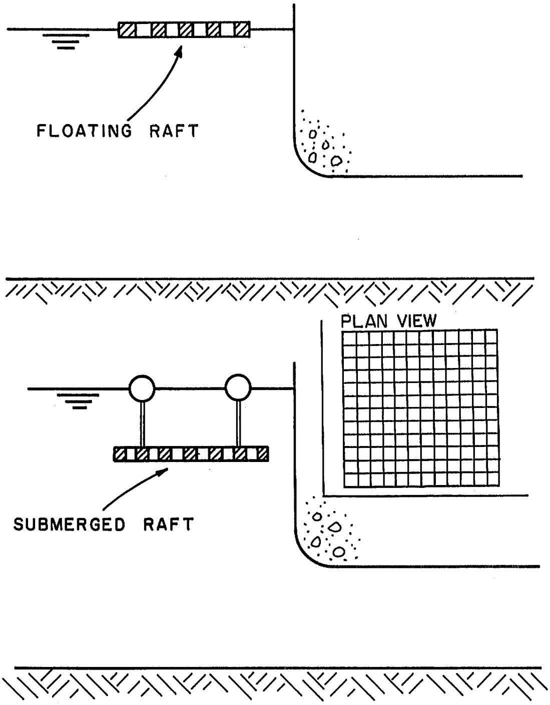


FIGURE 5.14 Floating and submerged raft vortex suppressors. *(From Ziegler [15].)*

Sediment, Ice, and Trash

Various devices and designs are required to handle or divert sediment, ice, and trash that the flow carries downriver. Since a large portion of that flow will be discharged through the hydroplant, a similar portion of debris will tend to be pulled through as well.

Log Booms. Any large object floating downriver will be swept away from the hydroplant intake by a log boom such as that illustrated in Fig. 5.18. Ice, logs, and trash are normally deflected by such booms, oriented to allow the material to pass over the spillway. It is desirable to be able to walk along the boom to release floating debris that is caught on the boom and for inspection purposes. The logs are attached to two steel cables that are often stretched between an anchor block and a winch.

Sluicing Facilities. Hydroplants on rivers that contain a large quantity of silt and sand in suspension normally have a sediment-sluicing facility included in the intake, because if the sediment passes through the turbines it can erode the runner, increase replacement frequency, and create a loss of efficiency. In addition, many rocks and boulders that roll along the bottom would be caught on the trash racks unless sluiced out of the intake.

Not all hydroplants need sluicing facilities. The increased costs of operation,

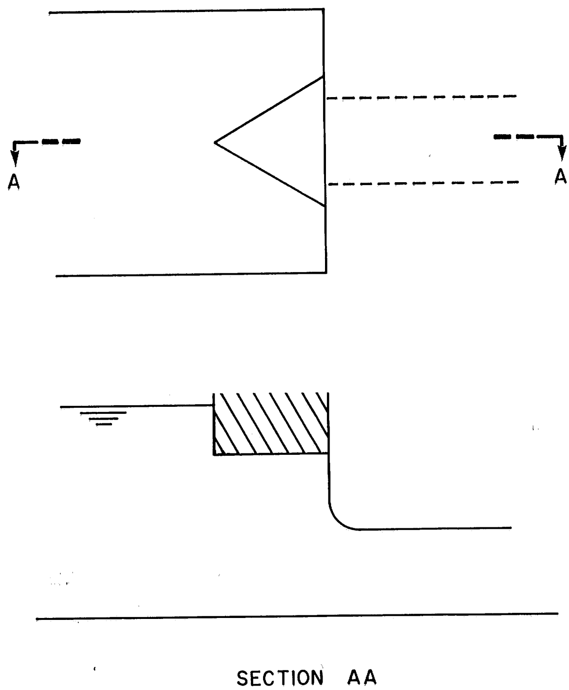


FIGURE 5.15 Wedge-type vortex suppressor. *(From Song [16].)*

replacement, and maintenance should be weighed against the cost of building the facility plus the lost energy from sluiced water. It is often more cost-effective to replace turbine runners more frequently and remove sediment accumulation than to build and operate a sluicing facility. An example of a sediment-sluicing facility is shown in Fig. 5.19 [22], designed to remove all sediment of 0.008 in (0.2 mm) in diameter and larger, with a cross-sectional mean velocity of 0.25 ft/s (0.08 m/s). The accumulated sediments are removed by periodic purging through the bottom outlet. Important aspects of the design are [22]: (1) The transition at the entrance should be gradual to evenly disperse the flow and avoid a high-velocity short-circuiting of the basin, and (2) the floor slope should be sufficiently large to allow sediments to be purged hydraulically.

Sediment can also deposit in the reservoir, reducing storage capacity, increasing head loss by restricting cross-sectional area, and eventually restricting intake area. In this case, it is prudent to design low-level outlets near the intake [23] to sluice out the region in front of the intake and to retain reservoir storage capacity. A review of the existing literature [24] resulted in the preliminary design curves given in Fig. 5.20. These curves are developed from reservoirs that have been built or modeled to successfully sluice sediment inflow. Sluicing gates should be at the bottom of the reservoir with a height of between 5 and 8 ft (1.5 and 2.5 m).

Upper-level sluices are used to carry off ice and trash. They do not usually work well for ice, however. The trash that collects on the trash racks or is raked off may be passed on downstream by one of these sluices.

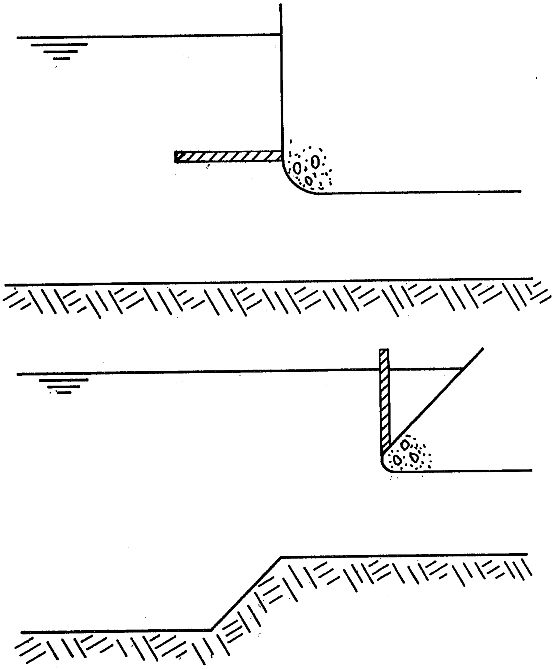


FIGURE 5.16 Extended plates used to dissipate vortices. *(From Blaisdell and Donnelly [18].)*

Trash Racks. A trash rack is required to protect the turbine runner from impinging objects. The opening of the racks is usually specified by the manufacturer, depending upon the clearance between wicket gates and between runner blades. Typically, the spacing varies between 1.5 and 4 in (4 and 10 cm). The racks are made up of a number of long vertical plates, 3 to 4 in (7.5 to 10 cm) parallel to the flow and 0.25 to 0.5 in (0.6 to 1.2 cm) thick, perpendicular to the flow.

Submerged trash racks are generally trouble-free and have no racking accommodation. At low-head installations, it is generally not feasible to submerge the trash racks far enough, and they are arranged at the water surface to accommodate raking. The racks are raked by hand or with an automatic trash rake. They are sloped at 15 to 20° off vertical toward the powerhouse so that most trash will work up to the water surface. The velocities approaching the trash racks should be less than 3 ft/s (1 m/s) if they are raked by hand and 5 ft/s (1.7 m/s) if they are automatically raked. They should also be designed to withstand a certain head difference, at least 6 ft (2 m), caused by blockage of trash, leaves, or ice.

Frazil ice and anchor ice present a special problem for trash racks. These types of ice form in the water column, rather than at the water surface, where the water turbulence is sufficiently high to inhibit formation of an ice sheet. The frazil and anchor ice stick to the trash racks in passage and can block an intake completely in a short time. There are a number of techniques that have been used to deal with frazil ice formation on trash racks, such as

1. Shut down the hydroplant when frazil ice begins to form. This may be the economic solution if frazil ice formation is a sufficiently rare occurrence.

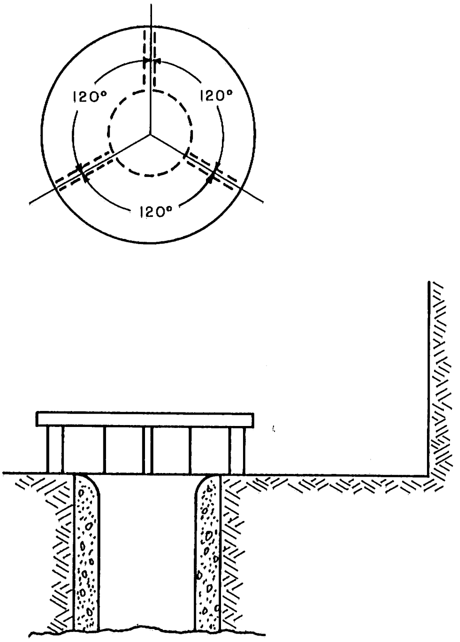


FIGURE 5.17 Covered-intake-type vortex suppressor. *(From Blaisdell and Donnelly [18]; Blaisdell [19]; and Humphreys et al. [20].)*

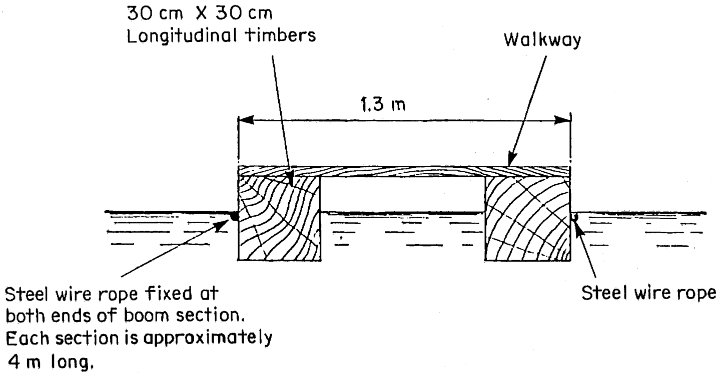


FIGURE 5.18 Illustration of a typical log boom section (1 m = 3.28 ft).

2. Design compressed-air bubblers to place at the bottom of the intake, and turn these on when frazil ice forms. The air bubbles carry warm water from lower levels up with them, which will remove most of the ice. The bubblers may also be used to keep sheet ice away from the racks.
3. Raise the trash racks out of the water during the period of ice formation. One problem is that the turbines lose the protection afforded by the trash racks.

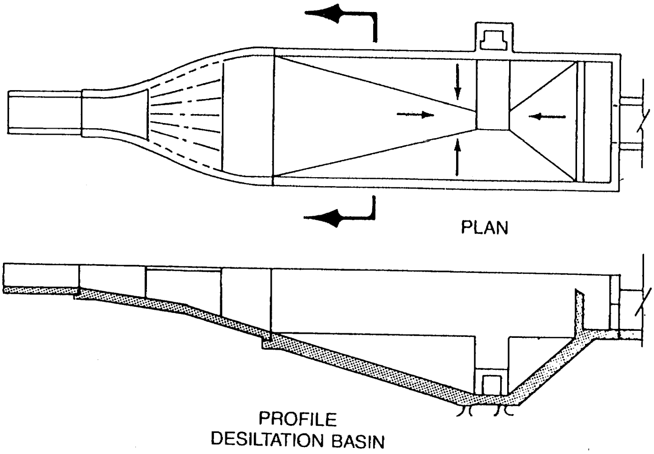


FIGURE 5.19 Illustration of a sediment sluicing facility. Note the gradual expansion to avoid short-circuiting and the bottom slope to assure sediment purging. *(From Moore [22].)*

Floating trash does not normally move downriver during the period of ice formation, however.

4. Paint the bars with a "slippery paint" and vibrate the racks to inhibit ice formation. This technique is currently in the experimental stages.

Fish Protection

Most intakes must also be designed for fish protection. Often regulatory agencies will specify a certain degree of fish protection at the intake. The wide variety of approaches to fish protection can be classified into three categories, as follows:

No Fish Protection. The costs of installing and maintaining fish protection devices at an intake can exceed any losses associated with the fishery. If mitigation is desired, it may be most cost-effective to have it take the form of stocking fish, partially supporting a fish hatchery, etc. Thus the intake would have no fish protection.

Partial Fish Passage. This option would pass fish through the turbines when they fit through the trash racks. Normally, the trash racks are designed with velocities allowing larger fish to swim away from the trash racks. This is especially effective for low-head installations with sedentary (nonmigratory) fish species. The low-head hydroplants have a propeller turbine, with a relatively low fish mortality, or a Francis turbine, with a somewhat higher fish mortality, but one that can be improved through the design process [25]. In addition, since the fish do not migrate, only a portion of the population will be located in the vicinity of the intake.

To determine whether the fish will be able to swim away from a given intake, Anderson's [26] literature review of fish swimming speed may be used. There are three levels of activity, burst, steady, and sustained swimming speeds.

Burst speed can be maintained for only a few seconds and is predominantly dependent upon fish length. The relation can be approximated by

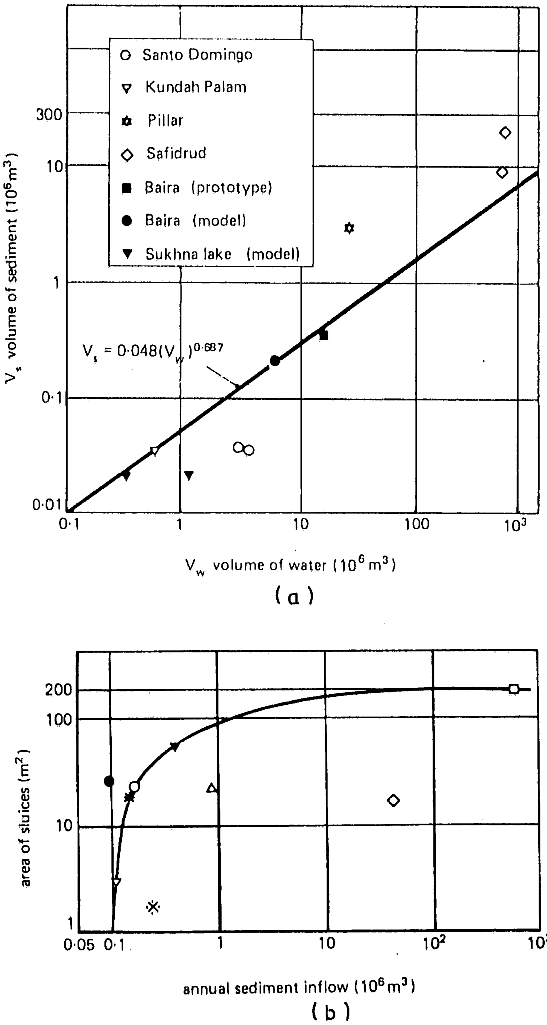


FIGURE 5.20 Guidelines for volume of water (above) and area of sluices (below) versus volume of sluices for reservoir sluicing developed from existing and modeled projects. *(From Paul and Dhillon [24].)*

$$S_b = a_1 X^{a_2} \tag{5.11}$$

where S_b = burst speed, cm/s (in/s)

X = fish length, cm (in)

a_1 = 14.8 $cm^{0.12}$/s (13.3 in $^{0.12}$/s)

a_2 = 0.88

The coefficients a_1 and a_2 for Eq. (5.11) were fit from data on dace, goldfish, trout, barracuda, porpoise, and dolphin.

The steady speed can be maintained for periods ranging from a few minutes to a few hours. The maximum steady speed (called the critical speed) is a function of fish length, water temperature, dissolved oxygen concentration, and the concentration of toxic substances [26]. For high dissolved oxygen concentration and low toxics concentrations, it has been given by the formula [26]

$$S_s = a_3 X^{a_4} e^{a_5 T} \tag{5.12}$$

where S_s = maximum steady speed, cm/s (in/s)
X = fish length, cm (in)
T = temperature (°C)
$a_3 \simeq 7.4\ \text{cm}^{0.4}/\text{s}\ (5.1\ \text{in}^{0.4}/\text{s})$
$a_4 \simeq 0.6$
$a_5 \simeq 0.04\ °\text{C}^{-1}$

The sustained speed that can be maintained indefinitely is generally between one and three body lengths per second.

The intake should be designed so that fish that do not fit lengthwise through the trash racks can swim away from the intake. Otherwise, the trash racks will continually collect dead fish, to the consternation of local anglers and regulatory officials. The length of a head-race canal, for example, that meets this criterion may be determined as

$$L = (S - V)\,\Delta t \tag{5.13}$$

where L = length of head-race canal
S = appropriate speed for time period of interest, determined by the time of travel to a calm region
V = flow velocity in head-race canal
Δt = time period for either bursting, steady, or sustained swimming speed

Eliminate Fish Passage through Turbines. The other broad option is to bypass virtually all fish around the turbines. This approach is usually expensive and is sometimes used when there are migratory fish species present, where the entire population of the fish species will migrate through one or more dam or powerhouse structures. If, in addition, an impulse turbine is used, with its high fish mortality, survival of the fishery often depends upon successfully passing a significant portion of the fish.

With this type of operation, the intake design is often similar to those of thermal power plants that are designed for fish protection, except that the fish are passed around the hydroplant using the existing head rather than pumped back into the river. Fish protection systems in use are shown in Table 5.5 [27]. The ASCE has published a manual on the design of intake structures for fish protection [28] that is applicable to relatively large (>10 MW) hydroplants. More recently, Taft [27] has reviewed the fish protection technologies for hydroelectric plants. Much of the following is summarized from these two reports.

The most common type of intake system to bypass fish is the *diversion concept*, where a component of the intake structure is designed to take advantage of a natural behavioral response of fish to an object in the flow. Fish tend to orient themselves facing into the current, even when they move downstream [27]. They

TABLE 5.5 Summary of Fish Protection System Advantages and Disadvantages

Device	Biological/engineering variables affecting performance of device	Advantages	Disadvantages or limitations
		Behavioral barriers	
Electrical screens	Temperature, velocity affect response; water quality/materials affect life/performance of electrodes	1. Relatively inexpensive 2. Downstream migrant salmonids have been successfully guided; dc shocker effective for passing shad 3. Can be easily retrofitted	1. Results variable; species-specific 2. Electrode maintenance essential for proper operation 3. Potential for injury/death to large fish exists 4. Field data needed to verify claims of improved systems 5. Not generally accepted by regulatory agencies
Air bubble curtain	Temperature, turbidity, light intensity, velocity and orientation affect response; water quality/materials may affect performance	1. Relatively inexpensive 2. Can be combined with other behavioral barriers (e.g., strobes) 3. Can be easily retrofitted	1. Variable biological effectiveness 2. Potential for clogging from debris/rust 3. Field data for salmonids needed
Hanging chains	Velocity, turbidity, flow conditions affect efficiency; debris load may affect performance	1. Inexpensive 2. Can be easily retrofitted 3. Can be combined with other behavioral barriers (e.g., strobes)	1. Species-specific; variable biological effectiveness 2. Need to prevent debris buildup
Strobe light	Length, turbidity may affect efficiency; flash duration of strobe influences efficiency	1. Relatively inexpensive 2. Effective for warm-water species 3. Can be easily retrofitted 4. Has potential for enhancing other fish protection systems	1. Effectiveness for repelling salmonids unknown 2. Design/operational problems with experimental underwater strobes have been encountered 3. Not adequately field tested to date
Mercury light	ID most variables	1. Relatively inexpensive 2. Excellent biological effectiveness for warm-water species 3. Can be easily retrofitted 4. May improve existing fish protection systems	1. Field data for salmonids needed 2. Engineering/design work needed to adapt lights for underwater use

TABLE 5.5 Summary of Fish Protection System Advantages and Disadvantages *(Continued)*

Device	Biological/engineering variables affecting performance of device	Advantages	Disadvantages or limitations
		Behavioral barriers	
Incandescent light	Length, size, turbidity, velocity, angle, intensity and flow conditions affect efficiency/performance	1. Inexpensive 2. Can be retrofitted	1. Results equivocal; generally not effective 2. Species-specific response
Sound	ID most variables	1. Inexpensive 2. Can be retrofitted	1. Results equivocal; generally not effective 2. Species-specific response 3. Habituation to sound source has been experienced
Popper	ID most variables	1. Relatively inexpensive 2. Good biological effectiveness for warmwater species 3. Can be retrofitted	1. Field data for salmonids needed 2. Reliability of device questionable
Hybrid barriers	Velocity/illumination affect efficiency; design and orientation of device affect performance	1. Relatively inexpensive 2. High biological effectiveness with warmwater species 3. Can be retrofitted 4. Has potential for improving existing fish protection systems	1. Not adequately field tested to date
Water jet curtain	Angle affects deflection; ID most variables	1. Relatively inexpensive	1. Disturbance to normal flow patterns 2. Has not been adequately field tested 3. Potential for clogging from rust/debris
Chemicals	Volume affects efficiency	1. None	1. Large volume of chemical required 2. Potential for build-up in aquatic environment 3. Not generally accepted by regulatory agencies 4. Not adequately tested
Visual keys	Turbidity/illumination affect efficiency/performance; ID most variables	1. Relatively inexpensive 2. Can be retrofitted	1. Species-specific; generally not effective 2. Illumination/low turbidity essential for operation

Diversion devices			
Angled screens (coarse-mesh)	Length, size, condition, temperature, velocity, screen angle and flow conditions affect efficiency	1. Total efficiency high 2. Mortality generally low 3. Has been tested in a variety of situations/conditions	1. Species/lifestage-specific 2. Approach velocity affects efficiency 3. Potential for clogging of fixed screens by debris 4. Difficult to backfit 5. Relatively expensive 6. Requires fish transport
Horizontal traveling screens	Velocity, debris load affect efficiency; continual maintenance problems have been encountered in full-scale system	1. Diversion efficiency can be high if properly operated/maintained	1. Unresolved engineering problems have been encountered 2. Clogging a potential problem 3. Relatively expensive 4. Has not been adequately tested
Angled rotary drum screens	Mesh size, approach velocity, screen orientation and bypass criteria and debris load affect efficiency/performance	1. Biologically effective if velocity is low, screens are kept clean and fish escape routes exist	1. Potential for debris/icing problems 2. Not effective where escape routes are not available 3. Moderately expensive
Inclined plane screens	Velocity, screen angle, species, season and site-specific conditions affect efficiency	1. Effective in diverting salmonids upwards in water column to bypasses at several hydro facilities 2. Screens self-cleaning in high velocities	1. Potential clogging problems 2. High cost 3. Unresolved engineering problems have been encountered
Submerged traveling screens	Percentage of flow intercepted by STS and porosity affects efficiency; debris load also affects performance	1. Effective for species which concentrate in upper levels of water column	1. Total screening of intake flow not achieved 2. Potential debris clogging problems 3. Mechanical difficulties have been encountered
Louvers	Length, size, angle, ratio of approach to bypass velocity, slat spacing and flow conditions affect efficiency	1. Adequately tested in a variety of situations/conditions 2. Can be retrofitted	1. Diversion efficiency not acceptable to some agencies 2. Size/species-specific 3. Potential operational/reliability problems due to clogging 4. Relatively expensive

TABLE 5.5 Summary of Fish Protection System Advantages and Disadvantages *(Continued)*

Device	Biological/engineering variables affecting performance of device	Advantages	Disadvantages or limitations
		Diversion devices	
Gulpers	Velocity and flow conditions affect performance	1. Biological effectiveness good	1. Not adequately tested to date 2. Need adequate attraction flow 3. Total screening of intake flow not achieved
		Bypass systems	
Orifice bypass	Location of orifice and debris affect performance	1. Biological effectiveness good to excellent if fish can be attracted or guided to orifice	1. Potential for debris clogging
Horn collector	ID	1. None	1. Failed
Ice/trash sluiceways	Spilling affects efficiency	1. Biological effectiveness for smolts and juveniles good 2. Limited maintenance	1. None
Controlled spill	Flow, head, spillway configuration influence effectiveness and survival potential	1. Accepted by fishery agencies 2. Potentially low mortality	1. Mortality may be high at some sites 2. Further mortality studies required
		Physical barriers	
Bar racks	Length, size, water velocity influence response/performance; rack spacing determines the size of fish excluded	1. Can be backfitted	1. Species-specific response 2. Influenced by water velocity 3. Potential for clogging from debris
Traveling/stationary screens	Velocity, debris affect efficiency/ performance	1. Biological effectiveness good generally	1. Cleaning system required to properly operate system 2. Escape/bypasses required for system 3. Relatively expensive
Rotary drum screens	Velocity, screen slot width, orientation, debris load affect efficiency/performance	1. Biologically effective if velocity is low, screens are kept clean and fish escape routes exist	1. Debris/icing potential problems 2. Has not been effective where escape routes are not available
Infiltration intakes	Siltation can reduce water flow; effective backflushing mechanism required	1. Biological effectiveness excellent	1. Not practical for hydro due to limited flow capabilities

Cylindrical wedge-wire and box screens	Velocity, slot size, debris load affect efficiency/performance	1. Biological effectiveness excellent	1. Need low through-slot velocity 2. Clogging potential problem for fine mesh 3. Expensive 4. Limited to small flow rates
Barrier net	Velocity and debris affect reliability/performance	1. Proven effective under conditions of low velocity and light debris loads 2. Relatively inexpensive	1. Clogging problems affect reliability 2. Proper mesh size needed to avoid gilling fish
		Collection systems	
Traveling water screens	Impingement duration, velocity, mesh size, fish length and life stage affect efficiency	1. For selected species, can remove impinged organisms with low mortality 2. Can be backfitted/modified to improve survival	1. Species must be capable of withstanding impingement 2. Survival is species-specific 3. Velocity should be limited to 1.0 fps or less 4. Fine-screening may require continuous operation and have potential for clogging 5. Relatively expensive
		Fish pumps	
Volute pump	Length and pump speed affect survival	1. Biological effectiveness good	1. Limitation on maximum size of fish which can be pumped (60 cm) 2. RPM's must not exceed 600 for optimum survival 3. Relatively expensive
Jet pump	Nozzle and pipe velocities affect survival	1. Has been used successfully to collect and transport fish	1. Must be coupled with screening system to direct fish to pump suction 2. Relatively expensive 3. Survival of fragile species may be low 4. Low hydraulic efficiency
Screw-impeller pump	Pump speed affects survival	1. Has successfully transported fragile species with low mortality	1. Pump speeds should be kept between 400–600 rpm's 2. Further field testing necessary 3. Relative cost
Gatewell collection nets	Few variables affect performance	1. Proven effective for collecting juvenile salmonids	1. Operation requires 3–4 person crew 2. Cost

NOTE: ID = insufficient data.
Source: From Taft [27].

sense the turbulence of and move away from objects in their path, such as boulders, piers, and screens, as they move downstream.

A screen or louver set at 25° or less to the approaching flow will cause the fish to move laterally to one side of the channel as they sense the turbulence created by the screen or rack on the other side. Then the fish are directed into a side channel and fed back into the river downstream of the intake. Sketches of angled traveling screens and louver intakes are given in Figs. 5.21 and 5.22.

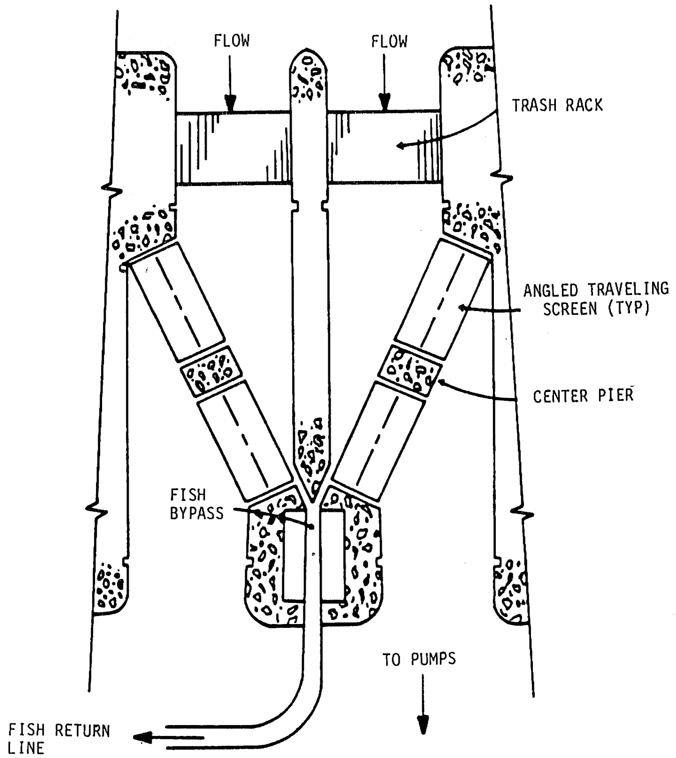


FIGURE 5.21 Conceptual angled screen design. *(From Taft [27].)*

The louvers designed for the San Onofre Nuclear Generating Station of the Southern California Edison Co. [29] had an optimum design of 1-in (2.54-cm) spaced louvers angled 20° to the approach flow. The approach velocity upstream of the louvers was 2.0 ft/s (0.6 m/s) with a bypass velocity of 2.5 ft/s (0.75 m/s). The bypass was designed for smooth flow because fish would not enter a bypass with highly turbulent flow.

Angled screens, using a standard through-flow screen, are also highly effective in diverting fish. Large power plants on Lake Ontario and the Hudson River [27] found the optimum performance with screens at 25° to the approach flow. Mean velocity upstream was 1.0 ft/s (0.3 m/s) with a similar velocity in the bypass. The minimum bypass opening was 6 in (15.2 cm).

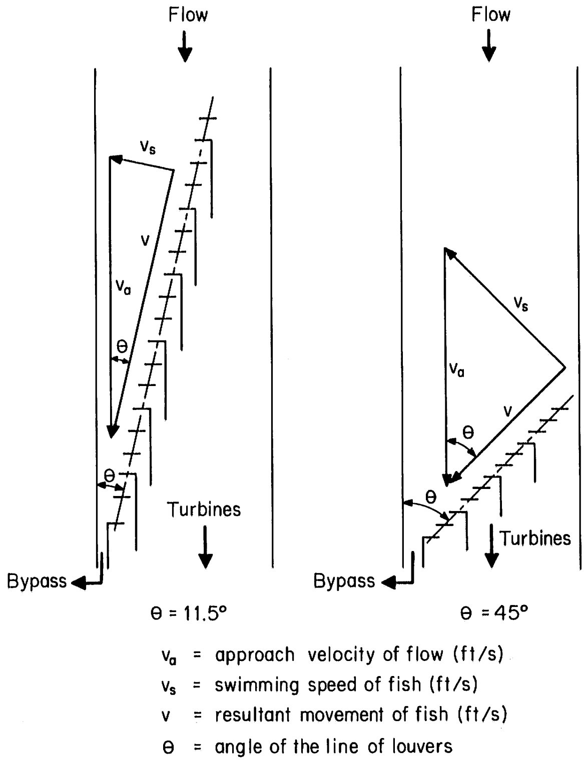


FIGURE 5.22 Conceptual sketch of louvers indicating the fish swimming velocity relative to approach velocity to avoid impingement.

Inclined-plane screens [30] are designed to divert fish upward into the bypass, as illustrated in Fig. 5.23. Design criteria are currently that velocities through the gross screen area should be approximately 0.7 ft/s (0.2 m/s). This criterion and the velocity in the penstock may be used to determine the angle to the approach flow. A high ratio of bypass velocity to approach velocity (10:3 at one plant) will

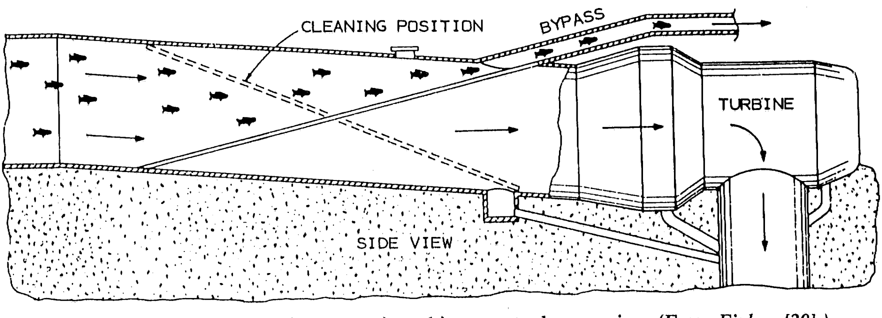


FIGURE 5.23 Pressure wedgewire screen in turbine penstock screening. *(From Eicher [30].)*

help maintain a high survival rate. Wedgewire screens are largely self-cleaning and are often used for this application. Punched flat plate screen is less expensive, but a tilting mechanism for cleaning must be included in the design.

Other fish diversion devices are discussed by Taft [28] for larger hydroelectric systems and Leidy and Ott [31] for small ones.

In the *fish barrier concept*, the velocities through a screen are kept very low by expanding the cross-sectional area. Thus, impingement of fish does not occur, and the mesh is sufficiently fine to exclude virtually all fish. Johnson Division of United Oil Products developed a cylindrical wedgewire screen to minimize both the velocities through the screen as well as the construction materials cost. This intake was tested and improved at the St. Anthony Falls Hydraulic Laboratory [32] and found to be very effective at reducing the intake velocities to a desired level. An illustrative sketch and an application are shown in Figs. 5.24 and 5.25. A stationary screen has been used for small hydroelectric systems, and barrier nets have also been used [28].

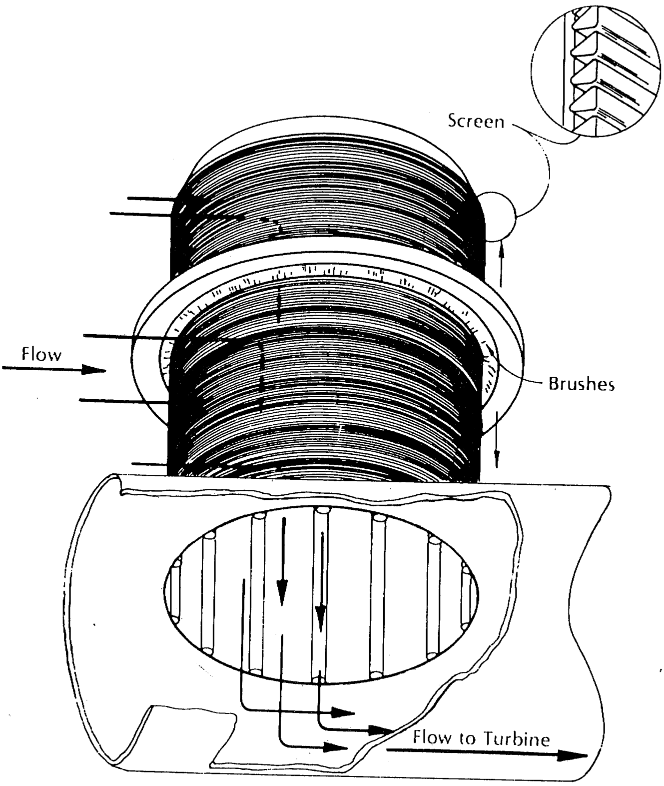


FIGURE 5.24 Illustrative sketch of the cylindrical wedgewire intake screen. *(From Leidy and Ott [31].)*

A third type of device in which the effectiveness is still being tested is behavioral barriers, designed to alter or take advantage of the natural behavior pattern

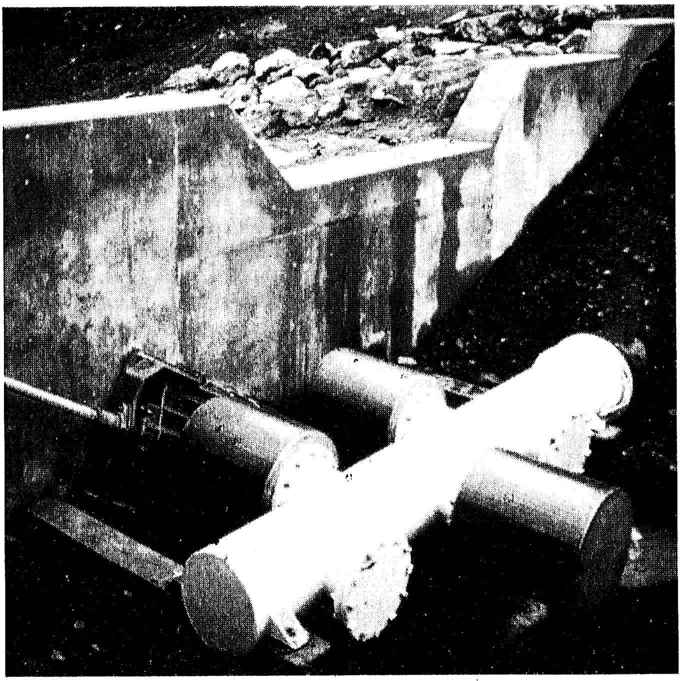


FIGURE 5.25 Cylindrical wedgewire screens for a 6 ft^3/s flow at Marathon Hydroelectric Project, Seward, Alaska. *(From Leidy and Ott [31].)*

of fish. They include electrical screens, air bubble curtains, hanging chains, lights, sound, water jet curtains, chemicals, visual keys, and any combination thereof. A few [28, 33] have potential, but most are not consistently effective.

5.4 GATES AND VALVES

Most intake-conduit systems have gates or valves for protection of the turbines and penstock. Even when the turbine may have the ability to shut off the flow, i.e., through wicket gates or needle valves, it is not uncommon that these gates or valves will not close properly. Thus, at least one additional closure mechanism should be built into the hydraulic conveyance system. Design considerations include the maximum pressure head the gate or valve can withstand, the tendency to have cavitation damage, head loss, leakage, required maintenance, and cost.

The following terminology is normally used to describe the various types of closure devices [34].

1. *Gate:* A gate is a closure device in which a leaf or closure member is moved across the fluidway from an external position to control the flow of water.
2. *Valve:* A valve is a closure device in which the closure member remains fixed

axially with respect to the fluidway and is either rotated or moved longitudinally to control the flow of water.

3. *Guard gate or valves:* Guard gates or valves operate fully open or closed and function as a secondary device for shutting off the flow of water in case the primary closure device becomes inoperable. Guard gates are usually operated under balanced-pressure no-flow conditions, except for closure in emergencies. Regulating gates and valves may be used to control the outlet flow of a reservoir or to control transient pressures that occur during turbine operation.
4. *Regulating gates and valves:* Regulating gates and valves operate under full-pressure and -flow conditions to throttle and vary the rate of discharge. Regulating gates and valves may be used to control the outlet flow of a reservoir or to control transient pressures that occur during the turbine operation.
5. *Bulkhead gates:* Bulkhead gates are usually installed at the entrance and are used to unwater fluidways for inspection or maintenance, and are nearly always opened or closed under balanced pressures.
6. *Stop logs:* Stop logs are installed in the same manner and perform the same function as bulkhead gates. A stop log may be considered as a section of a bulkhead gate which has been made of several units to permit easier handling.

Gates

The gates used most often in intakes and conduits are summarized in Fig. 5.26 [35]. Some of them will be discussed herein.

A *slide gate* consists of a leaf that is closed when positioned across the conduit and opened when pulled above it, into the bonnet.

Jet-flow gates were developed to reduce the cost of large outlet gates at the Shasta Dam for which tube valves were planned [34]. The gate consists of a wheel-mounted leaf moved vertically by a screw hoist. An upstream entrance to the gate is provided with a conical, converging transition ring, so that a contracted, circular jet that does not subject the housing to high pressures under any condition of discharge is formed in the vicinity of the leaf opening. The jet springs free of the wheel slots at the side of the gate leaf and decreases the impingement of the high-velocity water on the bottom portion of the conduit. Large air vents must be provided for supplying air to the conduit downstream from the gate so that when the gate is discharging at partial openings, the effects of cavitation resulting from subatmospheric pressures are minimized [36].

Radial gates are used in closed conduits under moderate head conditions and are generally employed when relatively large discharges are required. Early radial gates were often made of wood, but modern gates are made wholly of steel. Basically, the gate consists of a section of a cylinder connected to and supported by the two side beams, which are connected by hinge pins to the supporting structure. High-head radial gates are usually operated by means of a screw lift, which permits opening the gate and holding it at any desired opening, thereby allowing variable discharges. They are advantageous where large, unobstructed passages are desired, since no guides or seals protrude into the waterway. Radial gates are similar to spillway-tainter gates, except that the structural members are much heavier in order to resist increased water load, and seals across the top are

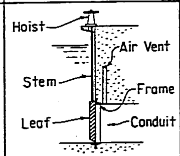
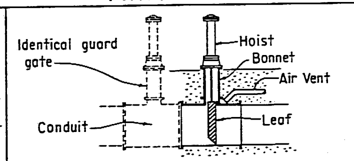
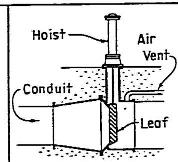
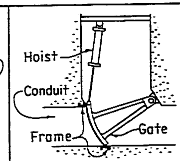
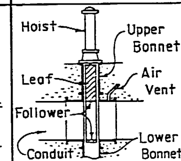
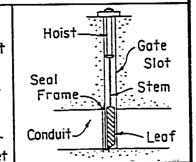
SERVICE CLASSIFICATION	THROTTLING GATES					GUARD GATES	
SCHEMATIC DIAGRAM FLOW DIRECTION →							
NAME	UNBONNETED SLIDE GATE	BONNETED SLIDE GATES "HIGH PRESSURE" TYPE	BONNETED SLIDE GATES STREAMLINED TYPE	JET-FLOW GATE	TOP-SEAL RADIAL GATE	RING-FOLLOWER GATE	WHEEL-MOUNTED OR ROLLER-MOUNTED GATES
MAX. HEAD (APPROX.)	75'	200'	500'+	500'+	200'–250'	800'	500'
DISCH. COEFFICIENT (a)	0.6 TO 0.8	0.85	0.87	0.60 TO 0.84	0.95	1.0	0.9
SUBMERGED OPERATION	NO	NO	YES (1)	YES (1)	NO	N/A	N/A
THROTTLING LIMITATIONS	AVOID VERY SMALL DISCH.	AVOID VERY SMALL DISCH.	AVOID VERY SMALL DISCH.	NONE	NONE	N/A	N/A
SPRAY	MINIMUM	MINIMUM	MINIMUM	SMALL	MINIMUM	MINIMUM	MINIMUM
LEAKAGE	SMALL	SMALL	SMALL	NONE	SMALL TO MODERATE	NONE	SMALL TO MODERATE
NOMINAL SIZE RANGE (b)	TO 12' WIDE & 12' HIGH	TO 6' WIDE & 9' HIGH	TO 10' WIDE & 20' HIGH	10" TO 120" DIA.	TO 15' WIDE & 30' HIGH	36" TO 120" DIA.	TO 10' WIDE & 30' HIGH
AVAILABILITY	COMMERCIAL STD. (1)	SPECIAL DESIGN	SPECIAL DESIGN	SPECIAL DESIGN	SPECIAL DESIGN	SPECIAL DESIGN	SPECIAL DESIGN
MAINTENANCE REQUIRED	PAINT	PAINT	PAINT (2)	PAINT	PAINT – SEALS (1)	PAINT	PAINT – RUBBER SEALS
COMMENTS AND NOTES: (a) Coefficients are approximate and may vary somewhat with specific designs. (b) Size ranges shown are representative, and are not limiting.	(1) Gates are readily available from several commercial sources They are not an "off-the-shelf" item, however.		(1) Air vents required (2) Use of stainless steel surfaced fluidways, will reduce painting requirements and cavitation damage hazard.	(1) Air vents required	(1) Seal replacement in 5-15 years is probable depending on design and use.		Normally wheel-mounted gates are used except for high heads.

FIGURE 5.26 Various types of gates for hydro development. Most gates may be used for either flow regulation from a reservoir outlet or as guard gates for the hydroplant. The ring-follower and wheel-mounted gates are designed strictly as guard gates (1 ft = 0.305 m, 1 in = 2.5 cm). *(From Kohler [35].)*

required in addition to those on the sides and bottom. The tunnel downstream from the gates must be designed to always flow partly full when the gates are fully opened, and ample air passages must be provided to the downstream side of the gates [36].

Ring-follower gates are used as a relatively inexpensive guard gate for circular conduits of up to 800-ft (245-m) head. The gate has a leaf composed of a bulkhead to block the flow when the gate is closed, and a circular opening which aligns concentrically with the conduit when the gate is open, as shown schematically in Fig. 5.27. The name comes from the fact that a ring portion follows the movement of a bulkhead portion. The gate requires a bonnet both above and below the conduit, and it is not suitable for partially open operation because of the resulting cavitation damage. The head losses with the gate, however, are virtually zero [34].

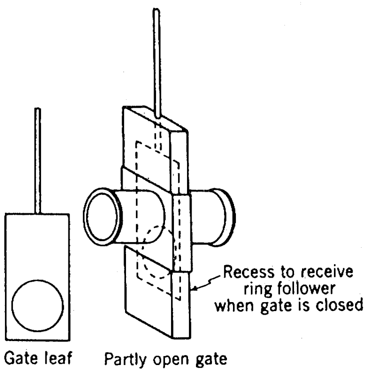


FIGURE 5.27 Schematic diagram showing the operation of ring-follower gates. *(From Linsley and Franzini [37].)*

Wheel-mounted and *roller-mounted gates* are, as their names imply, gates that are mounted on wheels or rollers. They are normally the primary shut-off gates for a conduit and are designed to close by gravity alone, so that a power outage does not preclude gate closure. To close under their own weight, the weight of the gate must be sufficient to overcome bearing friction forces [34].

Bulkhead gates or stop logs are placed over inlets or outlets under a balanced-pressure no-flow condition. They permit dewatering of the unit for subsequent inspection, maintenance, and repair. They are normally stored out of the waterway and placed in the gate slots with a crane when needed.

The head-loss coefficients for circular and rectangular gates are given in Fig. 5.28.

Valves

The most frequently used *valves* are summarized in Fig. 5.29 [35]. *Needle valves* are used at the outlet end of a conduit to control discharges under extremely high heads and are designed to discharge into the air, thereby eliminating the opportunity for cavitation within the conduit. They have a very precise flow control and are used almost exclusively as a flow-control device for high-head impulse turbines. The needle valve consists primarily of an outer casting housing a cylindrical inner mechanism so that an annular water passage is formed between the casting and the cylinders. The inner mechanism is arranged such that the discharge end of the annular passage is closed when hydrostatic pressure is supplied to a chamber inside the cylinders. Needle points are placed at the upstream and downstream faces of the cylindrical mechanism to guide the water flow into and out of the annular passage. Water issuing from the valve is always in the form of a jet, regardless of the valve opening, and control is easy to achieve. Usually needle valves are protected

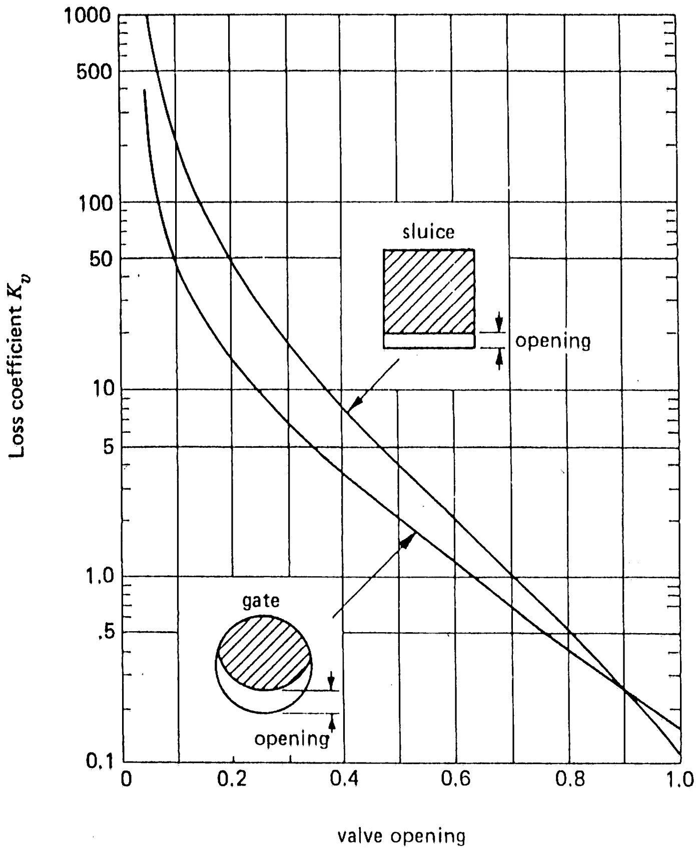


FIGURE 5.28 Head-loss coefficients for partially open rectangular and circular gates. *(From Miller [7].)*

by emergency gates located in the conduit upstream from the valve [36]. Although they have been used in many dams, such as the Hoover Dam, the use of needle valves in outlet works and as a pressure-regulating device has been supplanted by more economical and hydraulically efficient types of valves, such as the fixed-cone and hollow-jet valves.

The construction of *tube valves* is similar to that of needle valves, except that the movable needle is eliminated. It was designed to minimize the cavitation which developed at the downstream end of needle valves. Modifying the needle valve in this manner also resulted in a large weight savings and permitted its use in a conduit with less danger of damage from cavitation. The tube is normally moved by means of a motor-driven, screw-type actuator.

Tube valves are ordinarily better adapted to submerged discharges or for use inside a conduit, since an insufficient air supply does not appear to produce all the cavitational effects inherent in other types of valves. When located in the center portion of the conduit, tube valves have long bodies with a 30° nozzle and are provided with air inlets to aerate the jet immediately downstream from the valve seat [36].

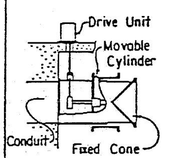
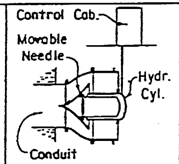
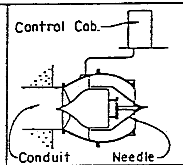
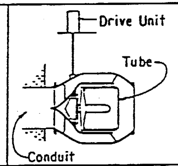
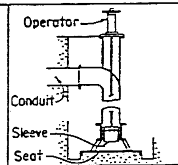
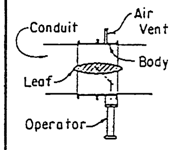
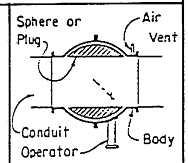
SERVICE CLASSIFICATION	THROTTLING VALVES					GUARD VALVES	
SCHEMATIC DIAGRAM FLOW DIRECTION →							
NAME	FIXED-CONE VALVE	HOLLOW-JET VALVE	NEEDLE VALVE	TUBE VALVE	SLEEVE VALVE	BUTTERFLY VALVE	SPHERICAL AND PLUG VALVES
MAX. HEAD (APPROX.)	1000'	1000'	1000'+	300'	250'+	750'+	1800'+
DISCH. COEFFICIENT (a)	0.85	0.70	0.45 TO 0.60	0.50 TO 0.55	0.80	0.7 TO 0.8	1.0
SUBMERGED OPERATION	YES (1)	NO (1)	NO	YES	YES (1)	N/A	N/A
THROTTLING LIMITATIONS	NONE	AVOID VERY SMALL DISCH.	NONE	NONE	NONE	SEVERE	SEVERE
SPRAY	VERY HEAVY (2)	MODERATE	SMALL	MODERATE (1)	NONE	NONE	NONE
LEAKAGE	NONE	NONE	NONE	NONE	NONE	NONE TO SMALL (1)	NONE
NOMINAL SIZE RANGE (b)	8" TO 108" DIA.	30" TO 108"-DIA.	10" TO 96" DIA.	36" TO 96" DIA.	12" TO 24" + DIA. (2)	12" TO OVER 12' DIA.	12" TO OVER 10' DIA.
AVAILABILITY	COMMERCIAL STD. (3)	SPECIAL DESIGN	SPECIAL DESIGN	SPECIAL DESIGN	SPECIAL DESIGN	STD. AND SPECIAL (2)	STD. AND SPECIAL (1)
MAINTENANCE REQUIRED	PAINT	PAINT	PAINT (1)	PAINT	PAINT	PAINT-SEALS (3)	PAINT
COMMENTS AND NOTES: (a) Coefficients are approximate and may vary somewhat with specific designs. (b) Size ranges shown are representative, and are not limiting.	(1) Air venting required. (2) Spray rating will change to moderate if a downstream hood is added. (3) Valves are not "stock" items but standard commercial designs are available.	(1) Submergence to ℄ of valve is permissible.	(1) If water operation is used, disassembly at 3 to 5 year intervals for removing scale deposits is usually necessary.	(1) Spray is heaviest at openings of less than 35%. At the larger openings the rating would be better than moderate.	(1) Valve is designed for use only in fully submerged conditions. (2) Larger sizes seem feasible and will probably be developed.	(1) Rubber seated valves have no leakage when new. Metal seats will have some leakage. (2) Sizes to 36" or 48" are fairly standard. Larger sizes and high pressures are usually special. (3) Metal seals may require periodic adjustment.	(1) Sizes to about 24" are fairly standard. Larger sizes and high pressures are special.

FIGURE 5.29 Throttling valves and guard valves (1 ft = 0.305 m, 1 in = 2.5 cm). *(From Kohler [35].)*

The *fixed-cone valve* (also known as the Howell-Bunger valve, after its inventors) is a widely used regulating valve. It is a very good energy dissipator and aerator, in addition to providing a flow control. A cone is held in place, pointed upstream, by radial ribs. Then a cylindrical gate opens and closes onto the cone. The discharge is controlled by the opening between the cylindrical gate and the cone. Flow leaves the valve at approximately the 45° angle of the cone; thus a spectacular spray (which may be a problem) occurs downstream. To restrict the angle of spray, a hood (or a continuation of the pipe) is often provided for a short distance downstream of the valve [34].

The *hollow-jet valve* is essentially one-half of the needle valve, with the needle turned so that it moves in the upstream direction in closing. The downstream nozzle is eliminated by allowing water to discharge from the bell-shaped body in a tubular or hollow jet, the outside diameter of which remains unchanged regardless of the valve opening. The hollow-jet valve derives its name from the shape of its discharge—a hollow or annular jet dispersed over a wide area. For a given size, the valve will discharge 25 percent more water than a needle valve. The body of the hollow-jet valve is not subject to the pressure of water in the reservoir and can therefore be of lighter construction than other control devices. The water passageways are proportioned to prevent subatmospheric pressures and consequent cavitation. Splitters in the water-passage area are necessary to support the interior parts of the valve, providing a means for introducing air inside the issuing jet [36].

The *sleeve valve* is designed to regulate flows under a fully submerged condition. It is similar to a fixed-cone valve, except that the movable cylinder is around the outside of the pipe and the cone sits on a horizontal seat. Thus the water discharges at 90° to the conduit. The sleeve valve operates within a vertical stilling well, providing an excellent dissipation of energy [34].

Butterfly valves are most generally used as emergency valves located immediately upstream from hydroelectric generating units, where the penstocks are long, or as emergency closure valves for outlet works. Essentially butterfly valves consist of a circular leaf slightly convex in form, mounted on diametrically opposed pivot shafts. One of the shafts is extended and fitted with a crank which is attached to a hydraulic cylinder hoist providing the motivating force for rotating the valve leaf from an open to a closed position.

When used for emergency closure, the butterfly valve operates in either a fully open or fully closed position and never remains at intermediate positions. Although butterfly valves can be used as control gates for low heads, they can only be used as emergency closure gates for higher-head installations [36]. Head-loss coefficients for butterfly valves are given in Fig. 5.30.

The *spherical valve* (also known as ball or plug valve) consists of a large sphere in a housing with a cylindrical hole through it equal to the size of the conduit. When the valve is open, the cylindrical hole is aligned with the conduit. Turning the sphere 90° will eliminate the cylindrical passageway and shut the valve. The head losses of a spherical valve are insignificant, while the costs are normally greater than a butterfly valve.

5.5 TUNNELS AND PENSTOCKS

Tunnels and penstocks are the conduits that convey the water from the intake to the turbine. A tunnel is normally not built for significant pressurization, and thus

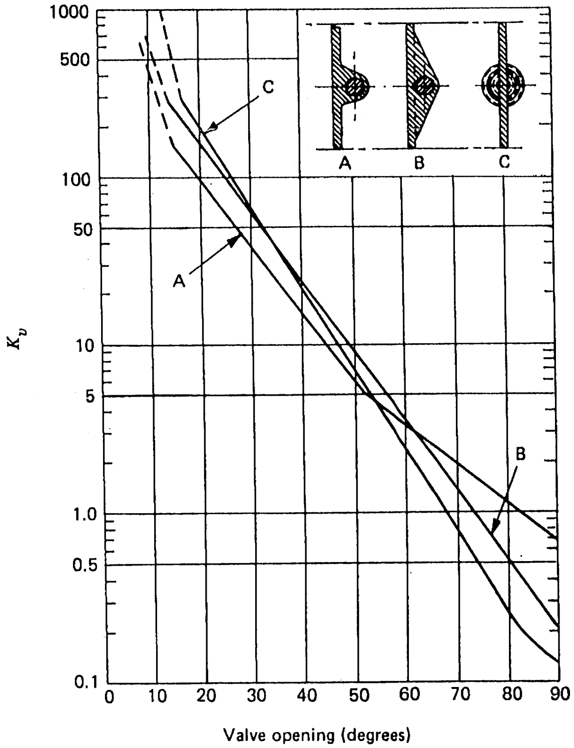


FIGURE 5.30 Head-loss coefficients for three types of butterfly valves. *(From Miller [7].)*

a surge tank is placed at the lower end. A penstock can be either pressurized or nonpressurized, depending upon site specifics and the design to handle unsteady pressure surges. A typical tunnel-penstock layout is given in Fig. 5.31. The length of the penstock is highly variable, from miles down the side of a mountain to nonexistent for low-head hydroplants.

Friction Head Losses

The primary source of penstock head losses is wall friction, bends, and bifurcations (splitting a penstock into two pipes), etc. Head loss due to wall friction h_f is given by

$$h_f = f\frac{LV^2}{D2g} \tag{5.14}$$

where f = Darcy-Weisbach friction factor
L = length of penstock
D = penstock inside diameter

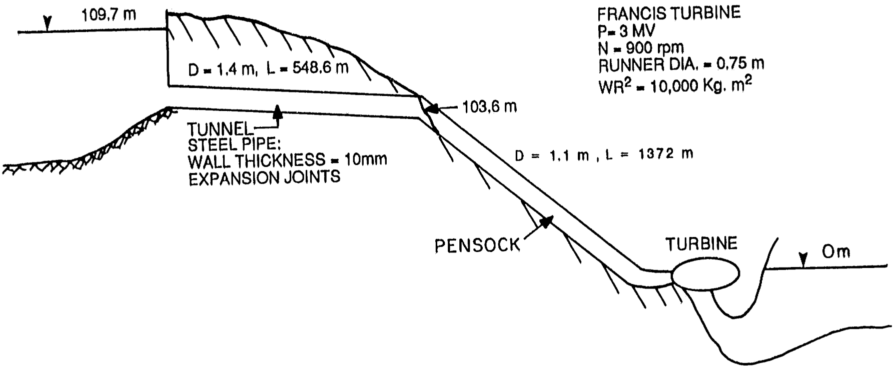


FIGURE 5.31 Layout of Rubi Mine Project, showing the tunnel and penstock.

V = cross-sectional mean flow velocity = $Q/(\pi D^2/4)$
Q = flow rate

The friction factor is a nondimensional number that gives the ratio of frictional forces and inertial forces, and therefore is the least susceptible to scaling problems and the most universal of the formulas. For noncircular conduits or open channels, $R_h = A/P$, where A is cross-sectional area and P is wetted perimeter, may be used. For a circular pipe, $R_h = D/4$.

The friction factor is given by the Colebrook formula as an implicit equation

$$\frac{1}{\sqrt{f}} = -0.869 \ln \left[\frac{\epsilon/D}{3.7} + \frac{2.523}{R\sqrt{f}}\right] \tag{5.15}$$

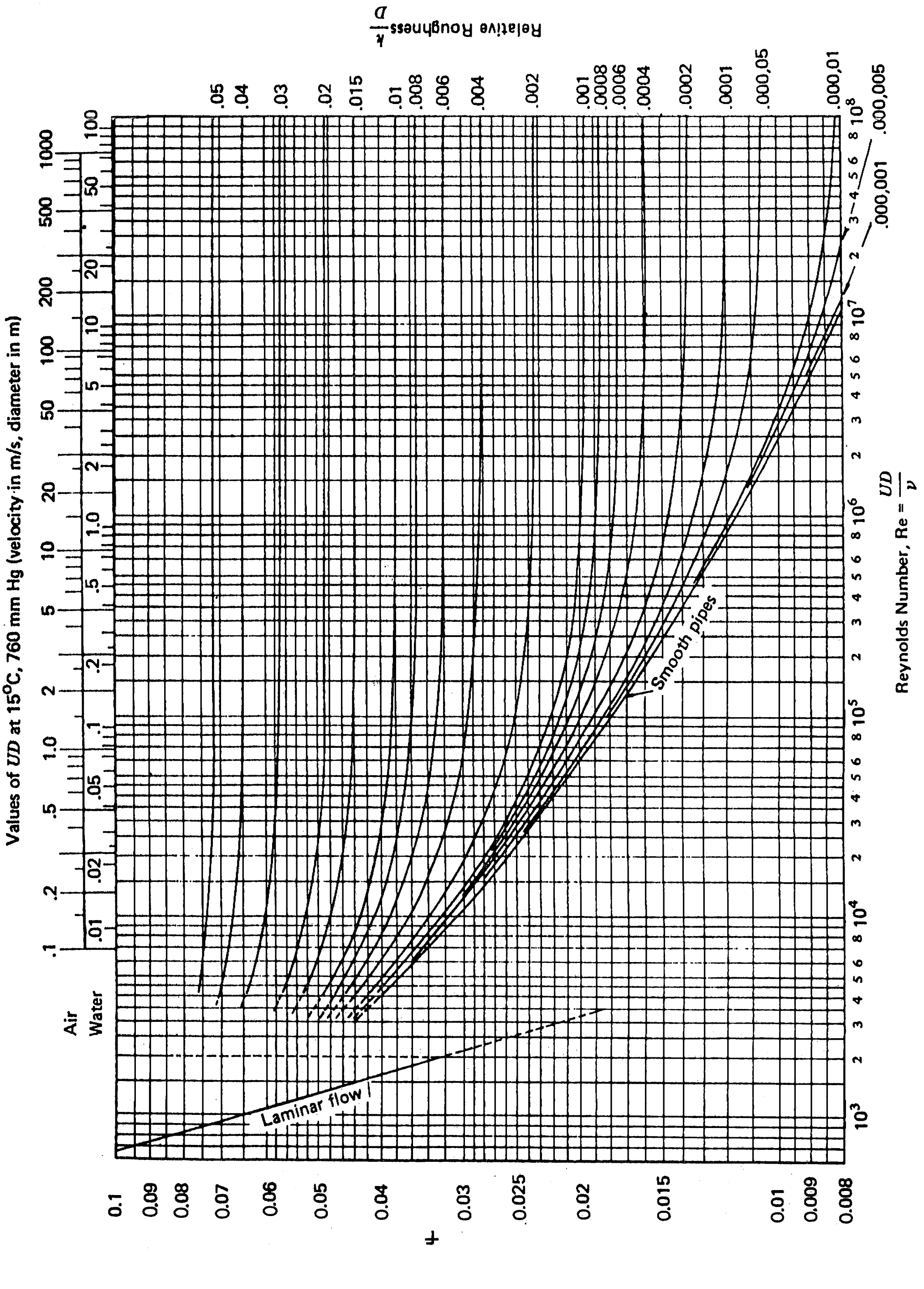
where R = Reynolds number (VD/ν or $4VRh/\nu$). An explicit formula that is accurate to within 1 percent is [38]

$$f = \frac{1.325}{\ln\{[(\epsilon/D)/3.7] + (5.74/R^{0.9})\}} \qquad \begin{array}{c} 10^{-6} \leq K_s/D \leq 10^{-2} \\ 5000 \leq R \leq 10^8 \end{array} \tag{5.16}$$

Values for f may also be obtained from a Moody diagram [39], as shown in Fig. 5.32. Values of f are plotted against the Reynolds number and relative roughness. Values of roughness ϵ are found in Table 5.6.

The surfaces listed are classified as good, normal, or poor examples of their respective categories, thus leaving to the engineer's judgment the actual value to be used in any particular scheme. The range of roughness covered accounts for the quality of the jointing and the variation in surface roughness to be found in pipes that are normally of the same material. It is advisable to check, by precise hydraulic tests whenever possible, the actual surface roughness achieved in a given project.

The roughness values bear some relation to the physical dimension of the roughness projections, and therefore a visual examination of a particular surface will give a guide to its roughness. Strickler's investigation of natural channels indicated that the ϵ value corresponded to the size which was exceeded by 10 per-



Values of UD at 15°C, 760 mm Hg (velocity in m/s, diameter in m)
Air
Water
Laminar flow
Smooth pipes
Relative Roughness $\frac{k}{D}$
Reynolds Number, Re = $\frac{UD}{\nu}$
f

TABLE 5.6 Recommended Roughness Values for Hydropower Tunnels and Penstocks

Classification (assumed clean and new unless otherwise stated)	Suitable values of ϵ mm*		
	Good	Normal	Poor
Smooth Materials			
Drawn nonferrous pipes of aluminum, brass, copper, lead, etc., and nonmetallic pipes of Alkathene, glass, perspex, etc.	NA	0.003	NA
Asbestos-cement	0.015	0.03	NA
Metal			
Spun bitumen or concrete lines	NA	0.03	NA
Wrought iron	0.03	0.06	0.15
Rusty wrought iron	0.15	0.6	3.0
Uncoated steel	0.015	0.03	0.06
Coated steel	0.03	0.06	0.15
Galvanized iron, coated cast iron	0.06	0.15	0.3
Uncoated cast iron	0.15	0.3	0.6
Tate relined pipes	0.15	0.3	0.6
Wood			
Wood stave pipes, planed plank conduits	0.3	0.6	1.5
Concrete			
Precast concrete pipes with O-ring joints	0.06	0.15	0.6
Spun precast concrete pipes with O-ring joints	0.06	0.15	0.3
Monolithic construction against steel forms	0.3	0.6	1.5
Monolithic construction against rough forms	0.6	1.5	NA
Fiber Composites			
Pitch fiber (lower value refers to full bore flow)	0.003	0.03	NA
Glass fiber	NA	0.06	NA
uPVC			
With chemically cemented joints	NA	0.03	NA
With spigot and socket joints, O-ring seals at 6- to 9-m intervals	NA	0.06	NA
Brickwork			
Glazed	0.06	1.5	3.0
Well pointed	1.5	3.0	6.0
Old, in need of pointing	NA	15	30
Unlined rock tunnels			
Granite and other homogeneous rocks	60	150	300
Diagonally bedded slates	NA	300	600
(values to be used with design diameter)			
Earth channels			
Straight uniform artificial channels	15	60	150
Straight natural channels, free from shoals, boulders and weeds	150	300	600

*1 mm = 0.04 in.

cent of the exposed bed material. In other conveyance facilities, the larger of the roughness elements probably have a similar predominating effect.

Figure 5.32 works well if one needs to determine the head loss associated with a given pipe at a given flow rate. However, when the allowable head loss is known and the pipe diameter needs to be determined at a given flow, or the discharge that a given pipe will pass needs to be known, Fig. 5.32 can only give the answer through a trial-and-error solution. The problem has recently been simplified in a very rational manner by Li [40], who inverted Moody's diagram for a determination of pipe diameter, given in Fig. 5.33, and the determination of pipe velocity, given in Fig. 5.34. Both graphs use the same parameters as Moody's diagram. Only the resulting parameter d or V, rather than f, is different.

Unlined *rock tunnels* have been built for flood flow diversion and for hydropower tunnels where the rock is of sound quality and not greatly jointed and fractured. These tunnels require special consideration to determine the roughness. A technique used by the U.S. Army Corps of Engineers given in Fig. 5.35 uses the cross-sectional area of the minimum excavation line to determine $\epsilon = K_s = K$ in the equation for fully rough frictional loss. It is compared with measured friction factors in the figure, a perfect fit being if all the data points fall on the line. Although the fit is not perfect, it is probably as good as can be expected for unlined rock tunnels, most of the measured friction factors being within ±20 percent of those predicted.

The friction coefficient will be influenced by

- Type of rock
- Method of excavation
- Direction of excavation
- Overbreak pattern ϵ
- Amount of lining
- Diameter and shape of tunnel D_m

The common techniques for reducing tunnel head loss are lining, increasing the tunnel cross section, or excavating a second tunnel. The approximate reduction of head loss which may be expected is given in Fig. 5.36.

Lower roughness values result for machine-bored tunnels. While the roughness varies between 60 and 600 mm for drill- and blast-excavated tunnels, machine-bored tunnels have a roughness of between 10 and 15 mm.

For *conduits with composite sections*, for example an unlined tunnel with a paved invert, a composite friction factor f_{comp} is required which takes into account the differences in roughness of the relatively smooth invert and the unlined walls and crown. The following formula will accurately predict the friction factor for composite sections:

$$f_{\text{comp}} = \frac{\Sigma(P_i f_i)}{\Sigma P_i} \tag{5.17}$$

where f_i = friction factor of surface i
P_i = perimeter of surface i

Friction calculations involve judgment in selecting roughness values. At high Reynolds numbers, an error of 100 percent in a roughness value causes about a 10 percent error in friction coefficient. As head losses vary inversely with at least

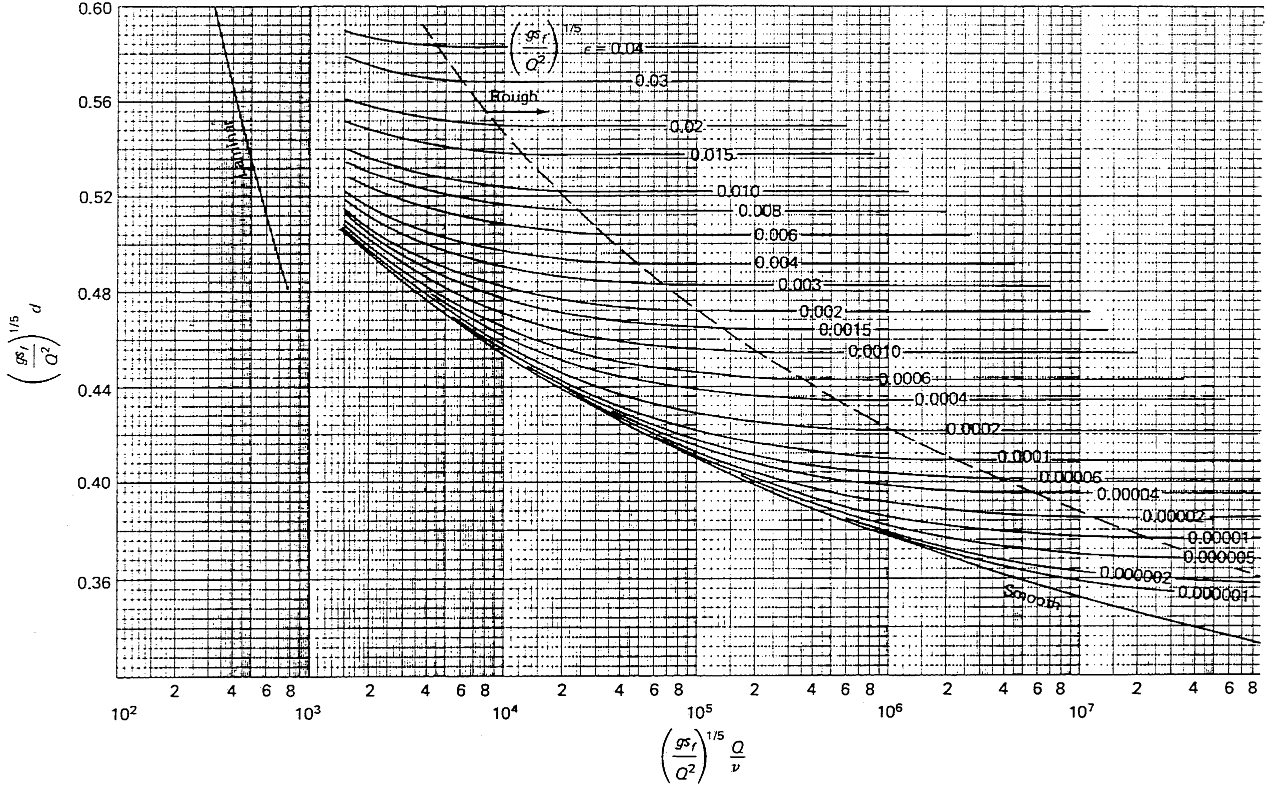


FIGURE 5.33 Diagram for determination of pipe diameter D. $S_f = h_L/L$ is the frictional slope of the energy gradeline. *(From Li [40].)*

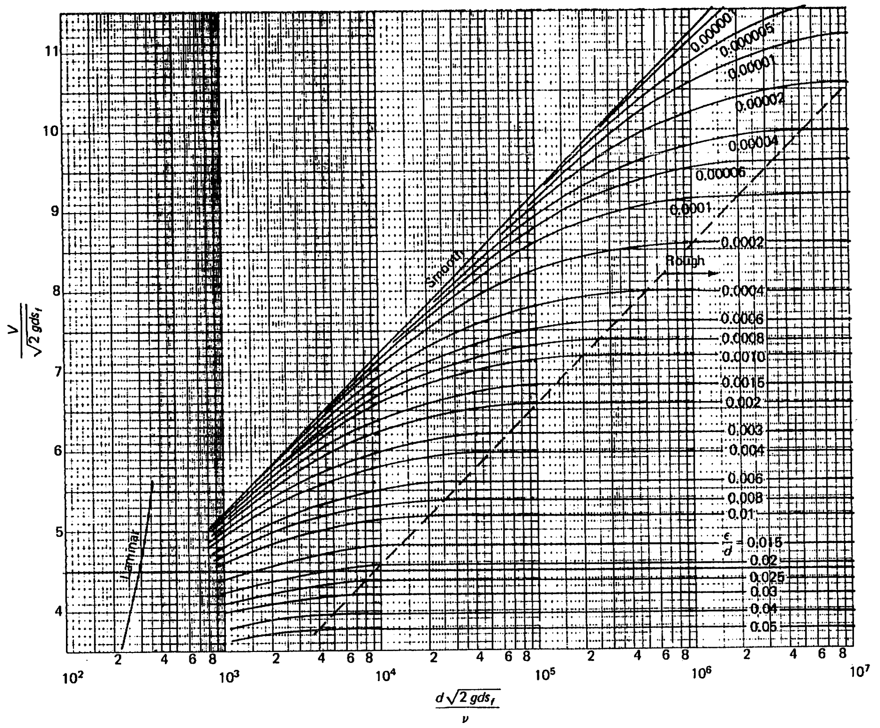


FIGURE 5.34 Diagram for determination of V. $S_f = h_L/L$. d = pipe diameter. S_f is the frictional slope of the energy gradeline. *(From Li [40].)*

the fourth power of the diameter, actual rather than nominal pipe diameters must be used.

For new pipes with estimated friction coefficients less than 1.2 times the smooth pipe friction coefficient, in which there is no fouling or deterioration of the walls, the head loss can be predicted with an accuracy of 5 percent, provided the pipe diameter is known to within 0.5 percent. Friction coefficients for similar pipes, but with estimated friction coefficients less than 1.5 times the smooth pipe values, can be predicted with an accuracy of about 10 percent.

Aging. The variation of the friction coefficient with age depends mostly on the composition of the water flowing in the conduit. Even thin coatings appearing on the walls can materially reduce the carrying capacity. Sliming due to algae growth on the walls has been known to triple the effective friction factor in a few years by reducing the nominal pipe diameter. The discharge capacities of tunnels and other water passages may also decrease with aging because of deposits and organic growths on the interior surfaces. These accumulations increase boundary-friction losses with resulting decreases in discharge capacities. Their general effect is to increase the roughness of the conduit wall material.

Typical allowances, where deterioration in service is expected, are 20 to 50 percent of new pipe values, but much higher allowances may be necessary where

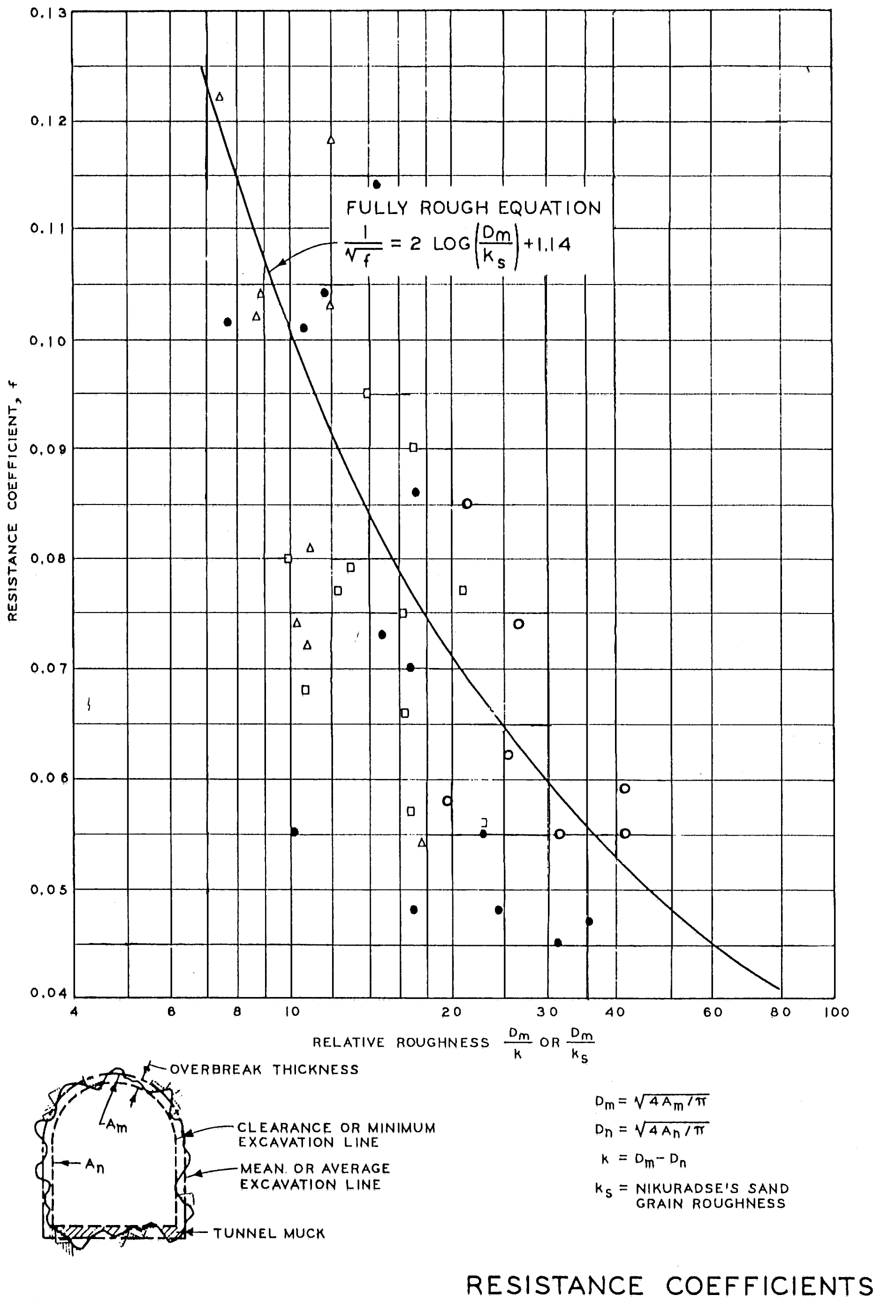


FIGURE 5.35 Comparison of unlined rock tunnel friction factors measured in the field with the technique used by the U.S. Army Corps of Engineers. *(From Army Corps of Engineers [41].)*

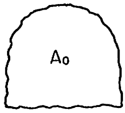
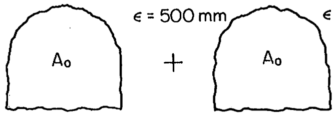
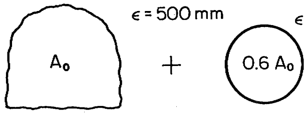
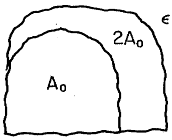
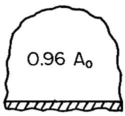
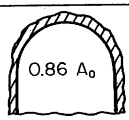
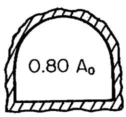
Method of head-loss reduction	Cross section and roughness, ϵ	Head loss as a percent of original
1. None	 ϵ = 500 mm; A_0	100
2. Additional tunnel with same area	 ϵ = 500 mm; A_0 + A_0; ϵ = 500 mm	25
3. Additional bored tunnel with same head loss	 ϵ = 500 mm; A_0 + 0.6 A_0; ϵ = 7 mm	25
4. Doubling of tunnel area	 ϵ = 500 mm; $2A_0$; A_0	16
5. Lining of tunnel floor	 ϵ = 500 mm; 0.96 A_0; ϵ = 1 mm	78
6. Lining of tunnel roof and walls	 ϵ = 1 mm; 0.86 A_0; ϵ = 500 mm	63
7. Lining of entire tunnel perimeter	 0.80 A_0; ϵ = 1 mm	25

FIGURE 5.36 Alternative methods of reducing head loss.

growths, deposits, or slimes are expected. Precautions against deterioration in service are

- Good initial surface finish to minimize areas of low velocity where deposits can begin to form in the wakes caused by roughness
- Adequate initial protection to prevent corrosion and erosion

One of the most difficult aspects of head requirement calculations is deciding what allowance to add to the calculated value in order to allow for uncertainties in loss coefficients, departures from nominal dimensions, and deterioration in service.

The procedure for obtaining a friction coefficient is to estimate a roughness height and use this along with the pipe diameter and Reynolds number to obtain a friction coefficient. In practice, friction coefficients are often based upon experience. One should always remember what the end usage will be and what part the value selected plays in the operation. High and low conservative values are necessary for surge tank calculations, water hammer, and other similar calculations.

Optimum Penstock Diameter. The optimum penstock diameter must minimize the sum of construction costs, maintenance costs, and the costs associated with head losses, i.e., the power and energy that is not produced because of head losses in the penstock. All of these costs vary with time, especially the value of power and energy, so relatively complex computer calculations are necessary. This may be done by a solution of a system of linear equations [42–44], or by an empirical equation that is easily used but is not specific to the conditions present at the site [45, 46]. Sakaria [45] found that an equation he had previously developed for economic penstock diameter

$$D(\text{m}) = 0.71P(\text{kW})^{0.43}/H_n(\text{m})^{0.65} \tag{5.18a}$$

$$D(\text{ft}) = 4.44P(\text{hp})^{0.43}/H_n(\text{ft})^{0.65} \tag{5.18b}$$

where P is the rated capacity of turbine and H_n is the net head on the turbine, was accurate in predicting a single penstock diameter to ±10 percent of the final chosen value for construction on approximately 40 conduits.

Fahlbusch [46] analyzed 394 steel-lined and concrete-lined conduits for conventional and pumped-storage hydropower plants. For steel-lined conduits, he found that the economical diameter could be expressed as

$$D(\text{m}) = 1.12Q(\text{m}^3/\text{s})^{0.45}/H_n(\text{m})^{0.12} \tag{5.19a}$$

$$D(\text{ft}) = 0.85Q(\text{ft}^3/\text{s})^{0.45}/H_n(\text{ft})^{0.12} \tag{5.19b}$$

to within ±20 percent. In addition, Fahlbusch found that the economic diameter for a concrete-lined conduit could be expressed as

$$D(\text{m}) = 0.62Q(\text{m}^3/\text{s})^{0.48} \tag{5.20a}$$

$$D(\text{ft}) = 0.37Q(\text{ft}^3/\text{s})^{0.48} \tag{5.20b}$$

to within ±20 percent. For concrete shafts with large gradients, the value obtained from Eq. (5.20) needs to be reduced by 10 percent.

Minor Losses (Form Losses)

Concentrated energy losses are usually associated with flow separation followed by intense mixing and flow reattachment. At the point of separation, the static pressure varies markedly because the flow is converging. Contraction of the flow persists downstream until a minimum effective flow area, the vena contracta, is reached. At the vena contracta, the static pressure is constant across the flow. Following the vena contracta, large-scale turbulence spreads throughout the flow, causing it to expand rapidly to fill the full cross section, creating an adverse pressure gradient (e.g., pressure increases) [7]. Up to the vena contracta, there is little energy dissipation, but after it considerable dissipation takes place. These

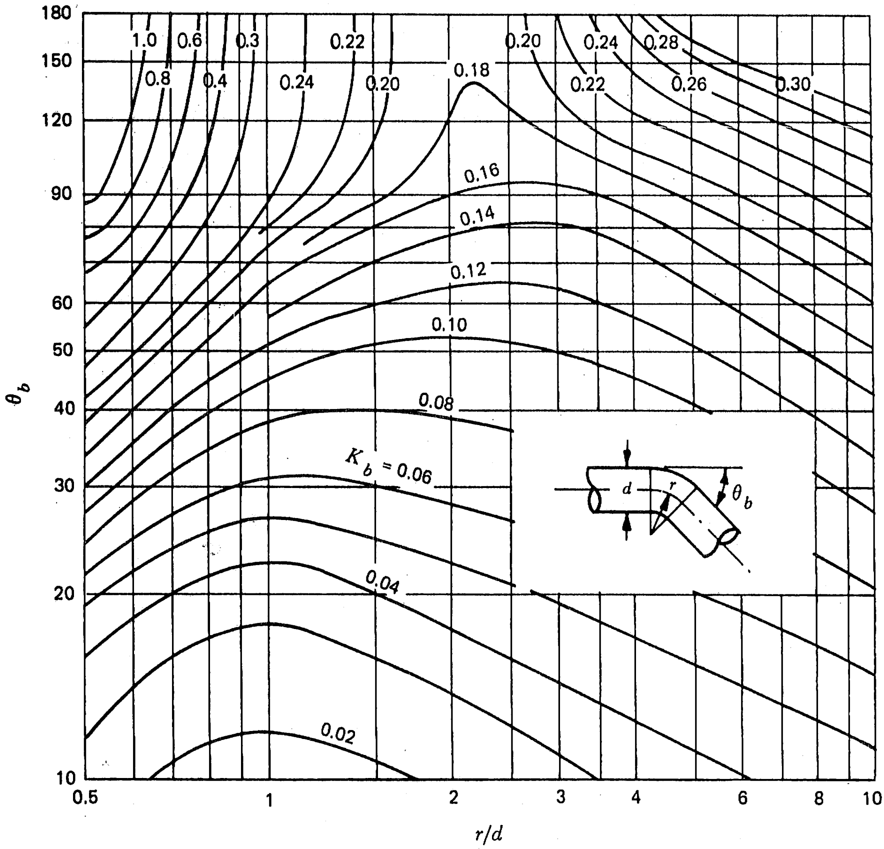


FIGURE 5.37 Bend performance chart, circular cross section (Re = 10^6). *(From Miller [7].)*

types of losses occur at junctions, bends, expansions, contractions, etc., in the conduit, and are called minor (or form) losses. In a long conduit that is fairly straight, frictional losses predominate and these form losses truly are minor. In shorter conduits, however, they no longer can be ignored, and frequently are the major source of head loss.

The head loss at a bend is given by

$$h_L = K_b \frac{V^2}{2g} \tag{5.21}$$

where K_b is a head-loss coefficient for bends. Charts that may be used to determine K_b are given in Fig. 5.37, with a Reynolds number correction in Fig. 5.38. For large pipes such as penstocks, a miter bend is often an economical design, and the increase in head loss may not be significant. Head-loss coefficients for single and composite miter bends are given in Figs. 5.39 and 5.40. Note that a three-segment composite bend has a head-loss coefficient that is very similar to the circular arc at r/d = 3.

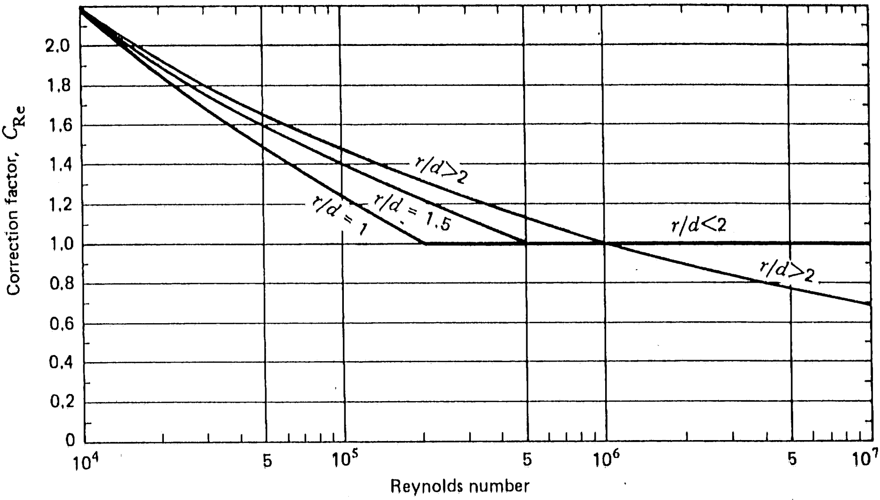


FIGURE 5.38 Reynolds number correction factor for Figs. 5.37, 5.39, and 5.40. *(From Miller [7].)*

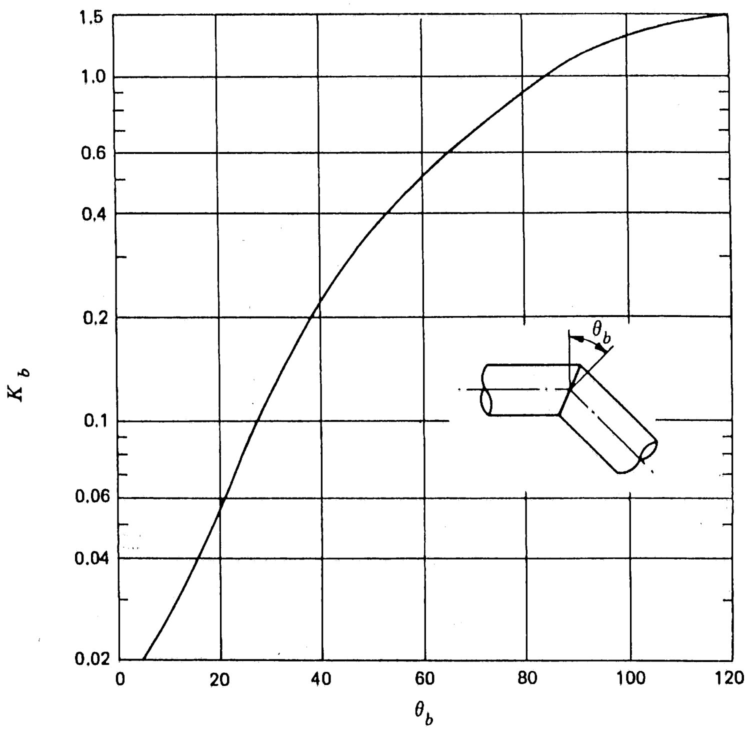


FIGURE 5.39 Miter bend loss coefficients (Re = 10^6). *(From Miller [7].)*

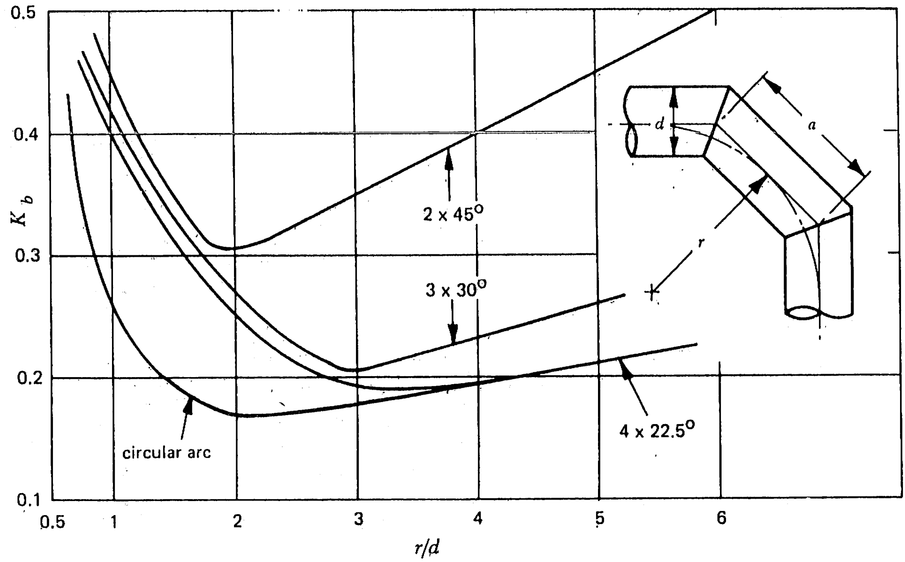


FIGURE 5.40 Composite miter bends (Re = 10^6); $r = a/2 \cot 90°/2n$ and n = number of bends. *(From Miller [7].)*

The head loss of an expansion is given by the equation

$$h_L = K_e \frac{(V_1 - V_2)^2}{2g} \tag{5.22}$$

where V_1 and V_2 are upstream and downstream of the expansion, respectively. Coefficients for a conical expansion are given in Fig. 5.41.

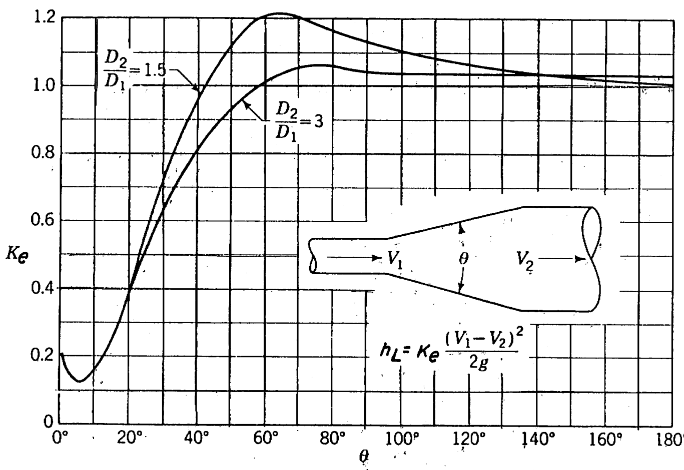


FIGURE 5.41 Loss coefficients for conical expansions. *(From Streeter and Wylie [6].)*

The head loss of a sharp contraction is given as

$$h_L = \left(\frac{1}{C_c} - 1\right)\frac{V_2^2}{2g} \tag{5.23}$$

where C_c = a contraction coefficient that depends upon area ratio, as given in Table 5.7.

TABLE 5.7 Contraction Coefficient versus Ratio of Areas. A_1 and A_2 Are Cross Sectional Area before and after the Contraction, Respectively

A_2/A_1	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
C_c	0.624	0.632	0.643	0.659	0.681	0.712	0.755	0.813	0.892	1.00

Dividing flow such as that occurring at bifurcations is the subject of a number of experiments because of the variety of arrangements and associated loss coefficients possible. The charts and figures required are too numerous to include here, but may be found in Miller [6], Idel'chik [7], and Williamson and Rhone [30].

Interaction of component losses is also a major topic of Miller [6] and Idel'chik [7]. A separation length of 25 pipe diameters is needed before the interaction is insignificant. The effect of component interaction, however, is to reduce head-loss coefficients by up to 30 percent. Thus, ignoring the component interaction will give a conservative estimate of head-loss coefficient.

Water Hammer

Water hammer is the pressure wave which occurs with a sudden change in penstock velocity, i.e., valve closure or turbine load rejection. It is normally associated with long penstocks, where the pressure wave does not return from the end of the penstock before the valve is fully closed. A pressure wave up to approximately 2100 ft (700 m) head is possible in hydroplants if valves are closed too rapidly. Negative pressures from a gate or valve opening require even more care. If the pressure in the penstock goes below the vapor pressure of water, the water will vaporize similar to water boiling or cavitation. This vaporized water is called column separation. The extreme vacuum will collapse penstocks and other conveyance facilities. A return of higher pressure to the region will return the vapor to the liquid state, and a column of liquid water is perceived to rush upstream at near the speed of the pressure wave. This column of water can have a devastating effect on control structures.

Water hammer calculations normally require a computer program for hydraulic transients, which contains the details of the turbine response, valve or gate closure, junctions, surge tank, etc. There are normally maximum and minimum acceptable pressures in the penstock system, and the computer program can be used to assure that these will not be exceeded. Then the program can be used to optimize design, i.e., to study the effects of a larger surge tank in a new location and a penstock of lower pressure capacity. Hydraulic transients are a potential problem if [6]

$$\frac{LV}{H} > 10 \text{ ft/s } (3.3 \text{ m/s}) \tag{5.24}$$

where L = length of penstock
V = flow velocity in penstock
H = net head

If the conditions specified in Eq. (5.24) are met, one needs to further investigate the possibility of water hammer problems. The most basic equation of pressure surge analysis may be written as

$$\Delta H = \frac{-a\,\Delta V}{g} \tag{5.25}$$

where ΔH = the change in pressure head associated with a given pressure wave
ΔV = the change in velocity which caused the pressure wave, i.e., closing or opening a valve, closing or opening turbine gates, load rejection, etc.
a = the speed of sound in water, which is the water hammer wave speed
g = the acceleration of gravity

Equation (5.25) indicates that the pressure head increases when the velocity is decreased, and vice versa. The water hammer wave speed is between 1000 and 4700 ft/s (300 m/s and 1400 m/s), depending upon the air content of the water, the relative wall thickness, wall elasticity, and the joints between the pipe sections. It is given by the formula

$$a = \left\{\frac{K/\rho}{(1 + K/E)[(D/d)C_e]}\right\} \tag{5.26}$$

where K = bulk modulus of elasticity for water, between 204×10^7 and 231×10^7 N/m^2, as given in Appendix D
E = Young's modulus of elasticity for the wall material, given in Table 5.8
d = wall thickness
ρ = density of water
D = inside pipe diameter
C_e = coefficient for reduced expansion at pipe joints; $C_e = 1.0$ for pipes with expansion joints

Most hydroplant penstocks are classified as thin-walled, e.g., $D/d > 25$. The coefficient of expansion C_e for thin-walled conduits is as follows [48]:

Conduit with frequent expansion joints

$$C_e = 1.0 \tag{5.27}$$

Conduit anchored against longitudinal movement

$$C_e = 1 - \epsilon^2 \tag{5.28}$$

where ϵ = Poisson's ratio, also given in Table 5.8.

Conduit anchored against longitudinal movement at the upstream end

$$C_e = 1.25 - \epsilon \tag{5.29}$$

Unlined tunnel

TABLE 5.8 Young's Modulus of Elasticity and Poisson's Ratio for Various Pipe Materials

Material	Modulus of elasticity E,* GPa	Poisson's ratio
Aluminum alloys	68–73	0.33
Asbestos cement, transite	24	
Brass	78–110	0.36
Cast iron	80–170	0.25
Concrete	14–30	0.1–0.15
Copper	107–131	0.34
Glass	46–73	0.24
Lead	4.8–17	0.44
Mild steel	200–212	0.27
Plastics		
ABS	1.7	0.33
Nylon	1.4–2.75	
Perspex	6.0	0.33
Polyethylene	0.8	0.46
Polystyrene	5.0	0.4
PVC rigid	2.4–2.75	
Rocks		
Granite	50	0.28
Limestone	55	0.21
Quartzite	24.0–44.8	
Sandstone	2.75–4.8	0.28
Schist	6.5–18.6	

*To convert E into lb/in^2, multiply the values given in this column by 145,038.
Source: From Chaudry [48].

$$C_e = \frac{d}{D} \text{ and } E = G \text{ (modulus of rigidity of the rock)} \tag{5.30}$$

Steel-lined tunnel

$$C_e = \frac{E}{E + G(D/d)} \tag{5.31}$$

where d = thickness of lining

A reinforced concrete pipe may be replaced by an equivalent steel pipe of thickness [47]

$$d_e = \frac{E_c}{E_s} d_c + \frac{A_s}{Z_s} \tag{5.32}$$

where d_e = equivalent pipe thickness
E_c = concrete pipe's modulus of elasticity
E_s = steel pipe modulus of elasticity
d_c = concrete pipe thickness

A_s = cross-sectional area of steel bars
Z_s = spacing of steel bars

Equation (5.32) may also be used for wood stave pipe, except that E_c/E_s = 1/60, e_c = thickness of wood staves, and A_s and Z_s refer to the steel bands. For both types of pipes, wave speed is then computed from Eq. (5.26).

Air entrained into the flowing water will greatly reduce the velocity of pressure (sound) waves in addition to the elasticity of the water. The propagation of sound through multiple air-water interfaces created by the air bubbles is logically slower than propagation through water or air alone. Figure 5.42 indicates that 0.1 percent air by volume will reduce the pressure wave speed by approximately 50 percent. Normally, for hydroplants, it is assumed that no air will be entrained into the penstock, as a conservative estimate.

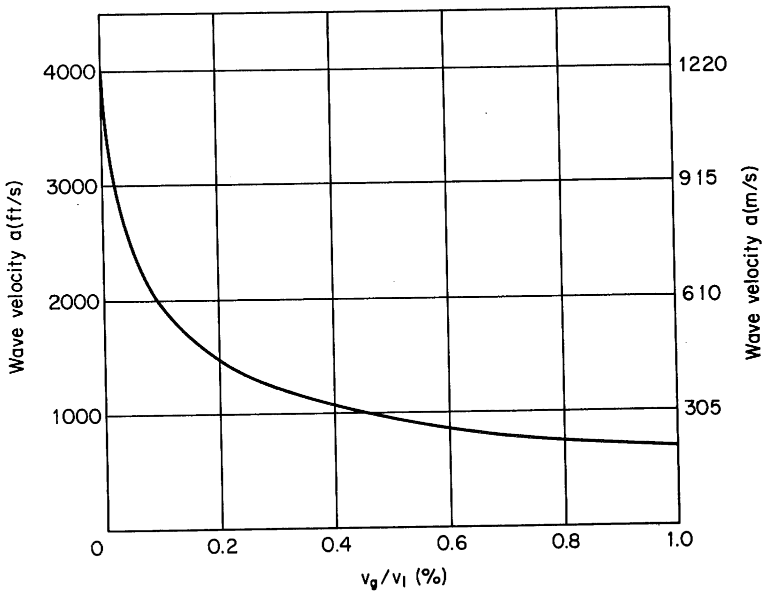


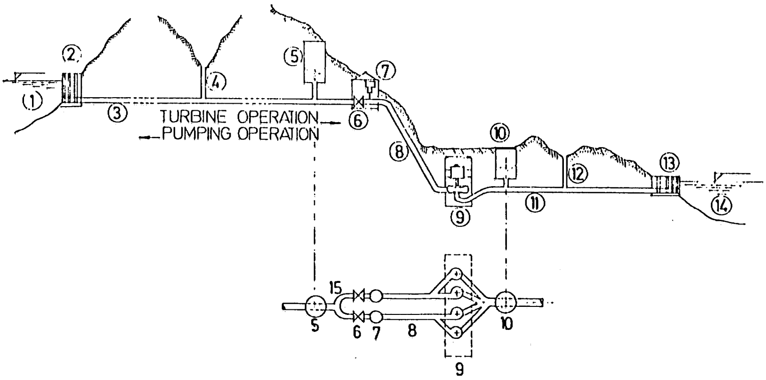
FIGURE 5.42 Propagation velocity a of pressure wave in pipeline for varying air content. V_g and V_l are the volume of gas and liquid, respectively.

For relatively short conduits, the pressure wave has reached the end of the pipe and returned before a significant ΔV occurs; thus water hammer is not a concern. For very long conduits, however, a fast change in velocity can create a significant water hammer, and plant design must include some means of reducing these pressures, such as longer valve closure and opening times, surge tanks, and pressure-regulating valves.

Reduction of Water Hammer Pressures

There are three types of devices commonly used to reduce hydraulic transients: surge tanks, air chambers, and valves. In addition, transients may be reduced by increasing penstock diameter, reducing the water hammer wave velocity (through entrained air or flexible piping), changing the penstock profile, increasing generator inertia, or slowly opening or closing valves and gates. Planning for extreme events usually precludes a long gate closure time.

Figure 5.43 illustrates a typical pumped storage facility or hydroplant with a number of devices to protect against water hammer pressures in the tunnel, penstock, and tailrace tunnel. Often, these devices are designed to mitigate transient pressures that occur in normal turbine operation, emergency operation, and extreme emergencies such as a broken linkage on wicket gate arms. The protection device, and the anticipated damage to the system, can vary for each. The deciding factors are the protection costs, operation costs, potential damage and subsequent outage costs, and the liability.



1. Upper storage basin
2. Intake/outlet structure
3. Tunnel
4. Aeration pipe
5. Surge tank on the tunnel
6. Tunnel valves chamber
7. Air release valves
8. Penstocks (in this case two)
9. Powerhouse
10. Lower surge tank
11. Tailrace
12. Aeration pipe
13. Outlet/inlet structure
14. Lower pool

FIGURE 5.43 Typical layout of a high head pumped storage project. *(From Pejović et al. [50].)*

A surge tank is an open standpipe connected to the penstock. Figure 5.44 illustrates the various types of surge tanks that are commonly in use. If a surge

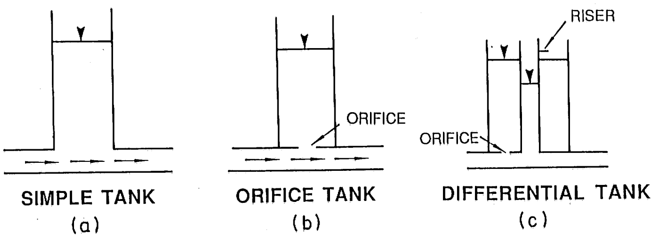


FIGURE 5.44 Types of surge tanks.

tank is properly sized, it will deflect water hammer, so that the only length of pipe to be used in water hammer analysis is between the surge tank and the turbine or valve. The surge tank should be located as near to the turbine as local topography permits. It should be sized so that (1) pressure waves are dampened, (2) the tank does not drain, and (3) the tank does not overflow. The actual sizing of a surge tank involves a more detailed engineering analysis with a computer program for hydraulic transients, although charts do exist for a preliminary design. A surge tank can also be placed at an angle. It will require more length to achieve the same height, but the effective cross-sectional area of the pipe, the open water surface, is larger than that of a vertical pipe.

Of the surge tanks in Fig. 5.44, the simple and differential types are most often used. The differential tank is roughly 30 percent smaller than the simple surge tank, and the decrease in pressure rise and fall is often not as great. The basic idea is to have an interior standpipe (riser) to dampen the high-frequency pressure fluctuations and a surge tank to dampen the lower-frequency fluctuations. A differential surge tank designed for a 6.4-MW hydroplant with 250 ft of head is given in Fig. 5.45 [51]. The response of the surge tank and riser water elevation to full-load rejection is given in Fig. 5.46.

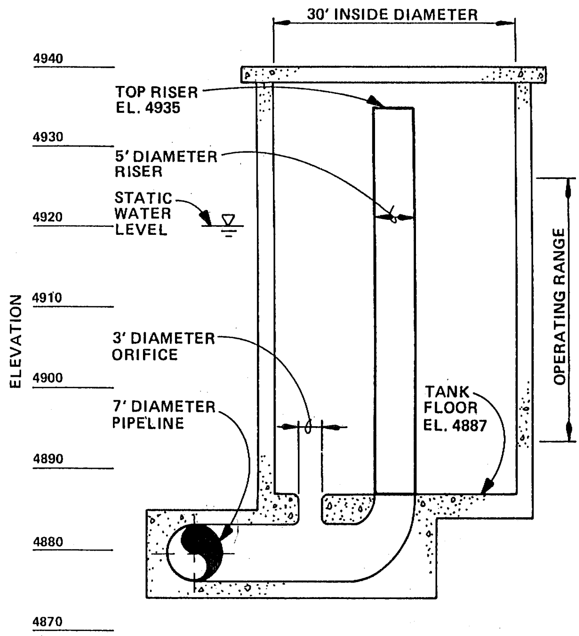


FIGURE 5.45 Differential surge tank. All elevations are in feet above mean sea level (1 ft = 0.305 m). *(From Otter [51].)*

The *air chamber* shown in Fig. 5.47 is a tank with compressed air at the top and water at the bottom. The primary advantage is that it can be located near the turbine, which may not be practical in the case of a surge tank because of an excessive height requirement. A differential orifice, which is shaped such that it produces more head loss for flow into the chamber than for flow out, is usually

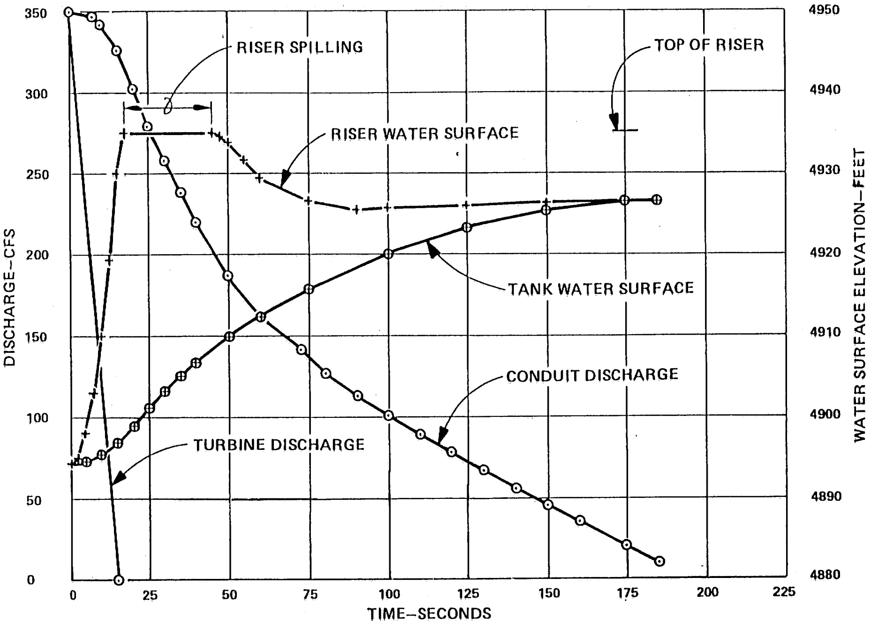


FIGURE 5.46 Response of differential surge tank and riser elevation to load rejection of a 5.2-MW hydro facility with a 15-s gate closure (1 ft = 0.305 m, 1 ft^3/s = 0.0282 m^3/s). *(From Otter [51].)*

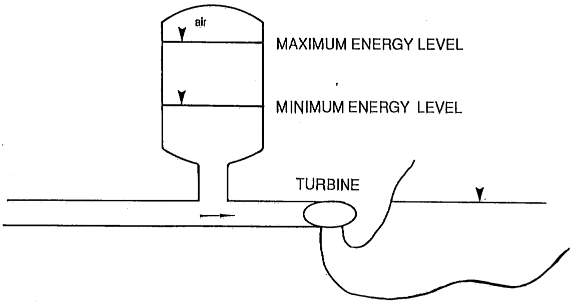


FIGURE 5.47 Illustration of an air chamber.

placed between the chamber and the pipeline. The outflow from the chamber should be as free as possible to prevent low pressures and hence column separation [48]. The air in the chamber expands or contracts as water flows out of or into the chamber. In addition to being of less height, the air chamber volume is somewhat smaller than a surge tank and it can be laid on the ground, e.g., the chamber has no height requirement. The main disadvantage is that the air volume is slowly reduced due to leakage or dissolution into the liquid, and an air compressor and auxiliary equipment must be supplied and continually maintained.

Air chambers have been used most extensively in Norway [52]. From 1973 to 1985, nine hydroplants with air chambers were installed. The advantage is reduced construction costs, especially with an underground hydroplant where an air chamber can be built with conventional tunneling methods [53]. The U.S. Army Corps of Engineers also designed both an air chamber and conventional surge shaft for the Snettisham project in Alaska [53]. The air chamber would have cost less because it eliminated the need for a 950-ft (289.5-m) high surge shaft, but was rejected because the savings over the conventional technique did not justify the use of an untested design.

A number of *valves* may be used to control transients by themselves or in conjunction with a surge tank or air chamber. A *safety valve* (overpressure pop-off valve) is a spring- or weight-loaded valve which opens when the pipeline pressure exceeds a certain limit. It offers no protection against low pressures. A *rupture membrane* may also be used for the same purpose. A chute is normally designed to carry off the water without undue erosion. A *pressure-relief valve* is similar to a safety valve except that the valve opening is proportional to the pressure (above a certain limit). Again, there is no protection against low pressures.

The *pressure-regulating valve* (PRV) is an automatically controlled throttling valve which is opened or closed by a servometer. A number of pressure regulating valves are shown in Fig. 5.29. The opening and closing times of this valve can be set. A line to the valve can be set parallel to the powerhouse, or the valve offtake can come directly from the scroll case, as illustrated in Fig. 5.48. A mechanical linkage to the wicket gate servo mechanism is an often-used alternative for controlling the valve opening. A 0.4-s time lag is typical in accounting for slop in the servo mechanism. Following load rejection, the valve is opened rapidly as the wicket gates or powerhouse valves are closed, such that the velocity in the penstock is relatively constant. The PRV is then closed slowly.

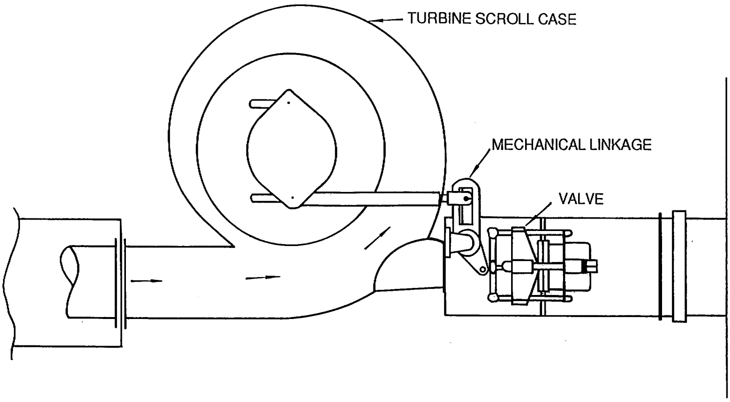


FIGURE 5.48 Pressure-regulating valve with offtake from turbine scroll case.

Ideally, the discharge characteristics of the PRV and the turbine can be matched to entirely eliminate pressure transients. This is not possible, however, because of the lag time between closing the wicket gates and opening the PRV. In addition, the turbine–wicket gate discharge characteristics are nonlinear, and not easily matched. These nonideal conditions should be taken into account in transient calculations. In most situations, PRVs will not provide protection against low pressures.

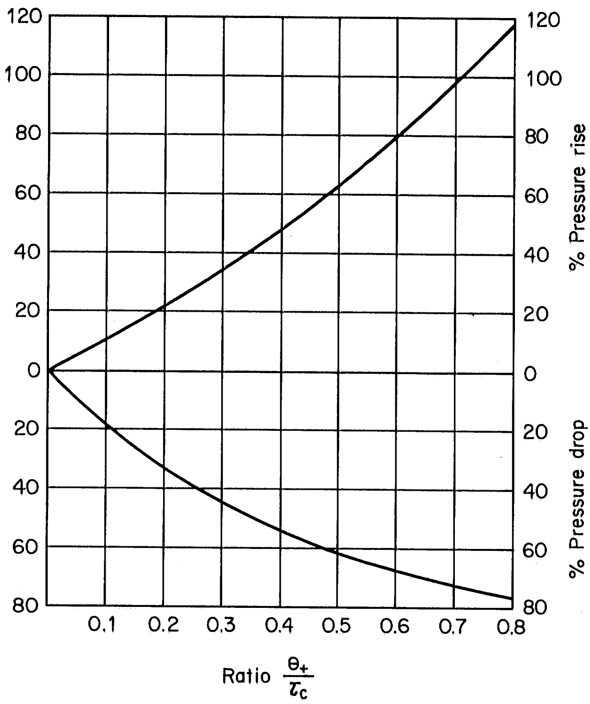


FIGURE 5.49 Maximum and minimum pressure rise and drop in a penstock that may be used as guidelines in preliminary design. *(From Brown [54].)*

Air-inlet valves and *aeration pipes* are used to admit air into the pipeline when severe underpressure occurs, preventing collapse of the pipeline. Aeration pipes are illustrated in Fig. 5.43. They are generally recommended as a safeguard against the extreme event. Once air has been admitted to the pipe, extreme care should be used in refilling the line because the entrapped air can result in unusual pressure transients, similar to those of water column separation.

The water hammer protection devices and their usage are summarized in Table 5.9.

Preliminary Design

Although computer flow computations are recommended for final penstock design, a preliminary design is possible for some components if simplifying assumptions are made.

A relatively simple calculation for maximum and minimum penstock pressures may be made from the chart given in Fig. 5.49. Given an equivalent servomotor opening or closing time, τ_e, approximately equal to the time obtained at the site under dewatered conditions, then compute the starting time of the waterway or water column θ_x

TABLE 5.9 General Characteristics of Protection Devices

Device	When to use	Protects against: High pressure	Low pressure	Vibrations	Extreme events	Operating reliability	Special demands	Problems in restarting	Frequency of application	Cost
Surge tank	High head with long tunnels/penstocks	Yes	Yes	Sometimes	Yes	Very good	None	None	Very often	Very high
Air chamber	High head with long tunnels/penstocks	Yes	Yes	Sometimes	Yes	Good	Air compressor	Routine control	Rarely	High
Increased inertia (MD^2)	All types	Yes	Yes	No	No	Excellent	None	None	Often	Medium
Regulated opening and closure	All types	Yes	Yes	No	No	Good	High-pressure oil installation	Routine control	Very often	Low
Pressure-relief valves	Medium/small plants	Yes	No	No	Sometimes	Fair	Frequent maintenance and control	Routine control	Not very often	Medium
One-way surge tank	Very long tunnel/penstock	No	Yes	No	Yes	Fair	Frequent maintenance and control	Routine control	Rarely	Medium
Air valves	Protection of penstocks	No	Yes	No	Yes	Fair	Frequent maintenance and control	Routine control	Very often	Low
Aeration pipes	Protection tunnel/penstock	No	Yes	No	Yes	Good	None	None	Often	Low
Rupture membrane	As a last measure of protection	Yes	No	No	Yes	Good	Arrange outlet for water discharge	Replace membrane	Rarely	Low

Source: Adapted from Pejović et al. [50].

$$\theta_x = \frac{LV}{gH} \text{ (s)} \tag{5.33}$$

where L = length of section to surge tank, air chamber, or turbine
V = velocity in section
H = head before change in rate of flow
g = acceleration due to gravity

Assuming a rigid, steel-lined pipe without frictional loss (the conservative case), the maximum and minimum pressures can be estimated from Fig. 5.49. The percent pressure rise is as a percent of net head. Frictional head loss in the penstock should be added to that value to estimate the maximum pressure. The percent pressure drop is as a percent of gross head (headwater elevation minus tailwater elevation). Frictional head loss is subtracted from that value to estimate the minimum pressure which could occur in the penstock [54].

More detailed preliminary design charts which consider the elasticity and thickness of the pipe, the anchoring of the conduit, and the frictional head loss, are given in Chaudhry [48]. Chaudhry also gives preliminary design charts for simple surge tanks and air chambers. Creager and Justin [55, p. 737] give design charts for simple and differential surge tanks. Relatively straightforward, menu-driven computer programs that can handle a limited number of cases have also been developed for hydroplants [56].

5.6 TOTAL HEAD LOSSES

The total head loss for an intake-penstock system is the sum of all the individual head losses. If there are two penstocks, each should be done separately.

The steps in obtaining an estimate of system head losses at a given flow rate are

1. Define the geometric parameters of the system and components, i.e., pipe diameter and length, bends, etc.
2. Determine the flow parameters, such as velocities and Reynolds numbers, from the flow rates and cross sections.
3. Select appropriate loss coefficients, as discussed herein.
4. Calculate individual component losses and correct as necessary for interactions between components.
5. Sum the individual system losses plus the static lift or pressure differential across the system, to establish the turbine head available.
6. If choosing the most economical size penstock, for example, estimate construction and operating costs, estimate the value of power not generated because of head losses, choose a new penstock diameter, and return to step 1.

When geometric and flow parameters are known, the selection of appropriate loss coefficients is the main task. In situations where the flow or pipe and components size has to be found, the simplest and usually the quickest method is to adopt a trial and approximation procedure. Equations (5.18) through (5.20) will help in selecting an economical penstock diameter. However, the final selection should be based on the particular economics of the site.

5.7 HYDRAULIC MODELS

The design of hydraulic conveyance facilities are influenced by site-specific conditions. Although standard designs exist for many types of hydraulic structures, site conditions often limit their use. The possibility of a poor design is increased when the engineer cannot use the standard design or previous experience. A hydraulic model study can be performed to verify that the proposed design functions properly. The model may also be a tool to improve structure performance or to reduce anticipated construction costs.

Any structure involving fluid flow is a candidate for a hydraulic model study. Structures often modeled are spillways, intakes, outlets, other control structures, bridge piers, and other flow obstructions. A spillway model might be used to investigate stilling basin performance, pressure distribution, and fluctuations on the spillway face, erosion above and below the spillway, and air entrainment. An intake model study may include investigation of head losses, resonance frequencies to avoid fluid-structure interaction, erosion around the intake, and the possibility of vortex formation. Models of outlet structures are used to indicate erosion patterns and, if applicable, demonstrate the diffuser performance of the outlet. Other structures, such as bridges, drop shafts, weirs, and flow diversions frequently require a hydraulic model study.

A typical hydraulic model is a small-scale representation of a planned structure designed to simulate the flow around or through the prototype structure. For example, Fig. 5.50 shows a model of the Guri Dam in Venezuela. The actual prototype is shown in Fig. 5.51. The model was used to study the characteristics of flow over and below the spillway and the erosion below the spillway.

Physical versus Computer Models

The hydraulic flow around a structure can be very complex, requiring three-dimensional modeling, and often consideration of two-phase flow (water and air or sediment), which is difficult to represent mathematically. Computer modeling is possible and is comparable in cost to a physical model study, but it requires more time to implement because the boundaries of the prototype must be input and tested in the computer model. The time required for computer program development and verification is often not practical to the design engineer with a tight schedule. Computer models are, therefore, generally restricted to special applications on specific details of hydraulic structures and are not used to model the entire flow field.

Computer models are, however, especially applicable when complex flow structures, such as water supply networks and storm sewers, cannot be accurately simulated with a physical model. Furthermore, some hydraulic problems, such as watershed runoff, groundwater flow, and lake and ocean hydrodynamics, are too large to be scaled down to a physical model and are, therefore, most appropriately studied with a combination of field measurements and computer modeling.

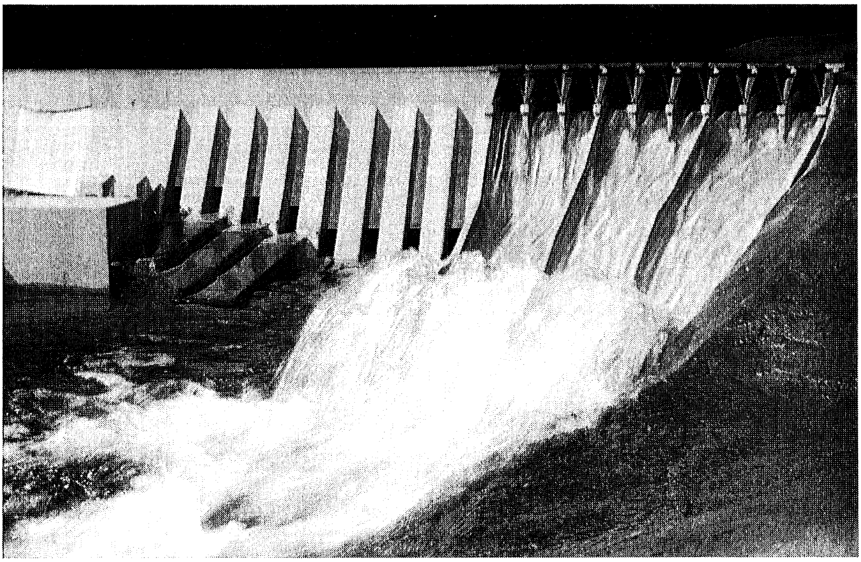


FIGURE 5.50 A reduced-scale hydraulic model of the Guri Dam in Venezuela simulated the characteristics of the flow over and below the spillway and the erosion below the spillway. *(From Anderson and Charbonneau [57].)*

Need for a Hydraulic Model Study

A model study should be performed whenever the risks associated with the design justify the model cost. Designers should ask three questions:

1. What is the possibility the structure will not perform adequately?
2. What costs are associated with inadequate structure performance?
3. What potential cost savings can be achieved as a result of a model study?

A model study may be justified based either on the potential cost consequences of a poor design or upon cost savings which could be realized based on modeling results. Postconstruction trial-and-error solutions of hydraulic problems in the prototype can be very expensive and may be unsuccessful. If possible, a hydraulic model study should be considered in the early stages of project design to avoid unnecessary costs and time delays.

The cost of a hydraulic model study may vary over a wide range beginning at approximately $25,000 (in 1982 U.S. dollars), depending upon the size and complexity of the structure and the number of problems to be investigated. The cost of the model study will increase if complex morphology is involved; if sediment transport or erosion is to be modeled; if sophisticated, detailed measurements are required; or if more than one specific aspect of the flow around the structure is to be investigated. The latter case may actually require more than one model for accurate investigation.

Even when a hydroplant has no unique design components that demand a model, a model study may often be justified on the credibility it brings to the hy-

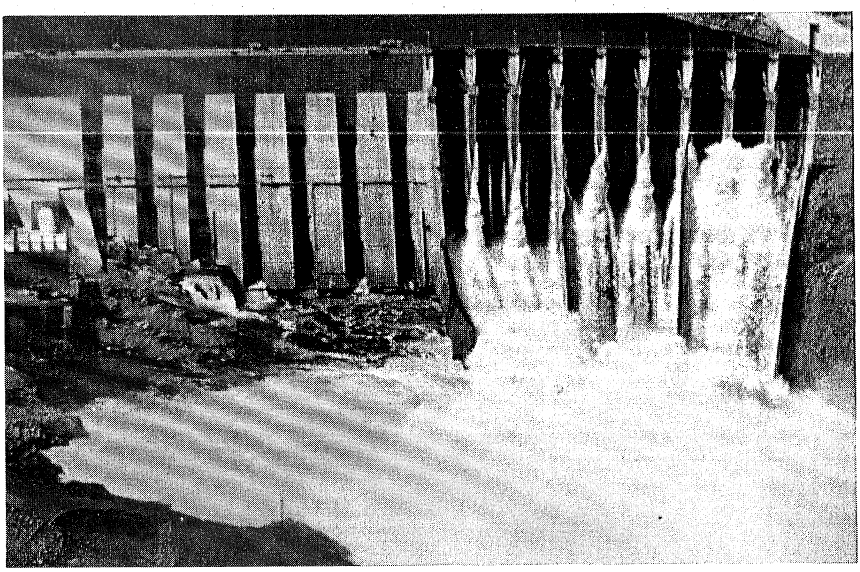


FIGURE 5.51 Guri Dam shortly after completion illustrates the prototype flow characteristics. The model and prototype are very similar except that there is more air entrainment in the prototype. *(Photo courtesy of Harza Engineering Co.)*

draulic design of a project. Table 5.10 gives the hours of plant output at $0.05/kWh (U.S. dollars) that will pay for a typical model study. The indication here is that most hydroplants above 5 MW can justify the cost of a hydraulic model

TABLE 5.10 Hours of Hydroplant Output That Will Pay for a Typical Hydraulic Model Study

Plant capacity, MW	Typical model study cost,* $(U.S., 1988)	Equivalent hours of plant output @ 5¢/kWh $(U.S.)	Conclusions
40	$100,000 (intake and outlet)	50	Yes! Good insurance.
20	60,000 (intake)	60	Yes! Good insurance.
10	60,000	120	Likely, check time schedule.
5	60,000	240 (10 days)	Probably, depending upon design considerations.
1	30,000	600 (25 days)	Only if important questions remain.
0.5	30,000	1200 (50 days)	Consider options such as the cost of trying various designs on site.

*These model study costs are toward the lower end of the possible scale. Model cost can escalate rapidly if additional detailed measurements are requested, if sediment movement is to be modeled, if a large region is to be modeled, etc.

study based upon only a few days of output, while a design problem can keep a plant off-line for months.

Performance of Model Studies

To implement a model study, the client provides the laboratory with design flow conditions as well as preliminary design drawings of the structure and surrounding morphology. The preliminary design drawings are reviewed by laboratory personnel, and decisions are made concerning the extent of modeling necessary. A model scale is selected based upon several factors: space available for the model, flow requirements, type of measurements to be made, cost, and scale effects.

The scale effects must be minimized to ensure that the flow conditions and other parameters adequately represent those of the prototype. The model studies are scaled so that the ratios of important dimensions, velocities, and forces remain constant with the prototype (the laws of similitude). Significant forces in a hydraulic model study include inertial forces (the force required to stop the flow), gravity, viscous forces such as friction, pressure forces, and surface tension. Perfect similitude requires that the ratios of the inertial force to each of the other forces remain constant between model and prototype. Although perfect similarity is obviously never achieved, effective model studies emphasize the most important forces. The model should be designed so that forces of lesser importance will not distort the results.

In a hydraulic model with a free-surface flow, such as a hydroplant intake or outlet, inertial and gravity forces usually predominate. Thus, model-prototype dynamic similitude is most closely approached by keeping the Froude number constant:

$$\text{Froude number} = \text{Fr} = \frac{V}{\sqrt{gL}} = \frac{\text{inertial force}}{\text{gravity force}} \tag{5.34}$$

where V equals flow velocity, g equals acceleration of gravity, and L equals a characteristic length dimension, such as depth of flow.

The Froude number in the model must be equal to that of the prototype if dynamic similarity is to be achieved:

$$\frac{V_m}{\sqrt{gLm}} = \frac{Vp}{\sqrt{gLp}} \qquad \text{or} \qquad \frac{V_m}{Vp} = \sqrt{\frac{L_m}{L_p}} \tag{5.35}$$

where m and p denote model and prototype, respectively. Once the geometric scale is established, the velocity and discharge ratios can be determined.

The significance of viscous effects (friction) must also be considered in the selection of geometric scale and model flow velocity. Viscous effects are governed by a characteristic velocity and length, and the kinematic viscosity of the fluid ν. The ratio of inertial force to viscous force is called the Reynolds number, as follows:

$$\text{Re} = \frac{\text{inertial force}}{\text{gravity force}} = \frac{VL}{\nu} \tag{5.36}$$

Although it is not possible to maintain the same Reynolds number for the model and prototype, the viscous forces may be of secondary importance to gravity forces, if the Reynolds number is sufficiently large. Thus, a minimum allowable Reynolds number at various locations in the model is usually the criterion that determines the model scale.

Frequently, similitude of friction and inertial forces (which is not quite the same as Reynolds similarity) is sufficient. This is accomplished by having a constant friction factor f between model and prototype

$$f = \frac{\text{bottom friction}}{\text{inertial force}} \tag{5.37}$$

When the prototype has a sufficiently large roughness, the model roughness may be adjusted to equate friction factors.

Example Model Studies

The Guri Hydroelectric Project, located on the Caroni River in Venezuela, is a large facility with a capacity of 10,600 kW and 531.5 ft (162 m) of available head. The dam is a concrete gravity structure with an earth- and rockfill tie-in to the right bank. The project was developed in a number of stages, involving at least five distinct model studies. Figure 5.50 gives a comprehensive 1:197 dam model and spillway section. The spillway chutes in the prototype are approximately 656 ft (200 m) long. Water surface profiles and pressure distribution on the spillway face and gate structures for various discharge and gate-opening combinations were examined in the model [57]. The study also included experimental observations of the flow pattern downstream of the spillway and in the large scour hole and also included development of tailwater rating curves and spillway bucket design.

Other model studies of the Guri Hydroelectric Project included flow investigations for river diversion schemes and a study on spillway cavitation in a depressurized, free-surface water tunnel [58]. It is common for large projects to require several individual model studies because of the extremely large flow volumes and flow velocities involved.

The Rapidan Hydroelectric Project, near Mankato, Minnesota, is a retrofit of new axial-flow turbines into an abandoned powerhouse. The redevelopment included two 2.5-MW turbines, half the number originally installed and with a greater total capacity. Each intake will therefore pass a significantly higher flow at greater velocities with the same submergence and approach conditions as the previous smaller units.

Because of the large angle of the approach flow, intake vortices were a design concern, even though prescribed guidelines had been met. A 1:11.7 Froude scale model showed severe intake vortices at all operational reservoir elevations [59]. At the lower reservoir elevation, the vortex was so severe that adjacent water collapsed into the air core, as shown in Fig. 5.52.

A flow improvement study recommended the following additions to the final design: (1) semicircular additions to the pier noses, (2) an antivortex plate with large diameter holes to give a 51 percent porosity installed directly behind the trash racks, (3) a solid wall extended at a 90° angle to the rightmost pier, and (4) a slotted wall extended parallel to the rightmost pier. The resulting improvement in the flow was dramatic, as shown in Fig. 5.53. All of the four additions were required to eliminate vortex problems.

The 8.5-MW St. Cloud Hydroelectric Project, on the Mississippi River in Minnesota, required a model study to test warranty requirements, test for free-

FIGURE 5.52 The model of the Rapidan hydroplant intake has severe vortices at all operational reservoir levels. At the lower reservoir elevation, the vortex was so severe that adjacent water collapsed into the air core. *(From Lindblom and Gulliver [59].)*

FIGURE 5.53 Four separate design modifications were required to eliminate vortices in the Rapidan intake model. *(From Lindblom and Gulliver [59].)*

surface vortices at the intake, reduce head losses, and generally improve the flow into the hydroplant [60]. The model is shown in Fig. 5.54. Intake vortices were observed and eliminated with a submerged grid, high velocities coming into the hydroplant were reduced by increasing the upstream excavation, and the I-beams supporting the trash racks were boxed in to reduce head loss and improve the flow entering the turbine. The total reduction in head loss was 3 in (7.3 cm). When compared to the 16.5-ft (5-m) head, this improvement in head loss was equivalent to a 1.5 percent increase in efficiency. Finally, two warranties were also tested in the model. The contractor had specified that two-unit operation would give at least 95 percent of the sum of single-unit operation. This was determined to be 99.4 percent, well within the guarantee. In addition, the turbine manufacturer had specified that flow in the gate slots be uniform with less than 5 percent cross flow. Flow velocities were measured at this location, and it was found that flow was beginning to split around the generator pit such that the warranty requirements were impossible to meet. This evidence and other velocity profiles indicating a high-quality flow persuaded the turbine manufacturer to drop this flow specification.

The 48-MW Jim Falls Hydro Redevelopment Project on the Chippewa River in central Wisconsin required a 1:60 Froude scale model, shown in Fig. 5.55, to observe and improve flow conditions [61]. Extensive rockfill modifications were made in the head race to eliminate vortices formed at the turbine intakes. Flow conditions were greatly improved by making the approach channel more uniform in depth. On-site placement of excess rock material instead of off-site disposal resulted in project cost savings. Changes to the geometry of the tailrace diffuser improved flow conditions and reduced the amount of excavation required. The model also revealed the need for a stilling basin at the base of the spillway. A

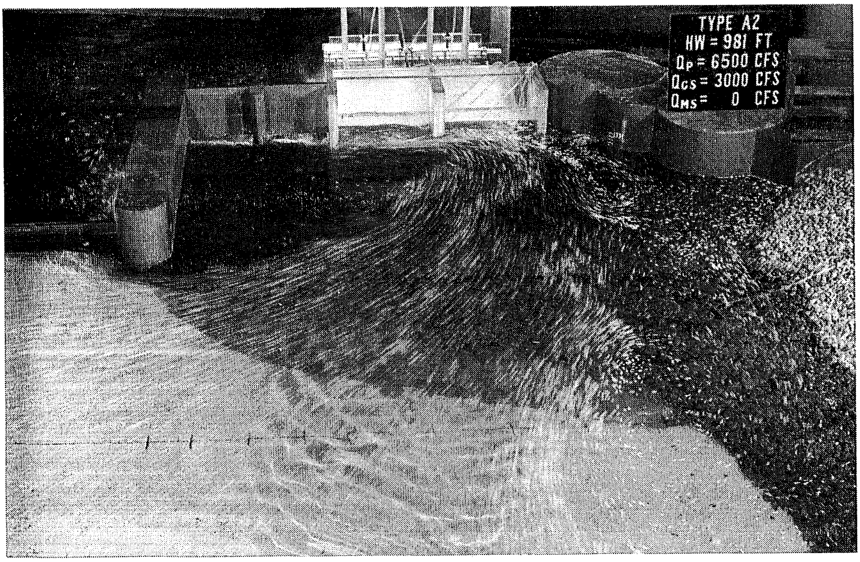


FIGURE 5.54 Water surface streaklines and surface waves before upstream modifications of the St. Cloud Hydraulic model. *(From Gulliver et al. [60].)*

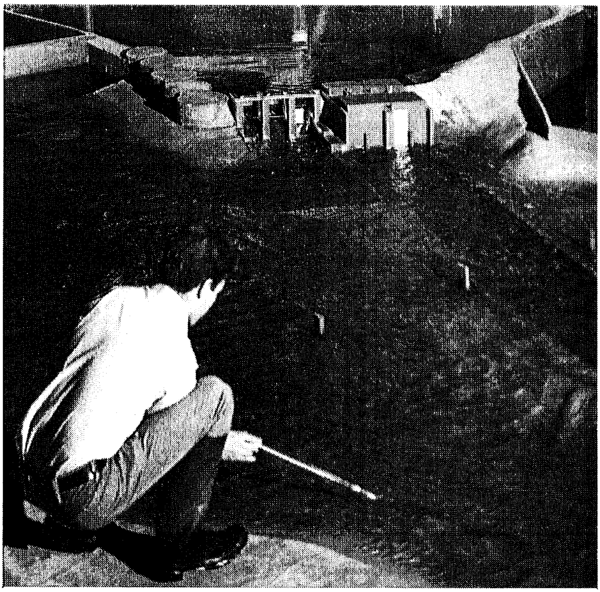


FIGURE 5.55 Observing flow patterns with dye in the Jim Falls hydraulic model. *(From Stefan et al. [61].)*

stilling basin was designed and tested. Baffle blocks were added to provide stability to the location of the hydraulic jump.

5.8 CASE STUDY

The proposed Rubi Mine Hydroelectric Project, illustrated in Fig. 5.31, has approximately 360 ft (110 m) of gross head with a design discharge of 128 ft^3/s (3.62 m^3/s). The plant design output would be approximately 3 MW. Other initial specifications are given in Fig. 5.31. The value of the power has been estimated at $2000 (U.S.) per kilowatt. The following questions are typical of a preliminary design study:

1. What intake submergence is recommended?
2. What size penstock is most cost-efficient?
3. Is there a water hammer problem? If so, what types of devices might be cost-effective for mitigation of this problem?

For intake submergence,

$$\mathrm{Fr} = \frac{V}{\sqrt{gD}} = \frac{(4)\ 3.62\ (\mathrm{m^3/s})}{\pi(1.4\ \mathrm{m})^2 \sqrt{9.8\ \mathrm{m/s^2}(1.4\ \mathrm{m})}} = 0.63$$

Figure 5.12 indicates that Fr should be less than 0.5 to reach a "reasonably safe" zone. This is, therefore, quite marginal. However, a preliminary minimum submergence of $S/D = 0.7$, or $S = 1$ m (3 ft) will be suggested.

Table 5.10 indicates that a hydraulic model study of the intake region could be paid for with approximately 10 to 20 days of full output operation. However, a turbine at the end of a long penstock is not as sensitive to intake vortices because the swirl is reduced by pipe wall friction [12] In this case,

$$\frac{\text{Swirl at turbine } (x = 1921 \text{ m})}{\text{Swirl at intake}}$$

$$= e^{-0.02(459.6/1.4)} e^{-0.02(1312/1.1)} = 5.8 \times 10^{-15} \qquad (5.10)$$

or virtually any swirl that enters the intake would be eliminated before the flow reaches the turbine. Thus, no hydraulic model would be recommended specifically duc to the possibility of intake vortices unless alterations in the field, should air entrainment vortices that could choke the flow occur, would be expensive.

The penstock diameters given by Eqs. (5.18) and (5.19) are 1.10 (3.3) and 1.15 m (3.45 ft), respectively, assuming a net head of 100 m (300 ft) (i.e., assuming 10 m (30 ft) of head loss). The initial specification of 1.1 m (3.3 ft) is in this range. This, however, should only be used as a starting point for the calculations. The head loss in the intake-tunnel-penstock system is given as

$$h_L = \left[k_i + \frac{f_t L_t}{D_t}\right] \frac{8Q^2}{\pi g D_t^4} + \left[\epsilon k_b + \frac{f_p L_p}{D_p}\right] \frac{8Q^2}{\pi g D_p^4}$$

where ϵk_b is the sum of bend loss coefficients and the subscripts t and p represent the tunnel and penstock, respectively.

As the penstock diameter is increased, head loss will be reduced and benefits of additional head are increased. The additional power from this additional head is then computed.

Based upon Eq. (5.1)

$$P(\text{kW}) = e \frac{\gamma(\text{N/m}^3)\, Q(\text{m}^3/\text{s})\, H_n(\text{m})}{1000} = 30 \text{ kW/m}$$

and assuming an efficiency of 0.85 and net head of 100 m, 1 m of head loss will result in approximately

$$\frac{P(\text{kW})}{H(\text{m})} = 0.85 \frac{(9800)(3.62)}{1000} = 30 \text{ kW/m}$$

of lost power. Then

$$\frac{\$2000}{\text{kW}} [30 \text{ kW/m}] = \$60{,}000 \text{ per meter of head loss}$$

is the equivalent cost of 1 m of head loss. This information will be used in head-loss computations below.

For water hammer, the relation

$$\frac{LV}{H_n} \cong \frac{2000 \text{ m } (3.80 \text{ m/s})}{100 \text{ m}} = 76 \text{ m/s}$$

which is much greater than the guideline of 3 m/s. Thus, water hammer needs to be considered further.

Pressure control, head losses, and the total intake-tunnel-penstock system cost were evaluated for four pipe diameters, 1.1, 1.2, 1.3, and 1.4 m, as summarized in Table 5.11. System construction costs were taken from those given in Chap. 6, equipment suppliers, and prior experience. The *effective penstock system cost* is the total cost of the penstock system minus any effective cost savings due to a reduction in head losses over the 1.1-m-diameter penstock. A menu-driven microcomputer program (SURGE) was used to evaluate three pressure-controlling devices and the water hammer pressures in the penstock system [56]. A simple surge tank located at the tunnel penstock junction, an air chamber located just upstream of the turbine, and a pressure-regulating valve located on the turbine scroll case were evaluated for a normal load rejection closure time of 20 s. The flywheel moment of inertia was already designed at a relatively high value.

The vapor pressure of water is at approximately −.32 ft of head, assuming atmospheric pressure is zero. Water column separation will occur if the pipe pressure approaches vapor pressure. Pressure at the junction is head −103.6. If this value is negative, there is vacuum in pipes at the junction. For all the options considered, the head at the junction never goes below 103.6 m, indicating that pressure is always positive in the system during normal load rejection. The reason for this is the relatively long governor closure time of 20.0 s during load rejection, versus the approximately 4 s it takes for a pressure wave to reach the reservoir and return to the turbine.

The options studied, the results generated, and the cost computations are given in Table 5.11. Figure 5.56 gives the result shown on the graphics screen for case no. 1a, 1.1-m-diameter penstock, without a pressure-controlling device. A maximum pressure rise of 99 percent of the net head requires a more sturdy and expensive penstock and tunnel lining to avoid rupture. The head and discharges for the best option studied, case no. 3b, 1.3-m diameter with a pressure-regulating valve (PRV), are shown in Fig. 5.57. The PRV placed at the turbine with typical lag time of 0.4 s and linear gate openings was very successful. Maximum pressure

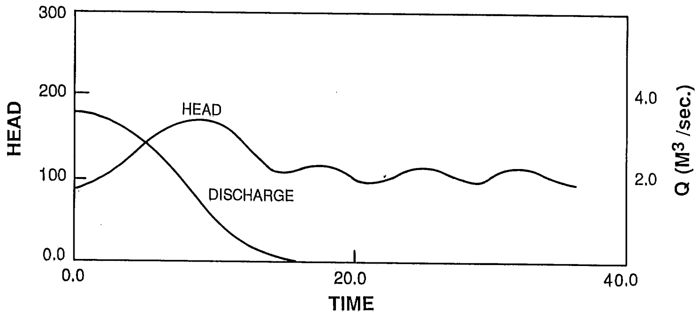


FIGURE 5.56 Graphic output of SURGE for case no. 1a, 1.1-m-diameter penstock without pressure regulating device. *(From Gulliver and Awate [56].)*

TABLE 5.11 Summary Results of Water Hammer Analysis for the Case Study*

Case no.	Penstock diameter (m)†	Pressure-controlling device	Design head (m)†	Maximum pressure		Rotational speed		Total cost penstock system, $ million	Effective cost penstock system $ million
				m	% increase	r/min	% increase		
1a	1.1	None	88.4	175.9	99.0	1225	36.1	2.80	2.80
1b	1.1	Pressure-regulating valve	88.4	111.8	26.5	1118	24.2	2.30	2.30
1c	1.1	Simple surge tank (area = 50 m^2)	88.4	157.0	77.6	1204	33.8	2.70	2.70
1d	1.1	Air chamber (area = 100 m^2) (air volume = 300 m^3)	88.4	132.1	49.5	1091	21.2	2.58	2.58
2a	1.2	None	95.1	166.5	75.1	1209	34.3	3.05	2.62
2b	1.2	Pressure-regulating valve	95.1	115.4	21.4	1119	24.3	2.61	2.18
2c	1.2	Simple surge tank (area = 50 m^2)	95.1	149.0	56.7	1187	31.9	2.96	2.53
2d	1.2	Air chamber (area = 100 m^2) (air volume = 300 m^3)	95.1	140.5	47.7	1096	21.8	2.93	2.50
3a	1.3	None	99.2	158.3	59.6	1191	32.3	3.27	2.57
3b	1.3	Pressure regulating valve	99.2	117.0	18.0	1114	23.8	2.88	2.18
3c	1.3	Simple surge tank (area = 10 m^2)	99.2	142.7	43.9	1168	29.8	3.13	2.43
3d	1.3	Air chamber. (area = 100 m^2) (air volume = 200 m^3)	99.2	147.9	49.1	1099	22.1	3.24	2.54
4a	1.4	None	101.7	151.8	49.2	1177	30.8	3.59	2.73

*Base year 1982 for costs.
†1 m = 3.27 ft.
Source: From Gulliver and Awate [56].

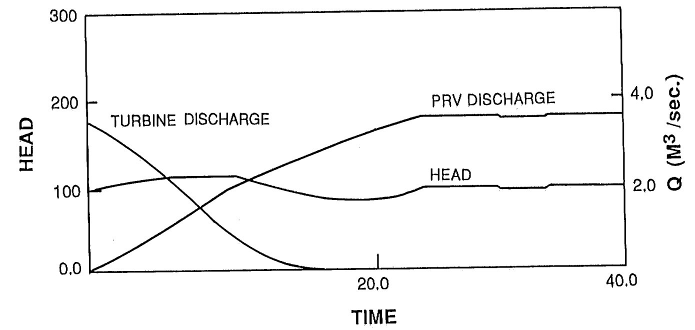


FIGURE 5.57 Graphic output of SURGE for case no. 3b, 1.3-m diameter penstock with a pressure-regulating valve. *(From Gulliver and Awate [56].)*

rise is 18 percent and increase in rotational speed is 23.8 percent over the steady-state condition. A simple surge tank located at the junction of tunnel and penstock resulted in a more costly installation. Surge tank location was chosen such that an extremely tall structure would not need to be built. However, because the location was not the best for pressure regulation, the penstock would need to withstand higher pressures, resulting in a more costly, thicker pipe wall.

For a relatively small hydro project, such as in this example, it can be cost effective to increase the pipe diameter. As the size of the hydroplant increases, the increase in pipe diameter is less beneficial. Water hammer computations should also be performed for other types of operations, such as start-up, an increase or decrease in power demand, and emergency situations. Often, wicket gates on a turbine will close automatically with a break in the linkage. This closure is so fast as to be virtually instantaneous, where Eq. (5.25) may be used to develop a conservative approximation of water hammer pressures,

$$\Delta H = \frac{-a\Delta V}{g} \tag{5.25}$$

For our example preliminary design, $a \cong 1400$ m/s, and

$$\Delta V \cong \frac{-(4)\ 3.6\ \mathrm{m^3/s}}{\pi(1.3\ \mathrm{m})^2} = -2.72\ \mathrm{m/s}$$

and

$$\Delta H \cong 388\ \mathrm{m}$$

which is a significant water hammer pressure. In addition, the return pressure wave from the reservoir may not be sufficiently damped to avoid column separation and the resultant penstock collapse. One option to deal with these emergency conditions would be to place a rupture membrane near the turbine for protection against high pressures and an aeration pipe at the tunnel-penstock junction for protection against low pressures. The rupture membrane could simply be a weak section of pipe, where the discharge would be routed around the powerhouse on a rock face or concrete apron.

5.9 CONCLUSIONS

Hydraulic conveyance facility design is an important aspect of hydropower facility development with significant implications on the design of other civil works. Many of the problems encountered at hydropower facilities are related to an inadequate hydraulic design.

One primary objective in designing hydraulic conveyance facilities is to minimize the sum of facility cost and value of power lost due to headlosses. Thus, headloss calculations are an important part of hydraulic design.

Proper operation is also an objective in hydraulic design. Two of the primary concerns in the operation of hydraulic conveyance facilities are intake vortices and water hammer. Other concerns include sediment scour and deposition, trash maintenance, and altered flow patterns (i.e., for navigation).

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CHAPTER 6

POWERHOUSE DESIGN AND SMALL HYDROPOWER PROJECT COST ESTIMATES

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6.1 BACKGROUND

The major equipment in a powerhouse is the turbine-generator unit. The remainder of the equipment serves to control, protect, and provide services to the main unit. The powerhouse provides support and housing for the turbine-generator unit and all auxiliary equipment. The powerhouse is located at the end of the waterway and as low as possible below the elevation of the reservoir water. This gives the maximum possible head for the turbine. Other factors which may influence the location of the powerhouse include foundation conditions, tailwater elevations, accessibility, and valley width. The intake structure and powerhouse for a low-head plant are usually combined into one structure.

The outline of the concrete; location of floors, walls, and hatches; and clearances between major equipment must be determined in consultation with the equipment supplier and the mechanical and electrical engineers associated with the project. Preparing and evaluating several trial layouts may be required before a final plan is determined. Equipment must be procured and the manufacturer's outline drawings must be obtained before the plans for construction can be finalized.

6.2 TYPES OF POWERHOUSES

The general types of powerhouses in use are indoor, semioutdoor, outdoor, and underground. The indoor powerhouse contains the turbine-generator unit and all auxiliary equipment under a roof. The building is of sufficient height to permit the handling of equipment by means of an indoor crane. In the semioutdoor plant, the generator floor is fully enclosed, but the equipment is handled by an outdoor gan-

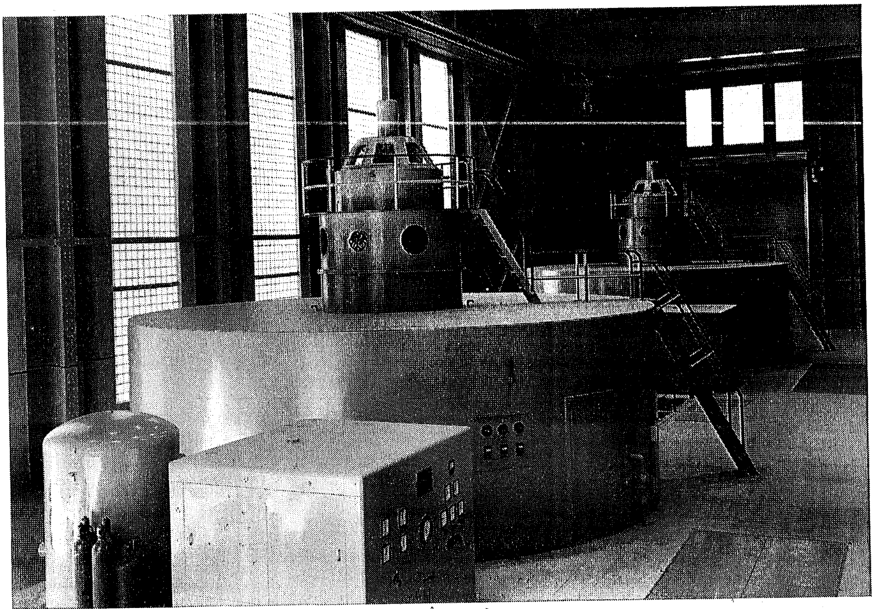


FIGURE 6.1 View of an indoor powerhouse at Cresta on the North Fork of the Feather River, California, two units totaling 67.5 MW. Equipment is handled by an indoor crane. (*Courtesy Pacific Gas and Electric Company*)

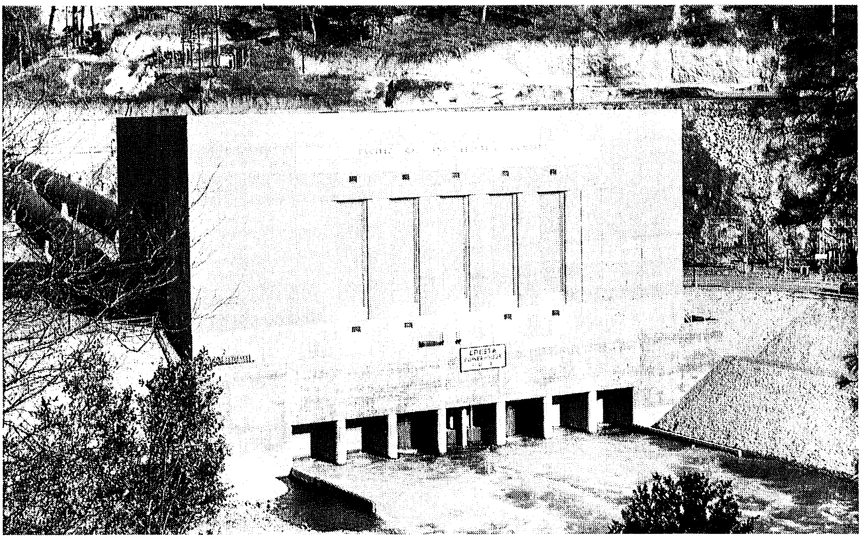
try crane or mobile crane through hatches in the roof. The outdoor powerhouse usually has the generator in a steel cylindrical housing set on top of the powerhouse structure that contains the turbine and powerhouse equipment. The cylindrical housing takes the place of a superstructure. The equipment can be handled by either a gantry crane or a mobile crane. Underground powerhouses are usually associated with large hydrogeneration units and pumped storage units. The rock shell is formed by the excavation from the walls and roof of the powerhouse.

The type of powerhouse selected is based upon an economic analysis which considers first costs and operation and maintenance costs. Figure 6.1 shows the interior view of an indoor powerhouse, Figure 6.2*a*, *b*, and *c* shows the exteriors of three types of powerhouses.

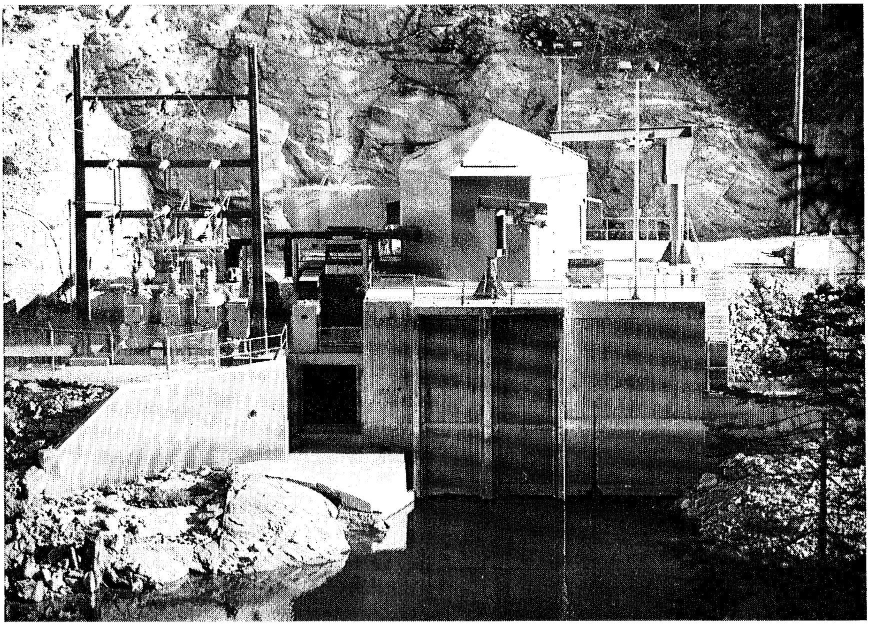
6.3 GENERAL ARRANGEMENT

The powerhouse arrangement is dependent upon the type of turbine-generator selected for the project. The selection of the turbine-generator is dependent upon the head available, the magnitude of the flows to be utilized, and the physical project arrangements best suited for those particular site conditions. Several different arrangements are tried to determine the most economical layout.

In multiunit plants, the hydroelectric generating units are almost always placed in a single row approximately normal to the direction of flow to the powerhouse. This arrangement facilitates the design of the water passages and the handling of equipment by a crane. An erection bay is usually provided for

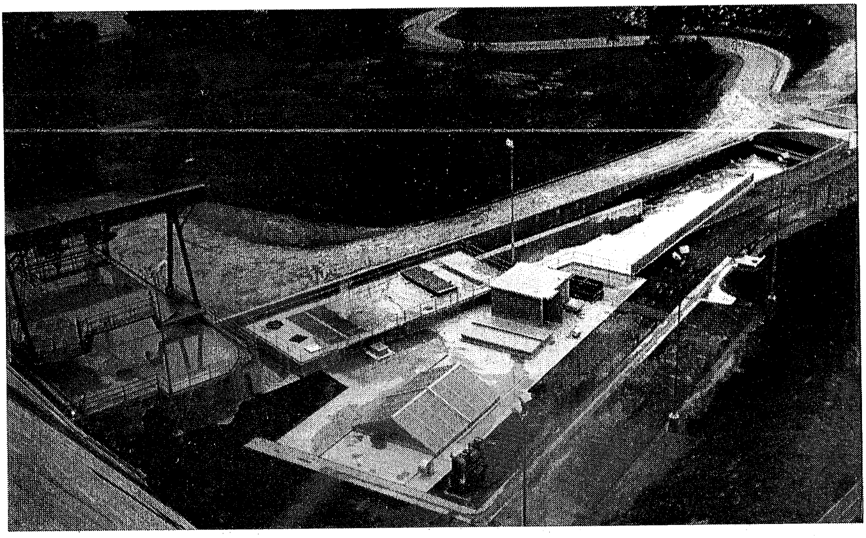


(a)



(b)

FIGURE 6.2 (*a*) Exterior view of the Cresta Powerhouse. (*b*) View of the outdoor powerhouse (Sly Creek) at Lost Creek, a tributary of the South Fork Feather River, California. The generator is located in the octagonal steel housing on top of the powerhouse.



(c)

Figure 6.2 (*c*) Exterior view of a semi-outdoor powerhouse (Madera) at Friant Dam, San Joaquin River, California. Equipment is handled by an outdoor crane through hatches in the roof.

multiunit plants at one end of the structure to provide space for the equipment to be unloaded or repaired. The spacing of the units is governed by the width of the draft tube. However, clearances between the units and major pieces of support equipment should be at least 4 to 5 ft (1.3 to 1.6 m). For multiunit plants an aisle at least 8 ft (2.6 m) wide from one end of the powerhouse to the other should be provided.

6.4 INTERIOR ARRANGEMENT

The space within a powerhouse is occupied by turbine, draft tube, generator, governor, electrical equipment, and mechanical equipment. The space needs to be carefully planned so that the auxiliary equipment is located near the main equipment with which it is associated. There should be adequate clearance to provide convenient working space for installation and maintenance. For the smaller turbine-generator unit, erection areas and overhead cranes are normally omitted. Some equipment, such as control room, battery area, and motors that operate dewatering pumps, should be located above maximum tailwater.

If the powerhouse is to be attended, a soundproof control room will be required. The control room should be planned for convenience in operation and maintenance of the equipment. Items to be included in the control room should be operator desks, control consoles, drawing cases, plus a small kitchen, supply room, and clothes closet. Toilet and wash facilities for control room operators must also be provided, directly accessed from the control room, if possible.

Hatches for removing equipment at various levels are required for maintenance. A determination should be made of the cost of providing a crane large enough to remove the largest piece of equipment compared to the cost and frequency of renting a crane. Smaller lifting equipment, such as a jib crane, needs to be provided to lift draft tube gates and supplies for operation and maintenance. Possibly one jib crane could perform both functions. Figure 6.3 has been prepared to show a typical layout of equipment and allocation of space for a two-unit high-head impulse turbine powerhouse.

The following is a list of auxiliary equipment and systems usually required for a powerhouse.

Electrical systems:

- Main control board
- Generator excitation equipment
- Station service switchgear
- Storage battery and charger
- Generator voltage switchgear, buses, and surge equipment
- Generator neutral grounding equipment
- Lighting
- Cable trays
- Grounding mat

Mechanical systems:

- Turbine governing equipment
- Water supply system for cooling and service
- Lubricating oil and/or hydraulic oil
- Heating, ventilating, and air conditioning
- Compressed air
- CO_2 fire protection
- Dewatering, building, and sanitary drainage
- Lifting
- Turbine shutoff valve
- Draft tube gates

Switchyard:

- Transformer
- Circuit breakers
- Takeoff structure

6.5 SUBSTRUCTURE DESIGN

The powerhouse substructure supports the turbine and generators, as well as the superstructure. The substructure contains water passages, rooms, and galleries, and furnishes most of the mass needed for stability. It is important that the structure rest on sound material, unweathered and unshattered by blasting, to develop full resistance to sliding. Sound rock foundations may contain seams of clay or other unsuitable material, which must be removed and filled with concrete, or areas of broken rock which must be consolidated by pressure grouting.

The lowest feature of the substructure is the draft tube. It is usually designed of metal and provided by the turbine manufacturer. When it is constructed of concrete, the neat lines will also be provided by the turbine manufacturer. The draft tube portion of the substructure should be designed to withstand all loads

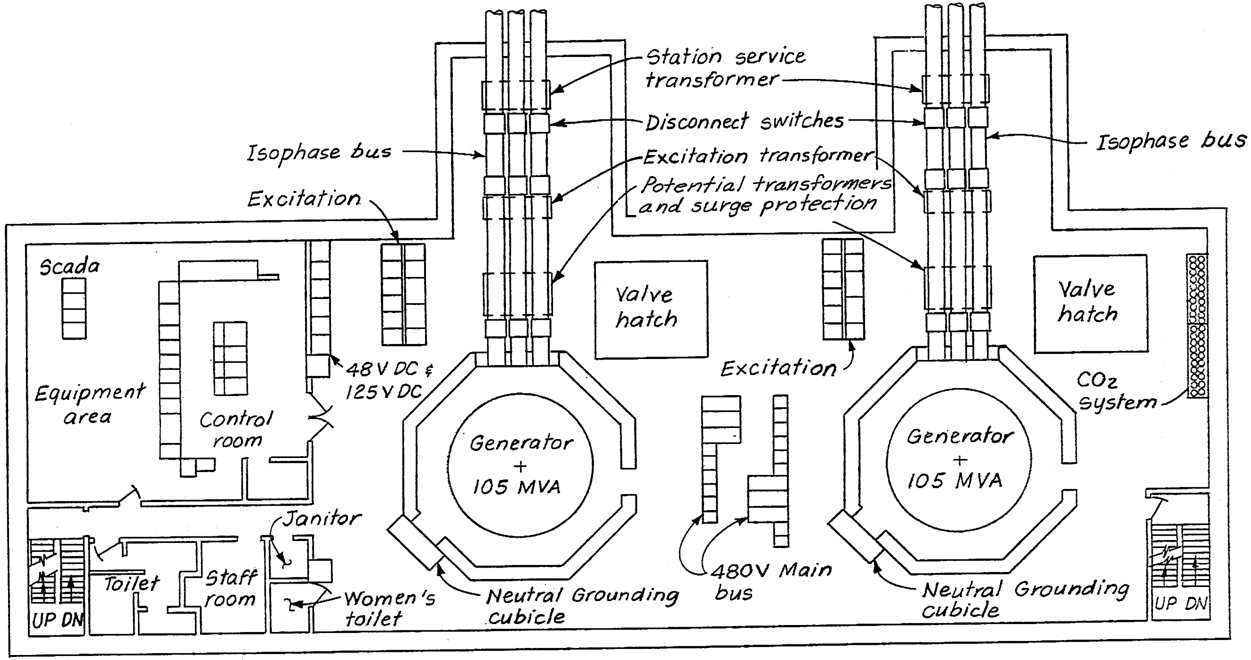


FIGURE 6.3 Typical layout of equipment and allocation of space, two-unit high-head impulse turbine.

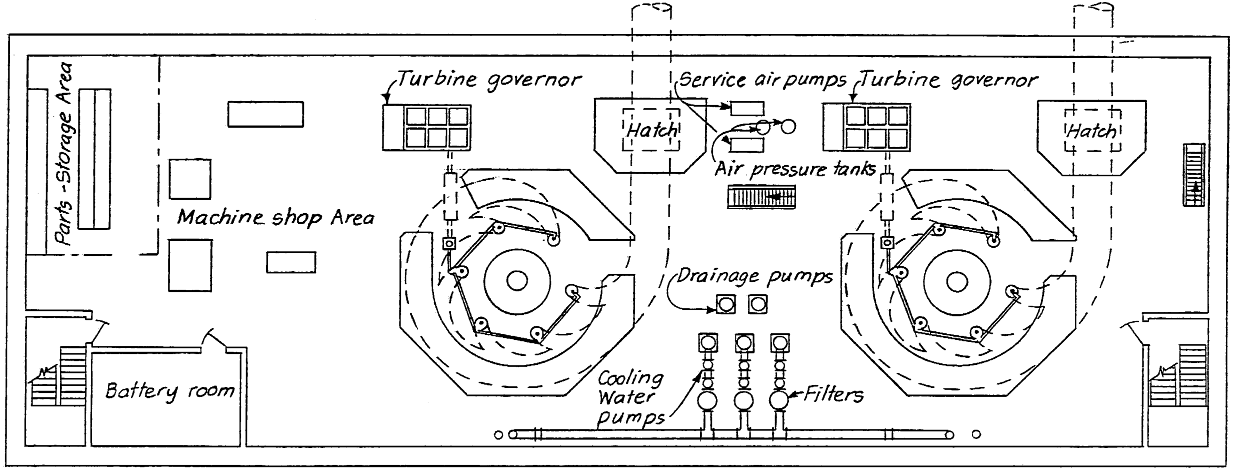


FIGURE 6.3 (*Continued*)

imposed upon it. Uplift under the floor of the draft tube should be considered in the design when relief drains under the structure are not provided.

The scroll case, or spiral case, carries water from the penstock, through the guide vanes and wicket gates, to the turbine. The scroll cases are designed to withstand the maximum water pressure plus water hammer. They are primarily constructed of metal and designed and furnished by the turbine manufacturer. Where concrete scroll cases are proposed, neat lines of concrete will be furnished by the turbine manufacturer.

Vertical-shaft turbines generally have their scroll case embedded in concrete. If the steel scroll case is embedded in concrete in an unwatered condition, care needs to be taken in the concrete preparation so that the scroll case will resist internal pressure by ring tension, and the force not be transmitted into the surrounding concrete. Reinforcing bars are often provided to resist any tensile stress in the surrounding concrete. Mastic cork filler can be provided in a nominal blanket over the area being concreted for larger units. It may not be practical for smaller units, given the shape of the spiral case and stiffeners on the steel exterior.

Where scroll cases are embedded in concrete under a pressurized condition, neither blanket nor drains are required. However, drains are sometimes provided to drain any water which may migrate through the concrete exterior and gather between the steel plate and concrete when they are unwatered. As a part of the construction procedure, it is recommended that field pressure tests on the scroll case be performed. Grouting preplaced aggregate has been used to surround spiral casings; in this method, large-size aggregate is deposited around the spiral casing, and colloidally mixed concrete and sand is injected. The advantage is that shrinkage is reduced because the aggregate particles have positive point contact.

6.6 CONSTRUCTION DETAILS

Joints

The purpose of joints is to facilitate construction, prevent unsightly cracks, and reduce the transmission of stress from one portion of the structure to another. Joints may be categorized as contraction or construction. Reinforcing steel should not cross contraction joints, but may continue across construction joints. Contraction joints are used to divide the structure into separate monoliths, mainly to reduce cracking from shrinkage due to cooling and settling of the concrete. Construction joints are required for dividing the structure into component working units for concrete placement. The joints should be located and designed so as not to affect the continuity of the structure. They should be placed to minimize cracking caused by the initial shrinkage of the concrete pour. Reinforcing steel should continue across the joint, and provisions should be made for transient shear. The basic lift heights should be as follows: 5 ft (1.6 m) for mass concrete pours, 10 ft (3.3 m) for piers and walls in excess of 18 in (45 cm) thick, and 5 ft (1.6 m) for walls under 18 in (45 cm) thick.

Waterstops and Waterproofing

Waterstops across contraction joints are necessary to prevent leakage and to obtain dry working conditions. Waterstops are also recommended for most construction joints that appear below the level of high tailwater on the power plant—also to prevent leakage. Extruded polyvinylchloride (PVC) and molded rubber have proved to be highly practical and durable materials for waterstops. Waterstops should be placed as close to the surface as possible to create a continuous barrier about the plane to be protected. All laps or joints should be completely cemented together.

The outside concrete surfaces of the powerhouse, including exposed faces of the draft tube, need to be waterproofed to the level of the highest tailwater, as does any surface in contact with earth backfill. The material should be nonstaining and crystal-forming waterproofing.

6.7 SUPERSTRUCTURE DESIGN

The superstructure of a powerhouse may be one of three types: indoor, semioutdoor, or outdoor. The indoor type completely encloses the generators and the erection bay and has an inside crane runway supported by the walls of the structure. The semioutdoor type consists of a continuously reinforced slab over generators and erection bays supported by heavy transverse walls. Sliding hatches in the roof over the generators provide access for handling equipment with an outdoor gantry crane. In the outdoor type, each generator is protected by a light steel housing. This is removed by the outdoor gantry crane when access to the machine is necessary for other than routine maintenance.

Choice of superstructure type should be dictated by consideration of first cost, with all equipment in place, cost of maintenance, building the equipment, and protection from the elements. For the semioutdoor and outdoor powerhouses, the designer carries out the design of the substructure and superstructure as one structure.

Indoor Design

The frame of the superstructure on an indoor powerhouse may be structural steel, reinforced concrete, or a combination of both, depending upon the economy. Concrete walls are needed below the elevation of maximum tailwater. Concrete exterior walls should be of massive uniform thickness rather than column and spandrel wall construction. The heavy walls are easier to build and should be designed to withstand stresses from all possible loading and to consider space requirements for embedded items. Openings in the walls should be confined to necessary access doors and to ventilation louvers or small windows above the crane rails. Wall pours should not be more than 10 ft (3.3 m) in height. Horizontal rustications should be used at the pour joints on the exterior side where practicable to prevent unsightly spalling. Locations of joints and pour heights will therefore depend partly on the exterior architecture. Contraction joints in the same vertical plane as those in the substructure should be provided in the superstructure walls.

While self-supporting concrete walls are usually preferred in the generator bays, their justification is dependent upon economy and tailwater limitations. Where economically feasible, steel framing with curtain walls or insulated panels is used where the generator floor is above maximum tailwater. The use of steel framing may, in some cases, be desirable to permit the early installation and use of the crane runways and cranes. This advantage should be evaluated, however, only in terms of overall cost. The steel framing of each monolith should be a separate unit, with no steel crossing the contraction joints except the crane rails.

With the exception of the generator room and erection bay, all floor systems should be made of reinforced concrete with 6-in (15-cm) minimum thickness supported on structural steel beams, or should be of reinforced concrete beam and slab construction. The thickness of the structural slab will, in many cases, be determined by the number and size of electrical conduits it must encase. Separate concrete fill placed on top of the structural slabs to encase conduits is uneconomical, but can be used where large numbers of conduits must be accommodated, or where reinforcement in the structural slab is closely spaced. Flush sockets should be provided at hatchways through floors for the installation of temporary railings for protecting personnel when the covers are removed.

The roof framing for the powerhouse will usually consist of steel girders or rigid frame bents supporting purlins which, in turn, support the roof slab or deck. The upper surface of the roof slab or deck should be shaped to provide drainage slopes. Insulation, embedded in hot bitumen, should be applied over a vapor-sealed course to the roof slab or deck.

6.8 POWERHOUSE DETAILS

There are rules governing the design of certain powerhouse features related to the safety of operators and workers. The rules pertain to fire doors; escape hatches; details of stairs, ladders, walkways, railings, battery rooms, etc.; and must be included in the design. All small pieces of equipment should be placed on concrete pedestals about 6-in (15-cm) high to prevent possible water damage, and to improve appearance. A gutter should be provided along the floor and outside walls to intercept any seepage through the wall and carry it to a sump. Hooks should be embedded in concrete walls and roofs to facilitate the movement of equipment in the powerhouse.

6.9 DESIGN LOADS

The powerhouse structure should be designed for the maximum dead, live, hydrostatic, wind and earthquake loads which may be imposed. Typical loads used in powerhouse design are shown in Table 6.1. These loads should be modified for specific layouts, sites and localities.

Dead loads considered in the design consist of the weight of all permanent construction and fixed equipment. Live loads on the structure may take the form of a uniform load per square foot of floor, or a moving concentrated load of truck wheels. The live load may include the load caused by a vehicle or crane, wind levels, construction loads, earthquake loads, and water loads, snow loads, or loads due to generator short circuit torque.

TABLE 6.1 Typical Powerhouse Structural Design Criteria

Powerhouse structure—design loads		
Dead loads	lb/ft^3	kg/m^3
Concrete	150	2,400
Steel	490	7,860
Water	62.4	1,000
Live loads	lb/ft^2	kg/m^2
Turbine floor	500	2,450
Generator floor	500	2,450
Other floors	200	980
Stairways and walkways	100	490
Loads imposed by specific items of equipment or temporary construction equipment.		
Impact load		
Moving live loads will be increased by 20% to account for impact loads.		
Hydrostatic pressure		
Powerhouse will be designed for full hydrostatic pressure from maximum spillway discharge or maximum tailwater.		
Uplift pressure applied to 100% of the foundation area.		
Wind load	lb/ft^2	kg/m^2
Vertical surfaces	20	98
Cylindrical surfaces	16	78
Towers & supported equipment, elevators	44	215
Earthquake load		
Buildings, structures, and equipment will be designed in accordance with the requirements of the Uniform Building Code for a horizontal and vertical acceleration of 0.10g.		
Snow load		
100 lb/ft^2 (490 kg/m^2) of horizontal surface.		

6.10 STABILITY ANALYSIS

A stability analysis should be performed for each level of the powerhouse. The analysis should include the following features: turbine valve open; turbine valve closed; water hammer; minimum, normal, and maximum tailwater; spiral case and/or draft tube full and empty; uplift and no uplift; wind and/or earthquake forces; and partially constructed structures. The vertical forces include the dead weight of the structure, fixed equipment weights, and weights, if any, of earth, water, and uplift. The horizontal forces include headwater, tailwater, earth, wind, and earthquake. Forces should be considered from the pressure of water in the penstock and valve upon closure [1, 2]. Headwater pressures should be included in the case of type D layouts. Foundation bearing pressure should be checked during the stability analysis calculations.

The U.S. Bureau of Reclamation, in its publication on powerhouse structural design [2], has an excellent checklist of 36 forces for stability analysis. Some of the powerhouse. The largest piece of equipment is usually the generator stator, which can be delivered in one-half or one-third sections to reduce the size of the tunnel. The grade of the access tunnel must permit vehicular traffic. The location of the transformers, coupled with the length of generator bus to the transformers, needs to be carefully planned. The housing of transformers underground requires special provisions to prevent fire from occurring in the main powerhouse chamber. Transformers located outside the powerhouse greatly lengthen the generator bus and increase electrical line losses between generator and transformer.

There are also special geological considerations associated with underground powerhouses. The location of the powerhouse should be thoroughly explored with drill holes and exploration tunnels to gain the most favorable rock conditions. In order to expedite construction of powerhouse, penstock, and tailrace tunnels, several access tunnels may be warranted. As excavation proceeds, rock bolts are utilized for rock support and reinforcement for walls and ceiling area. The purpose of rock bolting is to reinforce a mass of jointed rock into a structural entity capable of providing its own permanent support system across the excavation. Rock bolts are sometimes supplemented by chain-link fabric and then covered with gunite. In recent years, reinforced gunite, that is, gunite with small needle-shaped steel fibers, has been used successfully to support rock surfaces. After completion of rock bolting in the roof of the cavern, a suspended ceiling is often placed to intercept any water which might drip from the roof. The seepage is collected in a wet sump and pumped out of the powerhouse.

Cranes are usually installed as soon as possible after completion of the excavation to facilitate the remainder of the powerhouse construction. Crane beams are installed and supported on steel columns set on foundations independent of other power-plant structures. Some designers attach the crane rails to a bench in the excavated wall of the powerhouse, and the bench is reinforced with rock bolts.

After completion of the excavation, the structural design of the powerhouse proceeds similarly to a surface powerhouse. Since the powerhouses are often located far below maximum tailwater, more attention is given to emergency closure gates and sump water pumping to prevent accidental flooding of the powerhouse.

Underground powerhouses are economical in pumped storage design. These advantages result in shorter high-pressure conduits, deeper setting of the turbine and pump units, and use of units of higher speed and smaller physical size. Figure 6.5 is an interior photo of the underground powerhouse at the Helms pumped storage plant in California.

6.12 PROCEDURE FOR DEVELOPING A COST ESTIMATE

A detailed procedure and cost curves have been developed to obtain a preliminary cost estimate for a small hydroelectric project. To proceed, the project type should be determined and the basic components of the project defined (see Table 6.3). A key element to be determined is the type of turbine to be used. Figure 4.37 in Chap. 4 can be used as a guide in selecting the turbine type suitable for use with the design flow and head.

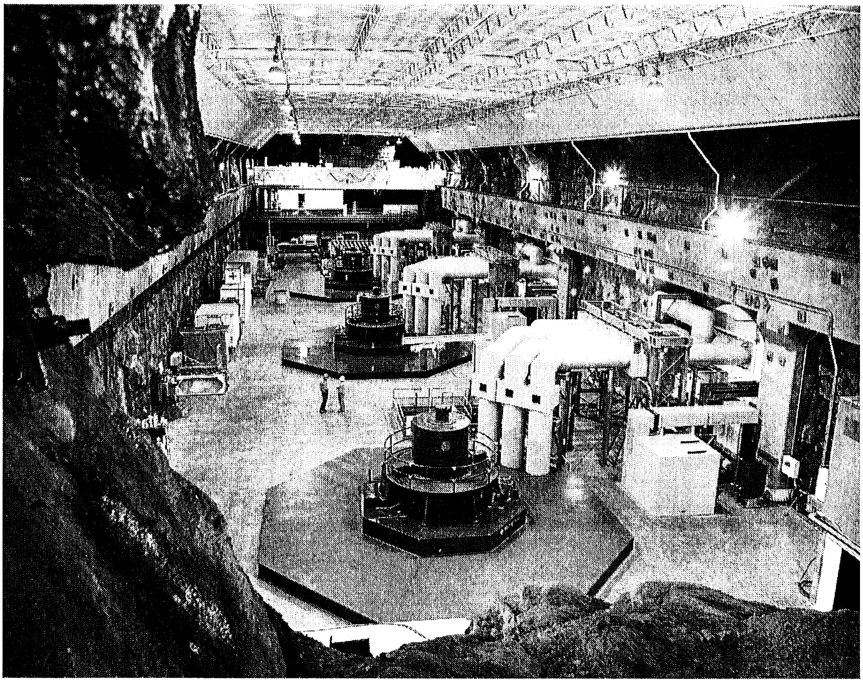


FIGURE 6.5 Helms pumped storage power plant located on North Fork of Kings River, California, three units, each producing 400 MW. (*Courtesy Pacific Gas and Electric Company*)

TABLE 6.3 Project Data Sheet

Project:__________ Date:__________

Description	Unit	Project data	Remarks
Location:		______	______
Cost zone coefficient		______	______
Project type (A,B,C,D)		______	______
Average annual flow	______	______	______
General items:			
Land required:			
For power plant	______	______	______
For reservoir	______	______	______
For transmission line	______	______	______
Access road required:			
Permanent	______	______	______
Construction	______	______	______
For transmission line	______	______	______
Facility to be relocated:		______	______
Care of water–condition:		______	______

TABLE 6.3 Project Data Sheet *(Continued)*

Project:______________________ Date:____________

Description	Unit	Project data	Remarks
Head data:			
Headwater elevation			
Maximum	______	______	______
Minimum	______	______	______
Tailwater elevation at design discharge	______	______	______
Gross head:			
Maximum	______	______	______
Minimum	______	______	______
Design	______	______	______
Design net head	______	______	______
Turbine			
Type	______	______	______
Design discharge	______	______	______
Unit capacity	______	______	______
Waterways:			
Intake structure required	No	______	______
Penstock:			
Length:			
main	______	______	______
branch	______	______	______
Bifurcation required	No	______	______
Maximum velocity	______	______	______
Diameter	______	______	______
Bypass structure required	No	______	______
Bypass discharge	______	______	______
Tailrace length	______	______	______
Transmission line:			
Voltage	kV	______	______
Capacity	MW	______	______
Length	______	______	______
New site developments:			
Embankment dam:			
Height W.S. above streambed	______	______	______
Length	______	______	______
Spillway:			
Region	______	______	______
Drainage area	______	______	______
Existing facilities:			
Degree of difficulty of alteration	______	______	
Remedial work required	______	______	

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For the purpose of reference in the cost-estimating section, the most common types of small hydroelectric development have been classified for use as described below. These types are illustrated in Fig. 6.6.

Type A—Canal Drop Layout

This type of development will usually occur where an existing canal contains a drop structure dissipating the head. Operation of the canal hydroelectric system will normally require some form of coordinated control over the canal intake gates in order to meet both the hydroelectric and the previously existing water demands. Canal spillways or a synchronous bypass may also be needed to handle the flow when there is a power-plant load rejection. A penstock may require a butterfly valve just upstream of the unit and a slide gate in the intake structure. Multiple units operating from one penstock require both the intake slide gate and the butterfly valve for each unit to provide adequate security and to ensure that energy can be generated when one unit is down for maintenance.

Type B—Concrete Dam Layout

A hydropower development of this type may be constructed downstream of a concrete arch, gravity, or buttress dam. Where the dam already exists, the cost of development will normally be reduced by utilizing an existing outlet works conduit as part of the penstock. Connection to an existing outlet works conduit can usually be completed without incurring major costs, provided an alternate outlet is available for making releases during construction and maintenance.

Type C—Embankment Dam Layout

This type of development is similar to type B in that utilization is made of an outlet works conduit. Many existing earthfill and rockfill dam outlet works, however, have the outlet regulating valve located at the intake or near the centerline of the dam cross section, with the downstream section of the conduit unpressurized and free-flowing. This arrangement requires a new section of penstock to be placed inside the existing conduit downstream of the regulating valve. Construction of the powerhouse is similar to that required for type B. An alternative means of making releases must be provided during construction and maintenance.

Type D—Weir-Type Layout

A development of this type will usually require the construction of the combined dam and powerhouse structure. A spillway structure will also be required unless some form of spillway already exists. A major item in the construction of this type of plant is the cofferdam and dewatering of the foundation. It may be that for small plants some form of prefabrication may be both feasible and economical.

Cost-Estimating Technique

Cost estimates are required at all stages of project development. They vary only in the amount of data available and detail in which they are carried out. Estimates

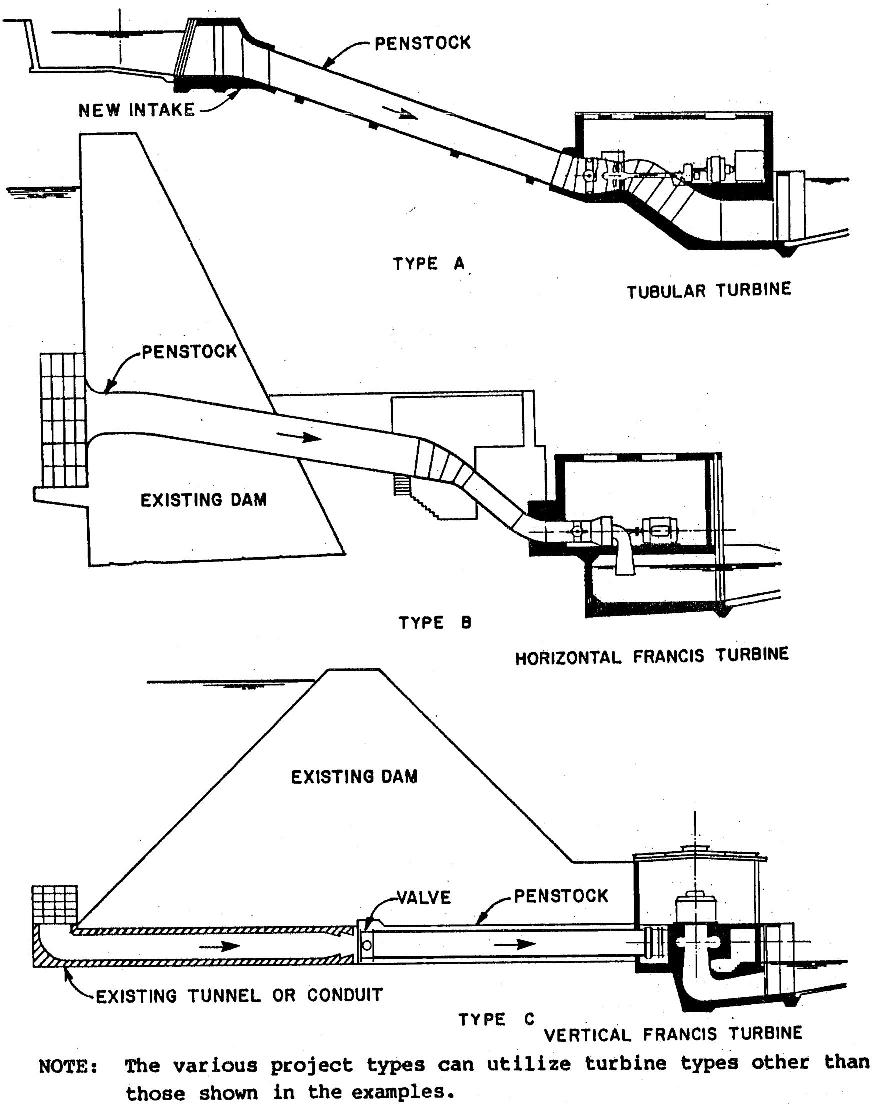
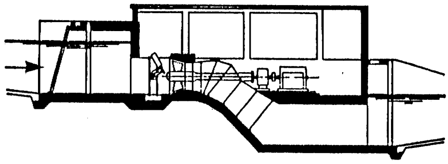
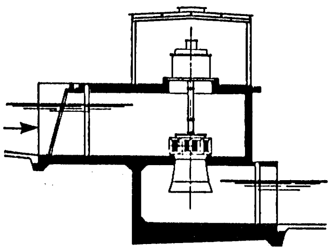


FIGURE 6.6 Standard project types. (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

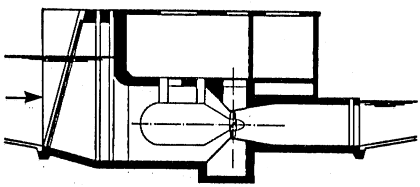
made at the level of a feasibility study and beyond usually are based on a quantity takeoff using plans drawn up to describe the project, and unit prices appropriate for each quantity item. For estimates of the type required for preliminary analysis, costs can be based on generalized statistical data. This is the approach used in this chapter. All costs are given in U.S. dollars. The magnitude of various costs will vary by country. However, the general trend versus plant capacity and head will hold true.



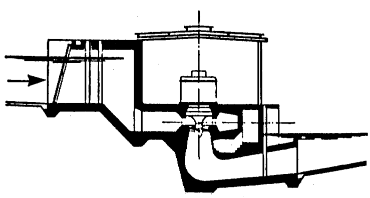
TUBULAR TURBINE



OPEN FLUME TURBINE



BULB TURBINE



VERTICAL PROPELLER TURBINE

TYPE D

FIGURE 6.6 *(Continued)*

Cost curves were prepared using data obtained from previous projects, manufacturers' data, and from quantity–unit price estimates of typical layouts prepared to cover the required range of projects. The procedure and cost curves were originally prepared by Hugh Brown, Chief Engineer, Tudor Engineering Company, for an Electric Power Research Institute Report entitled "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites" [3]. Permission was granted by EPRI for use of the materials. The cost curves have been escalated to a base period datum of January 1987 and revised from the recent actual construction costs of projects.

The curves are expressed in terms of three basic parameters: power-plant capacity (kilowatts), turbine design head, and the turbine design discharge. Other dimensions, depending on the physical arrangement, may also be necessary.

The construction cost estimate comprises direct costs and indirect costs. For purposes of estimating the direct costs the project is divided into the following components, some of which may not be required in specific layouts.

General:

- Land and land rights
- Relocations
- Mobilization
- Care of water
- Access roads

New dams:

- Dam
- Spillway

Existing facilities:

- Remedial work
- Connection to or modification of existing facilities

Power-plant addition:

- Approach channel
- Intake and equipment
- Penstock or canal
- Powerhouse
  - —Turbine, generator & governors
  - —Powerhouse
  - —Powerhouse accessory electrical equipment
- Bypass structure
- Tailrace
- Switchyard

Transmission line

The total of the costs of these items is referred to herein as the *direct cost*. To this cost is added the indirect costs, which are amounts to cover the costs of contingencies, engineering, administration, and construction management, the total being referred to as the *construction cost*. Further addition of an amount for interest accrued during construction gives the grand total, referred to as the *capital cost*.

The general procedure to obtain an estimate of the direct costs is first to list the components of the proposed project (see Table 6.3) and then to obtain the cost of each item using Tables 6.4 and 6.5. The costs can then be compiled on a form as shown in Table 6.6. An explanation of the tables or other procedures for obtaining costs appears on the tables. For addition of a power plant at existing facilities, it may be noted that the major portion of the cost is for mechanical and electrical equipment. The proportion of the costs of each of the project components is illustrated in Fig. 6.7. For this reason and because of the variety of turbine types available, the cost of the power plant and equipment has been prepared in greater detail than for the other items. Also described in this chapter are the methods for obtaining the indirect costs, escalation, and the recurring annual costs.

Table 6.6, prepared to facilitate compilation of the costs, lists the various components of the project. Costs can be applied to the appropriate items depending on the makeup of the particular project, and totaled in Table 6.7 to obtain the final project construction costs.

6.13 GENERAL COSTS

General

Items in this category are essentially site-specific and have to be identified separately for each project. These items are

- Land and land rights
- Water rights
- Relocations
- Mobilization
- Access roads and bridges
- Care of water

Some items will frequently not apply or be of small cost significance, especially for additions to existing facilities. The items are discussed herein.

Land and Land Rights

This item includes the costs of acquisition or use of project lands. The cost is obtained as the number of acres of land to be acquired at the price of land per

TABLE 6.4 Generalized Unit Prices (January 1987 U.S. $)

Item	Price, USCS units	Price, SI units
Land clearing:		
Light	\$2000/acre	\$810/hectare
Heavy	\$5000/acre	\$2025/hectare
Earthwork:		
Earth excavation	\$5.00/yd^3	\$6.55/m^3
Rock excavation	\$25.00/yd^3	\$32.75/m^3
Backfill	\$10.00/yd^3	\$13.10/m^3
Concrete, moderately reinforced	\$375/yd^3	\$490/m^3
Penstock steel	\$1.50/lb	\$3.30/kg
Access road:		
Single-lane, paved, new	\$145,000/mi	\$90,000/km
Two-lane, paved, new	\$295,000/mi	\$185,000/km
Single-lane, unpaved, new	\$90,000/mi	\$56,000/km
Existing road, upgrading	\$50,000/mi	\$31,000/km
Bridge:		
Prestressed I-girder type, new	\$75/ft^2	\$7.00/m^2

TABLE 6.5 Construction Cost Indexes of the Bureau of Reclamation, January 1987

Composite index	1.60
Dams	
Earth	1.41
Structures	1.27
Spillway	1.51
Outlet works	1.61
Concrete	1.60
Steel penstocks	1.65
Canals	1.50
Earthwork	1.49
Structures	1.54
Conduits (tunnels, concrete-lined)	1.70
Power plants, hydroelectric	
Building and equipment	1.66
Structure, reinforced concrete, and improvements	1.55
Equipment	1.72
Turbine and generators	1.74
Accessory electrical and miscellaneous equipment	1.63
Pipeline, concrete	1.64
Switchyards	1.60
Transmission lines	
Wood poles, 115 kV	1.47
Steel tower, 230 kV	1.70
Roads	
Secondary	1.81
Bridges, steel	1.64

Base: 1977 = 1.0

acre at the project site. Since land costs vary considerably according to location, an approximate estimate of the price of the land should be obtained from local sources.

Water Rights

The cost of rights for use of water to develop power will often be of little significance since power use is nonconsumptive, unless there is a diversion of water from the drainage area. It is not possible to provide generalized costs for this item suitable for use in preliminary estimates. The main purpose for its inclusion herein is to identify an item which may affect the project cost and which will have to be defined subsequently, if the project appears to be attractive.

Relocation

Included in this item would be the cost of relocation or replacement of facilities such as buildings, roads, and bridges affected by the project. For addi-

TABLE 6.6 Cost Compilation Sheet

Description	Unit	Quantity	Unit cost	Amount January 1987	Cost indexes 1987	Cost indexes 19( )	Escalated amount 19( )
1. New dam and spillway:							
Embankment dam							
Spillway							
Reservoir clearing							
Subtotal							
2. Intake:							
Intake, Type A							
Intake gate & hoist	No						
Subtotal							
3. Penstock:							
Penstock							
Penstock branch							
Bifurcation	No						
Subtotal							

TABLE 6.6 Cost Compilation Sheet *(Continued)*

Description	Unit	Quantity	Unit cost	Amount January 1987	Cost indexes 1987	Cost indexes 19( )	Escalated amount 19( )
4. Power plant (type A, B, C, and D projects)							
Total cost, single unit	LS	____	____	____	____	____	____
Equipment adjustments:							
For governor	LS	____	____	____	____	____	____
For fixed blade/mounting	LS	____	____	____	____	____	____
For inlet valve	LS	____	____	____	____	____	____
For spiral case	LS	____	____	____	____	____	____
Subtotal unit							
Bypass structure	LS	____	____	____	____	____	____
Tailrace	LF	____	____	____	____	____	____
Switchyard		____	____	____	____	____	____
Civil features	LS	____	____	____	____	____	____
Equipment	LS	____	____	____	____	____	____
Subtotal							
5. Special costs:							
		____	____	____	____	____	____
		____	____	____	____	____	____
		____	____	____	____	____	____
Subtotal							
Subtotal—Items 1 through 5							

TABLE 6.6 Cost Compilation Sheet *(Continued)*

Project cost summary	
Item	Base date: /19( ) Costs
1. New dam and spillways	________
2. Intake	________
3. Penstock	________
4. Power plant	________
5. Special structures	________
6. Existing facilities	________
7. General costs	________
8. Transmission line	________
9. *Total direct cost*	________
10. Contingencies (20%)	________
11. Subtotal	________
12. Engineering, administration and construction management (20%)	________
13. *Total construction cost*:	________

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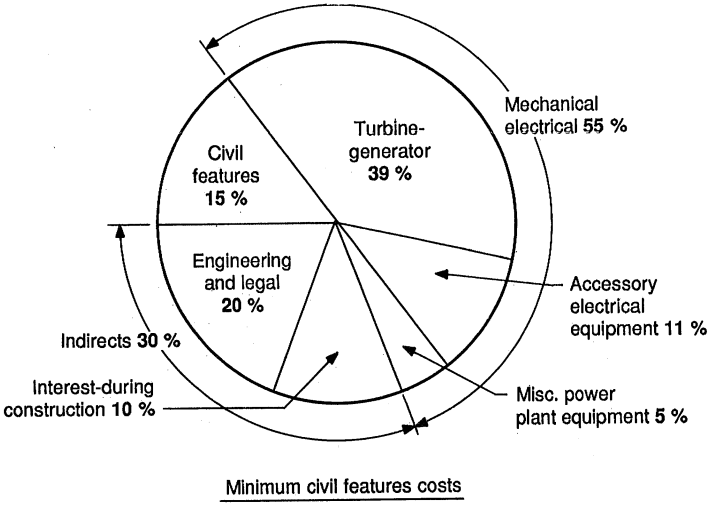
tions to existing facilities, the cost of this item should be small. For construction of a new reservoir or at a new site, the cost may be significant. Where significant, an approximate estimate should be made depending on the nature of the items to be relocated. Where a reasonable estimate cannot easily be made, a judgment may be made as to the possible cost, and the value reviewed against the results of the financial or economical analysis as a special study to be performed.

Mobilization

The cost of mobilization of the contractor is sometimes included in contract rates as a separate item and sometimes it is merged in the cost of the other items. In this chapter, a separate allowance is made for mobilization which can be estimated as 1.5 percent of the direct cost exclusive of general costs. At isolated sites or locations difficult to access, an additional allowance should be added.

Access Roads and Bridges

The estimated construction costs per mile of the new paved access road, single- or double-lane, is shown in Table 6.4. This cost is based on a 2-in (5-cm) asphalt



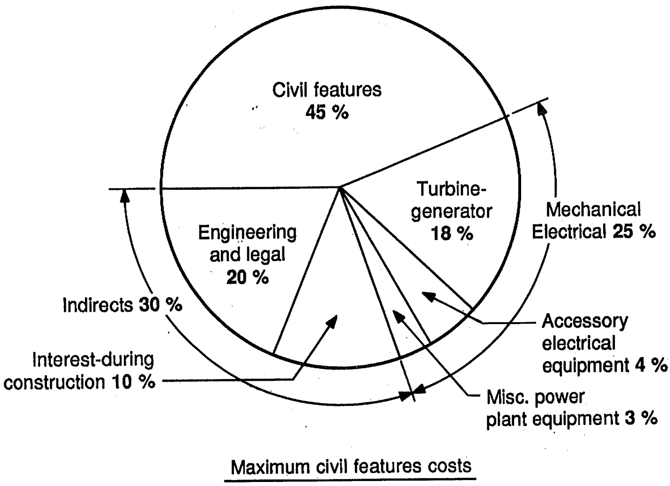


FIGURE 6.7 Approximate proportions of project capital costs. (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

concrete pavement, a 4-in (10-cm) subbase, and a 4-in (10-cm) base. Costs for a single-lane unpaved road and for upgrading an existing road are also given in Table 6.4.

For approximate costs of bridging, the cost per square foot of a single span access road bridge constructed of standard prestressed I-girders is given in Table 6.4. This cost includes excavation, substructure on piles, and superstructure.

TABLE 6.7 Example of Estimate Project Data Sheet

Project: Power plant at earthfill dam — Date: July 1987

Description	Unit	Project data	Remarks
Location:			
Cost zone coefficient		1.0	
Project type (A,B,C,D)		C	
Average annual flow		—	
General items			
Land required:			
For power plant	acre	1.0	
For reservoir		—	
For transmission line	acre	20.0	
Access road required:			
Permanent		—	
Construction	mile	0.20	
For transmission line		—	
Facility to be relocated:		None	
Care of water—condition:		Moderate	
Head data			
Headwater elevation			
Maximum	ft	3530	
Minimum	ft	3470	
Tailwater elevation at design discharge	ft	3375	
Gross head:			
Maximum	ft	155	
Minimum	ft	95	
Design (0.9 × 155′)	ft	143	
Design net head	ft	143	
Turbine			
Type		Francis	
Design discharge	ft^3/s	485	
Unit capacity	kW	5000	
Waterways			
Intake structure required	No	—	
Penstock:			
Length: Main	ft	620	
Branch	ft	60	
Bifurcation required	No	1	
Maximum velocity	ft/s	12	
Diameter	ft	7.5	
Bypass structure required	No	1	
Tailrace length	ft	—	
Transmission line			
Voltage	kV	34.5	
Capacity	MW	5000	
Length	mile	2.0	

TABLE 6.7 Example of Estimate Project Data Sheet *(Continued)*

Project: Power plant at earthfill dam			Date: July 1987
Description	Unit	Project data	Remarks
New site developments:			
Embankment dam:			
Height W.S. above streambed		—	
Length		—	
Spillway:			
Region		—	
Drainage area			
Existing facilities:			
Degree of difficulty of alteration		Moderate	
Remedial work required		Minor amount	

Care of Water

Where the construction requires provision of cofferdams or other facilities to prevent encroachment of the river or to permit connection to existing reservoirs or conduits, some allowance must be made to the cost.

The requirements for care of water can be highly variable. A reasonable estimate can only be made on the basis of a properly devised plan. For preliminary estimates, an allowance equal to the percentage of the total project direct cost exclusive of the general costs can be used, as shown below.

Condition	Percentage of total direct cost less general costs
None required	0
Moderate	2
Complicated	6

6.14 DAMS AND RESERVOIRS

Existing Facilities

The costs associated with utilizing existing facilities are in two categories:

1. Alterations: Those associated with the modifications, demolitions, replacements, etc., required to adapt the power plant to the existing structures.
2. Remedial work: The work necessary to repair or upgrade the existing facilities to render them suitable for further use as part of the power project.

For alterations, the work is highly variable and the cost is most readily accounted for as a percentage of the total direct project cost excluding general costs, as indicated hereunder. This requires a judgment to be made as to the extent of the work involved.

Amount of work involved	Alteration cost as percentage of direct cost less general costs
Moderate	5
Major	8

Remedial work may not be required, but alterations will always be required, except for some type A or D projects (see Fig. 6.6). The estimate for alterations does not include additional works to improve the project, such as raising an existing dam or adding gates to an existing spillway to raise water levels. Such alterations would have to be allowed for separately as a special cost.

The costs of remedial work cannot be generalized. It can only be evaluated by assessing the actual work required. Where such work is required, a provisional amount can be used in the cost estimate and reviewed as a special study.

New Dams and Spillways

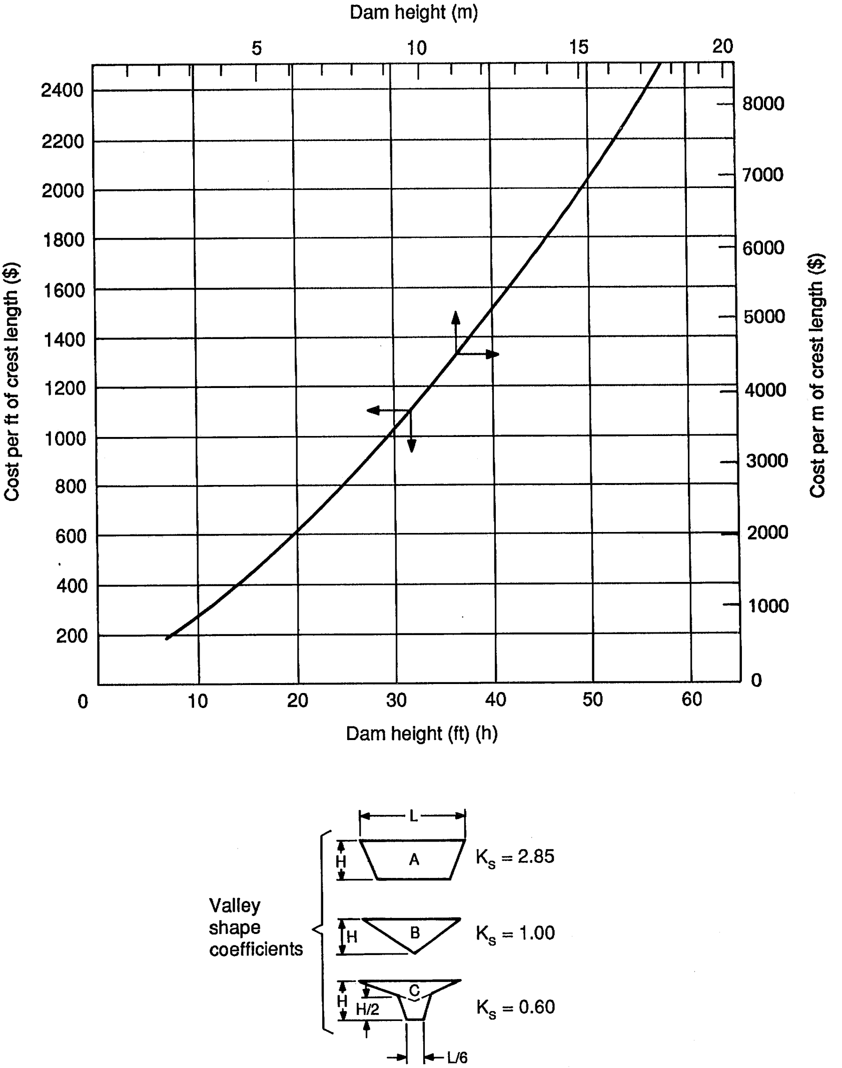
Since the volume of material in a dam is dependent on the height, length, shape of the valley, and the types of materials in the dam and foundations, the final cost is highly site-specific and only approximate costs can be given for providing a dam at a new site proposed for development. A cost curve is shown in Figure 6.8 for an embankment type dam based on conditions described on the figure.

To use this curve, it is necessary to know the height of the normal maximum headwater level above streambed, the crest length, and the shape of the valley. Since the costs are directly proportional to the valley shape factor, an inspection of these factors, which vary from 0.60 to 2.85, as shown in Fig. 6.8, will indicate the degree of approximation in these costs. The costs shown are based on data used by the U.S. Army Corps of Engineers [4] for investigations similar to the initial studies discussed here.

The costs in Fig. 6.8 include 20 percent for excavation, foundation treatment, drainage, and other minor items.

To obtain the spillway cost in the United States, use can be made of Fig. 6.9. These costs are based on work by the U.S. Bureau of Reclamation and the United States Geological Survey [5]. The costs are estimated using drainage area factors developed for 17 regions in the United States and the height of the maximum reservoir surface above the streambed. For spillways with costs beyond the range of Fig. 6.9 or in areas outside the United States, a preliminary estimate of a design flood and a preliminary spillway design and cost estimate design should be made.

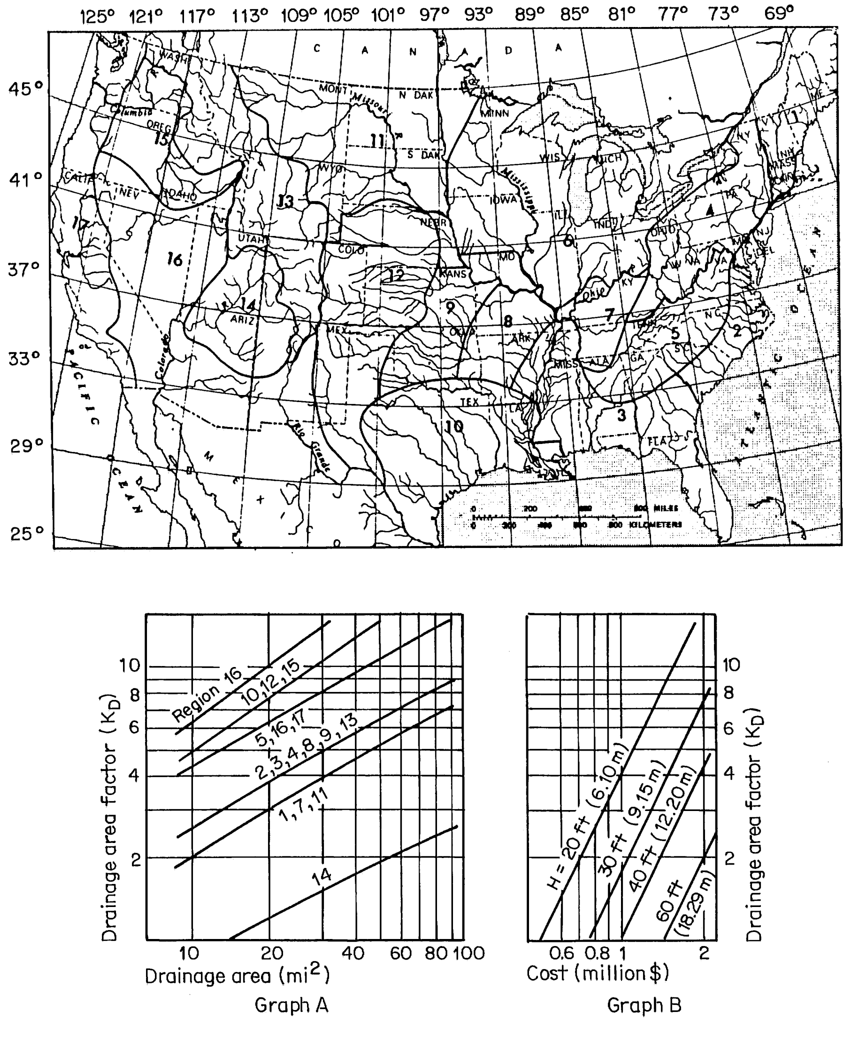
For a more comprehensive study, dam and spillway costs would have to be developed from layouts and quantity estimates.



Notes:

1. Date of costs is January, 1987, U.S. dollars.
2. Total costs obtained by multiplying the cost per foot by the dam crest length.
3. The cost must by multiplied by the valley shape coefficients shown. See also this Figure.
4. The cost is based on a unit price of $10 per cubic yard ($13.10/m³) for embankment fill.
5. Costs include 20 percent for excavation, foundation treatment, drainage, and other minor items.

FIGURE 6.8 Embankment dam costs. (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)



Notes:

1. Date of costs is January, 1987, U.S. dollars.
2. Identify appropriate region from map.
3. From Graph A obtain the drainage area factor, K_D, using the curve for the appropriate region and the specific drainage basin area.
4. From Graph B obtain the estimated spillway cost using K_D and H, the height of the maximum reservoir surface above the streambed.
5. To change mi^2 to km^2, multiply mi^2 by 2.59.

FIGURE 6.9 New spillway costs. (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

6.15 INTAKE AND WATERWAYS

General

The items, some or all of which may be included to provide the cost of the intake and waterways, are as follows:

1. The approach channel, or canal, leading to the intake or modifications thereto
2. The intake structure, or modifications to an existing structure to provide the intake facilities
3. The trash rack and guides
4. The headgate, guides, hoist, hoist support, and power supply
5. The penstock connecting the intake to the powerhouse (or modifications to an existing penstock) and valves
6. A bypass structure and valve (or gate)

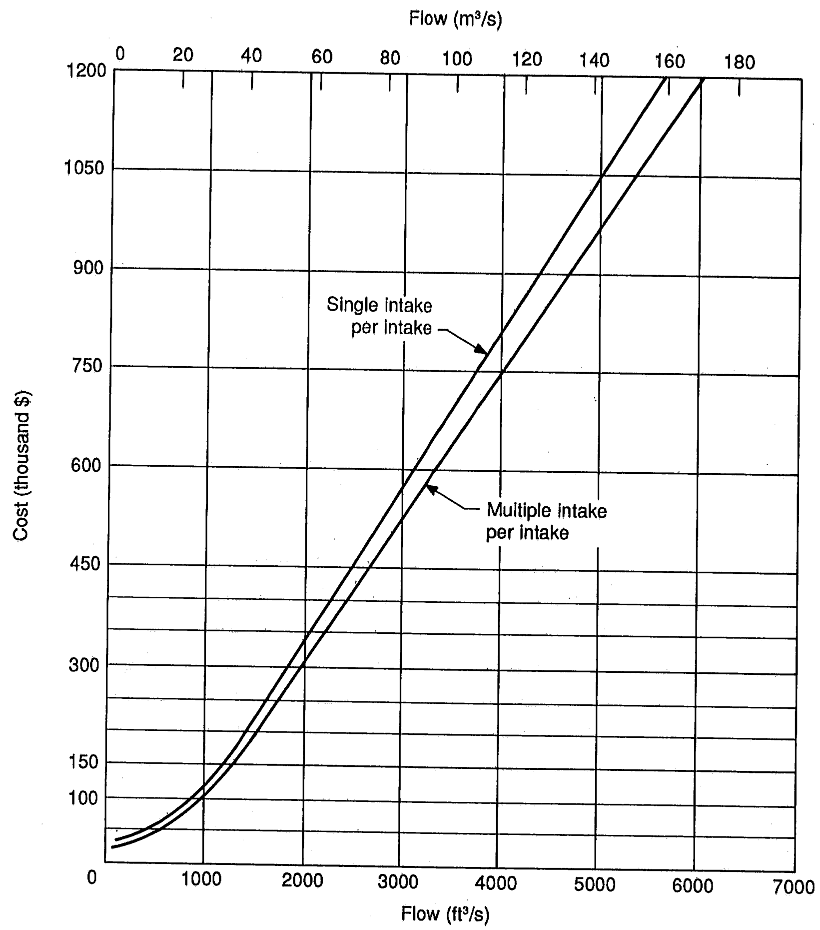
Intake

Where a large amount of earthwork is required to provide or modify a channel or canal to carry water to the power intake, it will be necessary to estimate the volume of excavation and fill required. The cost can then be estimated using the generalized unit prices shown in Table 6.4 for common and rock excavation and for embankment fill. However, in most instances only a short approach will be necessary and allowance for this cost is included in the intake cost discussed hereunder.

The cost of providing a new concrete intake for type A projects is shown in Fig. 6.10. Items included in this cost are the concrete structure, excavation, backfill, trash racks and guides, and a short length of approach channel. Where an existing intake is utilized, the costs of modification are assumed to be included in the alteration work described previously. Where significant variation from these conditions exists, a separate estimate should be made, based on the specific site conditions. The costs of providing head gates, if required, are shown in Fig. 6.11. Two types of gates are considered, namely, slide gates and wheel gates. The costs include provision of a hoist and its support. A nominal cost is included for provision of electric power to the hoist.

Penstock and Valves

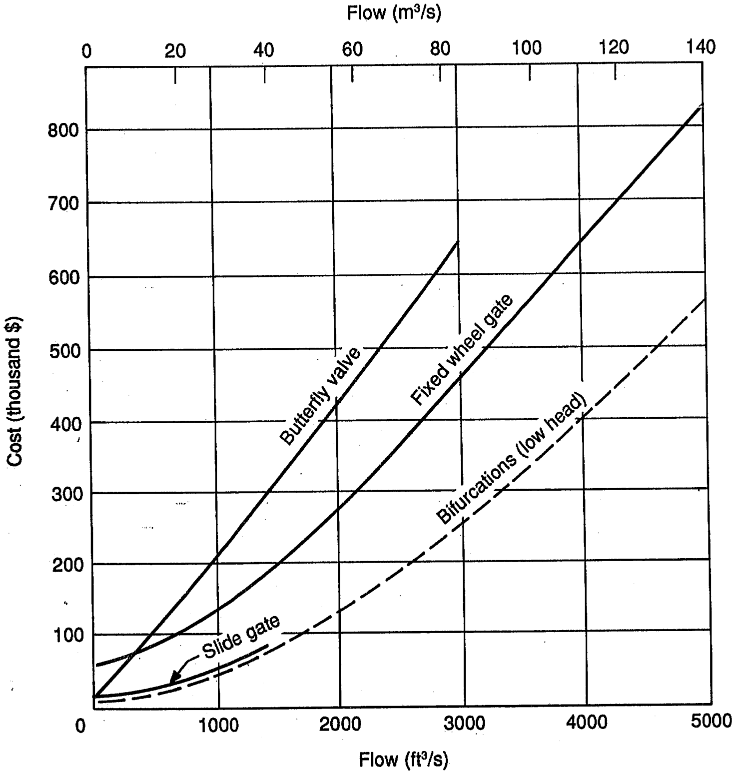
The costs of newly installed steel penstock are shown in Figs. 6.12 and 6.13. The length of penstock to be used is the actual developed length of steel conduit between the intake entrance and the upstream face of the powerhouse. To obtain the cost from this figure, the maximum turbine discharge should be used and a permissible penstock velocity selected. Where an economic velocity has not been determined, a velocity of 12 ft/s (4.4 m/s) can be assumed. The diameter can then be calculated for use in the figure. For total dynamic heads (static head plus water hammer pressure) in excess of 250 ft (80 m), an



Notes:
1. Date of costs is January, 1987, U.S. dollars.
2. The estimated cost includes excavation, concrete, reinforcing steel, backfill, trash rack, miscellaneous metal, and embedded metal.
3. No cost is included for an intake gate. The gate cost can be obtained from Figure 6.11.

FIGURE 6.10 Intake costs. (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

additional allowance should be made for increased plate thickness. This allowance has been made in Fig. 6.13 for heads from 200 to 2000 ft (65 to 650 m). The costs include supply and erection of the penstock, complete with supports. An allowance has been made to account for concrete footings and a

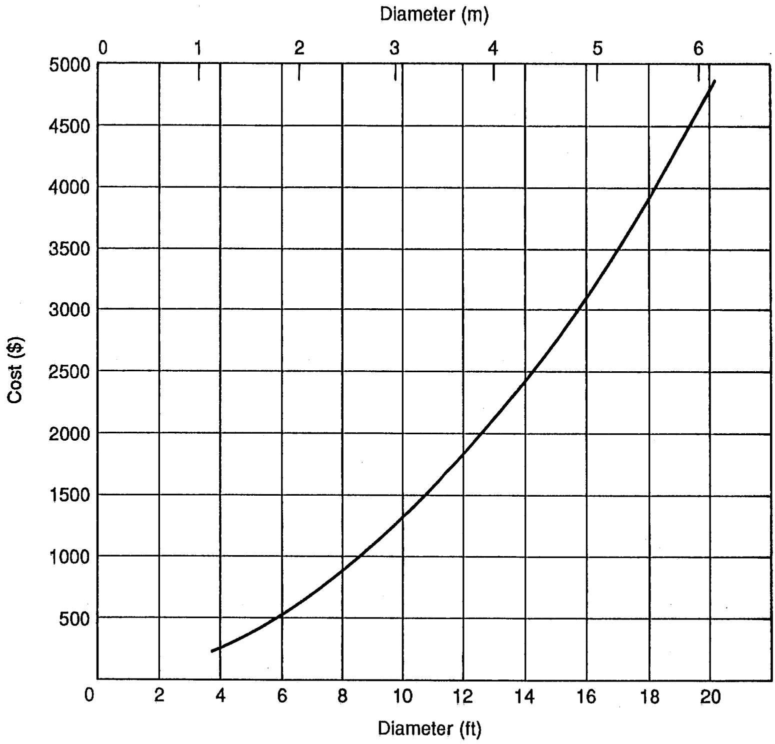


Notes:
1. Date of costs is January, 1987, U.S. dollars.
2. The estimated cost of the fixed wheel gate includes a hoist, gate frame, guides, and installation.
3. The estimated cost of the slide gate includes an operator, gate frame, guides, and installation.
4. The estimated cost of the butterfly valve includes a hydraulic operator and installation.
5. The bifurcation cost must be added to the cost of the equivalent length of steel penstock.

FIGURE 6.11 Valve, bifurcation, and gate costs. (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

nominal amount of excavation and surface treatment along the penstock length. No allowance has been made for external loading, which might result from water or earth pressure.

Where a bifurcation, or wye, should be necessary, its cost may be estimated as shown in Fig. 6.11 and 6.14. The cost of the penstock downstream of

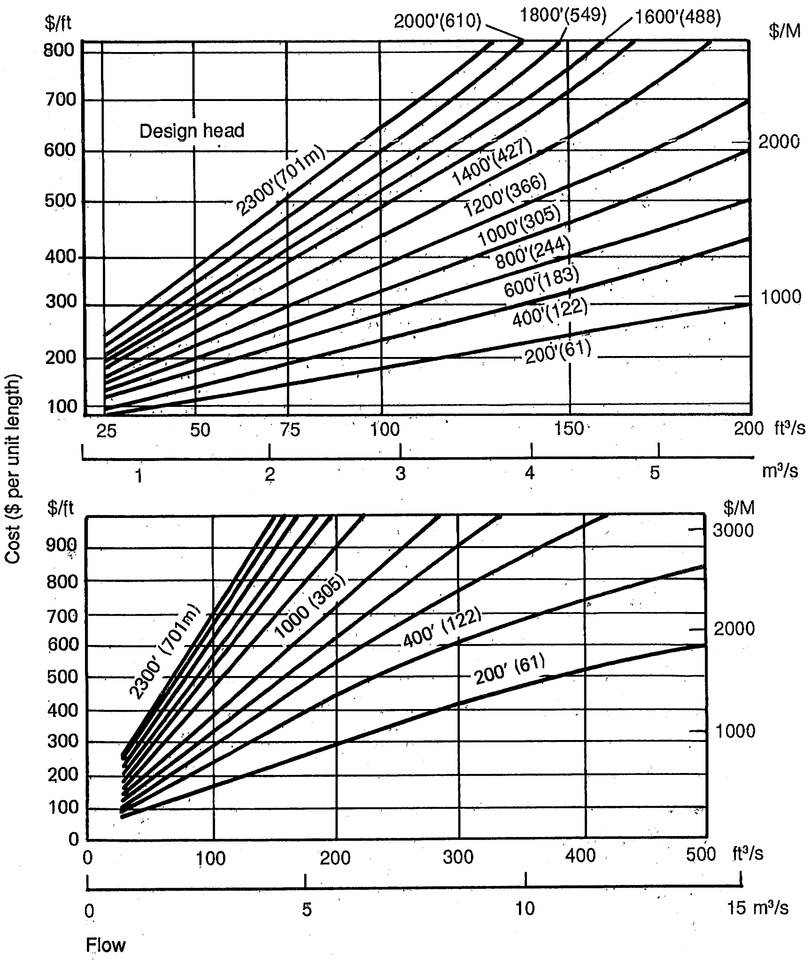


Notes:
1. Date of costs is January, 1987, U.S. dollars.
2. Cost includes penstock, excavation, and concrete supports.
3. Costs based on the minimum handling thickness for steel penstock.
4. If penstock gradient is greater than 15°, and 1% cost for each degree greater than 15°.
5. Values, bifurcations, and anchor blocks not included.
6. Do not use for heads greater than 250 ft (80 m). See Figure 6-13.

FIGURE 6.12 Penstock costs (low head).

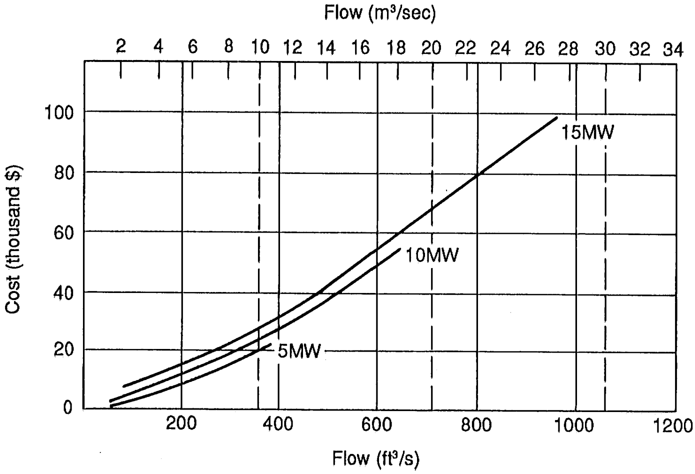
a bifurcation should be based on the flow which could occur in each branch. In the case of multiple units, this would be the maximum flow for one turbine. If a bypass valve is employed, the flow would be the discharge requirement for that valve.

If a bypass structure is required, its cost may be estimated from Fig. 6.15, based on the discharge capacity required and the type of bypass. The discharge may be the turbine discharge capacity or some larger or smaller discharge determined by downstream requirements. If an existing valve can be reutilized, an allowance can be made for removal and reinstallation instead of using a new valve, as in Fig. 6.15.



Notes:
1. Cost base is January 1987, U.S. dollars.
2. Costs based upon minimum handling thickness as a function of maximum design head (static head and design pressure rise up to 2300 ft (701 m).
3. If the penstock gradient is greater than 15°, add 1% of total cost for each degree greater than 15°.
4. Costs include a steel penstock, supports, couplings, site clearing, and excavation.
5. The cost of valves and bifurcations and anchor blocks are not included.

FIGURE 6.13 Penstock costs (high head). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)



Notes:
1. Cost base is January 1987, U.S. dollars.
2. Costs based upon minimum handling thickness as a function of maximum design head (static head and design pressure rise up to 2300 ft (701 m).
3. Bifurcation angle 45°. Area of each leg of bifurcation is 50% of main penstock above bifurcation.
4. Costs include a steel shell, crotch girder, and C clamps and installation.
5. The cost of valves and bifurcations and anchor blocks are not included.

FIGURE 6.14 Steel penstock bifurcation costs (high head). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

6.16 POWER PLANTS

General

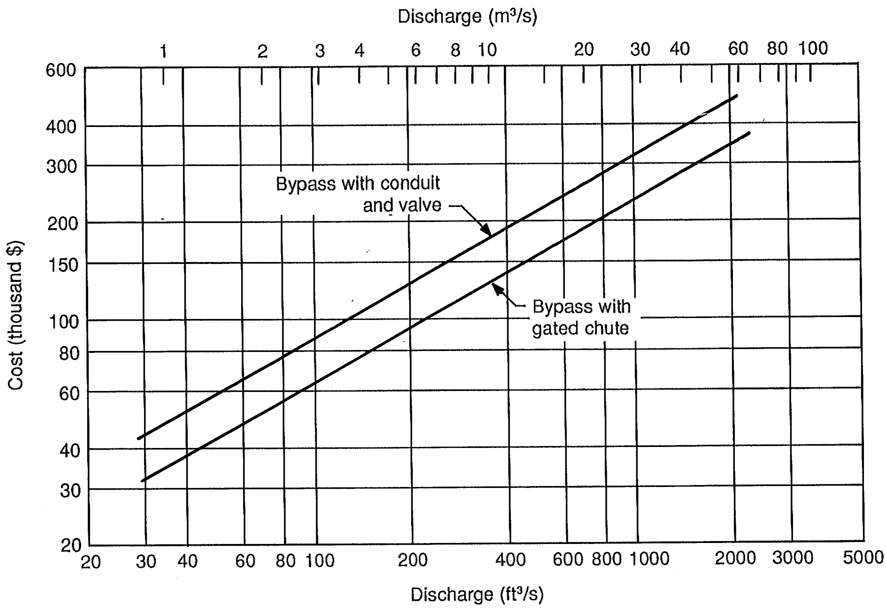
The items included to obtain the power plant cost are as follows:

1. The civil works associated with the powerhouse and tailrace
2. Turbines or generators, control equipment and valves
3. Accessory electrical equipment
4. Miscellaneous powerhouse equipment
5. Switchyard and equipment

Where the powerhouse forms part of a dam, the complete bay comprising the dam and powerhouse is categorized as a type D project (see Fig. 6.6).

Powerhouse and Equipment

To simplify obtaining the preliminary estimates, the equipment costs have been combined with powerhouse costs, based on standardized conditions to

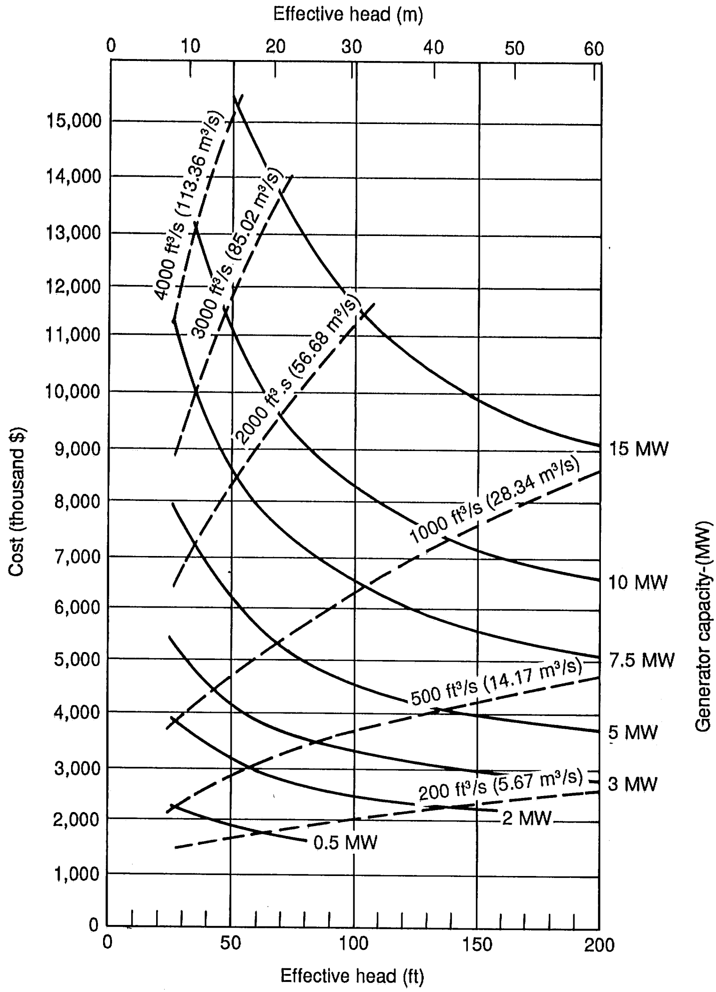


Notes:
1. Cost base is January 1987, U.S. dollars.
2. Cost of bypass with conduit and valve includes excavation, foundation preparation, reinforced concrete bypass structure, backfill, steel liner to dissipation chamber, miscellaneous and embedded metals, piping, fixed cone dispersion valve, and operator.
3. The bypass with a gated chute consists of a gated intake, chute, and stilling basin. Costs include excavation, foundation preparation, reinforced concrete structure, backfill, trashracks, miscellaneous and embedded metals, slide gate, and operator. For flows greater than those given, use a multiple-gated intake.

FIGURE 6.15 Bypass structure costs. (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

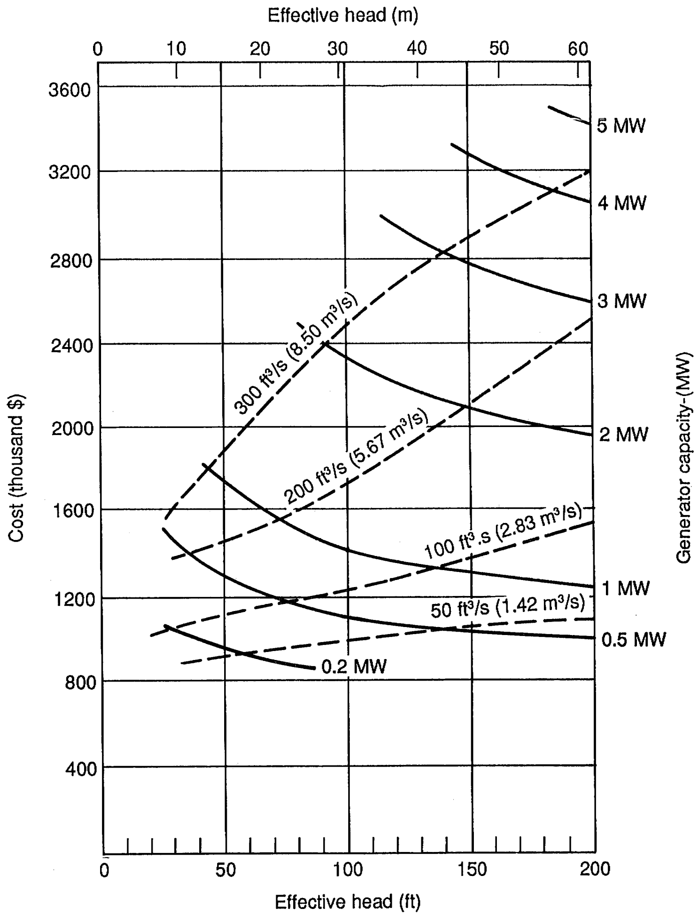
obtain composite power-plant costs for different turbine types, as shown in Figs. 6.16 through 6.25. Included in these costs are the civil works, turbine, generator, governor, accessory electrical equipment, and miscellaneous powerhouse equipment. Not included are the costs for an extended tailrace and for the switchyard, which are dealt with separately. The costs of the power plants will vary depending on the type of turbine used. Costs are included for Francis, impulse, propeller, tubular, and bulb units for use with type A, B, or C projects. For type D projects, costs have been included for open-flume, tubular, and propeller units. The cost of excavation and protective treatment of the tailrace transition is included in the composite costs. The turbine flows corresponding to the head and capacity have been plotted on these curves also.

These composite costs are based on single-unit plants. If it should be desired to investigate multiple-unit plants, the costs given in the charts for each unit can be reduced by 10 percent. The costs do not include a structure or valves for bypassing water around the turbine unit, if the unit is not operating.



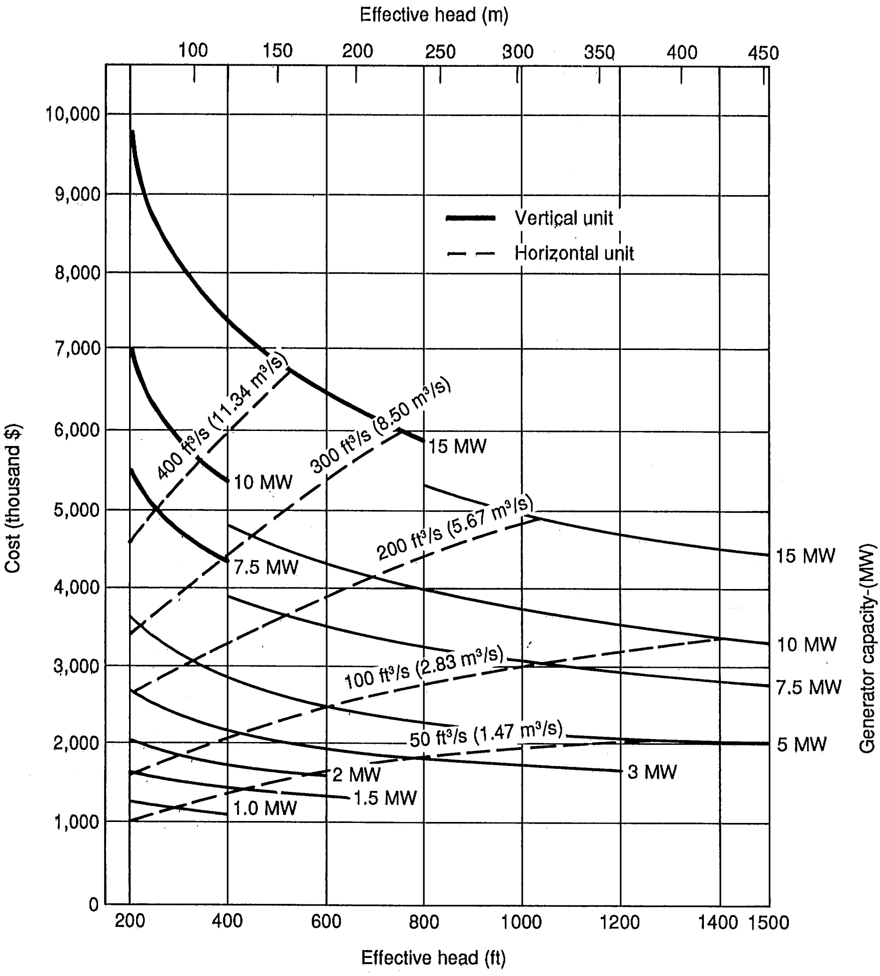
Notes:
1. Date of costs is January 1987, U.S. dollars.
2. Costs include turbine and generator, site civil works, powerhouse civil works, miscellaneous power plant equipment, accessory electrical equipment, and turbine shut-off valve.
3. Costs are for a single-unit powerhouse.
   Turbine discharges shown are based on a turbine efficiency of 0.85.

FIGURE 6.16 Powerhouse and equipment costs—vertical Francis units (type A, B, C projects). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)



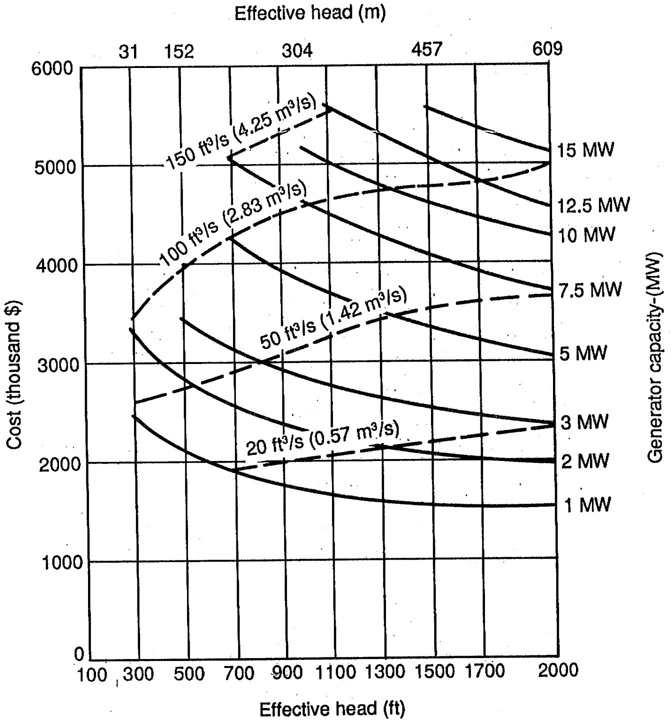
Notes:
1. Date of costs is January 1987, U.S. dollars.
2. Costs include turbine and generator, site civil works, powerhouse civil works, miscellaneous power plant equipment, accessory electrical equipment, and turbine shut-off valve.
3. Costs are for a single-unit powerhouse.
4. Turbine discharges shown are based on a turbine efficiency of 0.85..

FIGURE 6.17 Powerhouse and equipment costs—horizontal Francis units (type A, B, C projects). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)



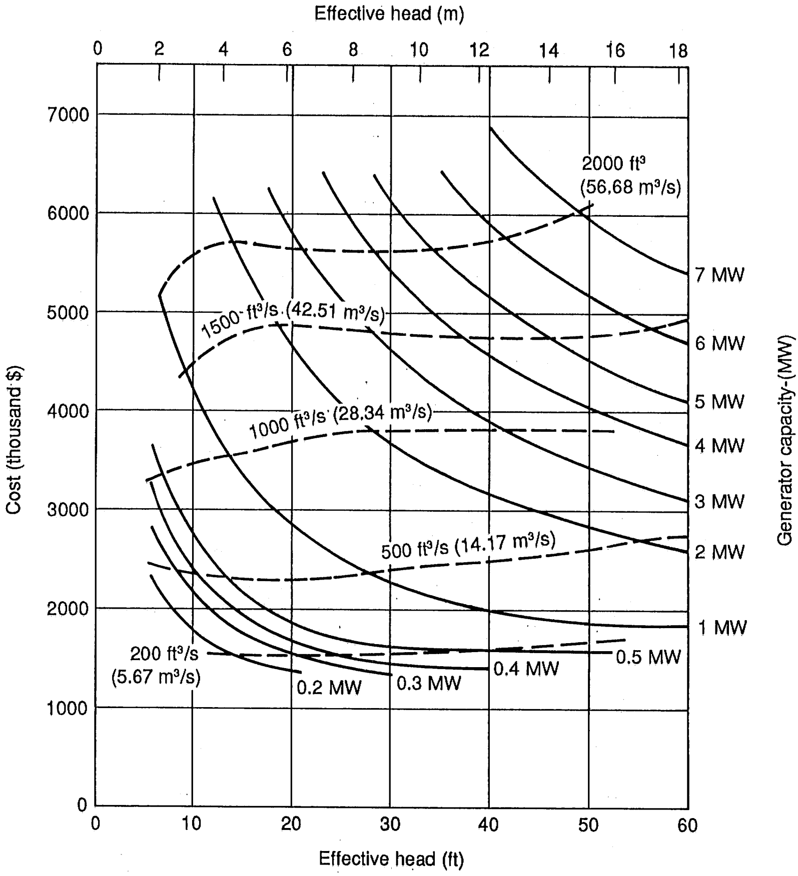
Notes:
1. Date of costs is January 1987, U.S. dollars.
2. Costs include turbine and generator, site civil works, powerhouse civil works, miscellaneous power plant equipment, accessory electrical equipment, and turbine shut-off valve.
3. Costs are for a single-unit powerhouse.
4. Turbine discharges shown are based on a turbine efficiency of 0.85.
5. Site conditions may dictate a vertical mounting within the conventional horizontal range. Add 8% to horizontal unit cost to obtain vertical unit cost.

FIGURE 6.18 Powerhouse and equipment costs—vertical and horizontal Francis units (type A, B, C projects). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)



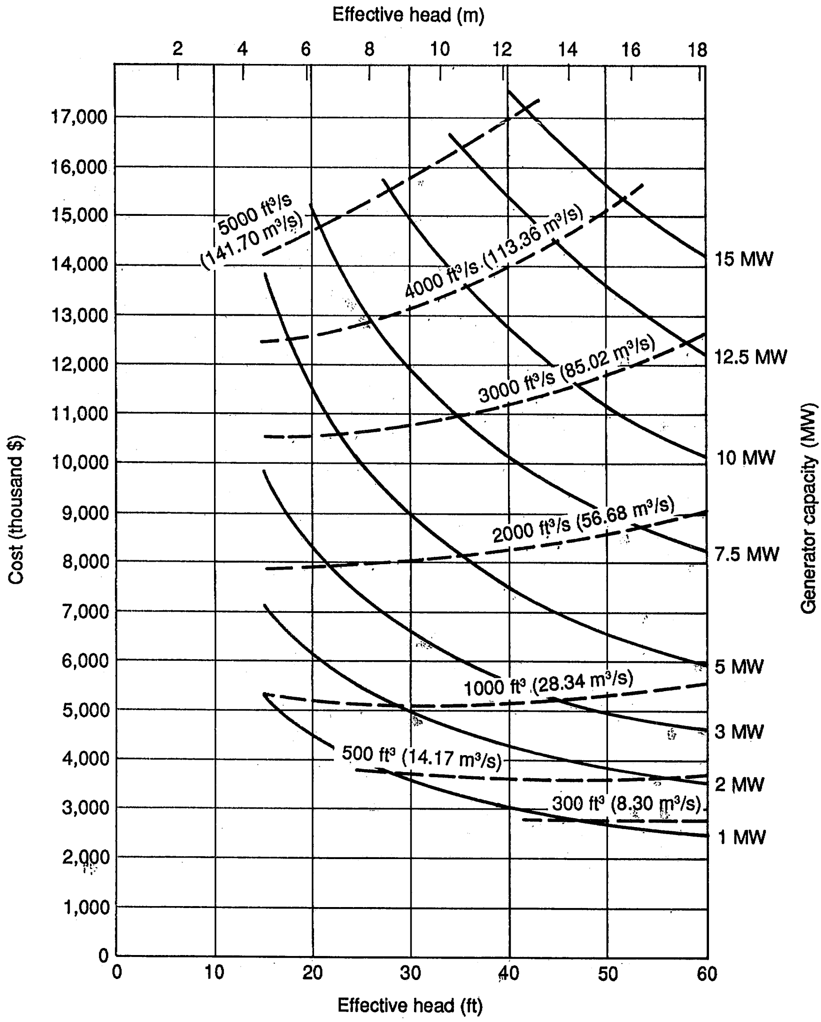
Notes:
1. Cost base is January 1987, U.S. dollars.
2. Costs are for a single-unit powerhouse with a typical indoor arrangement.
3. Costs include turbine and generator, site civil works, powerhouse civil works, miscellaneous power plant equipment, accessory electrical equipment, and turbine shut-off valve.
4. Turbine discharges shown are based on a turbine efficiency of 0.85.
5. Costs based on a single or double-nozzle horizontal impulse turbine directly connected to the generator.

FIGURE 6.19 Powerhouse and equipment costs—impulse units (type A, B, C projects). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)



Notes:
1. Date of costs is January 1987, U.S. dollars.
2. Costs include turbine and generator, site civil works, powerhouse civil works, miscellaneous power plant equipment, accessory electrical equipment, and turbine shut-off valve.
3. Costs are for a single-unit powerhouse.
4. Turbine discharges shown are based on a turbine efficiency of 0.85.

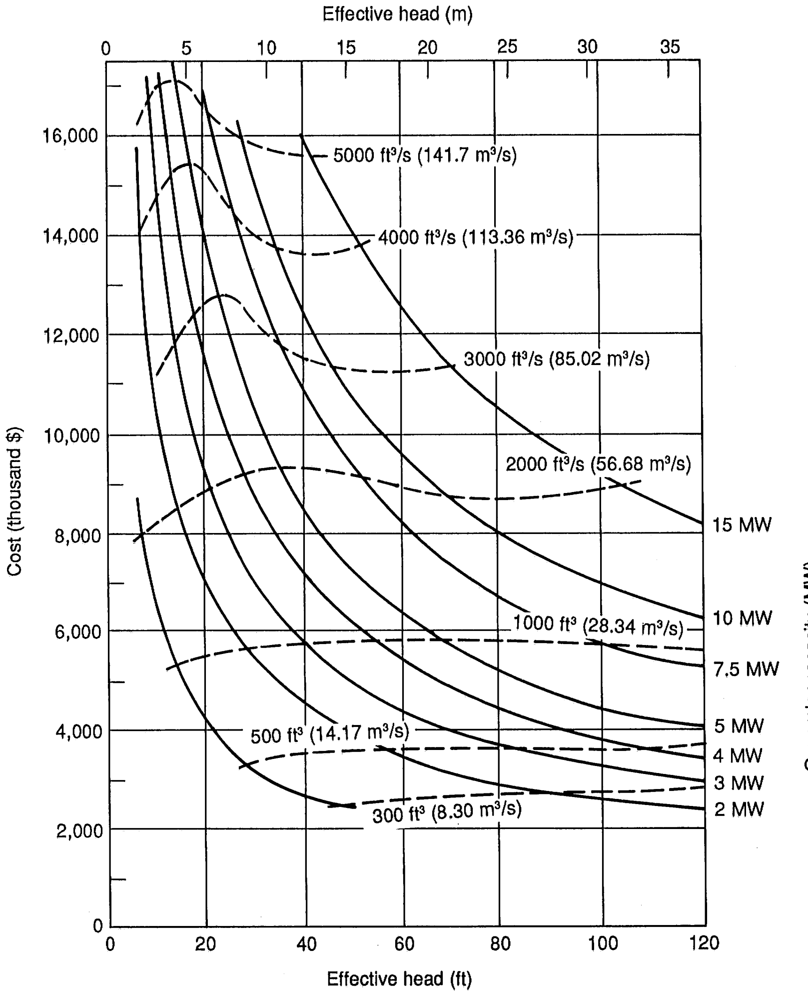
FIGURE 6.20 Powerhouse and equipment costs—tubular units (type A, B, C projects). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)



Notes:

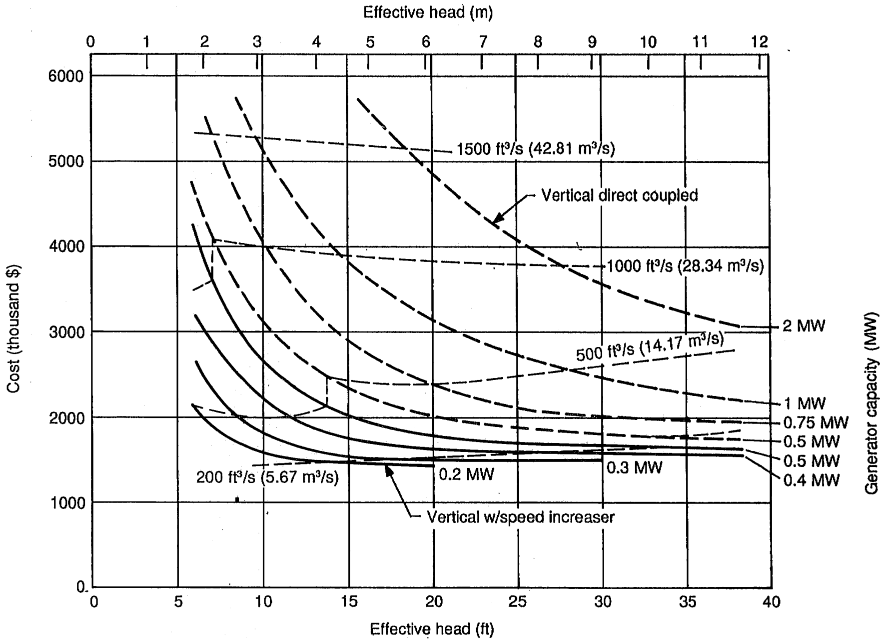
1. Date of costs is January 1987, U.S. dollars.
2. Costs include turbine and generator, site civil works, powerhouse civil works, miscellaneous power plant equipment, accessory electrical equipment, and turbine shut-off valve.
3. Costs are for a single-unit powerhouse.
4. Turbine discharges shown are based on a turbine efficiency of 0.85.

FIGURE 6.21 Powerhouse and equipment costs—bulb units (type A, B, C projects). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)



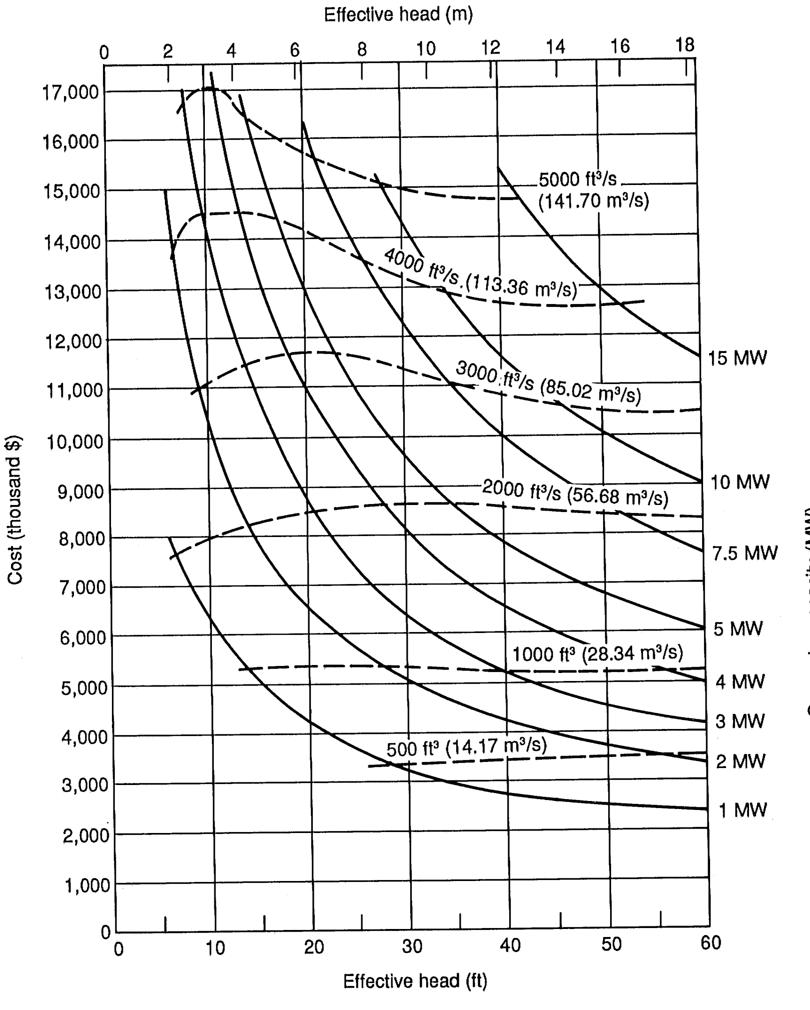
Notes:
1. Date of costs is January 1987, U.S. dollars.
2. Costs include turbine and generator, site civil works, powerhouse civil works, miscellaneous power plant equipment, accessory electrical equipment, and turbine shut-off valve.
3. Costs are for a single-unit powerhouse.
4. Turbine discharges shown are based on a turbine efficiency of 0.85.

FIGURE 6.22 Powerhouse and equipment costs—vertical propeller units (type A, B, C projects). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)



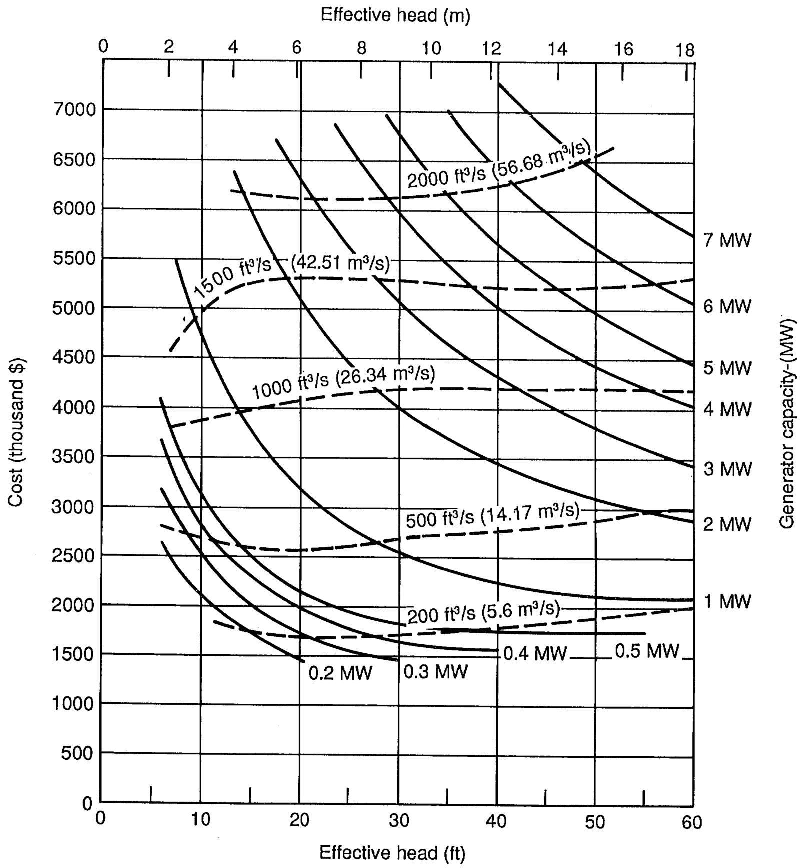
Notes:
1. Date of costs is January 1987, U.S. dollars.
2. Costs estimate include turbine and generator, site civil works, powerhouse civil works, miscellaneous power plant equipment, accessory electrical equipment, headworks structure, trashracks, intake gates, and operators.
3. Costs are for a single-unit power plant.
4. Turbine discharges shown are based on a turbine efficiency of 0.85.
5 Costs are based on a typical vertical propeller turbine unit with fixed blades.

FIGURE 6.23 Powerhouse and equipment costs—open-flume units (type D projects). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)



Notes:
1. Date of costs is January 1987, U.S. dollars.
2. Cost estimate includes turbine and generator, site civil works, powerhouse civil works, miscellaneous power plant equipment, accessory electrical equipment, headworks structure, trashracks, intake gates, and operators.
3. Costs are for a single-unit power plant.
4. Turbine discharges shown are based on a turbine efficiency of 0.85.

FIGURE 6.24 Powerhouse and equipment costs—vertical propeller units (type D projects). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

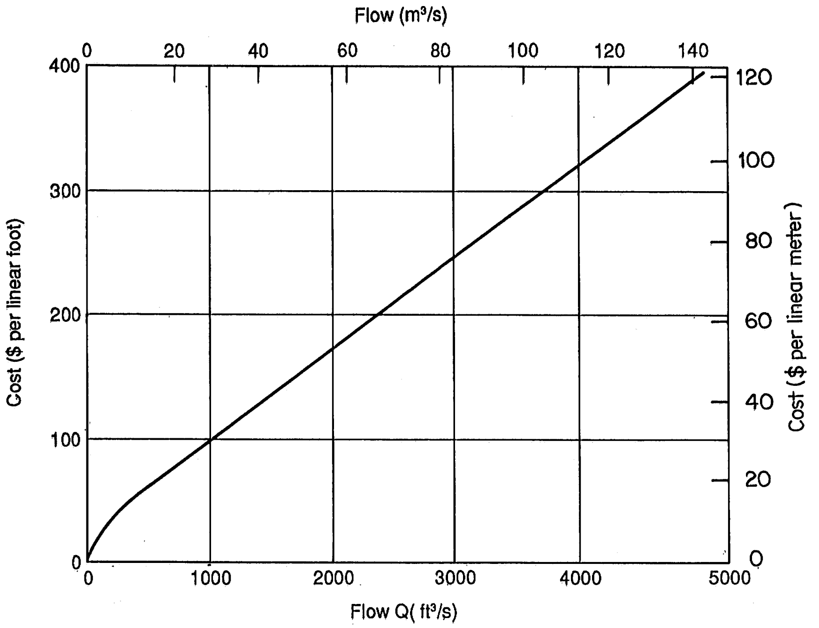


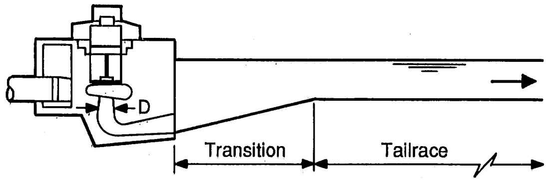
Notes:
1. Date of costs is January 1987, U.S. dollars.
2. Cost estimate includes turbine and generator, site civil works, powerhouse civil works, miscellaneous power plant equipment, accessory electrical equipment, headworks structure, trashracks, intake gates, and operators.
3. Costs are for a single-unit power plant.
4. Turbine discharges shown are based on a turbine efficiency of 0.85.

FIGURE 6.25 Powerhouse and equipment costs—tubular units (type D projects). (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

Tailrace Excavation

The cost of excavating a tailrace depends on the average depth and length of the excavation, as well as the magnitude of the turbine discharge to be carried. The cost will include an initial amount to provide for a transition from the draft tube exit to the normal tailrace channel section, plus an amount to cover the cost of providing the extension of the channel to its junction with the existing watercourse. The graph on Fig. 6.26 provides an estimate of the cost of the tailrace channel extension based on





Notes:
1. Date of costs is January 1987, U.S. dollars.
2. Excavation is assumed to be 50 % rock and 50 % common.
3. Tailrace transition excavation costs are included as part of the powerhouse costs. The tailrace length commences at the end of the transition.

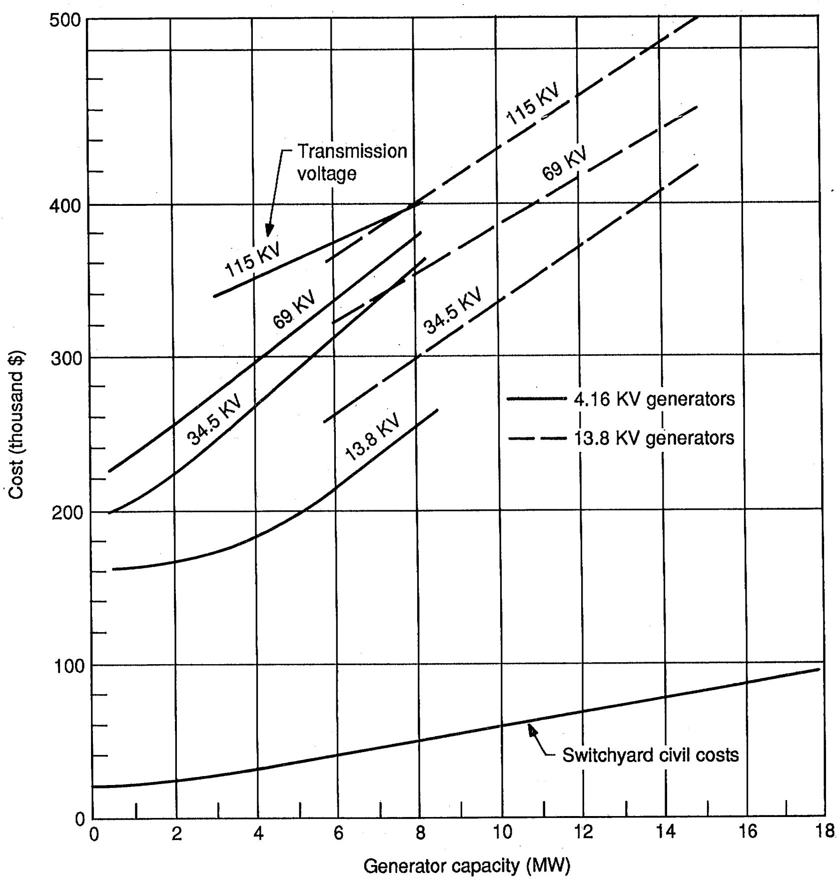
FIGURE 6.26 Tailrace excavation costs. (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

the turbine design discharge. The cost of the transition excavation is included in the power-plant cost. The length of the tailrace must be estimated and used in Fig. 6.26 to obtain the cost of the channel extension.

Switchyard

Switchyard civil costs can be obtained from Fig. 6.27. These costs assume a normal amount of grading and fencing. Some additional allowance would be necessary for extra excavation, in steep terrain, or for special foundations.

The cost of switchyard equipment can also be obtained from Fig. 6.27. The cost data provide for installations up to a maximum of 115 kV. The costs include



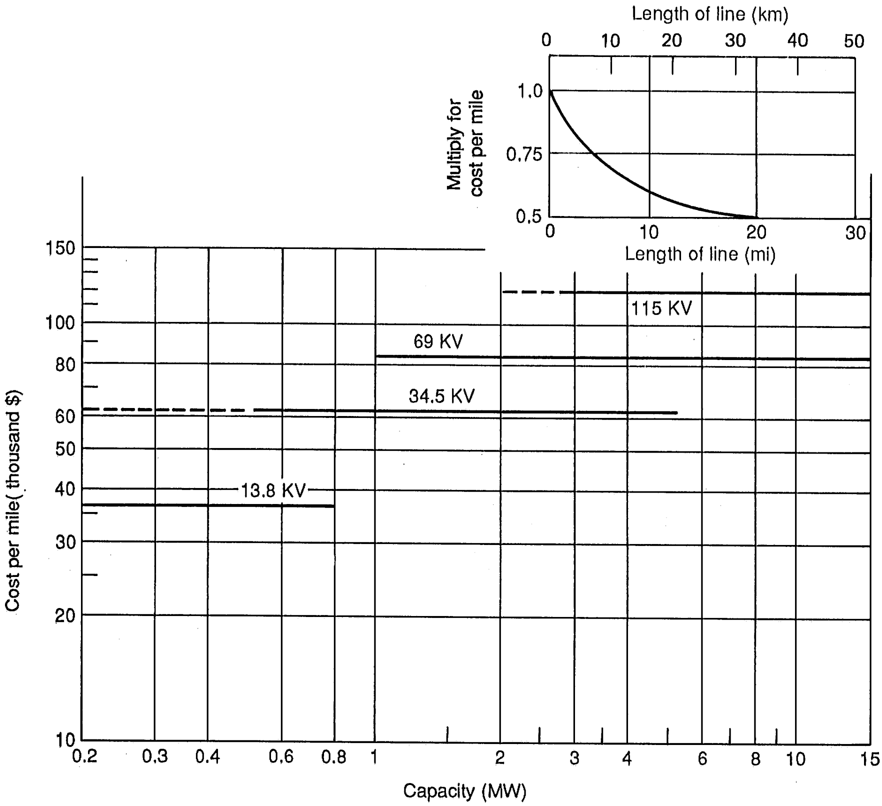
Notes:
1. Date of costs is January 1987, U.S. dollars.
2. Costs are for a single unit.
3. For multiple units add $100,000 multiplied by (N-1), where N is the number of units. Total switchyard cost is the sum of the civil cost and the equipment cost corresponding to the transmission voltage required.

FIGURE 6.27 Switchyard costs with generator voltage circuit breakers. (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

the main transformer. An additional cost should be added if a line circuit-breaker arrangement is preferred.

6.17 TRANSMISSION

The costs of transmission lines are given in Fig. 6.28. Included in the costs are wood poles, conductors, insulators, and connecting hardware. The costs of tapping into an existing transmission line or substation are absorbed in the costs



Notes:
1. Date of costs is January 1987, U.S. dollars.
2. The cost per mile of transmission line should be adjusted according to the length of the line required using the multiplier shown above.
3. The estimated costs include wood poles, hardware, insulators, conductors, and construction roads. No clearing, land, or right of way costs are included.
4. The estimated costs are based on single wood poles for the 13.8-and 34.5 kV lines and wood pile H-frames for the 69 -and 115--kV lines.
5. The estimated costs are for prairie-type terrain with favorable foundation conditions. For foothill terrain add 10 % of cost and for mountainous or swampy terrain add 30 % of the cost.
6. For cost per km, multiply cost per mile by 0.6.

FIGURE 6.28 Transmission line costs. (Copyright ©1983 Electric Power Research Institute, Report EM-3213, "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites." Reprinted with permission.)

shown. Included also is an allowance for construction of the line and normal access to the line. The costs should be varied as indicated to account for the type of terrain being traversed. Not included are the costs of land rights, relocations, clearing, and special access roads which must be included separately. The land area to be leased or purchased along the length of the transmission line would follow the utility's existing practice. Land costs or rights should be estimated according to prevailing rates in the area, and the type of land being traversed. The cost of clearing undeveloped land can be estimated as shown on Table 6.4. The cost of relocations, special access roads, or environmental mitigation should be considered according to the particular circumstances prevailing. The cost of access roads may be estimated from Table 6.4.

The voltage to be used may be selected with reference to the costs shown in Fig. 6.28, but will probably be determined by the voltage of the system into which the transmission line feeds. The total costs will be the sum of the number of miles of line times the unit cost in Fig. 6.28, plus the costs of land, clearing, and, if appropriate, special access roads.

6.18 OTHER FACILITIES

No allowance has been made directly for the cost of special facilities required as a result of environmental needs, since the requirements are so varied that it is not practical to do so. In some instances, the cost will result from provision of specific facilities such as outlet works, or new recreational facilities, while in others the cost may reveal itself in lost power output due to mandated methods of operation. Fortunately, for low-head, low-capacity plants at existing facilities, the requirements will usually, but not invariably, be not too costly to meet.

The item will therefore have to be addressed independently by the user. After establishing the nature of any problem which might arise, a provisional and rough cost can be used in the estimate. If the economic analysis indicates that the project is suitable, the environmental issue can then be addressed as a study, and the project reviewed after the results of the study are known.

6.19 COST ADJUSTMENTS

General

Adjustments should be made to the costs obtained as above to account for escalation in prices, and for cost variations apparent in different parts of the country. Cost adjustments can be made similarly for costs outside of the United States.

Escalation

While several types of cost index are published by various agencies, those published by the Bureau of Reclamation are commonly used for hydroelectric development. The current indices are readily available and are published quarterly for the months of January, April, July, and October in *Engineering News Record*, with a time lag of about two months. Prior to 1982, the indices were published using 1967 costs as a base. Due to the rapid rate of inflation in the late 1970s, *Engineering News Record* adjusted the indices to a 1977 cost base. The 1987 cost base is the base for the indices presented in this manual. Table 6.5 shows the values of the indices corresponding to January 1987, the cost level in this chapter, for applicable items of construction. The costs of a project that is being analyzed can be escalated to the date of the most recently published indices, by proportion, using those indices and the values in Table 6.5. Provision is made in Table 6.6 for insertion of the appropriate escalation factors.

6.20 INDIRECT COSTS

Contingency

A contingency allowance is normally added to the direct costs. This allowance is to account for items which are unforeseen or omitted and usually lie between 10 and 20 percent of the direct costs, depending on the degree of accuracy of the estimate. For a preliminary analysis, a value of 20 percent is appropriate. The percentage is applied to the total direct cost after adjustments for escalation and location have been made.

Engineering, Administration, and Construction Management

In addition to the contingency allowance, an estimate must be made of the development costs, that is, the costs for engineering, administration, and construction management. These include expenditures for a feasibility study, environmental impact report, license and permit applications, preliminary and final design, construction management, and administration.

For a preliminary analysis, these costs can be estimated as being 20 percent of the sum of the direct costs (after escalation adjustment) and the contingency amount.

The sum of the direct cost (adjusted), the contingency amount, and the development costs then gives the total construction cost.

6.21 RECURRING ANNUAL COSTS

Once completed, the project will incur annual costs for:

- Operation and maintenance (O&M)
- General expense
- Insurance

Operation and maintenance costs for small hydroelectric projects are difficult to forecast. Analysis of data for over 100 existing power plants throughout the country indicate a wide divergence in costs. Data obtained indicated that the costs could vary between 50 and 100 percent of the amounts given. It is expedient to estimate the O&M cost, general expense, and insurance as varying between 1 and 2 percent of the capital cost of the project. This assumes that the owner has an operating and maintenance organization and also operates and maintains other plants. This percentage should be increased for the owner of just one plant.

The O&M costs will vary depending on whether the plant's operation is remote or local. The direct costs of the power-plant equipment described in this chapter are for remote operation.

General expenses include miscellaneous expenditures required for administration, license, and permit requirements.

Insurance costs for small hydroelectric projects are undergoing changes as of this printing and may be higher than ever before. Historical costs have been used, however, in determining the amounts for insurance. These are included in the total annual recurring costs. For new dams, insurance may be prohibitively expensive. At existing facilities, however, the cost of insurance should have already been absorbed.

The annual costs should be escalated to the date of the estimate using an appropriate index, such as one published by the U.S. Department of Labor. The data of the weekly labor earnings of the machinery industry on the Producer Price Index, or data for commodity group 11-7, electrical machinery, on the CPI could also be used for this purpose.

6.22 EXAMPLE OF ESTIMATE

Here is an example for determining costs, using the procedure just described:

Assume that a cost estimate needs to be prepared for a new power plant in the southwestern United States. The plant is at the downstream end of the outlet works tunnel at an earth-fill dam. It will be connected by a penstock 610 ft (200 m) long to the tunnel plug, which is located at the midpoint of the tunnel. The maximum static head is 155 ft (47 m), and there is a reservoir fluctuation of 60 ft (18 m). About 1 acre (0.4 ha) of land is required for the power plant, and 20 acres (8 ha) for the transmission line. Construction requires a temporary access road about 0.2 mi (0.13 km) long. No facilities need to be relocated, and the care of water is considered moderately difficult. The mechanical engineers have selected a Francis-type turbine with a capacity of 5000 kW. A bifurcation and bypass structure are required. There is a 34.5-kV transmission line 2 mi (km) away which can be joined to the power plant.

The first step in determining the cost is to complete the project data sheet, Table 6.7 (pp. 6.27 to 6.28), and the cost compilation sheet, Table 6.8, as provided in the text. Finally, a project cost summary can then be prepared, Table 6.9. The July 1987 cost indexes shown in the example are estimates only.

Each cost estimate should be carefully reviewed for all items necessary for development. Some may have been overlooked. In retrofitting existing structures for power generation, the procedure for bypassing water during construction needs to be carefully evaluated from both a cost and a schedule viewpoint. Finally, cost estimates must be prepared several times during the planning and design phases as more information is acquired.

TABLE 6.8 Example of Estimate Cost Compilation Sheet

Description	Unit	Quantity	Unit cost	Amount January 1987	Cost indexes 1987	Cost indexes July* 19(87)	Escalated amount 19(87)
1. *New dam and spillway*:							
Embankment dam							
Spillway							
Reservoir clearing							
Subtotal							
2. *Intake*:							
Intake, type A							
Intake gate & hoist	No						
Subtotal							
3. *Penstock*:							
Penstock	LF	620	750	465,000			
Penstock branch	LF	60	750	45,000			
Bifurcation	No	1	1	30,000			
Subtotal				540,000	1.65	1.67	547,000
4. *Power plant (type C project)*:							
Total cost, single unit	LS			4,000,000	1.66	1.68	4,050,000
Equipment adjustments:							
For governor	LS						
For fixed blade/mounting	LS						
For inlet valve	LS						
For spiral case	LS						

TABLE 6.8 Example of Estimate Cost Compilation Sheet *(Continued)*

Description	Unit	Quantity	Unit cost	Amount January 1987	Cost indexes 1987	Cost indexes July* 19(87)	Escalated amount 19(87)
Subtotal unit							
Bypass structure	LS			220,000	1.65	1.67	223,000
Tailrace	LF						
Switchyard							
Civil features	LS			35,000	1.60	1.62	36,000
Equipment	LS			290,000	1.60	1.62	294,000
Subtotal							4,603,000
5. *Special costs*:							
Subtotal							
Subtotal—items 1 through 5							5,150,000
6. *Existing facilities*:							
Alterations to connect power plant (5% of subtotal)	LS						258,000
Remedial work (assumption)	LS						25,000
Subtotal							283,000

7. *General costs*:							
Land & land rights†	acre	1.0	6,000	6,000			
Water rights	LS						
Relocations	LS						
Mobilization (3% of subtotal)	LS			155,000			
Access road† (temporary)	mile	0.2	100,000	20,000			
Bridges	LS						
Care of water (2% of subtotal)	LS			103,000			
Subtotal				284,000			284,000
8. *Transmission line*:							
Line	mile	2	62,000	124,000	1.47	1.50	127,000
Clearing	acre						
Land	acre	20	3,000	60,000			60,000
Access road	mile						
Subtotal							187,000

*Estimate.
†Excludes transmission line.

TABLE 6.9 Project Cost Summary [Base date: July/19(87)]

Item	Costs
	$
1. New dam and spillways	—
2. Intake	—
3. Penstock	547,000
4. Power plant	4,603,000
5. Special structures	—
6. Existing facilities	283,000
7. General costs	284,000
8. Transmission line	187,000
9. *Total direct cost*	5,904,000
10. Contingencies (20%)	1,181,000
11. Subtotal	7,085,000
12. Engineering, administration and construction management (20%)	1,417,000
13. *Total construction cost*	8,502,000

REFERENCES

1. "Planning and Design of Hydroelectric Power Plant Structures," U.S. Army Corps of Engineers, EM 1110–2–3001.
2. "Design Standards No. 9, Buildings," U.S. Bureau of Reclamation Publication, 1972.
3. "Simplified Methodology for Economic Screening of Potential Small-Capacity Hydroelectric Sites," Report EM-3213, Electric Power Research Institute.
4. "Hydropower Cost Estimating Manual," U.S. Army Corps of Engineers, North Pacific Division, May 1979.
5. "Estimating Handbook," Series 150 Estimating and Appendix 150A, United States Department of Interior, Bureau of Reclamation, Engineering and Research Center, Denver Federal Center, Denver, Colorado 80225.
6. John R. Crippen, "Envelope Curves for Extreme Flood Events," *Journal of the Hydraulics Division*, American Society of Civil Engineers, vol. 108, no. HY10, October 1982.

CHAPTER 7

FACILITY DESIGN GUIDELINES AND CASE STUDIES

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In this chapter practical guidelines for the design of hydroelectric facilities are given, followed by three case studies which include the design of a new 11,000-kW hydropower facility for a remote mine in northwestern Saskatchewan, the design of a 1100-kW intake-penstock-powerhouse addition to an existing hydropower development, and the installation of a larger 2250-kW unit in an existing powerhouse. Both additions are located in Newfoundland.*

7.1 DESIGN GUIDELINES

The following design guidelines may be used as part of preliminary design or as verification that there are no major omissions in a final design.

Trash Racks

The velocity through the gross area (no deduction for area occupied by the rack) can vary over a range of 2 to 15 ft/s (0.5 to 4.4 m/s) and is a function of the unit size, rated head, and plant load factor.

A formula which can be used to determine a suitable velocity is

$$V = E_t k(1.6 - P)(1 + C/250)(\mathrm{kW})^{0.2} \tag{7.1}$$

where E_t = 0.16 for metric units
= 0.52 for USCS units
V = velocity through gross rack area, m/s or ft/s

*Data on the Charlot River development are published with the permission of Saskatchewan Power Corporation, and data on the Morris-Topsail Developments are published with the permission of the Newfoundland Light and Power Co., Limited.

k = water cleanliness factor based on judgment, 1.0 for clean to 0.5 for debris-laden
P = plant capacity factor, ranging from about 0.2 up to about 0.8
C = percentage of reservoir cleared
kW = turbine capacity

Intake Gate

The area of an intake gate should be about 25 percent more than that of the penstock or pipeline downstream. The height of the gate opening should be about 25 percent more than the gate width. These ratios will result in a gate having a width of about $0.85D$ and a height of about $1.05D$, where D is the penstock diameter.

To prevent vortices, the top of the gate must be submerged below the low-water level by a distance S. The value of S can be determined from the following formula, which is a refined version of data from Gordon [1].

$$S = E_i V D^{0.5} (1 + 0.5 \operatorname{Sin} A + \operatorname{Sin} B) \tag{7.2}$$

where E_i = 0.55 for metric units
= 0.30 for USCS
S = minimum upstream water level above top of gate in meters or feet
V = water velocity at gate, m/s or ft/s
D = gate height, m or ft
A = horizontal angle of approach of water to centerline of intake, degrees
B = vertical angle of conduit immediately downstream of gate to horizontal, degrees

Intake Air Vent

The area of the vent should be generous and should be sized on two criteria:

1. Maximum air velocity should be limited to about 80 ft/s (25 m/s) based on a flow of air equal to the rated flow for the turbines.
2. For smaller pipelines the air vent should be sized to allow access for inspection and maintenance. This will require a vent with a minimum diameter in the region of 3.5 ft (1.1 m). Of course, this criterion does not apply to a small penstock, less than about 3 ft (0.9 m) in diameter, where interior access is not possible.

Hydraulics

From a theoretical standpoint, an intake can have a bell-mouth shape. However, flat trash racks in front of a bell-mouth-shaped intake will tend to concentrate the flow toward the center, since the angular approach of the water at the extremities of the rack will produce high losses. The answer is a circular rack cage in front of the bell mouth, but this is expensive. Where a flat trash rack is used, the water acceleration from trash rack to gate must be more gradual. A common rule of thumb [2] is to use an acceleration of 0.5 ft/s per foot length of water passage (0.5 m/s per meter of length).

Intake Concrete

It is possible to design an intake within an earth dam, where the head-pond drawdown does not exceed about one penstock diameter, with a concrete volume of $15Q$, where Q is the maximum rated flow past the intake gate [3]. Hence,

$$\text{Concrete volume} = 15Q \tag{7.3}$$

Penstock

The economic diameter of a penstock is a function of head loss, cost, and the value of energy. A first estimate can be obtained from [4]:

$$D = E_p P^{0.43} H^{-0.57} \tag{7.4}$$

where E_p = 0.49 for metric units
= 3.18 for USCS units
D = diameter, m or ft
P = turbine rated capacity, kW
H = turbine rated head, m or ft

Head Loss

Total head loss from the intake trash racks to the turbine inlet should not exceed about 3 percent for base-load units, or 6 percent for peak-load units.

Water Hammer

Penstocks should be designed to withstand a negative water hammer of about 25 to 40 percent of head, depending on the governor opening time. To avoid negative pressures within the penstock, the grade line should be at least 2 to 3 penstock diameters below the minimum hydraulic gradient.

The positive water hammer will depend on both the set governor time and the type of turbine [5]. The minimum positive water hammer should be 25 percent of minimum gross head for impulse units, and for reaction units should be in the region of 25 to 50 percent of maximum gross head.

Powerhouse

For reaction units [6], the volume of concrete within a powerhouse substructure on a competent rock foundation should be about

$$V = E_h k\,(N + 0.5)d^{2.4} \tag{7.5}$$

where E_h = 1.0 for metric units

= 0.0757 for USCS units
V = volume, m^3 or yd^3
N = number of units
d = turbine throat diameter, m or ft
k = 140 for vertical-axis Francis, propeller, and Kaplan units
= 130 for horizontal-axis tube or bulb units

For high-head horizontal- and vertical-shaft impulse units [7], the concrete volume should be in the region of:

$$V = 50E[h\text{MW}/n]^{0.83}(N + 0.5) \tag{7.6}$$

where E = 1.0 with h in m and V in m^3
= 0.49 with h in ft and V in yd^3
V = concrete volume, m^3 or yd^3
h = turbine rated head, m or ft
MW = generator rating
n = unit synchronous speed, r/min
N = number of units

Where the powerhouse is built on a soft rock or other foundation, concrete substructure volume could increase to about twice the volume estimated with the above formulae.

The superstructure steel within a powerhouse must be designed to support the powerhouse crane. The crane must be capable of lifting the generator rotor, the weight of which [8] can be estimated from:

$$R = 50(\text{MVA})^{0.74}n^{-0.37}[1 + c(J - 1)] \tag{7.7}$$

where R = rotor weight, tonnes or tons
MVA = generator rating
n = synchronous speed, r/min
c = inertia coefficient, which varies from 0 to a maximum of 0.36
J = generator inertia ratio, defined as installed inertia divided by normal inertia

The spacing between units can be determined from the following formula [9].

$$S = 3.6d + E_s \tag{7.8}$$

where E_s = 1.6 for metric units
= 5.0 for USCS units
S = distance between unit centerlines, m or ft
d = turbine throat diameter, m or ft

Another dimension required when sizing a powerhouse is the diameter of the generator casing. This can be estimated from the following equation [10].

$$G = E_g J^{0.115}\text{kVA}^{0.23}n^{-0.575} \tag{7.9}$$

where E_g = 14.4 for metric units
= 47.1 for USCS units
J = generator inertia ratio, defined as installed inertia divided by normal inertia

kVA = generator rating, kilovoltamperes
n = generator synchronous speed, r/min

The required value for the generator inertia J can be determined from the following formula [10]:

$$J = k\text{MW}^{-0.25} n^{-0.125} T_g (1 + T_w/T_e) \tag{7.10}$$

where k = a factor which depends on the type of system and the nature of the load
MW = generator rating, megawatts
n = generator synchronous speed, r/min
T_g = total governor time for opening stroke, s
T_w = water column start-up time, s
T_e = effective governor opening time, s

The value of k varies as follows:

- For a unit which is very small relative to total system capacity and which will not contribute to the control of system frequency, k can be less than 0.55.
- For a unit which will contribute to the control of system frequency on a large system, k should be between 0.55 and 0.82.
- For a unit which will be the main source of frequency control on a small system, k should be between 0.82 and 1.10.
- For a unit which will be the main source of frequency control on a small system with large load swings, k must exceed 1.10.

7.2 CHARLOT RIVER—11-MW NEW HYDROPOWER DEVELOPMENT

Background

The 11-MW Charlot River development is the last in a series of three developments on the Charlot River which drains into Lake Athabasca in the northwest corner of Saskatchewan, Canada (Fig. 7.1). Upstream there is the single-unit 7.2-MW development at Waterloo and a small two-unit 4.6-MW development at Wellington Lake. All developments supplied power to a uranium mine owned by Eldorado Nuclear.

The first power plant at Wellington Lake commenced operation in June 1939 and provided power to a gold mine. The plant was closed in late 1942 and was reactivated in 1949 to power the adjacent uranium mine. As demand for uranium escalated, a second unit was added at Wellington, and the Waterloo unit was commissioned in 1961. The escalating power demand resulted in a decision to develop the remaining head on the Charlot River, hence in 1968 a prefeasibility report was produced for the development, which was followed by a feasibility report in 1969. However, development had to wait for an upturn in the uranium market. Meanwhile, the Wellington forebay was raised by 10 ft (3 m) in 1976, and the Wellington units were upgraded. Eventually, construction of the Charlot project commenced in 1978 and was completed in 1980, only to be shut down two

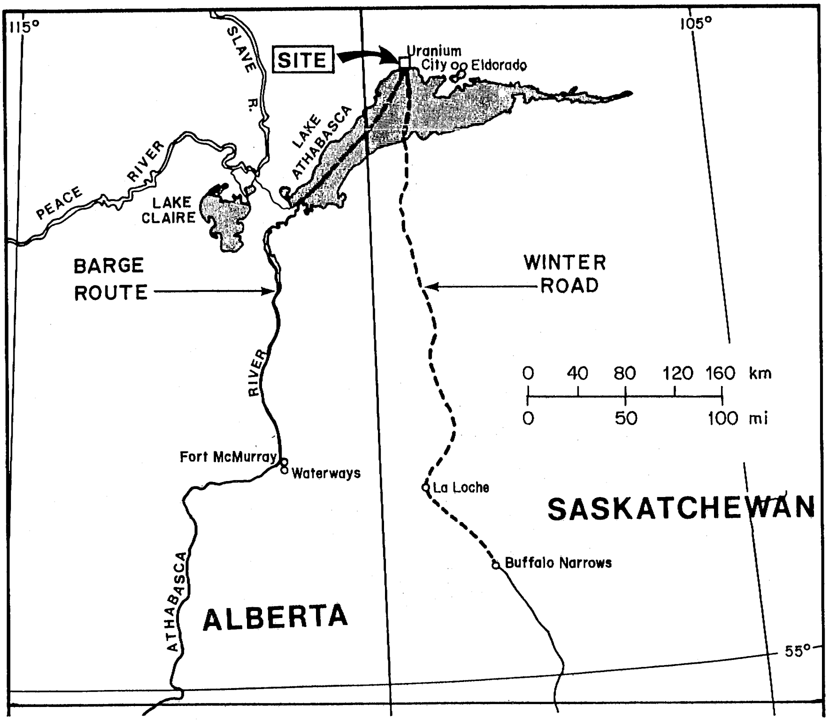

FIGURE 7.1 Charlot River project location.

years later when the demand for uranium collapsed following the incident at Three Mile Island in 1979. The power is now being used by other adjacent mines.

All three projects make use of water diverted from the Tazin River through a 1077-ft-long 12 ft × 16 ft (328-m-long 3.7 m × 4.9 m) tunnel into Mud Lake on the headwaters of the Charlot River (Fig. 7.2). All flood flows are discharged over the Tazin Lake diversion weir, so that only a continuous regulated flow of about 1,100 ft^3/s (31.4 m^3/s) is passed through to the relatively small Charlot watershed.

Feasibility Studies

Between the prefeasibility report issued in February 1968 and start of construction in January 1978, several reports were prepared, which outlined changes in design and project concept.

During 1968, the project was on a fast track, so that in September a site information document was issued and selected contractors were invited to visit the site in October. The feasibility report was issued in December, coincident with a temporary downturn in the uranium market. Despite attempts to downsize the project through staging the development by installing only one unit first, with provisions for the second, further work was halted for about 8 years.

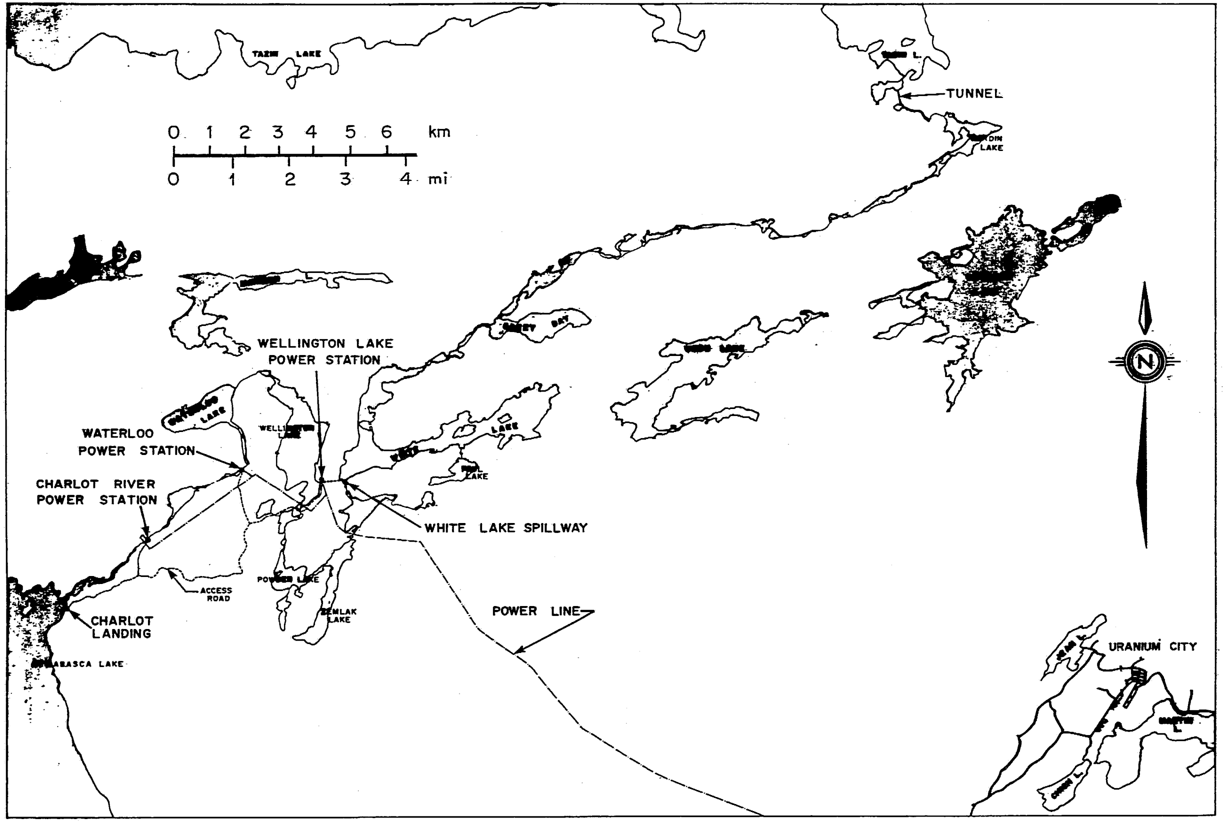

FIGURE 7.2 Site location on the Charlot River.

The project was reactivated in November 1976, and another visit by selected contractors took place in September 1977. The following is a brief outline of the reports produced during this 10-year period.

Prefeasibility Report

Work on the project began in September 1967 with a quick survey to establish ground control for some small-scale mapping using existing photography. Topography with 10-ft (3-m) contour lines was produced at a scale of 1 in–400 ft (1: 4800). Six months later, the prefeasibility report was issued outlining development comprising:

- A 62-ft (18.9-m) high embankment dam.
- 10-ft (3.0-m) diameter, 380-ft (116-m) long diversion tunnel in the left abutment.
- An 80-ft (24.4-m) wide timber stoplog spillway with a capacity of 5000 ft^3/s (143 m^3/s).
- A single-gated intake on the right abutment.
- A 13-ft (4.0-m) diameter penstock, 230 ft (70 m) long.
- An enclosed powerhouse with a 240 r/min 76-ft (23-m) net head Kaplan turbine and 10-MW generator.

Energy output was estimated at 52.8 GWh per annum. Sketches of the proposed layout are included in Figs. 7.3 and 7.4.

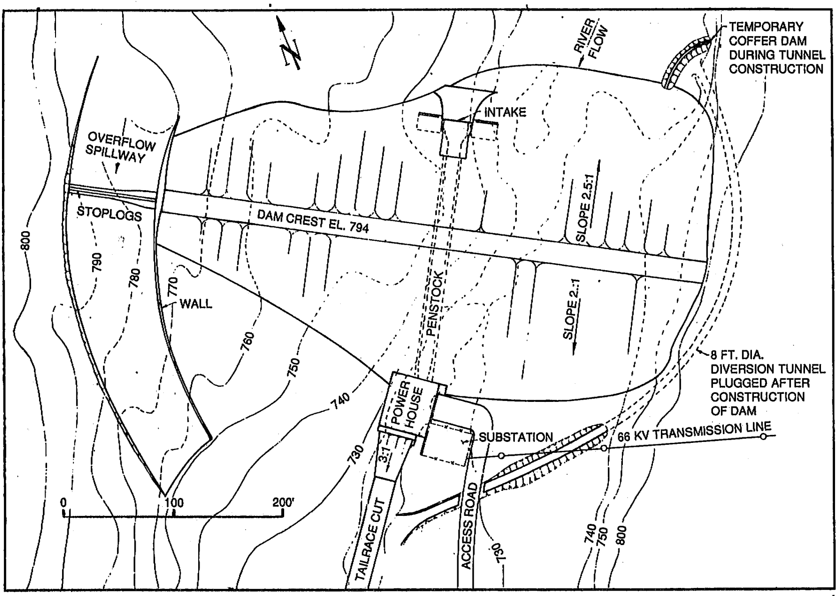

FIGURE 7.3 Prefeasibility project layout for Charlot River.

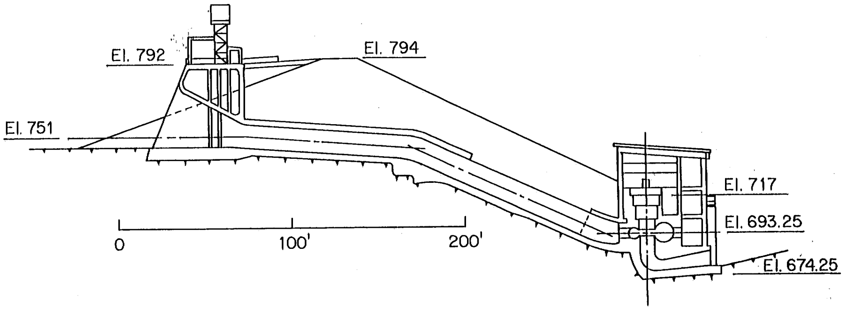

FIGURE 7.4 Prefeasibility Charlot River intake—powerhouse section.

1968 Site Information Document

By August 1968, six months after issue of the prefeasibility report, sufficient information had been received on a new dam site 1500 ft (457 m) further downstream, to outline a development comprising:

- A 94-ft (28.6-m) high embankment dam with a 10-ft (3-m) diameter unwatering conduit through the cofferdam.
- An 80-ft (24.4-m) wide stoplog spillway on the left abutment with a capacity of 5000 ft^3/s (143 m^3/s).
- A single-gated intake structure.
- A single 14-ft (4.3-m) diameter penstock with bifurcation.
- A powerhouse, containing two 5.5-MW Francis turbines.

The site investigation indicated that the bedrock consisted of hard durable granite-gneiss, with a major fault running parallel to the valley in the dam area. Difficulties were expected with the left abutment cliff due to a variable mixture of boulders consisting exclusively of massive rock spalls with cavernous voids.

As for construction materials, an abundance of pervious and impervious materials was found within 2 mi (3.2 km) of the site. This was the concept shown to contractors during their visit in October.

Feasibility Report

During the summer of 1968, the detailed site investigation was undertaken from mid-June through August, which accomplished the following work:

- 1862 ft (570 m) of drilling in 26 holes.
- Seven borrow areas investigated and 56 soil samples obtained.
- 2000-ft (600-m) length of valley was surveyed with cross sections at 25-ft (7.6-m) intervals to above dam crest level.

Due to the extremely rough nature of the terrain, topography was plotted at a relatively large scale of 1 in = 20 ft (1:240).

The feasibility report outlined a development comprising:

- A 100-ft (30-m) high embankment dam, with a 10-ft (3-m) diameter unwatering conduit through the cofferdam.
- An 80-ft (24.4-m) wide timber stoplog spillway with a capacity of 5000 ft^3/s (143 m^3/s).
- A twin-gated concrete intake structure on the right abutment.
- Two steel penstocks, 10 ft (3 m) in diameter, 320 ft (100 m) long.
- An enclosed powerhouse with two Francis units with 5.6-MW generators, operating under a net head of 92 ft (28 m) at 257 r/min.

Energy output was estimated at 61.5 GWh per annum for a plant capacity factor of 0.63. Construction was estimated to require two years.

1977 Site Information Document

For the second contractor's visit in September, a document was produced in which the basic layout as outlined in the feasibility report remained unchanged, except for the unwatering procedure. In previous concepts this would have been accomplished by first diverting flow through a 10-ft (3-m) diameter steel culvert through the cofferdam, and later through one of the 10-ft (3-m) diameter penstock pipes.

A detailed examination of this procedure indicated that it would cause conflict with work in the powerhouse. Hence additional concrete was added to the site quantities to allow for continuation of the culvert in a concrete encasement through the dam core, with a concrete plug below the core, in case this alternative was eventually selected.

Comments on Project Concept Development

The primary requirement was to obtain the maximum power from the remaining head on the river. This simplified project optimization since the upstream water level was fixed at the Waterloo plant tailwater level, the downstream water level was fixed by Lake Athabasca, flow was fully controlled by upstream storage, and finally the required spill capacity was fixed.

In all the project layouts, a balance of rock excavation for the spillway and intake, with dam fill requirements were kept in mind. However, the topography was such that all rock excavations could easily be used in the dam fill. Hence, for this site there are only two main work quantities, total concrete and total dam volume. These quantities are compared in Table 7.1. It is noteworthy that concrete volume remained remarkably steady, despite major changes in layout.

The major increase in dam volume over the prefeasibility estimate is due to moving the dam site about 1500 ft (460 m) further downstream to take advantage of an extra 12 to 16 ft (3.7 to 4.9 m) of head. It was found that the local datum was 9.16 ft (2.79 m) higher than geodetic, and that the level of Lake Athabasca (tailwater) was lower than expected from lower summer flows in the Peace River, due to storage of spring runoff flows at Bennett Dam in British Columbia.

The other change worth noting is in the powerhouse, where two Francis units

TABLE 7.1 Comparison of Quantities

Document	Total concrete, yd^3*	Dam volume yd^3*
Prefeasibility report (March, 1968)	4955	130,000
Site information (August, 1968)	5000	300,000
Feasibility report (January, 1969)	4165	301,000
Site information (September, 1977)	5500	305,000
Final, as built (January, 1980)	6380	294,300

*1 yd^3 = 0.765 m^3.

were substituted for a single Kaplan unit. A detailed cost comparison based on quantities and unit prices indicated a bias of 3.3 percent in favor of a single Kaplan unit, as shown in Table 7.2. However, this analysis did not take account of:

- The higher cost of transporting 14-ft (4.3-m) diameter penstock cans instead of the smaller 10-ft (3-m) cans.
- The additional difficulty associated with transporting larger components for a single-unit installation.
- The additional powerhouse unwatering cost due to the 20-ft (6.1-m) lower draft tube foundation for the Kaplan unit.
- The extra tailrace rock excavation associated with the deeper Kaplan draft tube.
- The higher security of the power supply with a two-unit plant.

TABLE 7.2 Cost Comparison: 1 Kaplan versus 2 Francis

	1968 estimated cost, $1,000	
Item	1 Kaplan	2 Francis
Intake	180	232
Penstock(s)	300	347
Powerhouse	413	355
Crane	60	35
Mechanical equipment	705	545
Electrical equipment	621	840
	2,279	2,354
Total difference $75,000		

It was felt that these factors benefited the two-unit installation to a value which exceeded the $75,000 cost difference, hence two Francis units were recommended. As for the power output, a comparison of the efficiencies indicated a preference for two units except when the total output is between 5.6 and about 8.2 MW, as shown in Fig. 7.5.

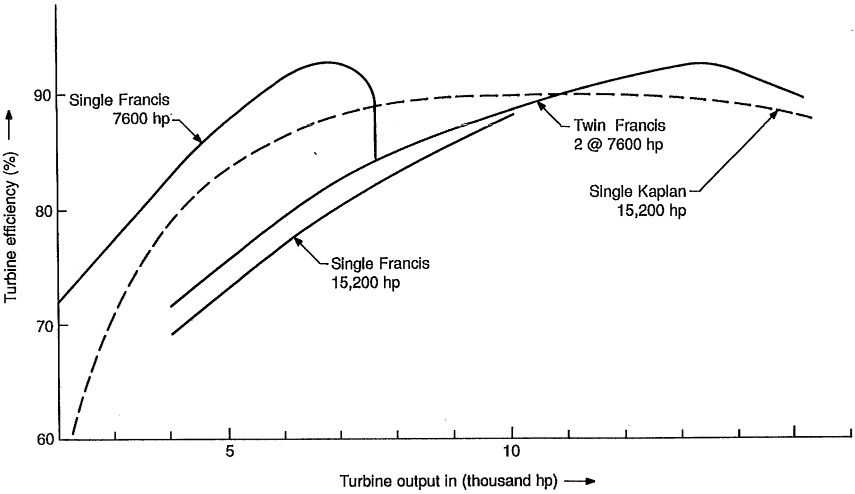

FIGURE 7.5 Comparison of Kaplan and Francis turbine efficiencies.

Computer Program for Feasibility and Design

When this project was designed in the late seventies, computer programs for power-plant costing were not available. Now, the selection of unit size, type, and number of units can be undertaken with the help of, for example, the POWDAC (hydropower design and cost) program developed at Monenco.

This is a computer program developed to provide detailed cost estimates at the prefeasibility level, verification of costs and quantities at the feasibility level, along with optimization of the power conduit and number of generating units. With the basic design data, powerhouse layout can be prepared in sufficient detail for prefeasibility work.

The program has been developed from the Monenco hydraulic design manual as an aid to the design and costing of the power facility. It includes intake structures of trash racks, gates, hoists, and penstocks; surface powerhouse excavation, concrete, and superstructure; powerhouse equipment including turbines, generators, cranes, draft-tube gates, hoists, valves, and dewatering pumps; and electrical costs, including control and communications, auxiliary equipment, step-up transformer, and switchyard.

The program, due to its length and complexity, has been written in a user-friendly format, posing a series of questions, the answers to which are then used to calculate structure quantities and equipment costs.

In order to compare the program output with an actual case history, POWDAC was used to calculate intake, penstock, and powerhouse quantities and costs for a single Kaplan and a two-unit Francis turbine development at Charlot. The results are shown in Tables 7.3 and 7.4.

A review of Table 7.3 indicates a close correlation between the quantities and dimensions calculated by the POWDAC program for two units, and the actual dimensions and quantities for the constructed development. A review of Table 7.4 reveals that the cost changes are all in the same direction when comparing POWDAC and the 1968 estimated costs from Table 7.2. However, there are substantial differences between some items, such as the intake and powerhouse me-

TABLE 7.3 Comparison of Dimensions and Quantities Computed by POWDAC and As Built

Item, unit*	POWDAC 1 unit	POWDAC 2 units	As built 2 units
Intake			
Concrete, yd^3	1,191	1,383	1,220
Trash rack, ft^2	286	286	340
Bulkhead width, ft	14.4	10.8	10.0
Bulkhead length, ft	19.0	14.1	14.2
Gate width, ft	13.1	9.8	10.0
Gate length, ft	16.4	12.1	11.5
Penstock			
Diameter, ft	14.8	10.8	10.0
Weight, lbs	345,000	455,000	272,439†
Powerhouse			
Concrete, yd^3	2,588	1,842	1,840
Re-bar, lb	240,000	170,000	145,000
Formwork, ft^2	31,950	22,650	20,495
Formwork, curved 1 way, ft^2	1,478	1,672	1,506
Formwork, curved 2 way, ft^2	569	600	602
Superst. steel, lb	152,000	128,000	143,000
Crane cap., lbs	121,000	70,400	83,600
Crane span ft	37.0	29.5	28.2
Misc. steel, lbs	37,400	33,000	41,800
Roof area, ft^2	5,487	5,594	4,810
Wall area, ft^2	8,284	7,423	7,728
Draft tube gate No.	2	2	2
width, ft	13.4	9.8	19.7
height, ft	11.1	8.2	6.6
No. and type of units	1 Kaplan	2 Francis	2 Francis
Speed, r/min	257.1	300.0	257.1
Throat diam., ft	8.36	6.07	5.83
Plant output, kW	10,800	11,200	11,200

*Metric conversion: 1 ft = 0.305 m; 1 lb = 0.454 kg; 1 yd^3 = 0.765 m^3.
†For 2 steel penstocks 198.5 ft long; remainder in concrete.

chanical equipment. The overall cost comparison still indicates that the cost difference is within the accuracy of the estimate.

Now, a few comments on design of the project.

Dam and Diversion Design

A general layout is shown in Fig. 7.6, and a section of the dam is shown in Fig. 7.7. The embankment slopes are slightly steeper than those detailed in the feasibility report. During construction, the left abutment talus deposit of large boulders caused some problems [11]. Rather than remove this deposit, the abutment contact was moved about 120 ft (37 m) further downstream. Also, the sandy silt used for the core material was found susceptible to frost lensing and cracking; hence special design measures were used in the area of the dam crest, as outlined

TABLE 7.4 Estimated and POWDAC Cost Comparison as Percentage Change Two Units over One Unit

Item	1968, % cost change	POWDAC, % cost change
Intake	+ 28.9	+ 14.3
Penstock(s)	+ 15.7	+ 27.5
Powerhouse	− 14.0	− 24.3
Crane	− 41.7	− 28.9
Mech. equip.	− 22.7	− 1.5
Elect. equip.	+ 35.3	+ 28.7
Total	+ 3.3	+ 5.6

by Solymar [12]. The fault in the dam was found to be 25 ft (7.6 m) deep, and was backfilled with lean concrete.

A 10-ft (3-m) diameter corrugated steel multiplate Armco culvert pipe was used to divert the flow of 1500 ft^3/s (43 m^3/s). Due to control of flows by the upper power developments, diversion was easily accomplished. The culvert pipe was encased in concrete through the upstream embankment shell and core. A simple steel drop gate was used to shut off flow after the dam was completed. The culvert pipe was left empty upstream of the core; through the dam core it was filled with concrete and grouted through the dam core and filled with sand downstream of the concrete plug.

Diversion capacity was based on 90 percent of full load from the upstream Waterloo plant in winter. For spring and summer it was possible to reduce Wa-

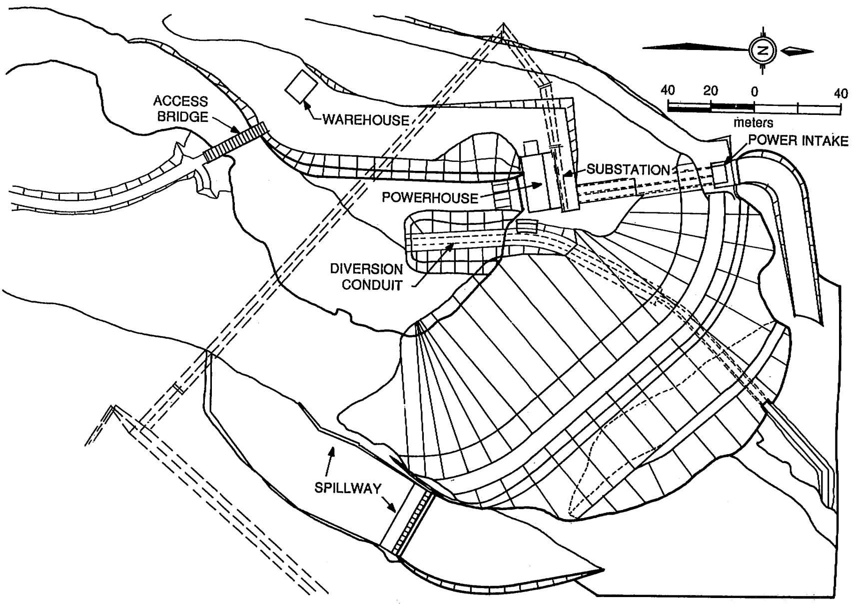

FIGURE 7.6 Final Charlot project layout.

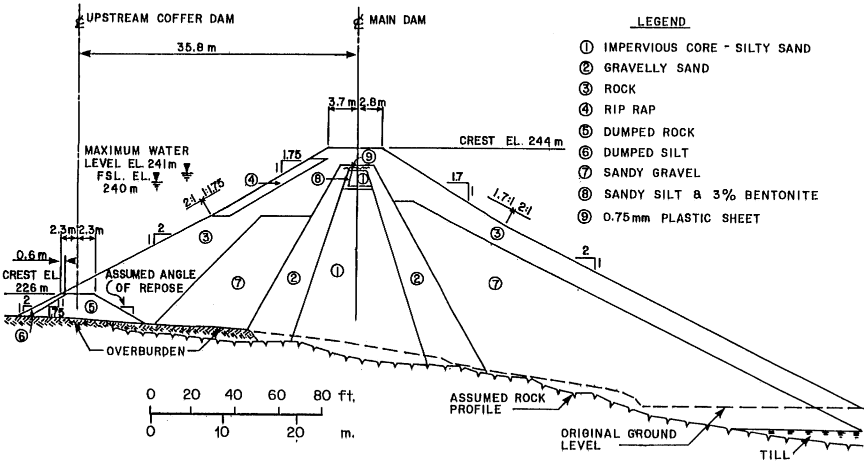

FIGURE 7.7 Charlot Dam section.

terloo plant discharge to provide capacity to pass the spring snowmelt or any rainstorm floods. In case flows exceeded this amount, a section of the cofferdam was left 12 in (0.3 m) lower, and the downstream slope covered with heavy riprap to act as an emergency spillway. This was used during the winter of 1978–1979, when demand for power was higher than expected. Minimal damage resulted.

Spillway Design

The simple stoplog spillway has a capacity of 5000 ft^3/s (143 m^3/s). Plant operators have been very successful with water management, hence it was expected that the spillway would rarely be used. Further details can be obtained from Bakar [13]. A small 1:100-scale hydraulic model cut in urethane foam was used to check the configuration of the downstream channel and waterfall back into the river. Based on the model results, the shape of the downstream rock channel was slightly modified to ensure that spilled waters did not erode the downstream toe of the dam. As mentioned previously, all excavated rock was used in the dam.

Intake Design

This is a simple bell-mouth-shaped concrete structure with gates and hoists supported above deck level by a steel frame. One of the first dimensions needed in designing an intake is the diameter of the penstock.

For Charlot the diameter based on Eq. (7.4) is

$$D = 3.18 \times 5600^{0.43} \times 92^{-0.57} = 9.88 \text{ ft} \qquad \text{use } 10.0 \text{ ft}$$

For a base-load power plant, as at Charlot, it is preferable to install a slightly larger penstock to reduce losses. For a peak-load power plant, with a load factor less than about 30 percent, a slightly smaller penstock would have been selected. With the penstock diameter, the gate size selected would be about 10 × 11.5 ft (3.0 × 3.5 m) high.

To obtain the gate velocity, flow must be known and can be determined from the well-known equation:

$$Q = 11.8 \text{ kW } H^{-1} e^{-1} \qquad \text{(USCS)}$$

$$Q = 0.102 \text{ kW } H^{-1} e^{-1} \qquad \text{(metric)}$$

where Q = turbine flow, in ft³/s (m³/s)
H = turbine rated head, ft (m)
e = combined turbine-generator efficiency (fraction)
kW = turbine output, kilowatts

For Charlot,

$$Q = 11.8 \times 5600 \times 92^{-1} \times 0.91^{-1}$$
$$= 790 \text{ ft}^3\text{/s}$$

Hence for a 10-ft-wide by 11.5-ft-high gate, velocity will be:

$$V = 790/(10 \times 11.5)$$
$$= 6.86 \text{ ft/s}$$

For the intake, angle A = 90° (Fig. 7.7) and angle B = 4.3° (Fig. 7.8), hence submergence computed from Eq. (7.2) is:

$$S = 0.3 \times 6.86 \times 11.5^{0.5}(1 + 0.5 \times 1 + 0.075)$$
$$= 10.52 \text{ ft (minimum)}$$

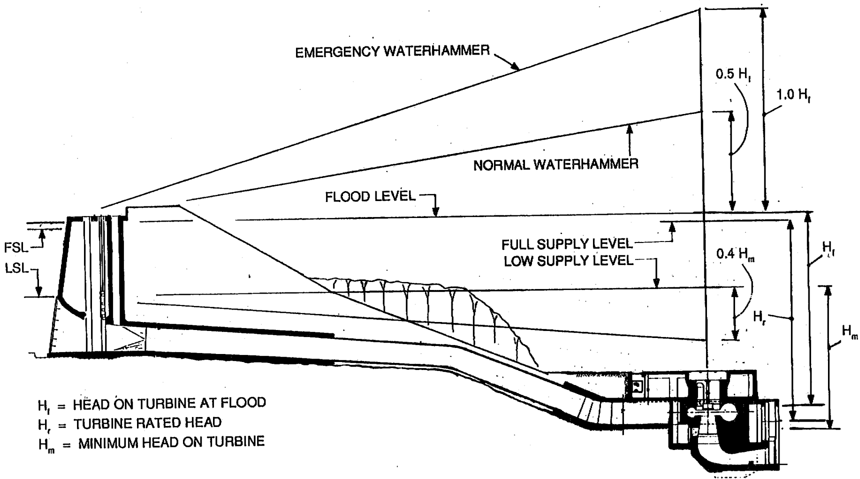

FIGURE 7.8 Schematic of penstock design water hammer.

Gate sill elevation can then be set as follows:

Low supply level	758.0
Less submergence	−10.5
Less gate height	−11.5
Gate sill level	736.0 (max.)

The sill was set just below this level at elevation 735.0.

At the entrance to the intake, a set of trash racks will be required. For an uncleared reservoir, the velocity through the gross area of the racks, based on Eq. (7.1), should be:

$$V = 0.52 \times 1.0(1.6 - 0.63)(1 + 0/250)(5600)^{0.2}$$
$$= 2.83 \text{ ft/s}$$

With a flow of 790 ft^3/s, rack area should be 790/2.83 = 280 ft^2. However, a larger area of 340 ft^2 was provided to extend the times between rack cleanings at this remote plant.

Trash-rack spacing between bars varies from 2.25 to 3.5 percent of turbine throat diameter, depending on the type of turbine and the manufacturer's experience.

At Charlot, the turbine throat diameter is 5.83 ft (1.78 m). Rack spacing would thus be in the region of 1.57 to 2.44 in (3.9 to 6.1 cm). Using a ⅝-in (1.6-cm) thick rack bar, the center-to-center spacing would be 2.25 to 3 in (5.6 to 7.5 cm). Based on advice from the turbine manufacturer, 3 in was selected. Due to the possibility of blockage by frazzle ice, the racks were designed for full blockage with the steel stressed to 90 percent of yield.

Volume of a two-unit intake, based on Eq. (7.3), would be:

$$\text{Concrete volume} = 15 \times 2 \times 790/27$$
$$= 880 \text{ yd}^3 \ (805 \text{ m}^3)$$

Total volume of concrete poured in the intake was about 1200 yd^3 (1010 m^3).

Referring to Fig. 7.9, it will be noted that the intake has an upstream sealing bulkhead gate, downstream sealing service gates, and full-width air vents large enough to permit access by ladder to the penstock. Since there is no turbine valve, the intake service gate is the second line of defense on flow control, hence it is capable of rapidly closing against the full turbine flow. Its hoist is placed upon a steel tower so that the gate can be lifted above deck level for inspection, maintenance, and repair.

Penstock

Initially the penstocks were to be fabricated in mild steel. However, a cost analysis indicated that for the first 70 ft (21 m), where the head is minimal, penstock steel thickness would be governed by minimum thickness requirements, and that it would be more economic to use twin reinforced-concrete box culverts. The steeper section down the slope to the powerhouse has two 10-ft (3-m) diameter steel pipes to the turbine spiral cases.

The penstock was designed for the following conditions [5] as shown in the schematic in Fig. 7.8.

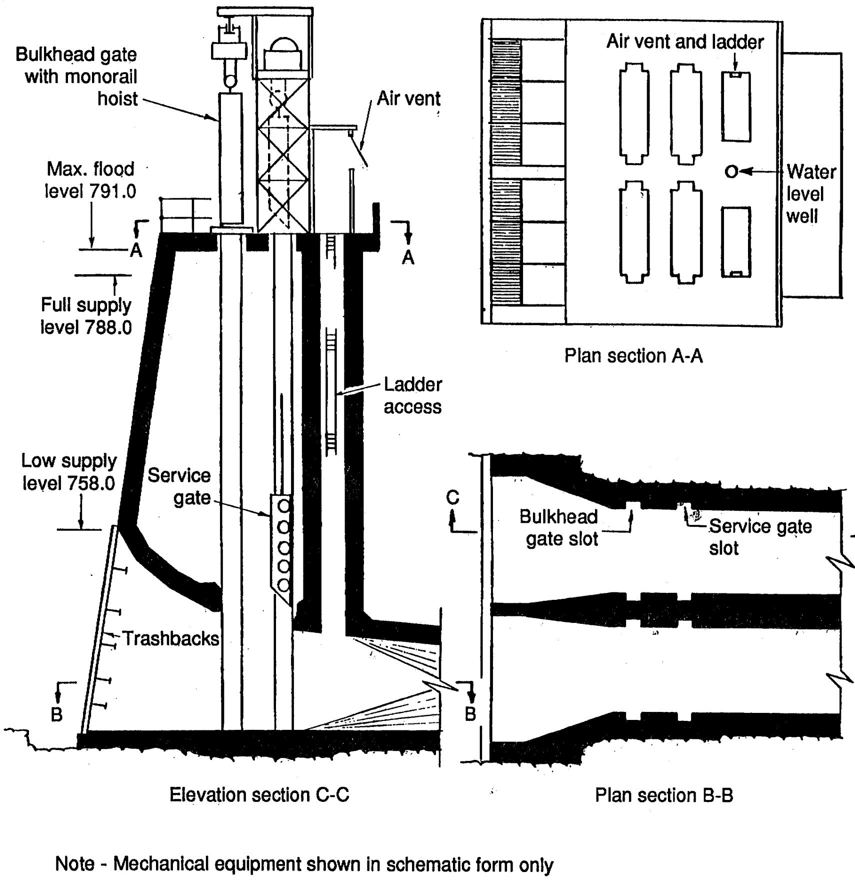

FIGURE 7.9 Charlot intake plan and sections.

- External pressure from the backfill with the penstock empty.
- Normal water hammer of 50 percent, with the allowable steel stress to be the lower of 60 percent of yield or 38 percent of tensile.
- Emergency water hammer of 100 percent, with allowable steel stress to be the lower of 96 percent of yield or 61 percent of tensile.
- A negative water hammer of 40 percent, with the penstock located about one pipe diameter below the minimum surge line.

The penstocks were buried to insulate the water from cold winter temperatures.

Powerhouse Design

The powerhouse contains two vertical Francis turbines with a capacity of 7500 hp (5600 kW) at 92-ft (28-m) rated head. Each is controlled by a mechanical gate shaft governor with black-start capability by manual opening of the wicket gates. Generators are rated at 5700 MVA at 0.9 power factor, with a winter load rating of 6550 MVA, and are equipped with on-shaft exciters. Terminal voltage is 6.9 kV, stepped up to 66 kV for transmission. The power plant operates unattended, with remote control from the mine by power-line carrier.

A plan and section are shown on Figs. 7.10 and 7.11. This is a typical powerhouse with a concrete substructure and insulated steel-siding superstructure. The generators are air-cooled with air drawn in from behind the powerhouse to below the generator, and then discharged through exhaust louvers in the walls and vents in the roof. Experience has indicated that air intakes should not be placed on the tailrace side of the building because of frost condensation and icing problems in winter.

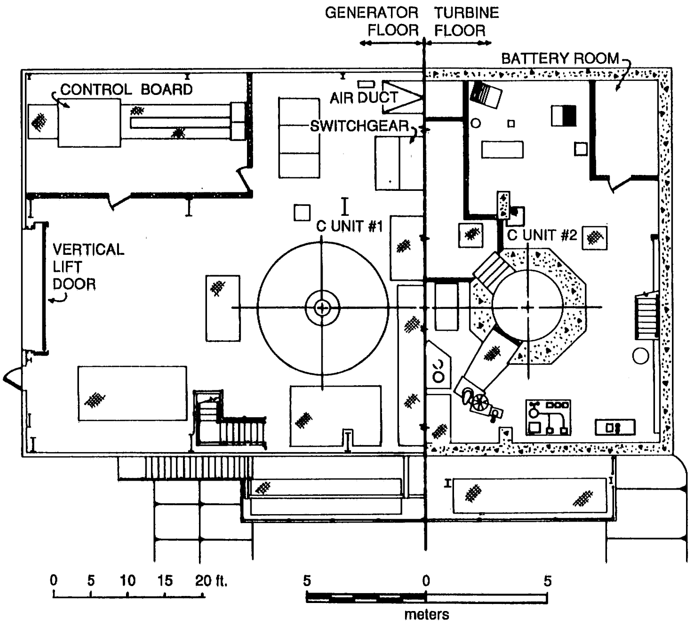

FIGURE 7.10 Final Charlot powerhouse plan.

Another point to note is that water piping and drains must not be embedded in, nor be in contact with exterior concrete, otherwise they will freeze in the cold northern climate. If water piping must traverse concrete, it should be heat-traced.

For the two-unit Charlot powerhouse, with a turbine throat diameter of 5.8 ft (1.8 m), the concrete volume computed from Eq. (7.5) is

$$\text{Volume} = 10.6(2 + 0.5)5.83^{2.4}$$

$$= 1823 \text{ yd}^3 \text{ or } 1395 \text{ m}^3$$

Actual concrete poured was 1860 yd^3 (1423 m^3).

It is interesting to note that if a single unit of 11.0-MW capacity had been used,

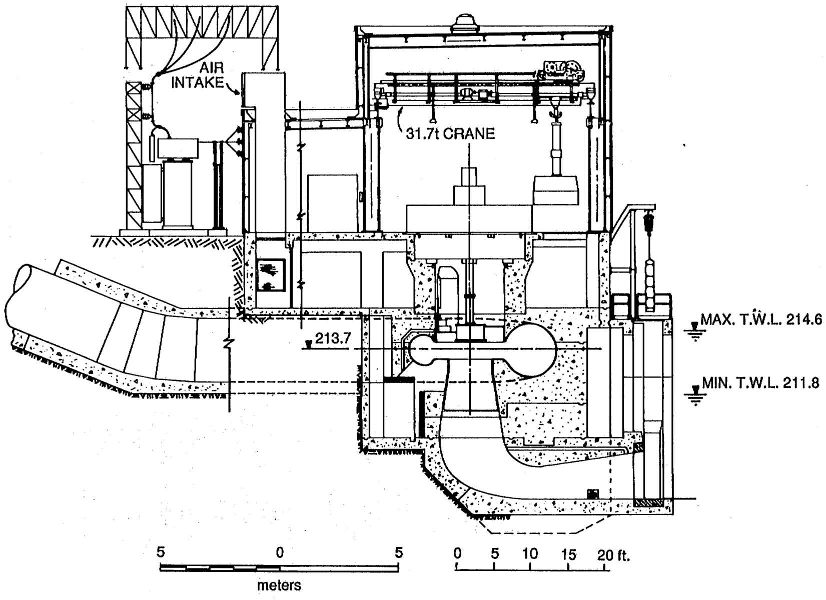

FIGURE 7.11 Final Charlot powerhouse section.

turbine throat diameter would be about 8.24 ft (2.5 m) (5.83 × 20.5) and powerhouse concrete volume would be:

$$\text{Volume} = 10.6(1 + 0.5)8.24^{2.4}$$

$$= 2510 \text{ yd}^3 \text{ or } 1920 \text{ m}^3$$

This is an increase of 38 percent, and is one of the reasons why two units were selected; less expensive on-site work in a costly environment.

Another factor which mitigates against a single unit is the large increase in crane capacity compared with a two-unit installation. An estimate of this increase in capacity can be obtained from the equations in Sec. 7.1 for which the unit speed must be known. Assuming that the turbine will have the same head and specific speed, the unit rotation speed can be calculated from information given in Chap. 3:

$$n_2 = n_1(P_1/P_2)^{0.5}$$

where n_1, n_2 = rotation speed of units 1 and 2
P_1, P_2 = power of units 1 and 2

For twice the power, $P_1/P_2 = 0.5$, hence the unit speed is

$$n_2 = 257.1 \times 0.5^{0.5} = 181.8 \text{ r/min}$$

Use 180 r/min, the nearest synchronous speed.

The units have 185 percent extra inertia, hence the inertia ratio J, defined as the ratio of installed inertia divided by standard inertia, will be 2.85.

Thus the rotor weight for an 11.4-MVA generator as computed from Eq. (7.7) is

$$R_w = 50 \times 11.4^{0.74}\ 180^{-0.37}\ [1 + 0.36(2.85 - 1)]$$
$$= 73.8 \text{ metric tonnes (72.6 tons)}$$

This is 91 percent more than the rotor weight of a 5.7-MVA unit (calculated later); hence there would be a large increase in crane cost and in the superstructure steel required to support the crane.

For Charlot, spacing between the units is calculated from Eq. (7.8) to be

$$S = 3.6 \times 5.83 + 5 = 25.99 \text{ ft}$$

Actual spacing is 26 ft.

The diameter of the generator casing, using Eq. (7.9), is estimated to be

$$G = (47.1 \times 2.85^{0.115})(5700^{0.23})(257.1^{-0.575})$$
$$= 15.97 \text{ ft (4.87 m)}$$

Actual casing diameter is 17 ft (5.2 m).

Turbine Speed Regulation

All three power plants generated electricity for a uranium mine, mill, and adjacent town. There was no other source of power, other than emergency diesels. The mine was equipped with several high-powered shaft hoists which ran through a cycle of power demand on hoist start and acceleration, followed by power generation on deceleration and stopping. On the small isolated hydropower system, this large and continuous variation in power demand resulted in large frequency variations in the order of ± 0.5 to 1.5 Hz. Extra rotating inertia and fast governor times were the only solution. Accordingly, 185 percent extra inertia was installed, which resulted in a 65 percent increase in rotor weight. Pertinent statistics and fastest allowable governor times were

Machine start time: $T_m = 7.63$ s. Effective governor $T_e = 3.00$ s.
Water start time: $T_w = 1.05$ s. Total governor time $T_g = 4.00$ s.

For Charlot, Eq. (7.10) was used to estimate inertia requirements giving

$$J = k \times 5.6^{-0.25} \times 257^{-0.125} \times 4.0(1 + 1.05 \times 3.0^{-1})$$
$$= 1.75k$$
$$= 1.75 \times 1.1 = 1.92 \text{ (minimum)}$$

Due to the very severe load swings being experienced on the system, further studies indicated that 50 percent more inertia would be needed to give a ratio of

$$J = 1.92 \times 1.5 = 2.85$$

If the power plant was instead interconnected to a large system, frequency control would not be a concern, hence the inertia ratio would be

$$J = 1.75 \times 0.55 \text{ (min.)} = 0.96 \text{ (minimum)}$$

In other words, a standard amount of inertia (J = 1.0) would suffice.

This extra inertia will increase the weight of the rotor, and hence will affect the capacity of the powerhouse crane.

The rotor weight for a 5.7-MVA generator was estimated from Eq. (7.7) as

$$
\begin{aligned}
R_w &= 50 \times 5.7^{0.74}(257.1^{-0.37})[1 + 0.36(2.85 - 1)] \\
&= 38.7 \text{ tonnes } (38.1 \text{ tons})
\end{aligned}
$$

The actual rotor weight is approximately 38 metric tonnes (37 tons). However, it should be noted that the rotor weight could be lower if the generator manufacturer elects to increase the diameter more than the rotor weight to obtain the same extra amount of WR^2 (weight times radius of gyration squared).

During commissioning, full-load rejection tests were undertaken. Speed rise was 32 percent, and water hammer 35 percent. Governor time was 6.4 s (effective) and 12 s total. Cushioning on closing starts at 11 percent gate opening and takes 3.4 s. On opening, cushioning starts at 88 percent gate opening and takes 5 s. No attempt was made to optimize these times.

Concluding Remarks

The project has had an interesting development history, with an initial rush, followed by cancellation, and then a second rush to finish the project as soon as possible. From an examination of the documents produced during the life of the project, one clear fact emerges: the initial assumptions should be conservative; otherwise major cost increases will occur when designers are faced with reality. Such was the case at Charlot. Conservative estimates of work volumes were made, even at the prefeasibility stage. For example, the dam volume did not change significantly, despite changes in section and location of the left abutment contact. However, in order

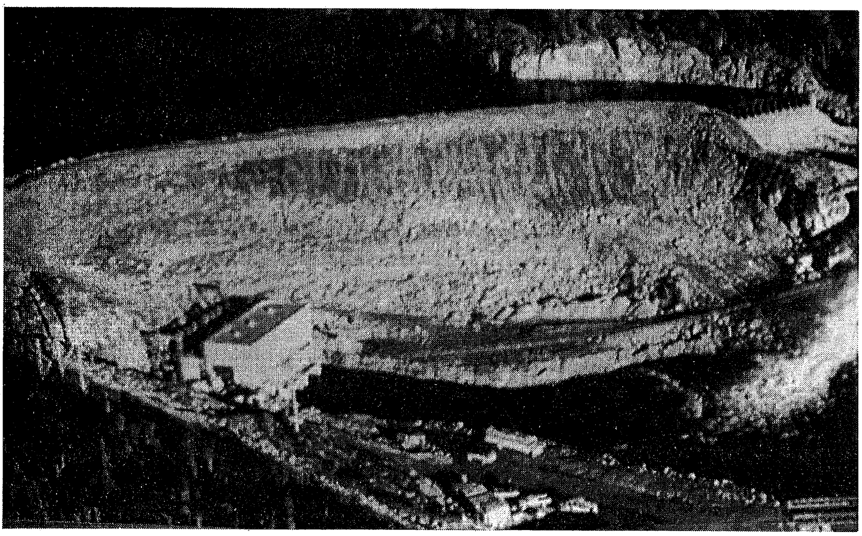

FIGURE 7.12 Aerial view of completed project at Charlot shortly after commissioning.

to keep quantities down, the design was refined and slopes were steepened slightly. An aerial view of the finished development is given in Fig. 7.12. The project was completed on schedule, and just within the budget of $25,000,000.

7.3 MORRIS—1100-kW ADDITION TO AN EXISTING FACILITY

Background

The Morris power plant harnesses 95 ft (29 m) of head between two storage ponds which form part of the Mobile development on the Mobile River (shown in Fig. 7.13), some 25 mi (40 km) south of St. John's, Newfoundland. The development was first proposed in 1949 during construction of the 9500-kW Mobile project, but was abandoned as uneconomical. The feasibility of the project was reassessed in 1981, when the Canadian government instituted an "off oil" incentive program which allowed a fast tax write-off of any new hydropower generation not exceeding 15 MW, or alterations of existing hydropower facilities which provide an increase in capacity not exceeding 15 MW. Equipment tenders were issued in 1982, and construction of the intake, penstock, and powerhouse commenced in April 1983. The unit was placed in service in December of the same year. Figure 7.13 shows the completed power plant.

When the Mobile development was built in 1949–1951, it was intended to harness head between Mobile Big Pond and Mobile First Pond. A 6900-ft (2100-m) long canal from a diversion weir below the outlet of Mobile Big Pond to the intake site had been built in 1948 to divert Mobile River water southward into the adjacent Tors Cove River watershed where there was an existing power plant.

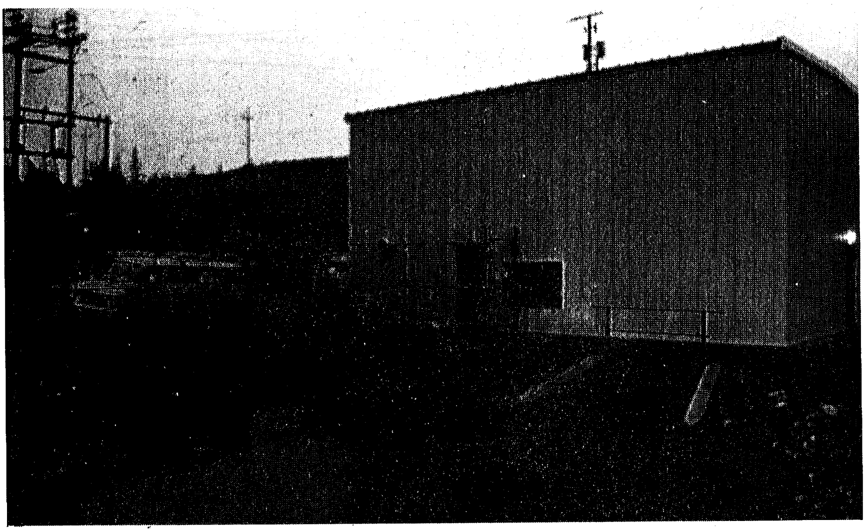

FIGURE 7.13 Spillway on left bank is in operation. Morris powerhouse and substation. View from left bank of tailrace.

With construction of Mobile, flow was returned to the Mobile River, and the canal was unused and left empty.

In 1981 a feasibility report was prepared which recommended:

- Cleaning and enlargement of the canal
- Construction of a concrete intake structure
- Construction of a 5.5-ft (1.7-m) diameter, 805-ft (245-m) long penstock to the powerhouse

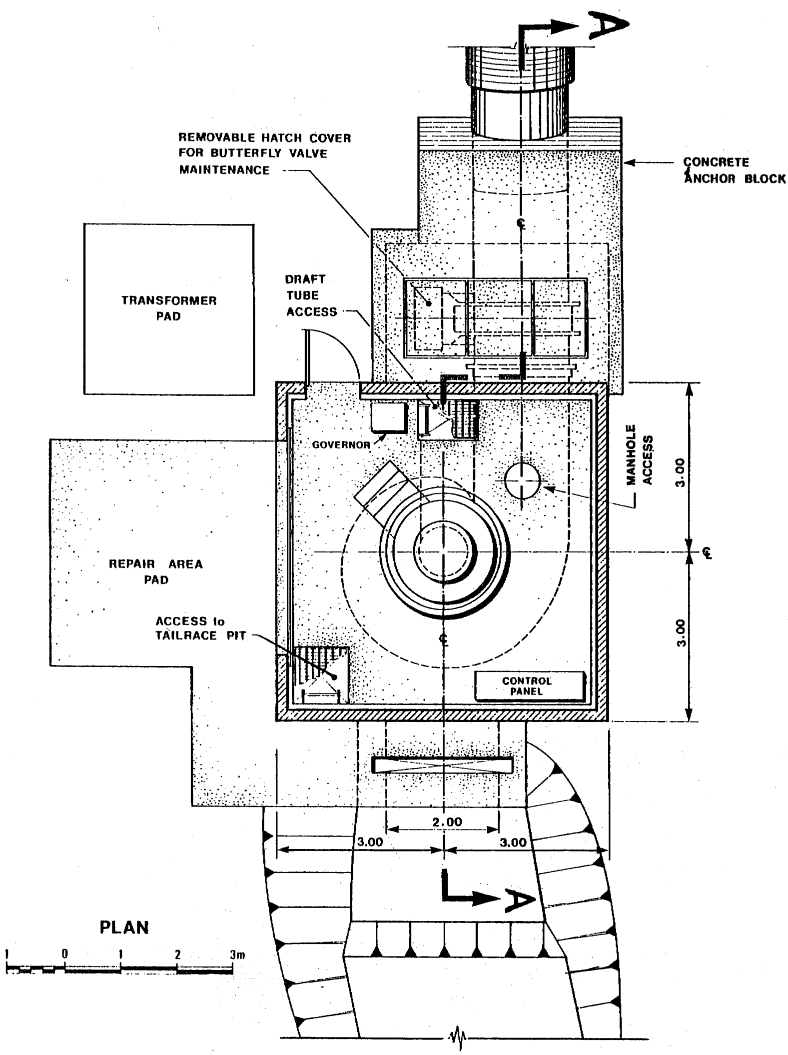

FIGURE 7.14 Morris project. Feasibility report powerhouse plan and section.

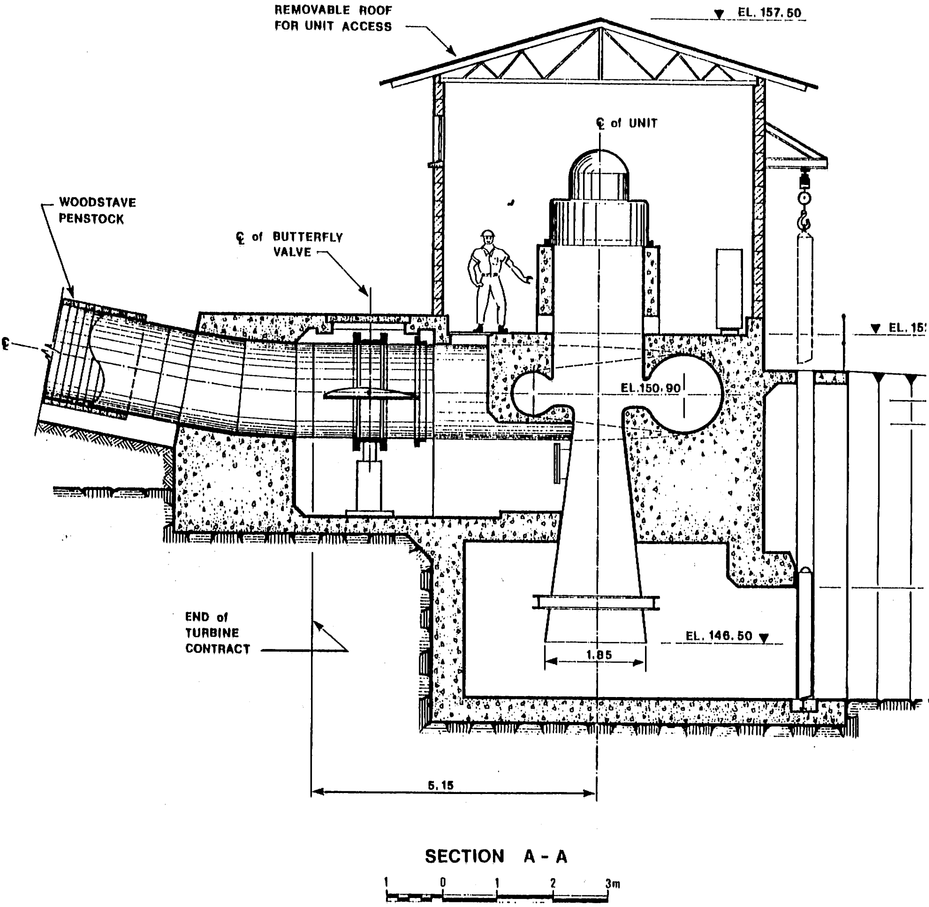

FIGURE 7.14 *(Continued)*

- Construction of a powerhouse to contain a single vertical-shaft unit (Fig. 7.14), based on a regulated flow of 150 ft^3/s (4.3 m^3/s) which would support an installation of 1000 kW.

Diversion Canal

Over the years the canal had silted up and become overgrown with alder bushes and pine trees. The new design called for a canal with a base width of 18 ft (5.5 m), side slopes of 2:1 and a grade of 2.5 in 10,000, designed to carry 150 ft^3/s (4.3 m^3/s) at a depth of 5.5 ft (1.7 m). The canal had to be deepened by about 2 ft (0.6 m) near the intake. During construction all material excavated from the canal was dumped on top of and on the downhill side of the side-hill canal embankment. Most of the material excavated from the canal was gravel and glacial till.

When the canal was filled, several leaks in the embankment appeared. The canal was dewatered, and an impervious blanket placed on the floor and dike slope in the leakage area. On refilling, all leaks had stopped except one where the canal traverses an old stream bed. Attempts with bentonite failed to stop this leak. It is currently running clear and is being monitored.

Intake

This is a simple concrete box structure containing stoplogs and trash racks over a 9-ft, 6-in (2.9-m) square opening shown in Fig. 7.15. Also, there are provisions for the addition of a gate. A timber gatehouse completes the structure. For economic reasons no attempt was made to streamline the water passages. The gatehouse also shelters a telephone and water-level transmitter used for turbine control.

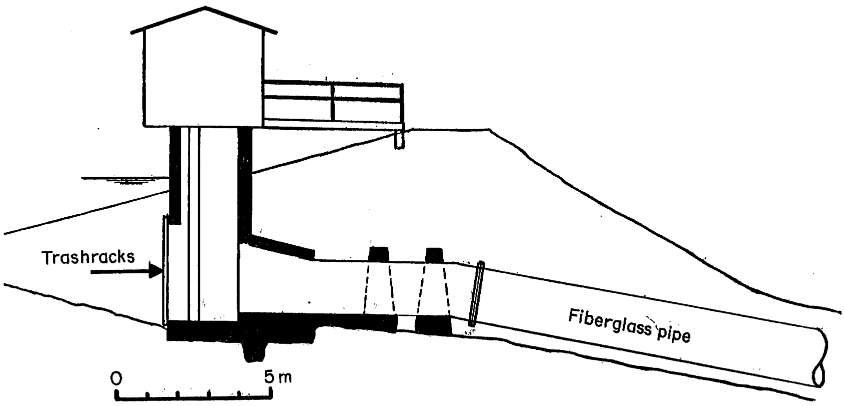

FIGURE 7.15 Morris intake—as-built section.

Penstock

Test pits along the penstock route indicated that the penstock could easily be buried in the 7 to 10 ft (2 to 3 m) of glacial till and sandy gravel predominant in the area. Accordingly, it was decided to have the general contractor prepare the penstock grade and place the backfill. Specifications for supply and erection supervision of the penstock permitted the bidder to select the pipe material. Penstock size, length, water-hammer head (25 percent), and allowable stresses were all specified for steel, woodstave, polyethylene, and fiberglass pipe. Bids were received for steel, woodstave, and fiberglass pipes, with the latter being the low bid by a small margin.

The fiberglass penstock is connected to the concrete intake by means of a steel thimble. Around the thimble there are two concrete cutoff walls shown in

Fig. 7.15 to inhibit seepage along the steel pipe-dike material interface. At the downstream end, the fiberglass pipe is butt-joined to the turbine inlet pipe with fiberglass overlay and two steel bands.

The fiberglass pipe was delivered in 40-ft (12.2-m) can lengths, and joined with a double O-ring rubber bell and spigot joint. Erection of the pipe proceeded without incident, despite an intense rainstorm which flooded portions of the grade. During backfill operations, site staff meticulously ensured that no sharp rocks or boulders came in contact with the pipe. The pipe was buried to a depth where the top is level with the ground, and then 12 in (0.30 m) of gravel was mounded up over the pipe. Construction equipment is not allowed to traverse the pipe.

Powerhouse

This is a simple concrete slab, steel superstructure, aluminum siding building with no insulation as shown in Figs. 7.16 and 7.17. The turbine-generator, valve, and column footings are all founded on rock. The repair bay is a slab-on-grade. Tailwater level is only about 2 ft (0.6 m) below the powerhouse floor, with increased levels controlled by the downstream dam. This facilitated the change to a horizontal-shaft unit. The powerhouse is equipped with a monorail hoist suspended over the unit centerline to ease equipment installation and maintenance.

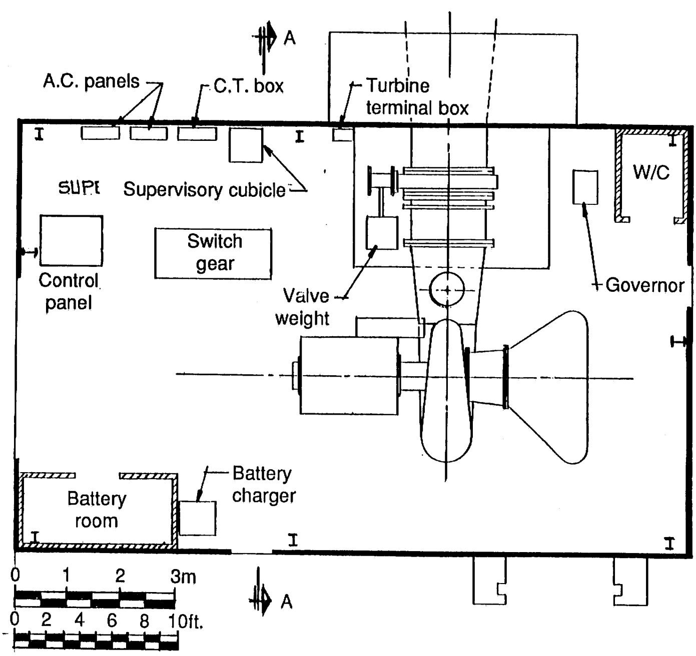

FIGURE 7.16 Morris powerhouse as-built plan.

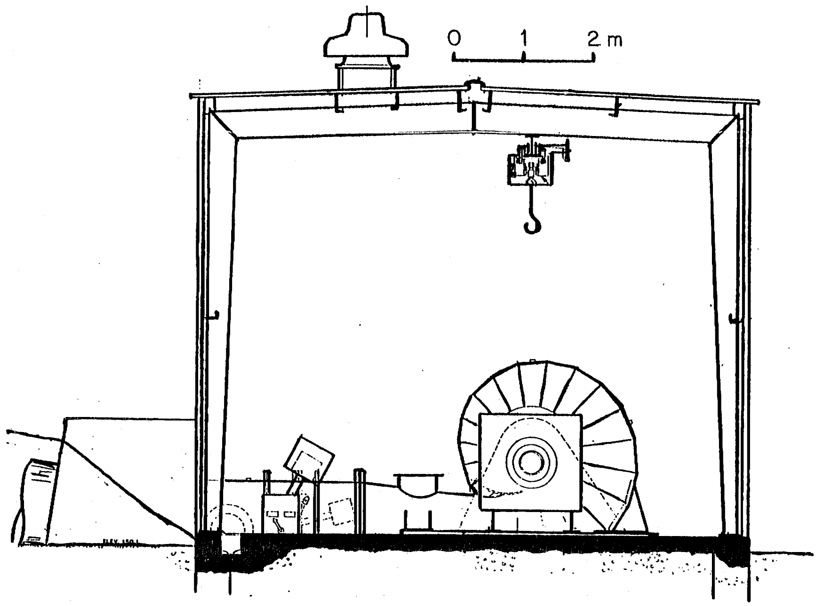

FIGURE 7.17 Morris powerhouse as-built section *AA*.

Tailrace

Since the project will divert water away from the Mobile River, fish spawning habitat will be lost. Accordingly, the Canadian Department of Fisheries and Oceans agreed to the construction of a fish spawning channel to be built into the tailrace. It is simply two shallow gravel bed channels with a control weir at the outlet.

Equipment

The powerhouse contains a single horizontal shaft 1100 kW, 600-r/min, 94-ft (28.6-m) net head Francis turbine, and 1436-kVA induction generator. Upstream of the turbine there is a 54-in (1.37-m) diameter flow-through butterfly valve with a gravity-closure counterweight. The butterfly valve is opened by means of an oil-pressured servomotor, and closed by weight for safety in case of loss of station service power. The equipment can be operated by remote control from St. John's. With an induction generator there is no governor. The wicket gates are moved by an electric-powered "controller" to whatever position is called for by the operator. It is interesting to note the change in powerhouse layout with a horizontal-axis unit (Figs. 7.14, 7.16, and 7.17). This required a larger powerhouse, but provided a more accessible unit.

The unit was commissioned in December 1983, after resolving some problems with bearing movement and backlash in the wicket gate controller. The unit is put on-line by first opening the turbine valve, and then opening the wicket gates to about 15 percent gate opening. Shortly after the induction generator starts to produce power, the wicket gates are opened to the desired position, which is usually about 85 to 90 percent of rated load. Pertinent statistics for the unit are

What start time T_w = 1.69 s
Machine start time T_m = 0.82 s
Valve opening/closing times = 130 s
Wicket gate opening/closing times = 22 s
Speed rise on full-load rejection = 70 percent
Water hammer on full-load rejection = 24 percent
Bypass valve opening/closing times = 30 s

When the unit was commissioned, a backlash problem caused by slack in the gearing became evident in the wicket gate operator. This was solved by slowing the wicket gate movement down from 22 s to 40 s.

The project was completed on schedule, and well within the budget of $2,500,000.

Concluding Remarks

The design was kept simple, and the structures were deemed to be sufficiently optimized when concrete volume for the intake approached 15 ft^3 of concrete per cubic foot of flow (15 m^3 per cubic meter of flow) and for the powerhouse as outlined in Ref. 7.

7.4 TOPSAIL—INSTALLATION OF LARGER UNIT INTO EXISTING POWERHOUSE

Background

The Topsail development, about 12 mi (20 km) west of St. John's, was commissioned in 1931. The development comprised several timber crib dams for storage, with the head pond controlled by a small timber dam weir. A small concrete intake with a timber gate directed water into a 36-in (0.9-m) diameter, 6250-ft (1900-m) long woodstave pipe down to the powerhouse near the shore. Installed capacity was 1250 kW [but was limited to 1000 kW by the 3-ft (1-m) diameter penstock] in one horizontal-shaft Gilbert-Gilkes-Gordon turbine with relief valve, flywheel, and generator.

By 1976 it was realized that the woodstave pipe was reaching the end of its service life. Moreover, water records indicated that there was a considerable amount of spill, and therefore lost generation. A report prepared in 1977 indicated that it would be economical to replace the 36-in (1.07-m) woodstave pipe with a larger 42-in (1.07-m) inside diameter woodstave pipe, and increase the capacity of the unit.

Replacement of the old woodstave pipe with a new larger woodstave pipe was completed in 1981 without major difficulty. The only problem encountered was where the pipe traversed a public highway through a culvert pipe. Bending the larger pipe to the required S bend proved to be too difficult, hence a steel thimble bend was added. As for adding a new larger unit, this required:

- The insertion of a steel plug cap into the downstream end of the woodstave pipe so that it could be maintained full of water
- Removal of the existing turbine-generator and auxiliary equipment
- Excavation and construction of a valve house behind the powerhouse
- Excavation of the powerhouse concrete to accommodate the larger embedded parts
- Installation of the new equipment
- Enlargement of the tailrace
- Cleaning of the canal upstream of the intake
- Replacement of plant transformers
- Installation of air-vent valves on the penstock
- Installation of a communication cable between forebay and plant

The most interesting aspect of Topsail is the hydraulics of the unit, hence this will be discussed in more detail.

Penstock

The woodstave penstock has a total length of 6280 ft (1915 m), with the upper third being on a relatively flat grade (Fig. 7.18) which limits the negative water hammer. Gross head on the unit is 341 ft (104 m), so that the penstock length-head ratio is high at 18.3. A rule of thumb states that whenever this ratio exceeds about 4, a surge tank or relief valve is needed. The downward-sloping terrain near the powerhouse prevents installation of a surge tank at a convenient location, hence a relief valve is used. With a full-load flow of 108 ft^3/s (3.09 m^3/s), the penstock length times velocity (LV) is 70,500 ft^2/s (6553 m^2/s), and normal water hammer is 25 percent of the design head.

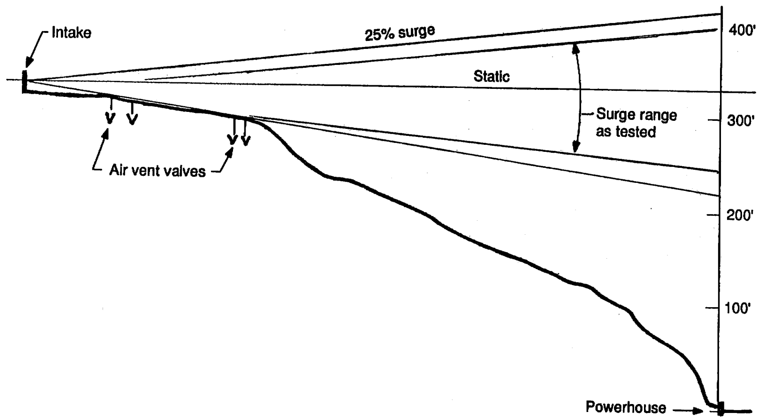

FIGURE 7.18 Topsail penstock profile.

Necessity for a Relief Valve

With a long penstock, the governor wicket gate closing time has to be slow in order to avoid excessive water hammer. With a slow governor time, longer than about 8 to 12 s, depending on unit inertia, the unit speed will increase to full runaway, usually about 60 to 80 percent above normal synchronous speed.

Medium- to high-head Francis turbines throttle the flow when operating at runaway, so that flow through the unit is reduced to somewhere between 40 and 60 percent of full-load flow. If this throttling of flow occurs over a short period of time, a runaway induced water hammer will result.

To counteract this water hammer, the relief valve is opened at a rate which will match the flow reduction, thus reducing water-hammer pressures to somewhere between 20 and 50 percent.

Sizing and timing of the relief valve is a very complex calculation, which requires the following data as stipulated by Rich [14], Allievi [15], Streeter [16], and Parmakian [17]:

- Penstock LV and allowable waterhammer
- Relief valve flow characteristic
- Penstock sound wave velocity
- Wicket-gate timing
- Wicket gate flow characteristic
- Inertia of generator and turbine

The calculation is best left to the turbine manufacturer, since only the manufacturer will have data on the flow characteristics. Two methods have been developed to control the relief valve, by linkage to the wicket gate operating ring—the classic method—and by breaker tripping with backup from a pressure sensor on the turbine spiral casing, as used at Topsail. Both are discussed below.

The Classic Control of a Relief Valve

With this system there is a lever connecting the wicket gate operating ring to the relief valve, with the wicket gate servomotors having sufficient capacity to also operate the relief valve. The linkage is designed so that when the wicket gates close, the relief valve opens, and vice versa. However, the relief valve is only required to operate when the wicket gates close, hence the lever mechanism contains a dashpot with a bypass which allows the wicket gates to slowly open without affecting the relief valve.

On load rejection, the wicket gates will close rapidly to a position just below that of the runaway flow, and at the same time the relief valve will open rapidly to discharge the portion of flow cut off by the wicket gates. After completion of this first rapid movement, both wicket gates and the relief valve close slowly to produce a minimum of water hammer.

This is where the first complication is introduced. The governor must be capable of:

- Slowly opening the wicket gates
- Rapidly closing the wicket gates over about half the gate opening
- Slowly closing the wicket gates over the remaining distance

This two-speed closure rate can be accomplished by check valves and solenoid-controlled double porting on the oil piping to the wicket gate servomotors.

However, there is another complication. The characteristics of the wicket gates and the relief valve can only be matched approximately for one wicket gate position. For example, suppose the turbine is operating at about 60 percent load and the load is rejected. The turbine should close rapidly to about the 50 percent gate position, and then slowly for the remainder of the stroke. Meanwhile, as the wicket gates close by 10 percent, the lever will open the relief valve by about 20 percent (50 percent wicket gate movement = 100 percent relief valve movement). However, the different discharge characteristics of wicket gate and relief valve are such that at the end of this movement, the flows will not match, and a positive or negative water hammer may occur.

The solution to this problem is to add a cam into the control circuit, which will change the range of rapid wicket gate movement, depending on the initial position of the wicket gates. As mentioned previously, the hydraulics are complex.

Topsail Control of Relief Valve

As inertia is added to the unit (by means of a flywheel), the rate of change of flow through the turbine is reduced, since the unit will take longer to reach runaway. Hence, wicket gate closing times and relief valve opening times can be lengthened, which gives more latitude in matching of flow characteristics. If sufficient inertia is added, unit acceleration can be reduced sufficiently to permit use of a single-speed governor. Hence, added inertia, as always, is beneficial to the unit.

At Topsail the contractor (Barber Hydraulics) was faced with the task of fitting a larger unit into a small powerhouse. There was space only for a turbine and generator with no flywheel as shown in Fig. 7.19. Moreover, the generator, being a standard synchronous motor, had an inertia which gave a machine start time (T_m) of only 1.78 s. This can be compared with a penstock water start time (T_w) of 6.16 s.

Since this small unit was not required for frequency control on the Newfoundland grid, and since a three-element electronic governor would be used for synchronizing and speed regulation, the contractor initially decided to install a relief valve operated by a pressure sensor, to limit water hammer, and to use a single-speed wicket gate movement, thus avoiding the complexity of two-speed control.

The decision represented a major departure from precedent, and was only possible due to the use of computers to calculate the system hydraulic transients. In the previously outlined classic control system, flow control in the turbine always resides in the wicket gates. It is never allowed to pass to the turbine runner. However, with a single-speed governor, slow closure of the wicket gates on load rejection means that flow will be throttled and controlled by the runner as the unit runs up to and reaches overspeed. Needless to say, it is only with a computer that a time history of flow through the runner can be calculated, as the runner increases in speed, and head changes as the flow is throttled.

Several computer runs indicated that a match at full-load rejection could be attained. However, much further work was required to obtain a match for load rejections from partial gate openings. Initially, it was hoped that this could be accomplished by changing the rate of relief valve opening with different gate po-

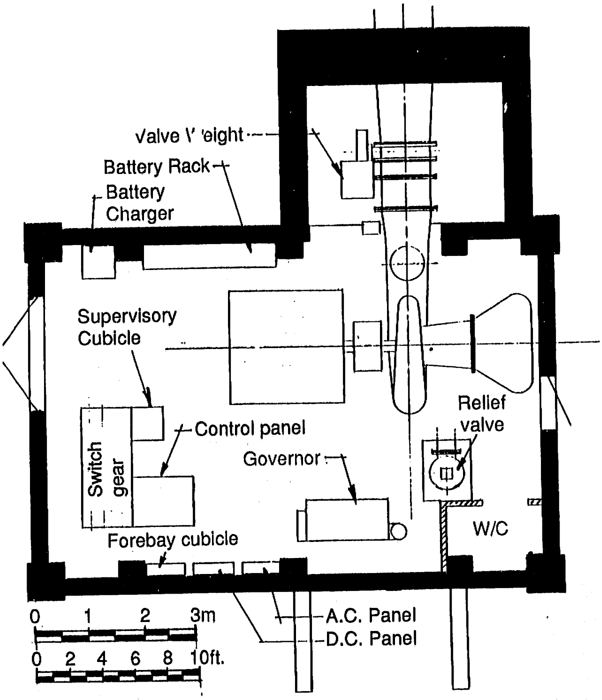

FIGURE 7.19 Topsail powerhouse plan.

sitions, but this was not possible. Eventually, the solution was attained by adding a cam, operated by the wicket gate ring, which limits the extent of the relief valve opening, depending on the initial position of the wicket gates. Also, it was found that using a pressure sensor to initiate opening of the relief valve was too late in the sequence of events to keep water hammer within the 25 percent limit. Since the only time runaway-induced water hammer will occur is when the unit becomes disconnected from the system, the controls were changed so that breaker opening triggers opening of the relief valve. The pressure sensor was retained as an emergency backup.

The advantages of this control system over the classic control system are:

- Less complex single-speed wicket gate timing
- Smaller wicket gate servomotors
- Very rapid movement of the relief valve permits use of minimum generator inertia

Of course, there are some disadvantages, the major one being the complexity of the hydraulic controls. Timings of the system are:

Wicket gate opening time 120 s	Relief valve opening time 3.3 s
Wicket gate closing time 120 s	Relief valve closing time 60 s

Plant Commissioning

The unit was commissioned in August 1984, about 7 months later than expected due to relief valve control problems. During commissioning load-rejection tests were undertaken in about 5 percent gate-opening increments, with records kept of all data, but most importantly, speed and pressure rises as indicated in Table 7.5. Pressure gauges were also installed at the knee in the penstock to watch for unacceptable low or high surges. One minor problem encountered with the unit during the automatic runup sequence (remote-controlled from St. John's) is that the unit takes about 4 to 5 min to synchronize, since with little inertia, the unit responds very rapidly to minor movement of the wicket gates at speed-no-load (design speed without any load on the generator). However, the unit is stable, and once locked into the system, operates without difficulty. A view of the unit is included in Fig. 7.20.

Concluding Remark

Although small, and seemingly inexpensive, a relief valve will add to the complexity of a generating unit. Moreover, the hydraulic calculations required to size the valve and determine the correct controls and timing of valve operation with the turbine unit are just as complex and extensive as those for a large unit. It can be argued that with modern computers this task should be simple. Such is not the case. Every situation is unique, hence the computer program has to be revised to match the configuration and operation of the controls, and the equations which

TABLE 7.5 Topsail Load Rejection Tests

			Penstock pressure, lb/in^2							
			Penstock knee			Powerhouse				
Test no.	% Gate	kW	Run	Min.	Max.	Run	Min.	Max.	Speed rise, %	Valve open, %
1	30.0	702	6.2	5.0	13.2	147	152	170	63.8	0
2	37.5	1142	5.9	4.5	16.0	145	150	172	75.7	0
3	40.5	1268	5.0	5.0	17.5	145	145	174	79.1	3.0
4	44.0	1382	5.0	5.5	18.0	143	145	175	83.3	3.0
5	45.5	1463	5.0	5.0	18.5	142	140	175	80.6	6.2
6	46.0	1468	5.0	5.0	18.5	142	140	175	83.3	6.2
7	50.0	1568	5.4	5.0	19.0	141	140	175	83.3	9.2
8	51.5	1626	4.2	5.0	19.0	140	140	175	84.7	9.2
9	54.0	1714	4.0	5.0	19.0	139	140	172	86.1	15.4
10	57.0	1799	3.6	5.0	18.5	137	140	170	86.1	21.5
11	60.0	1889	3.3	5.0	18.5	135	140	168	86.1	26.1
12	65.0	2013	2.6	5.0	15.0	132	135	160	83.3	47.7
13	72.0	2116	2.0	4.4	9.0	127	130	150	77.8	83.1
14	77.0	2177	1.5	4.4	9.0	125	130	140	76.4	95.3
15	82.5	2242	1.1	3.0	8.6	123	130	135	76.4	100.0
16	91.0	2272	0.4	1.6	8.2	120	125	135	77.8	100.0
Design max. or min.:				0.4	26	—	110	185	70.0	

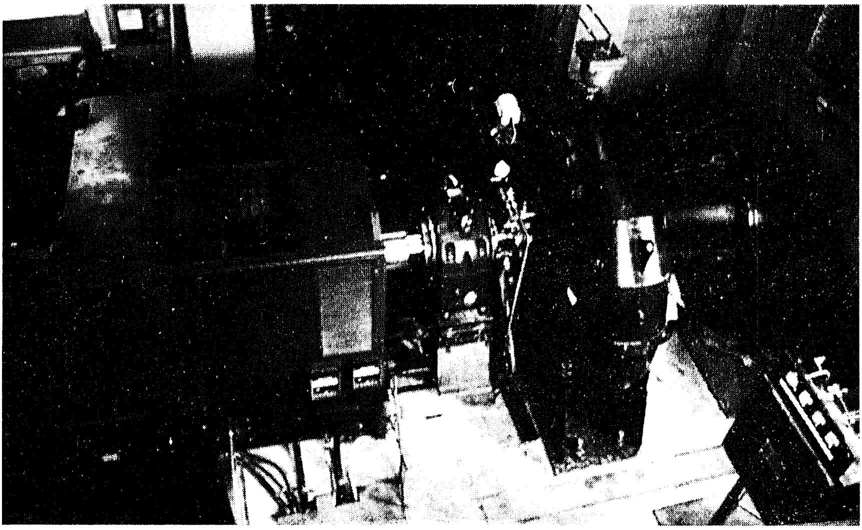

FIGURE 7.20 Topsail unit viewed from crane rail of new unit. Control panel for relief valve in bottom right corner.

represent the performance of the turbine runner under changing head, discharge, and speed must all be reprogrammed.

For units larger than 10 MW, a relief valve will normally add about 25 percent to the cost of the turbine, decreasing slightly as the unit becomes larger. Conversely, for smaller units, the relief valve can be expected to cost more as a percentage of the turbine cost. Fortunately, there may be a solution in sight. Some manufacturers are now developing Francis turbine runners which do not throttle the flow on runaway. However, they do have one disadvantage: efficiency is lost. It comes down to balancing the extra relief valve cost against the added generation.

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CHAPTER 8

ECOLOGICAL EFFECTS OF HYDROPOWER FACILITIES

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8.1 BACKGROUND

Hydropower engineering textbooks have not routinely dealt with the ecological effects of hydropower development. For years, hydropower was assumed to be ecologically "clean"—a nonissue for civil, mechanical, and electrical engineers that were the targeted audience for these books. However, in the last two or so decades, the perception of both the scientific community and the public of the "cleanness" of hydropower has changed. Extensive procedures have been legislated to regulate development and operation of hydropower plants. It is now incumbent upon the developer—usually an engineer or engineering team—to prove that significant ecological impacts will not occur, are not occurring, or can be effectively mitigated. Thus, it is important that engineers be conversant with ecological impact issues. Only then can they direct ecological consultants and interpret their results and recommendations, as well as interact effectively with biologists and ecologists in federal and state agencies. This chapter is a response to this real need.

The new environmental procedures referred to above have developed relatively rapidly and are complex. Not all hydropower developers have encountered the new licensing system. Others have not yet accepted the need for the extensive procedures for licensing or relicensing. Further, the recent accent on small hydropower has brought many new, often environmentally naive, developers into the field. The next three sections briefly provide examples of (1) reasons for the decline and fall of the "clean hydro" assumption; (2) legislative changes that have influenced hydropower licensing; and (3) the impact of ecological issues on hydropower generation. These topics are included to demonstrate to developers the importance of the discussions of potential impacts and of the general approach to hydropower licensing that conclude the chapter.

Throughout this chapter, the primary emphasis of the discussion will be on North American experience and application. This is mainly due to the large volume of literature detailing studies in temperate North America. The emphasis is in contrast to the fact that most of the remaining potential for hydropower development (about five times current capacity) exists in the developing countries, as discussed in Chap. 1. Although much of the discussion below can be applied to those countries by selecting analogs (e.g., fish species) from temperate climates, there are some qualitative differences. For example, changes in the prevalence of parasite transmission is a potential concern in tropical or subtropical climates [1]. Predicted parasite increases below the Aswan high dam have not yet been observed [2], but this may be because appropriate mitigative measures (piped water, sanitation) have been taken. Thermal stratification of reservoir waters also appears to be different and more complex in tropical climates. In such cases, direct use of information presented in this chapter may not be possible. On the other hand, legislation, policies, and practices outlined here may be indicative of future developments around the world. Procedures and methods developed in the past in North America (e.g., assessment of thermal electric power plants) have been incorporated in recommendations of international agencies.

8.2 THE "CLEAN" HYDROPOWER ASSUMPTION

What could be more environmentally sound than to harness water on its downhill path to the sea? Certainly the delays or short circuits that dams and reservoirs cause in the water cycle would not be likely to have dire ecological consequences. It is true that the effects of dams on migrating fish have been recognized in Europe for hundreds of years [3]. In fact, since about 1870, all dams in Scotland, England, and Wales have been built with fish bypasses to protect salmon and trout. However, recognition of this problem in North America lagged. For example, the first fish ladder on the Connecticut River was installed in the early 1900s. In addition, as steam electric generation increased in importance, the reputation of hydroelectricity as "clean" power grew. The social and environmental costs of fuel extraction, refining, and waste disposal were much more obvious. This focused environmental concerns away from hydropower; until the last two decades, the full range of potential effects of hydroelectric generation have not been considered seriously and systematically in North America. Thus, the spate of hydroelectric plant construction that began in the 1930s was viewed as an unmitigated boon to society.

Federal development on two river systems, the Columbia, shown in Fig. 8.1, and the Tennessee, helped focus attention on ecological effects of hydropower. In each case, multiple dams, the presence of commercially important aquatic resources, and the desire of the agencies (Bonneville Power Administration and Tennessee Valley Authority, respectively) to be publicly responsible helped foster extensive and intensive ecological studies. Results of these studies led to a reevaluation of the clean hydropower assumption.

The salmonid fishery on the Columbia River is a multimillion dollar per year industry. Salmon are anadromous; that is, the young hatch from eggs laid in gravel beds high upriver in freshwater, migrate to the ocean where they grow and mature, then migrate back upriver as adults to lay eggs and complete the life cycle. Upstream passage of the adults was recognized as critical to maintenance of sport and commercial harvests of fish. As a result, dams constructed in the 1920s and 1930s were built or retrofitted with fish ladders to allow passage. However, total catches of salmon gradually declined from peaks in the late 1800s [4]. The decline was surely due to many causes, such as overfishing, general environmental degradation, and river blockage. However, the general parallel between

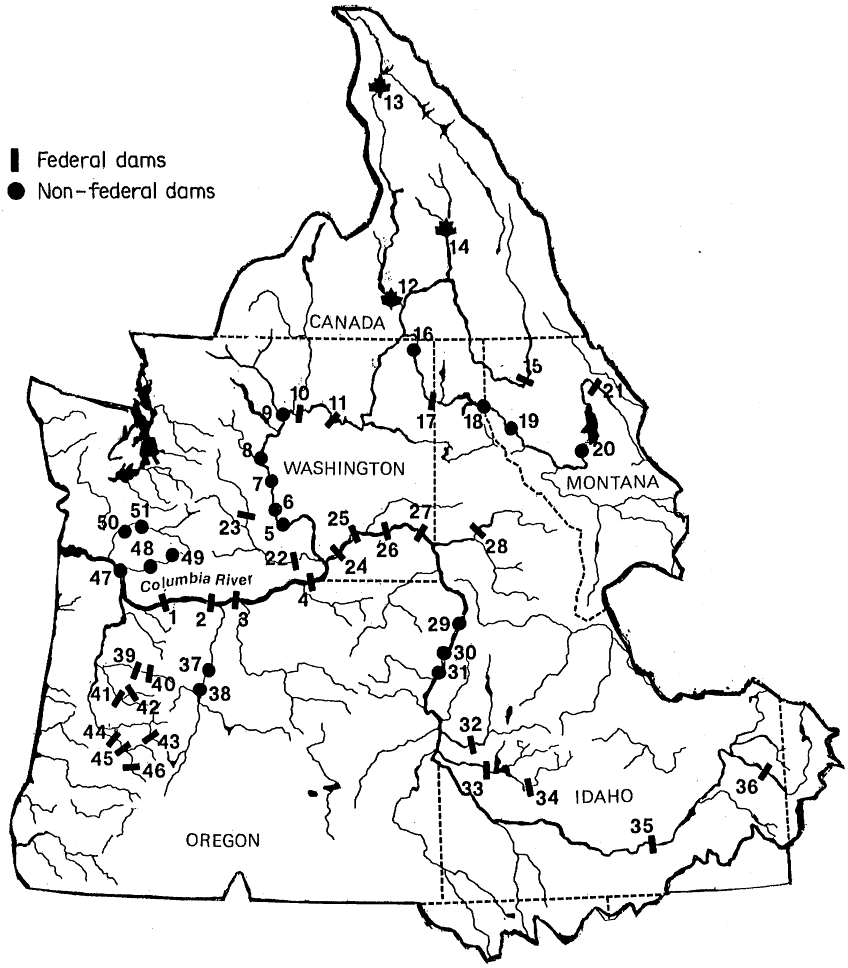


FIGURE 8.1 Federal and nonfederal dams of the Columbia River Basin. *(Courtesy of Bonneville Power Administration, "Downstream Fish Migration: Improving the Odds of Survival," May 1985, E & P-3-9, p. 2.)*

hydropower construction and fishery decline focused interest on such issues as loss of spawning habitat, mortality of young fish as they passed through turbines during their downstream migration, and effects of gas supersaturation resulting from spilling.

Mussel harvests from the Tennessee River and its tributaries, although of much lower absolute value than the Columbia River salmonids, were an important part of the depressed economy of the Tennessee Valley. These harvests first supported the pearl button industry. Later, when plastics became a cheaper alternative, they provided raw materials for cultured pearl production. According to Isom [5], harvests were "measured in the hundreds of tons per year" in the early 1900s. Between 1945 and 1958, harvests gradually declined from above 50 tons to below 10 tons per year. In addition, by 1967 the mussel fauna in the Tennessee River had declined to 44 species compared to about 100 species recorded before dam construction was initiated. Many of the endemic species (occurring only in the Tennessee River drainage) were confined to the tailwaters of dams. Isom [5] attributed most of the decline to impoundment, with overharvesting contributing in the later stages of the decline. This hypothesis is not testable, but impoundment almost surely contributed to the decline.

Studies on these two river systems have not been able to quantify the relative contribution of hydropower to the loss of the commercial resources. However, they did focus attention on potential effects of hydropower generation and on species other than migratory fish. Despite the fact that naive statements are still made about clean hydropower, such statements no longer represent the consensus view. Studies such as those on the Columbia and Tennessee rivers spelled the doom of the clean hydropower assumption.

8.3 LEGISLATION AFFECTING HYDROPOWER

The death knell of the clean hydropower assumption was sounded by the passage of new laws and reinterpretation of policies designed to attain the goals of existing laws. Comprehensive treatment of all of the federal laws, much less of the state laws, pertaining to hydropower development is beyond the scope of this chapter. A complete guide to the FERC licensing process, FERC-0100, "Application Procedures for Hydropower Licenses, License Amendments, Exemptions and Preliminary Permits," is available from the Superintendent of Documents, FERC, 825 North Capitol Street N.E., Washington, DC 20426. However, from an ecological perspective, several laws have major influences on hydropower development and operation. These are, in chronological order, the Federal Power Act (1934, 1965), the Anadromous Fish Conservation Act (1965), the Wild and Scenic Rivers Act, the National Environmental Policy Act (1970), the Federal Water Pollution Control Act (1972), the Endangered Species Act (1973), the Clean Air Act (1977), and the Electric Consumers Protection Act (1987). Except for the last, each of these has been amended. For example, one of the 18 amendments of the Federal Water Pollution Control Act is the Clean Water Act (1977). However, updated copies of most of the above laws can be found in the *Environment Reporter*, available in many university law libraries. Each of these laws is discussed briefly below.

The Federal Power Act (Public Law 66-280; 16 USC 791 et seq.) requires that almost all hydropower plants greater than 1.5-MW capacity be licensed. Private, municipal, and cooperative plants are all subject to this law even if the plants are located on federal dams. The Federal Energy Regulatory Commission (FERC) has been the licensing authority since 1977, when such authority was transferred from the Federal Power Commission. However, licenses are valid for periods ranging from 30 to 50 years. At the end of each licensing period, plants must be relicensed in order to continue in operation. Because most plants have been licensed since 1943, many will be subject to relicensing in the next decade, as shown, in part, in Fig. 8.2.

Criteria for license approval are safety, adequacy of structures, economic feasibility, and conservation of resources. In relicensing, performance in meeting the conditions of the existing license is also evaluated. Until passage of the Na-

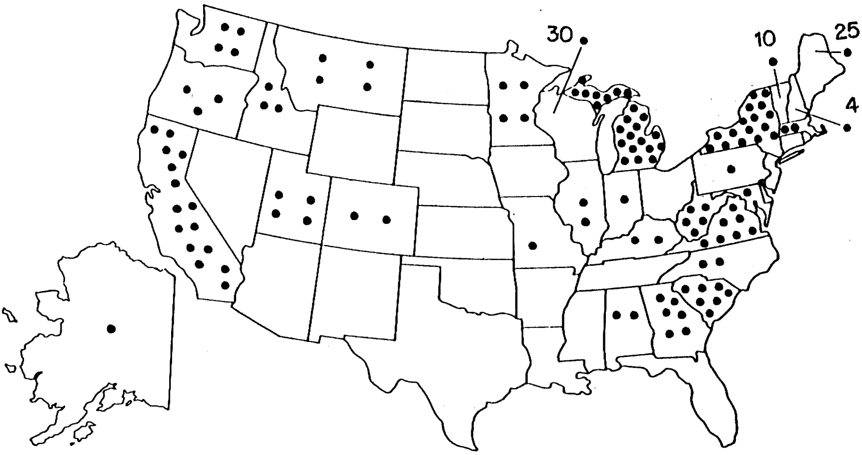


FIGURE 8.2 Investor-owned utility company hydroelectric projects up for relicensing by January 1, 1994. *(From Edison Electric Institute [6].)*

tional Environmental Policy Act (see below), conservation of natural resources was generally interpreted as referring to the water resources, i.e., does this project make efficient use of the water passing by the site? However, today the conservation of resources criterion is mainly focused on biotic (plant and animal) resources.

In practice, FERC generally accedes to recommendations of the U.S. Fish and Wildlife Service, the National Marine Fisheries Service, and state fish and game agencies. According to Kerwin [7] resource agencies submitted recommendations on almost 80 percent of the 300 licenses he studied. Less than 10 percent of those recommendations were rejected by FERC. By law (16 USC 811) FERC must require installation and operation of fishways to pass fish around dams if they are prescribed by the Secretary of the Interior. In reality, applicants for licenses must now demonstrate that the project will provide for protection and enhancement of fish and wildlife resources. And, with the passage of the Electric Consumers Protection Act (see below), fish and wildlife agency decisions about whether this criterion has been met have been placed on an equal level with the criterion of economic feasibility in final licensing decisions by FERC.

The Fish and Wildlife Coordination Act (PL 85-624; 16 USC 661 et seq.) was written to ensure consideration of fish and wildlife protection in actions of federal agencies. Thus, federal agencies must consult with state and federal fisheries and wildlife agencies to mitigate ecological impacts of those actions. From a practical standpoint, this law provides the basis for the developer-agency consultation process now required in FERC regulations.

The Anadromous Fish Conservation Act (PL 89-304; 16 USC 757 et seq.) was established to foster federal and state cooperation (and funding) to conserve, develop, and enhance migratory fish resources along the coasts and in the Great Lakes. The law promoted development of state and/or regional plans to restore fisheries resources depleted because of water resources development. Federal funds have been provided to enhance state efforts to provide free migration of fish over obstructions, to improve feeding and spawning conditions, and to construct and operate hatcheries. From the standpoint of hydropower development, the existence and implementation of these plans require additional efforts toward assessment and mitigation of impacts on migratory fish.

The Wild and Scenic Rivers Act (PL90-542; 16 USC 1271 et seq.) advocates protection of rivers or river segments of outstanding or remarkable value for the benefit of future generations. Such values can be due to scenic, recreational, geological, historical, cultural, or fish and wildlife resources. Development (dams are mentioned specifically) is precluded on designated rivers or river segments.

The National Environmental Policy Act (NEPA; PL91-190; 42 USC 4341 et seq.) was the first law to formally recognize that the environment is held in trusteeship by each generation for future generations. The law outlined a process to ensure "appropriate consideration (of the environment) along with economic and technical considerations." The environmental costs and benefits of any major federal action must be compiled in an environmental impact statement (EIS). Federal hydropower developments, such as those built by the Corps of Engineers or the Bureau of Reclamation and operated by the Bonneville Power Administration (BPA) or the Tennessee Valley Authority (TVA) fall under this law. However, the granting of a FERC license can also trigger NEPA review of a privately funded project if it is considered a major license action. The draft EIS is submitted to federal and state agencies (e.g., fisheries and wildlife) that have jurisdiction by law or special expertise related to any of the environmental impacts. The public can also comment officially on the report and/or participate in hearings on the project. In contrast to the FERC licensing process for private hydropower developments, there is no formal reconsideration ("relicensing") of a federal plant once it has passed NEPA review. However, the federal agencies (TVA, BPA) have historically worked with fisheries and wildlife groups to mitigate impacts discovered after the plant is in operation. Thus, the procedures involved in licensing and operating federal and nonfederal hydropower projects are more different in theory than they are in actual practice.

The objective of the Federal Water Pollution Control Act (PL92-500; 33 USC 1251 et seq.) is to eliminate pollution of water bodies in the United States. The courts have ruled that dams are not point sources (of pollution). However, FERC requires state certification that the project will not violate state water quality standards before a license will be issued. Furthermore, a dredge and fill permit must be obtained from the Corps of Engineers before the project is begun.

Endangered species and their critical habitat are protected by the Endangered Species Act (PL93-205; 16 USC 1531 et seq.). Under this act, federal agencies can deny licenses or permits and federal projects may be stopped if a project threatens either an endangered species or its habitat. Although the act is couched in terms of "can" and "may," occurrence of an endangered species at a project site can have severe repercussions. For example, the Tennessee Valley Authority was prevented from filling the reservoir behind Tellico Dam because of the presence of a small endangered fish called the snail darter. Even though the dam was fully complete, it took an Act of Congress to permit TVA to close the gates and begin reservoir filling. Subsequent investigations suggest that the decision did not result in the demise of the snail darter. However, costs of delays and litigation reduced societal benefits of project development.

The Clean Air Act (PL95-95; 42 USC 7401 et seq.) established the basis for setting air quality standards. It is incumbent upon hydropower developers to provide a construction (and operation) plan that will avoid exceedence of the applicable air quality standards.

Finally, the Electric Consumers Protection Act (PL 99-945; 16 USC 791a et seq.) requires that FERC give the same consideration to energy conservation, fish and wildlife, recreational opportunities and "preservation of other aspects of environmental quality" as to hydropower development in deciding whether to grant a license. This stipulation is ensured by two procedural requirements. First, FERC must consider the extent to which the project is consistent with any comprehensive plan for the water body. These can be developed by a federal agency or the state in which the facility is or will be located. Second, FERC must include in each license conditions that will "adequately and equitably protect, mitigate damages to, and enhance fish and wildlife affected by" the project. These conditions will be based on recommendations made by federal and state fish and wildlife agencies. If the commission disagrees with any recommendation, it must first try to resolve the disagreement. If this is not possible, FERC is required to publish the reason(s) why the recommendation is rejected and the commission position is correct. This cumbersome process makes it even more likely than in the past that fish and wildlife agency recommendations will be accepted by FERC.

8.4 IMPACT OF ECOLOGICAL ISSUES ON HYDROPOWER

Not all changes in ecosystems brought about by hydropower development can be, or are, viewed as bad. For example, valuable cold-water fisheries are now sustained in southern areas of the United States as a result of the cooler tailwaters of dams built to generate power. However, the most common results of attempts to prevent ecological effects of hydropower are increased capital costs for building or retrofitting mitigation devices or loss of generating capacity. Both of these results affect the economics of projects. They can reduce the profitability of existing plants or make development of marginally profitable sites unfeasible. Selected examples are given below to show that costs of ecological mitigation can be high.

The Bonneville Power Administration spent about $70 million for fisheries and wildlife programs in fiscal year 1985 [Anthony Morrell and Lee Miller, personal communication, BPA, Portland, Oregon]. About $26 million was spent directly for protection, mitigation, and enhancement of fish and wildlife in the Columbia River basin. The other $44 million resulted from lost revenues due to spilling and augmenting flows to enhance upstream passage of adults and downstream survival of young migrant fish. Similar expenditures are expected for at least the next few years. Additional costs undoubtedly accrue to the Corps of Engineers and the Bureau of Reclamation in this same river basin.

In 1981, 194 hydroelectric sites were in some stage of development in the New England states. Authors of a study conducted for the New England River Basins Commission [8] concluded that there was a "potential for conflict" over in-stream flow requirements at between 34 and 80 percent of the sites. In-stream flow requirements are the flows of water that must be released from an impoundment to maintain uses (e.g., recreational fishing below the dam). Thirteen of the 194 sites were investigated more thoroughly. In-stream flow requirements were estimated to cost between 1 and 76 percent of the capacity planned for these sites.

Since 1949, Northeast Utilities subsidiary companies have spent millions of dollars to enhance upstream and downstream passage of Atlantic salmon and

American shad [Ronald Klattenberg, personal communication, Northeast Utilities, Hartford, Connecticut]. A fish lift built in 1955 cost $1.1 million. A fish ladder system, shown in Fig. 8.3, built in 1979 cost $13.8 million. Additional millions of dollars have been spent by these same companies on other ladders and lifts, fish attraction systems, and other operation and maintenance for fish protection. The current hydropower licenses permit reconsideration of fisheries resource needs at any time.

In 1984, state fisheries' staff of the states of Montana, Idaho, Oregon, and Washington analyzed 861 small hydropower proposals in those states [Nicholas Iadanza, personal communication, National Marine Fisheries Service, Portland, Oregon]. Using a number of criteria, the proposals were divided into four classes according to predicted ecological impact, shown in Table 8.1. Most of the proposals did not contain enough information about the site or plant design to predict what the impacts might be. These proposals were placed in a class called "insufficient information." Of the rest, 51 percent either were expected to produce insignificant impacts or impacts that could be mitigated. The rest, 49 percent, were expected to produce impacts that would be unacceptable, even with state-of-the-art mitigation. In other words, it likely would be futile for hydropower developers to pursue licensing of almost half of these sites. In addition, economically marginal sites in the 20 percent that require mitigation might be in jeopardy. Projects at these sites would have to be carefully evaluated to ensure that they would remain economically profitable even with the added construction and/or operation costs.

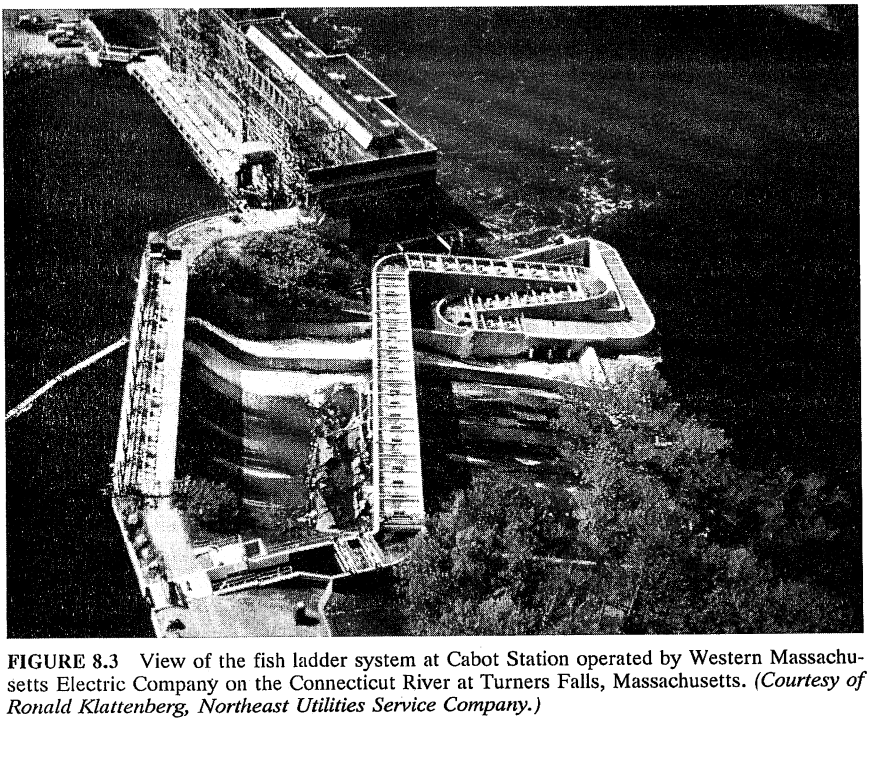


FIGURE 8.3 View of the fish ladder system at Cabot Station operated by Western Massachusetts Electric Company on the Connecticut River at Turners Falls, Massachusetts. *(Courtesy of Ronald Klattenberg, Northeast Utilities Service Company.)*

TABLE 8.1 Analysis of 861 Proposed Hydropower Projects in the Pacific Northwest Based on Their Potential Impacts to Fish and Wildlife Resources

	Number of projects			
	Impact insignificant	Impact can be mitigated	Insufficient information	Unacceptable impact
Montana	29	13	75	18
Washington	13	12	145	69
Idaho	47	37	203	35
Oregon	10	0[a]	122	33
% of total projects	12	7	63	18
% of projects' sufficient information to judge potential impacts	31	20	—	49

[a]Included in insufficient information category.
Source: CRIFC et al. [9].

A final example can be found in the conclusions of an analysis of 41 licensing case histories of small hydropower projects included in the National Small Hydropower Program of the U.S. Department of Energy [10]. The data cited from this report (Table 8.2) may not be typical of a randomly selected group of proposed projects. The projects were screened prior to being included in the program. Times are probably shorter than average. Percentage success is probably inflated. Comparative values, however, may be realistic. Developers that used a "good" licensing strategy were more likely to have their projects approved (89 versus 79 percent), and received approval almost eight months quicker (12.0 versus 19.9 months) than those using "poor" licensing strategy, as summarized in

TABLE 8.2 Analysis of Effects of Licensing Strategy on Licensing of 41 Projects in the National Small-Hydropower Program*

	Good licensing strategy†	Poor licensing strategy†
Project status		
Approved	24 (89)‡	11 (79)
Not approved	3 (11)	3 (21)
Average months to application acceptance	2.3	5.4
Average months to project approval	12.0	19.9
Number of projects with interventions	13 (48)	7 (50)

*Modified from Cunningham et al. [10].
†For definitions, see text.
‡Percentages in parentheses.
Source: Copyright ©1985, Electric Power Research Institute, EPRI report EM-4036, vol. 2, "Small Hydropower Development: The Process, Pitfalls, and Experience—Licensing Activities Summary and Analysis." Reprinted by permission.

Table 8.2. Insufficient consideration of ecological impacts and/or mitigations were identified as major components of poor licensing strategy in all cases.

These examples should serve to show that ecological issues can affect hydropower development. It behooves developers and operators of hydropower plants to anticipate the potential severity of ecological impacts, as well as the need for, possibilities for, and cost of mitigation in their planning. With such information, field studies can be focused where they will do the most good and cost-effective mitigation measures can be designed, smoothing the licensing or relicensing process.

8.5 PHILOSOPHY OF ENVIRONMENTAL IMPACT ANALYSIS

A list of sources of potential environmental impacts of hydropower development is given in Table 8.3. Extensive discussion of all impacts may be found in Rochester et al. [11], and many methods for mitigation of effects are found in Acres Consulting Services Limited [12]. Additional discussions of individual topics are referenced in the following subsections. For a given site, the number of topics that will require more than preliminary consideration will usually be much shorter. This is because severity of impacts from the different potential sources will depend on characteristics of both the power plant and the surrounding environment. Obvious plant characteristics include whether a new dam is to be constructed or an old dam modified or repaired; whether the powerhouse is on-stream or water is transported away from the natural stream bed to gain generation head (off-stream); whether power generation is derived from the normal stream flow regime (run-of-river) or is store-and-release, generating power only at certain times of the day or seasons of the year (peaking); or whether the water flow through the powerhouse is one-way or reversible (pumped storage). As will become clear, however, finer levels of design, such as turbine placement and/or type, can also exacerbate or mitigate ecological effects. But local densities of human populations, as well as types, quantities, and uses of biota in the existing water body, also play a role in determining which sources will need careful consideration and which can be dismissed at a given site.

Decisions made early in the design process can greatly affect which sources are likely to cause impacts. But each choice in siting, design, and operation involves trade-offs between electricity generation and costs of construction, operation, and maintenance. For this reason, it is important that a multidisciplinary design team be formed early in licensing. Ecologists and economists, as well as engineers, are needed on the team. Finally, team member interactions should be stimulated during early phases of decision making for the generation facility.

Potential ecological impacts from each of the sources listed in Table 8.3 are discussed in the following sections. The discussions are not exhaustive. Space limitations make this impossible. Also, the goal is not to turn hydropower developers and engineers into instant ecologists. Rather, the goal is to provide AID (anticipation, interpretation, decision) for hydropower developers. Hydropower licensing will proceed more smoothly if the developer or consultant can: (1) *anticipate* what impacts are likely to be significant at a given site or for a given design; (2) *interpret* the statements or recommendations of fish and game representatives and expert consultants; and (3) *decide* on the most efficient course of action in concert with design and operational alternatives and economics. There-

TABLE 8.3 Sources of Ecological Impacts of Hydropower Development

Terrestrial	Aquatic
Land use	Dredging
Materials handling	River blockage
Bird—transmission line interactions	Water-level fluctuation
Flow changes	Turbine operation
	Ponding

fore, the discussions below will attempt to highlight which effects might be expected, how to decide if such effects are likely to be severe at a proposed plant, and what modifications of structure or operation will help to reduce, minimize, or obviate the impact(s). Toward these ends, many of the references cited are reviews or summaries. They provide more in-depth information on specific topics, but skip the minute details that can often be more obstructive than constructive to decision making. With the discussions as background, a developer (with the aid of experts) may make more reasonable decisions about whether a potential impact can be dismissed as unimportant, requires further study, or requires mitigation.

8.6 SOURCES OF POTENTIAL TERRESTRIAL IMPACTS

Impacts of hydropower on the terrestrial ecology generally received little attention in the licensing process prior to the 1970s. The reasons have been partly historical—impacts on recreationally or commercially important aquatic populations were the first focus of hydropower impacts—and partly comparative—where large areas of terrestrial organisms are exposed to harm, losses are sometimes balanced by gains in aquatic habitat. However, regulators are placing increased emphasis on terrestrial impacts. Of most concern are those resulting from changes in land use, materials handling, interactions between birds and transmission lines, and changes in flow patterns below dams [11, 13–15]. Hydropower construction also can impact human populations directly, but these impacts will not be considered in this chapter.

Land Use

Land use impacts can be classed as direct or indirect. Direct effects include physical removal, burial, or flooding of terrestrial habitat. In each case, terrestrial vegetation and wildlife are killed or harmed as an immediate result of hydropower development. Animals drowned during filling of the reservoir exemplify such a direct impact. These impacts end when construction is completed. Indirect effects, including loss of habitat when the reservoir is filled, displace organisms for all of the hydropower plant operation period.

Direct Effects. These effects begin prior to construction and last through the construction period, including filling of the reservoir. Preconstruction activities,

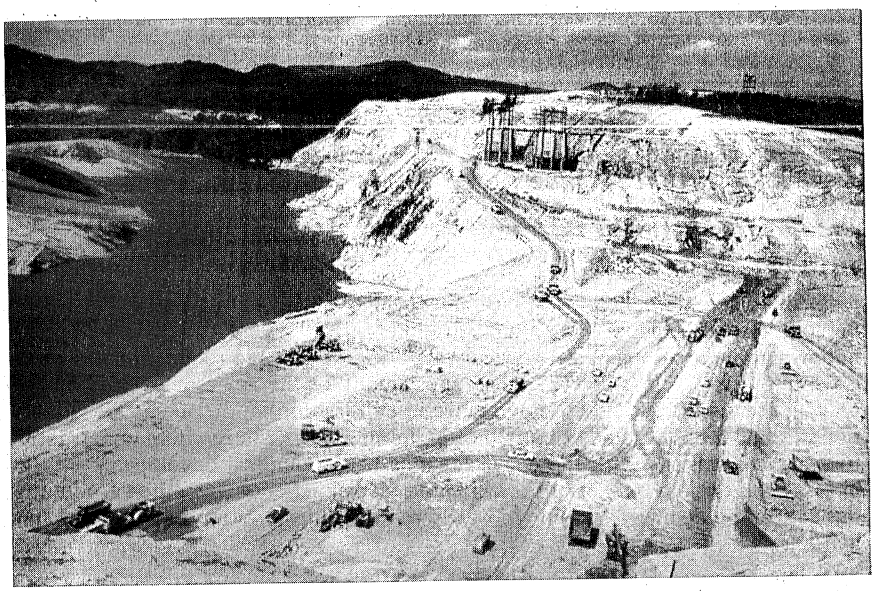


FIGURE 8.4 An early view of construction of the Bad Creek pumped storage hydroelectric plant in Oconee County, South Carolina. *(Courtesy of Dr. Arnold Gnilka, Duke Power Company.)*

which generally involve surveying the physical and biotic characteristics, as well as the geology and hydrology of the site, can impact terrestrial plants and animals. In order to conduct these surveys, machinery and personnel must gain access to the site. Where an old dam is being used, access roads and trails probably already exist. Drilling, coring, and mapping will probably only be needed to supplement existing information; thus, impacts would probably be slight and mostly temporary. On the other hand, access to remote sites and studies for larger projects will have a higher potential for impact. Direct physical damage, sometimes called "big-foot effects" by ecologists, probably have the highest potential for impact during the preconstruction phase. Increased erosion and sedimentation, human presence, and noise are other potential impacts, but are expected to be relatively minor and short-lived.

Potential direct effects during the construction phases are similar in kind but more extensive than in the preconstruction phase. Construction involves clearing and removal of vegetation from the site of the dam, powerhouse, transmission corridors, and associated structures. Supply of services—transportation ways, electricity, water, and sanitary waste disposal—can increase the area of impact. Finally, terrestrial organisms are killed when the reservoir is filled. (Animals are also displaced, but this is dealt with in the next section, "Indirect Effects.") Figure 8.4 shows an example of a hydropower project during early construction.

Indirect Effects. These effects are usually less immediately severe than direct effects. However, indirect effects gain in importance because they generally occur over a longer time. In some areas of the site (buildings, fenced areas, reservoir) habitat is permanently changed. Noise and simple presence of humans are obvious components of construction, but may continue due to machinery operation, the need for on-site personnel, and increased recreational use along access roads.

Road kills and losses of wildlife due to hunting and fishing may increase as well. The presence of the reservoir or diversion canals or conduits may change normal migration routes. Finally, increased erosion and deposition of transported soils may reduce or change habitat occurrence or value.

Impact Evaluation. Land use impacts can be evaluated on an areal basis. However, severity of impact is also dependent on sensitivity of local species and uniqueness of the area impacted. For example, impacts on stream-side (riparian) biota in low-rainfall areas would commonly be viewed as more severe than in high-rainfall areas, because such biota are scarce where there is little rainfall.

The Habitat Evaluation Procedures (HEP) developed by the U.S. Fish and Wildlife Service [16] are the most widely used methodologies for assessing the impacts of terrestrial habitat losses. In some cases, these methodologies use qualitative or quantitative (regression) analyses of changes in wildlife occurrence or density with habitat characteristics to develop predictive relationships. These characteristics can include vegetation, soil, water supply, etc. Such relationships are then used to predict biomass for representative species with and without the plant. It is common to analyze 10 to 20 representative species at a site. In other cases, the models rely on conceptual relationships between features of the habitat and presumed habitat quality (Habitat Suitability Index, HSI, or Habitat Quality Index, HQI). In these cases, the HSIs or HQIs are compared with and without the plant. The impact is the difference between the two values. The expense of HEP analyses and expected severity of impacts argue against such analyses for preconstruction activities; it is probably most appropriate to try to avoid impacts during this phase. HEP analyses are more likely to be warranted in the construction phase of a project.

There are, however, some problems with HEP analyses [17]. First, the methods assume that habitat limits the size of local wildlife populations and that the important components (and their interactions) of that habitat can be accurately defined for each species. For some species and localities, these assumptions are supported; for most species the assumptions have not been validated. In addition, it is largely unclear how noise and human presence impinge on the value of habitat otherwise available to wildlife populations. Second, almost all projects will have positive impacts on some species and negative or no impacts on others. For example, construction of transmission corridors in a large area of dense forest may increase the diversity of wildlife, such as deer and small game, that live at edges (ecotones) of forests. Conversely, the corridors decrease habitat available to animals that prefer more densely forested ranges. Because only a few species are used as surrogates for ecosystem response in the HEP analyses, it is easy to see that choice of these species can bias the results. Finally, many of the existing models have not been checked for accuracy or generality of prediction. Thus, it may be critical for project economics to have an expert who understands the ecology of the project site evaluate the appropriateness of specific HEP analyses. This should aid in ensuring studies that are as unbiased and objective as possible.

Impact Mitigation. Because of the subjectivity of the impact evaluation, the mitigation plan will be subjective also. Although it is not possible to deal with specifics here, methods have been developed to reduce effects of construction and maintenance of transmission line corridors [14, 15]. To summarize briefly, the methods involve selective cutting and application of growth inhibitors to species that could interfere with the lines or replanting of species that provide similar habitat to the natural plants but do not grow as tall. This increases the interval between maintenance periods and permits more natural communities to occupy the corridors. Many of these methods also would be useful at the dam or powerplant site.

Other potential land use impacts can also be reduced. The reservoir can be filled slowly to promote wildlife emigration, access routes can be planned to avoid streams and other sensitive or valuable habitats, proper techniques can be used for bridge or culvert construction to reduce siltation (see "Dredging"), saving trees and other vegetation and/or immediate replanting following construction at the plant site and along roads and transmission corridors can help reduce erosion, and natural or man-made screens can be used to reduce effects of construction and operation noise.

Species or habitats can be the focus of mitigation, so it is crucial to identify which concerns are paramount for design of the impact evaluation. For example, it is important that interested parties agree about whether mitigation or compensation will be "in kind" or "equal replacement," if and/or when negative impacts are predicted. "In kind" compensation deals with one species or habitat separately from all others. Predicted losses in one area must be compensated by equal gains elsewhere. "Equal replacement" compensation allows gains in one species or habitat to substitute for losses of others. The latter, even though comparative gains and losses are usually difficult to quantify, appears more desirable. The former will most certainly involve cash contributions and/or land banking. Nevertheless, if requirements are negotiated early in project planning, costs of studies and compensation can be incorporated into overall project benefits and costs at a time when they can be useful to the developer.

Materials Handling

During construction, many materials are transferred on- and off-site. Many of these materials are man-made, relatively inert, and stored only temporarily before incorporation into structures. However, others such as rock, earth, and chemicals can impact ecosystems.

Effects. Both direct and indirect effects on terrestrial organisms will result at source areas for construction materials and disposal sites for materials removed. During site preparation, fill may be needed from off the site to level the area. In addition, materials may be needed for earthen dams, canal walls, or cofferdams. Effects of removal of the materials from their source will be similar to those incurred in clearing the plant site (see "Land Use" above). Removal, transport, and disposal can also have negative impacts via production of dust. Other materials brought on-site such as diesel and gasoline fuels, herbicides, and insecticides may be subject to spillage and subsequently contaminate soils or streams following runoff. Where materials removed from the site are not disposed of in existing landfills, direct and indirect effects will also result from disposal of materials derived from dredging, tunneling, digging of canals and diversion structures, excavation of dam and transmission tower footings, and removal of cofferdams. Effects from chemical spillage are dependent on the rate and amount of transfer and fate. Other effects will be qualitatively the same as burying habitat or wildlife on site.

Materials Handling Impact Assessment. In general, impact assessment can be conducted as for land use impacts. HEP analyses may be appropriate. Except for accidents, chemical spills probably will affect only isolated areas on-site.

Materials Handling Mitigation. Mitigation involves reducing land areas affected and/or providing in kind or equal replacement compensation. Scheduling of construction so that materials removed in digging canals can be used to supply needed fill or be used in dam construction will help decrease effects. Using existing landfills or otherwise disturbed sites for borrow pits or disposal will reduce affected areas. Transport and storage of chemicals in containers as small as practicable will help to lower the risk of contamination in case of an accident. Chemical transfer areas can also be chosen so that topography aids in isolating spills, especially from water bodies and wetlands.

Bird Interactions with Power Lines

Effects. Birds can be killed or injured when they encounter transmission lines in flight or land on transmission towers [18, 19]. Large waterfowl and predatory birds (raptors) appear most vulnerable. For waterfowl, the primary impact is thought to be where transmission lines cross waterways. The bigger the waterway and the closer it is to major waterfowl flyways, the higher the potential for impact. At most sites, deaths due to impact with transmission lines are probably few. However, if a transmission line crosses a major flyway, observational assessment of local use may be worthwhile. Where critical, assessment can be quantified using a mobile laboratory [20]. One mobile laboratory, dubbed "the birdmobile," combines radar, electrooptical, and video technologies to study local and migratory bird behavior. It is shown in Fig. 8.5. By examining flight behavior before the site is developed or after the dam is built, transmission lines can be routed to minimize bird collisions.

Large predatory birds (raptors) can be electrocuted when they use transmission towers as perches and their wings contact both phase and ground wires, completing an electrical circuit [21]. Between 70 and 90 percent of mortalities involve golden eagles, so it is likely that protection will be needed only in areas where they are known to occur. Most mortalities are immature eagles that are less maneuverable than adults, but mortalities also increase where rabbits, rather than rodents or birds, are the primary prey. This is apparently because the towers are used as perches for "still" hunting. Because some towers provide better opportunities and capabilities for hunting, mortalities can vary widely from tower to tower.

Mitigation. Eagles can be protected in three ways. First, protection devices have been developed for many types of transmission poles and towers (illustrated in Fig. 8.6). Most of these devices are inexpensive. Nevertheless, they should not be required without evaluation of food types and hunting practices in the area even if golden eagles are present. Transmission corridors located in topographically low areas or routed around preferred prey habitat also help reduce eagle use of poles or towers and thus reduce mortalities [22].

It may not be possible to anticipate and/or mitigate all potential impacts before the plant is built. For example, Puget Sound Power & Light Company (PSP&L) found that an osprey built a nest (shown in Fig. 8.7) on one of its transmission towers. In order to avoid the possibility of affecting the adult pair or their young,

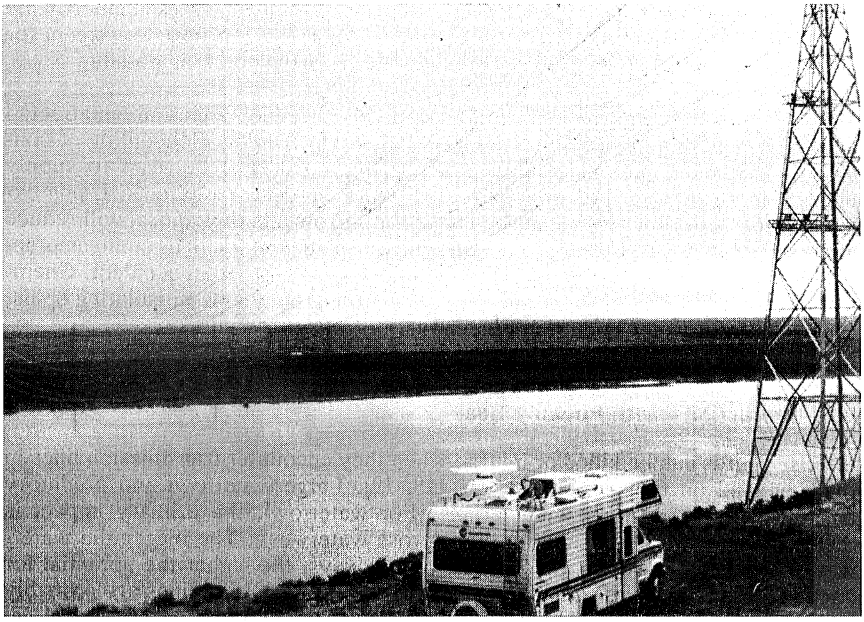


FIGURE 8.5 The mobile laboratory ("birdmobile") tracking bird flight patterns at Crow Butte, Washington, where a 500-kV transmission line crosses the Columbia River. *(From Gauthreaux [20].)*

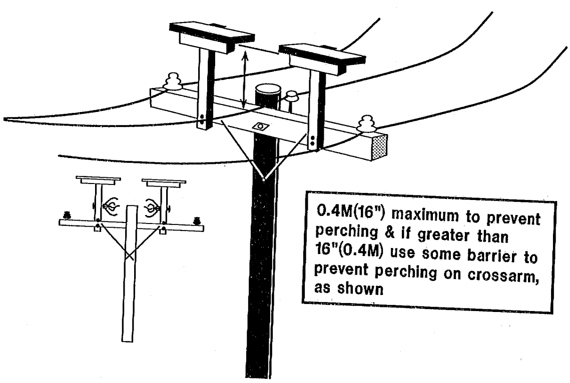


FIGURE 8.6 An elevated perch: one of several transmission tower modifications to prevent injury or death of large birds from contacting transmission lines. *(From R. R. Olendorff et al. [21], p 40.)*

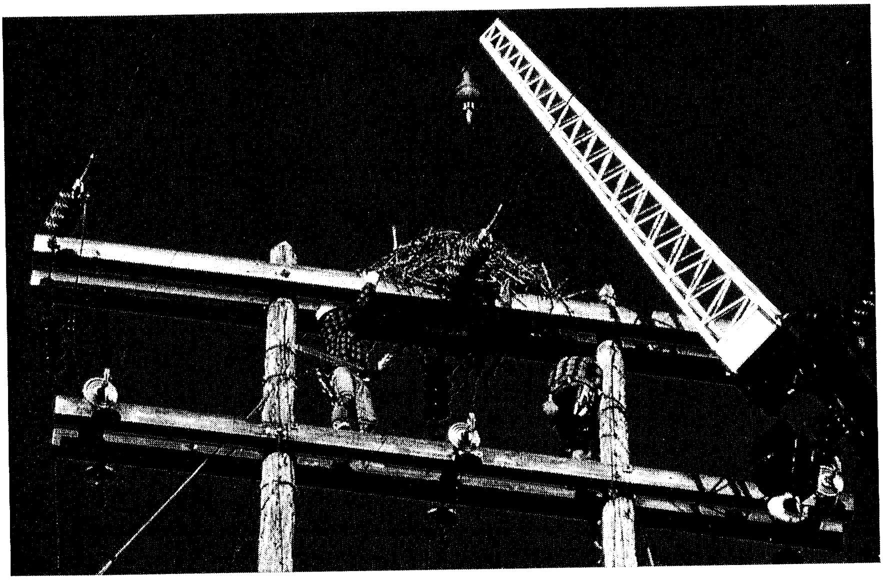


FIGURE 8.7 An osprey nest is removed from a transmission tower for relocation. Unique problems may require innovative solutions. *(Courtesy of Dr. Robert Clubb, Puget Sound Power & Light Company.)*

PSP&L, with the help of agency personnel, arranged to relocate the nest to a safer location. Thoughtful observation, analysis, and innovative solutions to potential problems may be needed to mesh power production or transmission and environmental protection.

Flow Reduction below the Dam

Effects. Flow changes during construction and operation can affect stream-side (riparian) vegetation and the animals (muskrat, beaver, etc.) that inhabit this vegetation. However, effects can range from strongly negative through strongly positive, depending on the stream and the structural geology of the valley in which it is located [23]. For example, some streams or stream reaches contribute to the surrounding aquifer (water table). These are called "losing" streams. Riparian communities (plants and animals) near these streams usually are strongly dependent on either mean or peak flows in the stream. Reductions in flow for any long duration would likely result in extensive changes in the riparian zone. Conversely, where "losing" streams are naturally intermittent or seasonal flows differ widely, hydropower flows may smooth the yearly hydrograph. In such cases, riparian communities would likely benefit. Finally, in streams such as those channeled into bedrock and that receive upslope seepage, called "gaining streams," stream flow may have little direct effect on riparian communities. In these cases, stoppages or changes in stream flow may be benign. A single stream may include sections (reaches) conforming to each of the above classes.

Other characteristics also are important in defining response of riparian communities to flow changes. Factors affecting vegetation include soil type, elevation, precipitation, isolation, and evapotranspiration. Time of flow change also may be important. Effects appear to be exacerbated when flow decreases occur at the time of seedling establishment, usually in the spring or early summer. Effects on both plants and animals also can be indirect; e.g., effects on muskrats or beavers may be mediated by increased predation if openings to dens or lodges become exposed during low water.

Assessment. The truncated list above should indicate the complexities of assessing, even on a qualitative basis, responses of riparian communities to changes in flows. Add to the above the potential effects of peaking power and the problem is made even more difficult. A recommended procedure was one of the outputs of a workshop held in 1984 [23]. Admittedly, the procedure involves subjective judgment of an expert or team of experts. Interpretation of visual observations of geological and hydrological features is used to classify stream reaches as "gaining" or "losing" and effects are then estimated based on a thorough knowledge of existing published data. One can expect controversy about results of this type of analysis, but it seems rational to admit that this is the best that can be done without further scientific progress.

Mitigation. Although any assessment is subject to question, there are techniques to mitigate terrestrial effects of flow change. Periods of no flow during construction of the permanent dam can be prevented or shortened. Flows during operation can be maneuvered to ensure that losing streams continue to be so at least intermittently. Finally, when peaking generation is required, water flows can be increased in steps to slow inundation of areas that were dry prior to generation to permit motile wildlife to escape.

8.7 SOURCES OF POTENTIAL AQUATIC IMPACTS

Aquatic impacts have been, and probably will remain, the prime focus of ecological assessments for licensing or relicensing hydropower projects. Dredging, river blockage, water-level fluctuation, turbine operation, and ponding all can elicit ecological responses. Each of these impact sources (Table 8.3) is discussed below.

Dredging

Dredging involves the physical removal of substrate (mud, gravel, etc.) from the bottom of a river or lake. Dredging is often needed to construct, maintain, or repair new generation facilities or to reclaim facilities that are in disrepair. Regardless of the objective, however, effects of dredging are due to substrate removal, increased suspended solids, downstream siltation, or changes in chemistry of water or sediments [24].

Effects. Removal of substrate by dredges can cause both direct and indirect effects. Animals and plants that live on or in the sediments (benthos) are removed from the aquatic environment and destroyed when they are disposed of with the spoils [25]. Because animals and plants usually occupy only the upper few centimeters of substrate, dredging results in an area devoid of benthos. Recolonization of this area can be relatively fast or take a long time. If the dredged area is small and the character of the substrate is not changed, motile organisms quickly immigrate from refuge (undredged) areas. Repopulation by nonmotile organisms such as plants may take somewhat longer, but substantial repopulation will likely occur after a single reproductive period (≤ 1 year). At the other extreme, if the area is large and the character of the exposed substrate is different from the material removed, recolonization may be very slow. This is especially true if there are no similar natural substrates nearby. In this case, even when recolonization is complete, the species composition of the benthos may be very different because benthic organisms often have narrow ranges of sediment characteristics in which they can live.

Increased suspended solids result from sediment resuspension during the dredging process itself or from runoff, if sediment disposal areas are not sufficient to contain the spoils during the disposal process or following rainstorms. Effects span the ecosystem structure. Decreased penetration of light into the water can reduce plant growth (primary production). Increased suspended solids in the water column can clog gills of fishes or feeding structures of bivalve mollusks (clams) and zooplankton (floating animals). Finally, decreased light penetration can inhibit predators, such as fishes, that are dependent on sight for location and capture of their food. Effects of suspended solids are exacerbated by small particle size, low sediment cohesion, absence of natural flocculants, and water-body characteristics that increase the time that the suspended solids remain in the water column, e.g., rapid and/or turbulent flow conditions. Under the opposite conditions effects would be less harmful.

Siltation of suspended solids is the alternative to transport. The influencing factors are the same; factors that increase suspended solids impacts reduce siltation problems (or transport the problems further downstream) and vice versa. The potential for impacts from siltation are therefore increased by large particle size, high cohesion of sediments, presence of natural flocculants, and water-body characteristics that decrease the time that suspended sediments remain in the water column, e.g., slow and nonturbulent flow. Small amounts of sedimentation are not usually harmful. In addition, effects of siltation may be mitigated by the presence of proximate refugia (areas where the native biota is not disturbed) from which recolonization can occur, if the resultant substrate quality has not been changed. However, if large volumes of sediment fall rapidly, effects can be deleterious. Benthos, macrophytes, and fish eggs and larvae can be buried and killed. Density, distribution, and species composition of benthos can be changed if sediment characteristics change. Finally, fish-spawning habitat, e.g., gravel, can be lost. In areas subject to heavy storms, the rapid runoff may help flush sedimented materials from the bottom and ameliorate effects. Nevertheless, siltation is an especially important consideration in streams where fish requiring gravel beds for spawning, e.g., salmonids, are present.

Environmental effects of changes in chemistry of water or sediments are generally due to lowered dissolved oxygen (DO), raised nutrients (nitrate, phosphate) or release of toxicants. Low DO resulting from oxidation of organic matter or chemicals can result in death or stress of aquatic biota. Severity of the DO decrease is dependent on the amounts and types of organics and other reduced chemicals in the dredged materials. Ecological response to any DO decrease depends on the sensitivity of species and life stages occurring in the region (see "Water Quality," below). For example, salmonids (salmon and trout) and other

FIGURE 8.8 Collection of sediment cores for chemical analysis. *(Courtesy of Oak Ridge National Laboratory.)*

cold-water species are usually more sensitive to effects of low DO than warm-water species. Early life stages (eggs, fry) of most species are generally more sensitive than adults. Nutrients can increase productivity of algae and/or bacteria depending on the content and form of these materials in the sediments. Finally, toxicants including methane, hydrogen sulfide, ammonia, organics, and metals can be released into the water column from the sediments, depending on oxidation and pH conditions in the water. Release of toxicants can lead to direct toxicity and/or bioaccumulation depending on chemical species, concentration and time of exposure for the various biota.

In general, where there has been no industrial development or high-intensity farming upstream of the site, toxicants will not be a problem except, perhaps, for early life stages of the most-sensitive species. However, in areas where industrial effluents are prevalent, or have been prevalent in the past, the only rational approach is to sample (Fig. 8.8) and chemically analyze sediment. Chemical concentrations can then be examined in light of known toxicity or bioaccumulation potentials to determine if problems are likely.

Assessment and Mitigation. Assessment and mitigation of the aquatic effects of dredging are site-specific to the extent that generalizations may not be very useful. Effects depend on the dredging and disposal method used; the area and volume to be removed; physical, chemical, and biological characteristics of the substrate removed, remaining, and nearby; hydrological characteristics of the water body; occurrence and intensity of preexisting stresses; and species and life stages of biota at or downstream from the dredging site. However, in many cases, areas to be dredged are probably not very large and will not contain significant levels of toxicants. In these cases, scheduling dredging during low-productivity periods (fall and early winter) and when important species are not reproducing, and maintaining refugia to support rapid immigration will probably be sufficient to control effect levels. In cases where large volumes of materials are to be dredged, sediments contain significant amounts of noxious chemicals, or susceptible endangered species are vulnerable, more extensive mitigation such as silt curtains or hydraulic dredges may be needed. Some care should be taken so that the mitigation technique fits the need. Hydraulic dredges have many advantages, as summarized in Table 8.4, in terms of prevention of immediate effects to aquatic organisms. However, hydraulic dredging increases the land area and containment necessary to prevent later transport [26]. Thus, unless hydraulic dredging is really needed, little gain in reducing aquatic impacts may be exchanged for increased terrestrial impacts and added dollar costs.

TABLE 8.4 Comparison of Economic and Environmental Costs of Hydraulic and Mechanical Dredging and Dredge Spoils Disposal (+ = greater/higher)

Source of cost	Hydraulic	Mechanical
Economic		
Disposal site acquisition and construction	+	
Transportation		+
Environmental		
Dredging site		
Substrate removal	+	
Suspended solids		+
Downstream siltation		+
Changes in water chemistry		+
Disposal site		
Habitat destruction	+	
Surface runoff, erosion	+	
Groundwater contamination	+	
Suspended solids	+	
Downstream siltation	+	
Changes in water chemistry	+	
Uptake by terrestrial biota		+

Source: Modified from Loar et al. [24], p. 10.

River Blockage

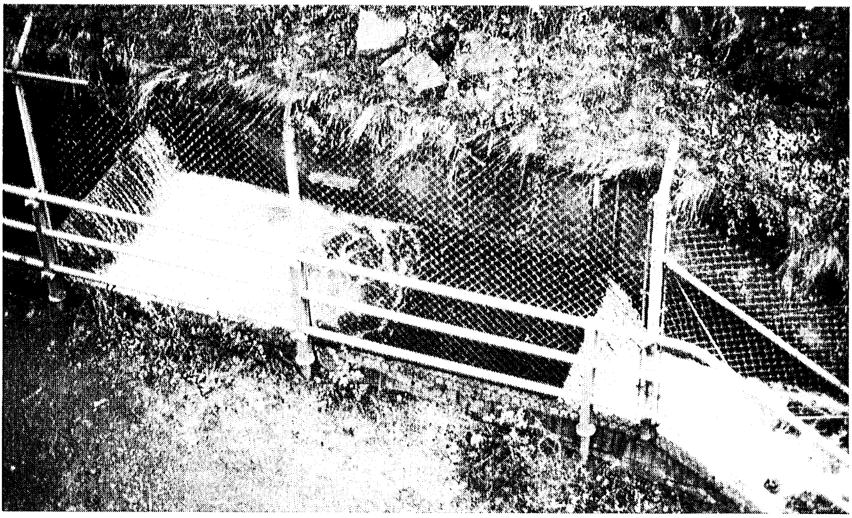
Effects. Hydrogeneration facilities can block upstream passage of fish. (Interception of downstream movement is discussed under "Turbine Operation," below.) The impact of such blockage is most clear for anadromous and catadromous fishes that are dependent upon upstream migrations to complete their life cycles. Anadromous fish, such as the American shad (*Alosa sapidissima*), striped bass (*Morone saxatilis*), and several salmon species, spend two or more years in the open ocean, where they grow and mature prior to swimming up major rivers to spawn. Catadromous fishes, such as the American eel (*Anguilla rostrata*) reproduce in the open ocean and the young migrate up rivers where they grow and mature in fresh water. Where these types of fishes occur, there seems to be little choice but to provide upriver access.

Whether there is a need to provide similar upriver access for fish that do not migrate between fresh and salt water is not clear. In recent years, concerns have spread to include other types of fishes. For example, some wholly fresh-water fish, such as sauger (*Stizostedion canadense*), swim upstream prior to spawning. Even resident nonmigratory fish have received some attention because of the net downstream flow of fish, especially in early life stages, because this could deplete upstream populations. How much net downstream flow of resident fish actually occurs and how significant such transport is remains unknown, but concern for providing upstream passage is unquestionably expanding to include a greater number of species. Still, without any data to indicate that upstream passage will benefit resident fish populations, it would appear that mitigation is unwarranted. This is an area where research is needed to determine whether there is a problem after all.

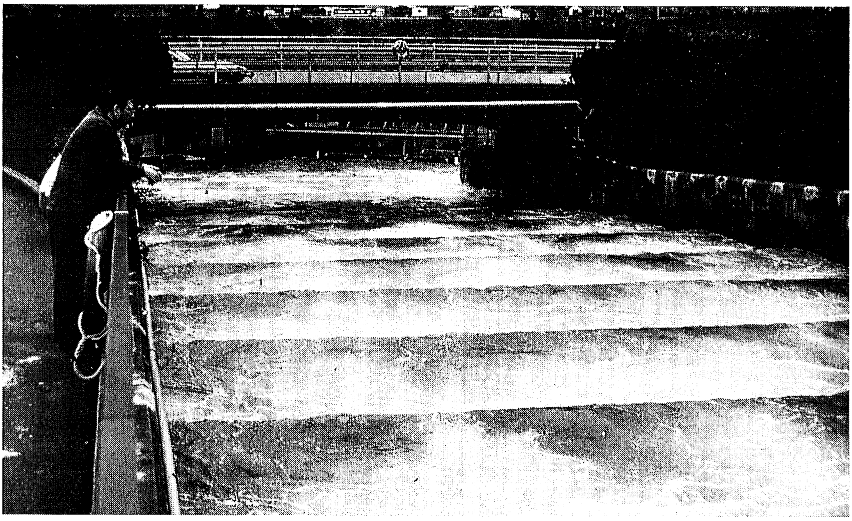
Assessment. For small hydropower facilities located on dams less than 20 ft (6.1 m) in total height, Reiser and Peacock [27] have proposed a method to assess whether and when the facility will block upstream passage. Knowledge of fish behavior and hydraulic characteristics near the dam is required. The reader is referred to the paper for further information. At larger dams where there are migratory fish, assessment is probably not worthwhile because mitigation will almost surely be required.

Mitigation. River blockage effects on upstream migration can be mitigated using fishways, fish locks, or fish lifts [28, 29]. Several types of fishways or fish ladders, e.g., pool and weir, vertical slot, Denil, have been designed. Each consists of a series of ascending levels with areas of reduced currents in between where fish can rest during the swim to the upper reservoir. Two pool-and-weir fishways are shown in Fig. 8.9. In a fish lock system, illustrated in Fig. 8.10*a*, fish are crowded into a small area near a closed entrance to the partially filled lock chamber. The chamber is filled and the entrance opened. In response to the pressure to migrate upstream, the fish move into and to the top of the chamber. The lower entrance is then closed and an upper exit port is opened allowing the fish to move into the reservoir. The fish lift, or elevator, system illustrated in Fig. 8.10*b* is similar to the fish lock except that fish are crowded and led into a water-filled container which is then raised to the exit opening.

Choice of the optimum upstream passage facility depends on trade-offs between capital, operating, and water loss costs as well as characteristics of the fish found at a specific site. Some of the relative costs are compared in Table 8.5. Unfortunately, capital cost estimates are dependent on assumptions used in calculating them. Site-to-site generalizations usually would not be even reasonably accurate. Under a single set of assumptions, Bell and Richey (in Hildebrand et al. [28]) estimated unit costs of $7000 to $30,000 and $5800 to $30,000 per ft (30.5 cm) of height required for fishways and fish lifts, respectively. This suggests that costs of fishways and fish lifts are in the same order of magnitude, but relative costs might change drastically depending on availability of space for and complexity of the system (see Fig. 8.3). Fishways are the simplest to operate and maintain and have the lowest on-site personnel requirements, but pass large volumes of water (including both attraction and transport flows) around, rather than through, the turbines. In addition, some fish swim poorly and/or avoid fishways (e.g., American shad). For these species, fishways are inappropriate. Locks and

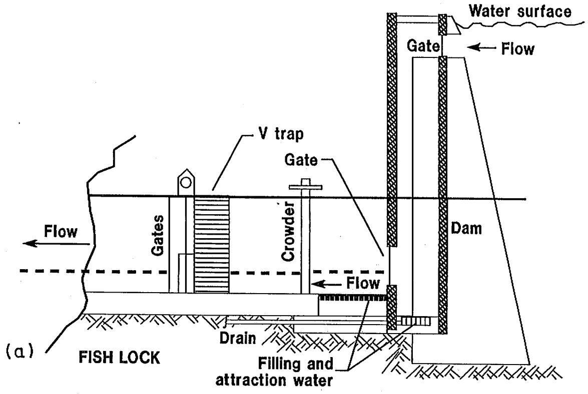


(a)



(b)

FIGURE 8.9 Old (*a*) and new (*b*) fishways at the Bonneville Dam on the Columbia River. Pools in the new fishway are wider, longer, and deeper and the vertical distance between them is reduced to increase fish passage. *(Courtesy of Dr. Donald Porcella, Electric Power Research Institute.)*



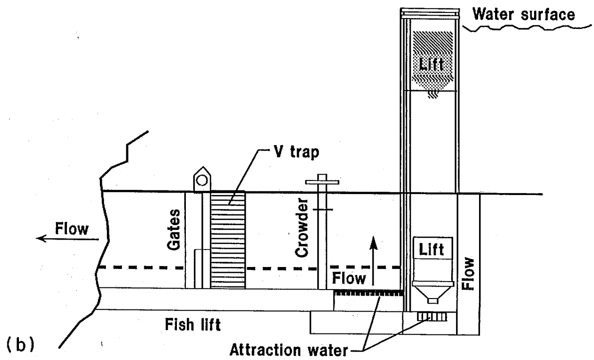


FIGURE 8.10 Fish locks (*a*) and lifts (*b*) are similar in design. *(Modified from Hildebrand et al. [28], pp. 44, 46.)*

TABLE 8.5 Comparison of Methods for Mitigating River Blockage

Method	Operating cost	Water loss	Species applicability
Fishway	Low	High	Narrow
Fish lock	High	Low	Broad
Fish lift	High	Low	Broad

lifts generally have higher operating and maintenance costs, but can transport most species and avoid much of the nongenerating water loss. Design of any one of these facilities at a specific site depends on swimming and/or jumping ability, behavior, and physiology of the fish.

Full consideration of the complexities involved in these responses and design modifications which would be required to account for them is beyond the scope of this chapter; a good summary appears in Hildebrand et al. [28]. However, the most important decision may be how long the upstream passage facility must operate each year. Flows to operate the facility and attract fish to it do not contribute to power generation. Therefore, operation before or after the migration period is economically costly without providing environmental benefit.

Water-Level Fluctuation

Effects. Water-level fluctuation is a potential problem at all store-and-release or peaking hydropower facilities, including pumped hydropower [30]. Most of us have seen evidence of the effects of water-level fluctuation in the relatively barren areas in reservoirs (drawdown zones) or immediately below dams during late summer or drought periods (Fig. 8.11). These areas are generally barren because neither aquatic nor terrestrial plants have time to fully colonize the areas during the limited periods when they are available to them. This can be particularly important for production in the reservoir because many organisms are dependent on aquatic macrophytes (rooted plants) for either food, substrate, or shelter. Some fish, such as carp, use the macrophytes themselves, along with any associated plants or animals, directly for food or consume the organic matter produced when the plants die and decay. This dieback process is particularly important in the annual cycling of nutrients within the reservoir. Further, snails and other browsing animals, which are important food for some fish, are dependent on periphyton (plants and bacteria), which grow on the rooted plants, for food. Periphyton can have production rates as great as or greater than those of the macrophytes themselves, so the total loss can be considerable. Finally, many fish lay their eggs on

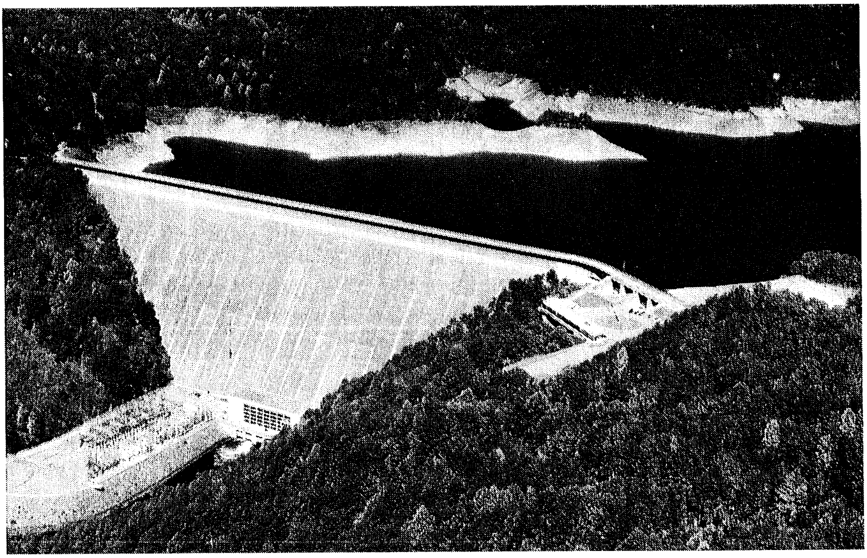


FIGURE 8.11 Seasonal and/or intermittent drawdown can result in unproductive shoreline areas. *(Courtesy of Charles Massey, Tennessee Valley Authority.)*

macrophytes or in the near-shore zones where these plants are found. The plants offer protection for the larvae or juveniles (as well as other small fish) from larger predatory fish during the particularly vulnerable period following hatching.

Benthic (bottom-dwelling) organisms and fish can also be directly affected by drawdown. Fish eggs and/or young in nests cannot migrate with the declining water level and will die if they are uncovered. The response of many benthic organisms that live in the bottom is to move deeper into the sediments until the water no longer covers the area. Death will result if the exposed "soil" dries out. If the uncovered period is short, however, the benthos may be little affected.

Motile organisms may also be affected indirectly by drawdown. Most fish and some macrobenthos, e.g., crayfish, are capable of migrating with the changing water level. Thus, they are not vulnerable to direct mortality from water-level fluctuation. However, they may be subject to increased mortality if food or suitable habitat is limiting or if mortality due to predators is increased because of crowding.

Because colonization is slow, the loss or partial loss of aquatic production of the shallow-water zone is not replaced by terrestrial production. But loss of the shallow-water zone in the reservoir is only one of the effects of water-level fluctuation. Below the reservoir, alternation of high and low flows, whether daily or seasonally, can produce scouring. (Effects of changing velocities and volumes of flow will be treated below in "Instream Flow.") Because of the dependence of benthic organisms on substrate type, the relative abundances of organisms can be shifted when scouring changes substrate characteristics. Both in and below the reservoir, exposure and inundation of the bottom can increase erosion and change the ability of the sediments to retain or release components. Results of erosion are often similar to those of dredging that have been discussed above (see "Dredging"). However, solubility of metals and some metal-nutrient complexes depend on, among other things, their oxidation state. As that state is changed by exposure to air or water, availability of nutrients, upon which the reservoir populations depend, and toxic metals can be changed. Both can affect local productivity. Finally, particularly in seasonally cycling reservoirs, flushing of nutrients, phytoplankton, and zooplankton can be a problem. If water flow through the dam is maintained constant or increased during low-water periods, losses will be increased. Reproductive rates may not be sufficient to maintain a substantial food base when the reservoir is refilled with stream water containing many fewer organisms. Effects could be observed in benthos and fish populations that are dependent on phytoplankton or zooplankton for food.

The effects of water-level fluctuation are site-specific, but increase as the amplitude and rapidity of the fluctuations increase. Water-level fluctuation tends to decrease the overall productivity of the water body, but effects on individual species can be very different. Thus, effects on a socially valuable species (e.g., commercial or sport fish) could be substantially greater or less than those on the ecosystem as a whole.

Assessment. Assessment of the ecological effects of water-level fluctuation depends on the answers to three questions. The first is, what is the extent and schedule of fluctuation? Where fluctuations are small and slow and do not occur during periods when organisms in the lake are dependent on near-shore habitat, effects will probably be insignificant. Conversely, where the fluctuations have opposite characteristics, many organisms may be stranded. Second, how much of the zone of productivity near the shore will be affected? Plant productivity is limited by light penetration. Therefore, if this zone of productivity is large, the loss due to water-level fluctuation may be only a small percentage of the total. Models are available [30] to predict the proportional loss of shore zone habitat due to water-level fluctuation. On the other hand, if there is a very rapid drop-off near the shore, the zone might not contribute significantly to reservoir productivity and large fluctuations in water level might have little overall impact. Third, what is the importance of the near-shore zone to important species in the reservoir? If important recreational or commercial fish species are not dependent on the shore zone for habitat (living, spawning) or food, water-level fluctuations might not be important. These questions cannot be answered without reference to a specific site for both operational and ecological characteristics.

Mitigation. Based on the answers to the above questions, the need for and extent of mitigation can be determined. In some cases, this might simply involve some operational changes. For example, in several of its multiple-use reservoirs, the Tennessee Valley Authority raises and maintains higher-than-normal water levels during spawning and nursery periods for some fish. Terrestrial vegetation that is inundated then can provide spawning substrate and protection from predators during these critical stages of the life cycle. Where effects appear likely to be more severe or extend over a longer time period, more extensive mitigation or compensation measures may be required.

Turbine Operation

Effects. During hydroelectric generation, water is passed from the reservoir above a dam past turbine blades which turn a generator. Fish inadvertently trapped in the intake water or actively seeking downstream currents during migration are exposed to several potential stressors, as they pass through the system illustrated in Fig. 8.12. These stressors are mechanical contact with conduits and turbine blades, shearing and cavitation near the blades, changes in pressure during passage through the system, and release into the tailwaters, and predation in the tailwaters [31, 32]. Losses of fish due to turbine passage may negatively impact fish populations, especially in river systems with multiple generating sites.

Field studies at operating sites and laboratory studies using models support conclusions regarding the relation of intensity of these stressors to relative mortality. However, quantitative predictions for specific sites and designs, especially for small hydropower generation, are controversial. There are several reasons for this. First, estimates of mortality for specific turbine designs are only tentative or do not exist. Data collected at field sites are plagued by introduction and collection problems (escapement, mortality). These problems make it difficult to interpret results or to have a high level of confidence in the actual mortality values obtained. Model studies, on the other hand, involve downscaling of facility size, which seems to preclude quantitative translation to full-scale facilities. Using a smaller turbine system model effectively increases the relative size of the fish; several studies have shown that larger fish are more prone to damage in passing through turbine systems. In addition, data which do exist generally do not cover the full scale of turbine operational conditions. For some types of turbines, especially those applicable for small hydropower (e.g., tubular turbines), tests have not been conducted at all. Second, in many studies it is not possible to identify a causal relation between mortality or injury and a specific stressor. Physical abrasion or contact with turbine blades is often obvious, but in other cases cause of injury or death is less clear. Third, it is difficult to isolate relationships between

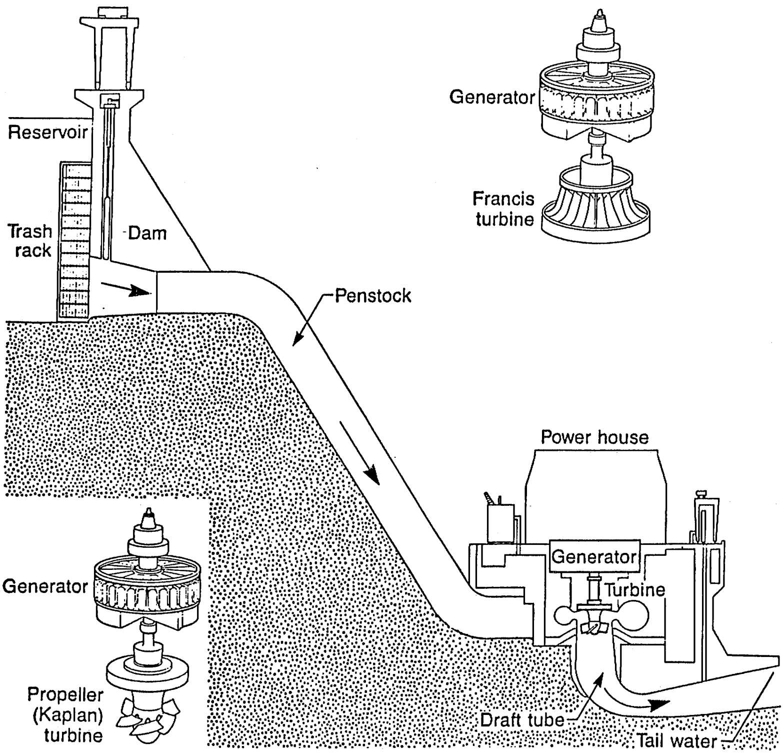


FIGURE 8.12 Schematic of a hydroelectric facility indicating path of fish through the system. Fish are harmed by contact with conduits and turbine blades, shearing and cavitation near the turbine blades, pressure changes during passage and release into the tailwater, and predation in the tailwater. Two typical turbine designs are indicated.

level of a stressor and level of harm because full control of each stressor in the system is not possible. For example, decreasing turbine speed may affect mechanical contact with the blades as well as shearing forces and cavitation. Therefore, it is unlikely that stressors can be tested over the full range of possibilities in the absence of stressor interactions. Fourth, and finally, different fish respond differently to the same conditions of operation. In general, it appears that small fish or fish with small scales are less sensitive than large fish or fish with large scales to stresses encountered during turbine passage.

There do appear to be some tentative generalizations that can be made regarding turbine passage. Turbine mortality at large hydropower projects varies widely from site to site. Mortality at small hydropower sites may well be less, but confirmation is lacking. The highest potential for turbine effects on fish populations is for migratory fish which move downstream under a behavioral pressure. Design changes can reduce turbine mortality; these include smoothing of conduit surfaces, increasing clearance spaces, decreasing speed of rotation of turbine blades, reducing the height of the turbine above the tailwater, increasing the depth of the entrance to the penstock, and decreasing turbine diameter. Finally, survival for a given site and design seems to be highest at maximum turbine efficiency, which is generally about 70 percent of full load.

Reduction of turbine mortality through choice of design and operational procedures, however, may not be a reasonable answer to environmental concerns. Some of the alternatives reduce unit-power production, forcing construction of other units to make up the difference. Impacts of construction and operation of these additional units should have to be considered in the benefit-cost determinations of the changes. In other cases, where fish populations are already heavily affected, reduction in mortality during turbine passage (as against prevention of it) may not be sufficient. In these cases it may be necessary to prevent fish from ever entering the generating system in order to attempt to preclude further impacts.

The last sentence specifically ended with the phrase "in order to attempt to preclude further impacts." Seagulls and anglers are commonly observed in larger numbers just below hydropower facilities than in other areas of the water body. Gulls can be seen swooping to the water to feed on the small fish that have passed through the turbines. The anglers harvest predatory fish that also appear to congregate to feed on the fish passing through the turbines. From work with other stressors [33], it is not unexpected that turbine-passed fish are more vulnerable to predation. It seems likely that losses to gulls are real; however, it is more difficult to estimate whether there is a real increase in losses of young fish to predatory fish or simply a concentration of those losses in a shorter length of the stream. Some losses to gulls have been prevented using nets, but how to prevent the losses to fish is a question. It may be that passing a dam will involve above-natural mortality no matter how the passage takes place.

Assessment. Accurate quantitative assessment of the effects of turbine mortality requires answers to each of three questions. What percent of a fish population passes through the turbines? What percent of those fish passing through the turbines is killed or harmed? What will be the response of the adult (i.e., fishable) population to those mortalities? Each of these questions is discussed briefly below.

At some sites percentage of fish passing through the turbines is easy to assess. These are sites where there is no bypass flow or overflow (spring runoff) and anadromous fishes are of prime concern. Anadromous fishes must pass the site in order to complete their life cycle. Therefore, all of these fish are exposed to turbine passage. However, in the more common case, bypass or overflows do occur. Because some salmonids migrate downstream in surface waters, worst-case estimates of exposure for these species can be based on the relation of turbine flow to surface bypass flow or overflow. More accurate estimates can be made by including fish distribution in the calculation. However, this may still overestimate exposure, because it assumes that fish approaching the dam will passively follow downward currents rather than seeking the surface. For anadromous species with unknown or broader depth distributions, prediction of exposure may be difficult or impossible, but surely less accurate.

The above discussion deals with anadromous fishes; less clear is the exposure of nonmigratory fishes. There appear to be no estimates or measurements of absolute numbers or fractions of populations that experience turbine passage. The potential for exposure is surely less for nonmigratory than for migratory fish and may be negligible. However, we can conclude now only that there is no evidence that turbine passage is major problem for nonmigratory fish.

Measurement of exposure after the plant is on-line is not always simple. It is dependent upon using sonar or more standard collection techniques to obtain relative counts of fish in the bypass or overflow and turbine waters. Plant designs do not always allow this.

The problems involved in predicting and/or measuring the percentage of fish that are killed or harmed during turbine passage were discussed above. Prediction depends on finding an analogous site (design, fish populations present, etc.). Even then, substantial error may exist because we have not yet quantitatively defined the cause-effect relationships. Measurement of damage depends on whether the plant design and site configuration will permit sampling at turbine intakes and discharges. If sampling of the whole water volume is not possible, as is usual, questions will remain about the accuracy of the estimates.

Objective prediction of fish population response to losses of fish during turbine passage is beyond the current capability of fisheries science. There is no question that fish populations can sustain themselves in the face of anthropogenic impacts. Many fish populations have been harvested at high rates (25 to 50 percent of the adults) for years, without diminution of fishery success [34]. These populations *compensate* for the added mortalities via increases in survivorship at other stages in the life cycle. Another more inclusive term for compensation is density-dependent population regulation. Density-dependent mortality means that mortality is directly proportional to density; at high densities, mortality rate (percent loss) is high and vice versa. This tends to stabilize population numbers.

On the other hand, compensatory capacity is finite. There are many examples of fish populations that have been fished, if not to extinction, to levels that would no longer support a fishery. Thus, it is important to be able to estimate compensatory capacity. Only then will we be able to predict fish population responses to turbine or other sources of mortality.

Many population models [35] have included compensation functions in various ways according to known mechanisms [36, 37]. The problem is that compensation theory has outstripped the empirical base from which it arose. We know, or think we know, how compensation works in general [38], but not how or how much it works in specific cases. There is no scientific consensus on how much compensatory capacity exists in any single fish population. Thus, any prediction of fish population response to turbine mortality will be controversial.

The above problems may, however, be extraneous. There is no real evidence to indicate that turbine passage mortality is a major problem for nonmigratory fish populations. On the other hand, migratory fish populations along the coasts of the United States are nearly all reduced compared to historical levels. State and federal fisheries agencies are involved in trying to restore them. It is not possible to determine the relative contribution of turbine passage mortality to the total impact. Nevertheless, consensus is that this mortality is not insignificant. Thus, it is doubtful that a hydropower plant will be licensed on a water body used by migratory fishes without some technology to bypass fish around the turbines. In other words, assessment may not be a necessary adjunct to decisions about whether mitigation measures are needed.

Mitigation. There are several alternatives that can be used to reduce turbine mortality [39–44]. The systems can be classified as increased spillage, collection systems, behavioral barriers, diversions, or physical barriers depending on operational characteristics. The last four require some method for safely transporting fish to the water body below the dam if migrating fish are involved.

The most cost-effective alternative may be increased spillage if the migration period is relatively short and occurs when river flows are normally high. The increased spillage flushes adults and/or young over the dam or through a bypass to the tailwaters. The cost of this alternative depends on the volume of water lost for power production. Costs of construction and/or labor for this alternative are usually low.

Collection systems involve capture of fish by screening and/or netting and transport by truck or barge to a downstream location. An example of a fish-hauling operation undertaken by the Bonneville Power Administration is shown in Fig. 8.13. Such systems have been used in the Northwest and Michigan with migratory fish. The systems would not work for resident fish and are labor-intensive. Thus, universal application of these techniques to hydropower sites would not be warranted.

Behavioral barriers utilize fish responses to external stimuli to keep fish away from the intake or to attract them to a bypass. Electric screens, bubble and chain curtains, light, sound, and water jets have been evaluated in laboratory or field studies. In general, results have been equivocal, but recent work [44, 45] suggests that strobe lights and/or "poppers" and several "hybrid" systems should be subjected to further testing.

It is certain that behavioral barriers will not prevent all fish from entering hydropower intakes. Fish behavior is notably variable both within and between species. It depends on such things as acclimation, environmental conditions (e.g., silt load can obscure stimuli), and competing behaviors such as feeding or predator avoidance. Where only a single species or a few related species are of concern and some turbine mortality is acceptable, certain behavioral barriers may be sufficient. However, substantial research will be required before such locations and methods can be identified more closely.

Diversion systems lead or force fish to bypasses that return them to the natural water body below the dam. Such physical systems include angled, inclined plane, and wedge-wire pressure screens, traveling screens, drum screens, and louvers. Each of these types of diversion systems appears to work in some cases. However, it is not yet possible to determine effects of design changes on efficiency to allow accurate selection of the best alternative for a given site. The passive (nonmoving) devices appear to offer the best alternatives if they can be kept clear of debris without much effort. If so, the reduced complexity and maintenance, as compared to a traveling screen, for example, would be a major advantage. A wedge-wire pressure screen [46], attached to the inside of a penstock by

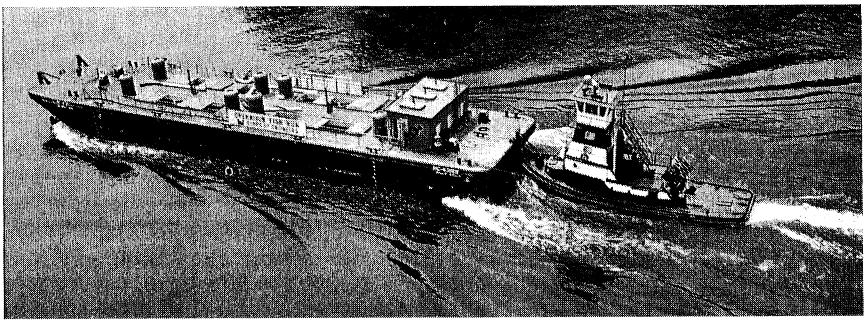


FIGURE 8.13 Collection and hauling of fish around dams is labor-intensive, but can be effective. *(Courtesy of Anthony Morrell, Bonneville Power Administration.)*

a fulcrum to allow intermittent back-flushing, appears to offer promise for solving problems with salmonids in the northwestern United States. Studies at the Brayton Point Steam Electric Generating Station also support the value of angled screens [47]. Further analyses of these and other diversion systems should be conducted to permit definition of the optimal type and design for specific hydropower sites.

Physical barriers (barrier nets, stationary screens, etc.) probably do not have much potential for application at hydropower plants. Sizing ("mesh" diameter and surface area) and labor-intensive maintenance, because of water-borne trash, lower the feasibility of their use. These barriers would also not solve problems involving downstream-migrating fish.

Ponding

Ponding, the backup of water behind a dam to form a pond or lake, can result in changes in water quality and water flow both in the reservoir and downstream. Size of the reservoir and operation of the dam help determine the extent and severity of potential impacts; the smaller the reservoir and the closer operation approaches true run-of-river (i.e., conditions prior to dam construction), the less likely that impacts will be significant. In some cases, interactions between flows and water quality may result in exacerbation or mitigation of effects, but current instream flow models do not generally deal with such interactions. Therefore, water quality and instream flow changes are treated separately below.

Water Quality Changes. Potential water quality changes due to ponding include temperature, dissolved gases, toxicants, nutrients, and turbidity [48]. In the reservoir, these changes generally result from suspended solids in stream water, sedimentation of these solids as the influent waters slow, lengthened proximity of nutrients and flora (both phytoplankton and macrophytes) because of the decreased turnover of water in the reservoir, and seasonal temperature stratification during which the mixing of warm surface waters and cold deeper waters are prevented.

Effects. Below-dam water quality is generally dependent on the position of the intake for power generation and the quality of water at that depth in the reservoir. By knowing or predicting the water quality at the turbine intake, one can estimate downstream water temperature, dissolved oxygen, toxicants, nutrients, and turbidity. Unfortunately, the intake pulls water from various levels of the reservoir, depending upon the velocity into the intake and the strength of stratification in the reservoir. Therefore, estimation is not simple. Water drawn in through the intake also replaces water which would have been spilled and exposed to the substantial aeration capacity of the spillway. Waters below hydropower plants can also become supersaturated with gases such as nitrogen. This can happen during spilling or during normal operation from deep-release dams. In the former case, air is entrained into the water as it falls and forced into solution under pressure in the plunge pool. In the latter, fully aerated waters under pressure are taken into the turbine system. Supersaturation in either case results when pressures decrease and/or temperatures increase downstream. Supersaturation can cause gas bubble disease in fish and other biota. Gas bubble disease is roughly equivalent to the "bends" experienced by deep-sea divers when they return to the surface too quickly. In fish, symptoms are bubbles under the skin or between fin rays, or, in extreme conditions, "pop eye." In the latter case, the eyes are forced from the sockets. Fish in this condition are unlikely to survive.

Water quality in a reservoir is often determined by occurrence of stratification. If the water depths exceed 9 to 15 ft (3 to 5 m) in temperate regions of the world, seasonal changes in insolation result in two reservoir seasons. During winter, the water bodies are generally well mixed. In spring, insolation warms the surface waters. The warm water floats on the cooler deeper waters which have higher density. As time passes, three distinct regions of the water body can be distinguished on the basis of temperature (Fig. 8.14), an upper layer (epilimnion), in which water is relatively warm; a middle layer (metalimnion), in which water temperature changes rapidly; and a lower layer (hypolimnion), in which water temperatures are relatively cold. The temperature (and density) gradient in the metalimnion effectively prevents mixing of epilimnetic and hypolimnetic waters. The deeper water is thus isolated from oxygen replenishment from the surface. This state continues until the stratification breaks down and whole-lake circulation becomes possible when the water body becomes isothermal in fall.

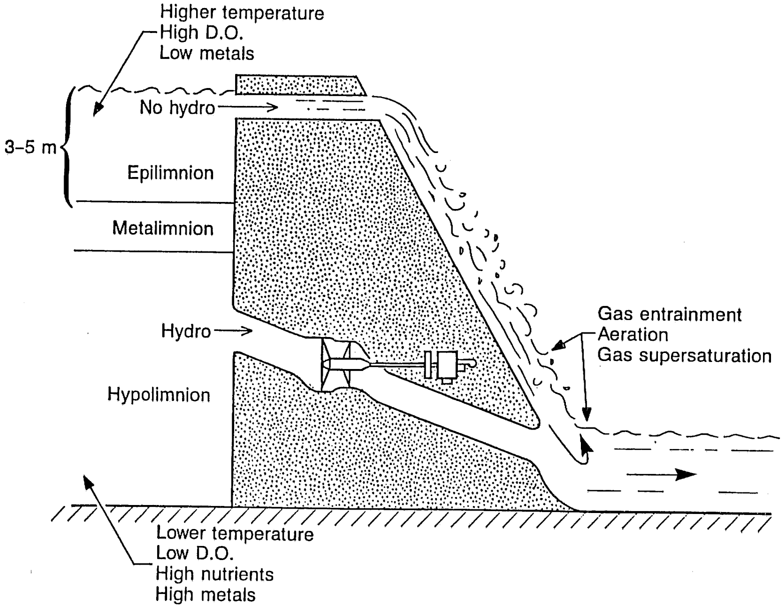


FIGURE 8.14 Water quality downstream from a hydropower site is related to stratification and location of the hydropower intake.

This stratification can result in other water-quality differences besides temperature. Decay of organisms in the hypolimnion combined with lack of oxygen inputs during stratification can result in declining dissolved oxygen and production of anaerobic conditions. This does not occur in the epilimnion, due to oxygen inputs from the air-water interface. On the other hand, nutrients and potential toxicants may be higher in concentration in the hypolimnion than the epilimnion. Nutrients such as nitrate and phosphate are released by decay in the hypolimnetic water. They remain in the water column because light levels are too low to support growth of algae and other plants which remove the nutrients from the water. In addition, reduction of insoluble iron phosphate complexes at the sediment surface causes release to the water column. Finally, other metals, both natural and anthropogenic in origin, are generally more soluble in reduced form. Thus, these metals may be released from the sediments into the hypolimnion as well.

Assessment. Assessment of water-quality effects depends on prediction or measurement of water-quality changes in the reservoir, followed by prediction of responses of plants and animals to those conditions. Models have been developed to predict many of the water-quality changes, but most have not been critically evaluated [49–54]. In addition, some of these models may not always be applicable to small reservoirs. Measurements of conditions in existing reservoirs or lakes in the area of the proposed site may provide reasonable estimates of conditions that can be expected.

Each of the water-quality changes has the potential for impacting the environment in the reservoir and the water body below the dam, but evaluation is not simple. In some cases, interactions of the changes is important for assessment. For example, the occurrence of colder water in the hypolimnion and/or tailwaters released below the dam may allow establishment of a "cold-water" fishery. In the United States, this has led to expansion of salmonid sportfishing to areas where natural waters were originally too warm. However, increased fishery potential may not be possible if, for all or part of the season of stratification, the dissolved oxygen in the water is below the level necessary for fish survival. On the other hand, high dissolved oxygen levels do not necessarily ensure positive effects in cooler tailwaters. In some cases, the temperatures produced may not be low enough to support a cold-water fishery. In this case, decreased growth and productivity of existing warm-water fish may result if temperatures are below optimum for the species in question. An example of fish temperature response is given in Fig. 8.15. Therefore, even for evaluations of the effects of lowered temperature alone, it is necessary to have knowledge of the fish which will be present or could be stocked into the reservoir or tailwaters. A summary of such data for a large number of species can be found in Coutant [56].

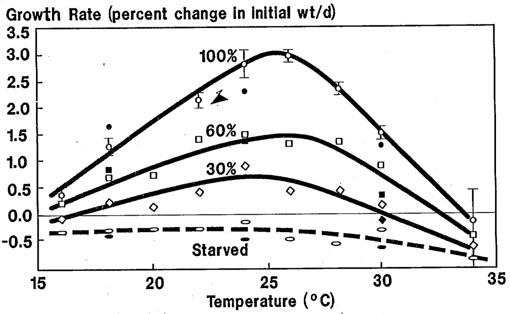


FIGURE 8.15 Growth rates of resident fish in the tailwaters below a dam can be reduced at different feeding levels if the power intake is located in the cold-water zone (hypolimnion) of the reservoir. *(From Cox and Coutant [55], p. 232.)*

Problems involving dissolved gases are twofold: low dissolved oxygen [57] and supersaturation [58, 59]. Fish are dependent on oxygen for respiration. Thus, low DO has the potential to affect survivorship, as well as sublethal responses such as growth and reproduction. Unfortunately, definition of a safe level of DO is not yet clear for either chronic or fluctuating conditions. Some fish do appear more sensitive than others, but choice between 4, 5, and 6 mg/L for chronic exposure is not strongly based in data; 5 mg/L DO appears to have somewhat stronger support, but may not protect all species in some areas and may be overprotective in others.

In 1986, the Environmental Protection Agency (EPA) issued revised water-quality criteria for DO that differed for warm- and cold-water species and life stages for each. These criteria are modified at intermittent intervals. Because of problems in defining safe limits (see above), the most efficient option appears to be to determine the limits in force at the time of licensing and to design the project to meet them.

Supersaturation is not likely to be a significant problem at low-head hydropower sites, but may occur at high-head sites. Supersaturation is largely determined by the amount of air entrainment and the depth of the emitted jet below tailwater level. The higher pressures at these submerged depths acting on the entrained air bubbles will cause supersaturation. As this is being written, the EPA has recommended a limit of 110 percent saturation of total gases to prevent effects. However, an American Fisheries Society committee recommended that where waters are deep, a higher limit would be warranted because fish existing in deeper water (higher pressures) are less susceptible to gas bubble disease.

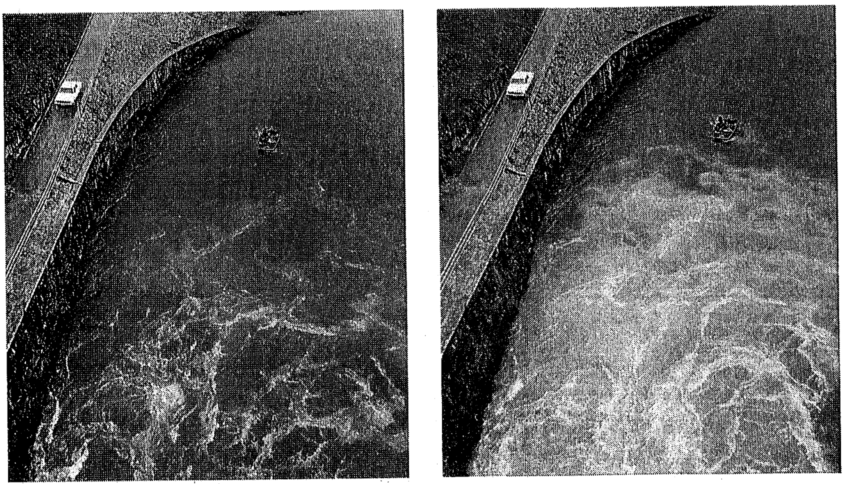
It appears that any limit set on present data will be a compromise. Hatchery mortalities, apparently due to supersaturation, have occurred at levels only a little above 100 percent saturation. Conversely, fish have been observed to behave normally and show no signs of stress in natural waters at saturation levels above those that caused effects in laboratory studies. More work will be required before accurate assessments of the impacts of supersaturation will be possible. For the present, it is only clear that levels above about 110 percent saturation should be viewed with caution and ameliorated if possible. Again, however, these limits may change in the future, so it will be necessary to determine the limits in force at the time of licensing and to attempt to stay below them.

Nutrient and turbidity increase will be treated together in this section, because turbidity can often be related to availability of foods for certain organisms. (See "Dredging" for direct effects of turbidity.) In general, productivity in the reservoir will be elevated over that in the natural stream prior to dam construction. Nutrient influx from streams and from newly inundated soils and vegetation combined with increase in light penetration due to sedimentation of suspended solids is a partial cause. In low-productivity areas, this increase may very well be viewed as positive because production of fish tissue may be enhanced. Conversely, in already highly productive water bodies, this increase may result in hypereutrophic reservoirs that are generally considered unaesthetic and limited in usefulness to humans. Site-specific considerations are thus critical for assessment of effects of reservoir formation on nutrient increase.

Alternatively, the same processes that make reservoirs more productive may also decrease productivity far downstream. An example of this apparently occurred in the Nile River delta region following construction of the Aswan high dam [2]. Since the dam was built, fertilizers have been required for continued high crop production in the delta, presumably because of reduced sediment and nutrient inflows. There has also been a decline in the sardine catch near the Nile delta. This has been attributed to decreased fertilization of the Mediterranean Sea. Short-circuiting of normal nutrient transport because of sedimentation behind the dam is the apparent cause. The overall evaluation of hydroelectric construction on nutrient transport in a drainage basin may thus involve more than just consideration of local effects on the reservoir and tailwaters. However, such effects are not very likely to be severe at most hydropower sites.

Effects of toxicants in both the reservoir and tailwaters are site-specific problems. Metals, ammonia, and hydrogen sulfide appear to be the major potential toxicants in most regions, but in some areas organics may also be problematical. As discussed above ("Dredging"), analyses of sediment samples can probably indicate the scope of the problem. However, it seems likely that, under natural pH conditions (>6.5), toxicants will not prove to have significant ecological effects in areas where there has been little industrial development or use of pesticides. For example, mercury concentrations have been found to increase in fish immediately following the filling of reservoirs at several sites in the United States and Canada. However, the levels of mercury are of concern because of potential effects on human health rather than on the fish *per se* [60]. Even this potential is expected to be transitory [61]. However, there is conflicting evidence about the length of time before mercury in fish tissues declines below public health standards [60, 62].

Mitigation. Although effects of water-quality changes resulting from hydroelectric development can be significant, it is possible to avoid or mitigate them. Run-of-river plants sited on shallow reservoirs have low potential for water-quality impacts. However, even where reservoirs are deep and operation is store-and-release, effects on reservoir and downstream water quality can be mitigated. Nutrient and suspended solid inputs to the reservoir can be flocculated using various chemicals that remove them from the water column and reduce recycling.



(a) (b)

FIGURE 8.16 Turbine aeration is used to increase dissolved oxygen in waters used for power generation at TVA's Norris Dam on the Clinch River: (*a*) without and (*b*) with turbine aeration. *(Courtesy of Charles Massey, Tennessee Valley Authority.)*

Aeration of the reservoir can add oxygen to the water and break up temperature stratification. These mitigation measures decrease downstream water-quality changes by changing reservoir water quality. There are also methods that do not directly change water quality in the reservoir, but do change it downstream. Turbine aeration has been used by the Tennessee Valley Authority and others to increase oxygen content in waters passing through several dams during hydropower generation, as shown in Fig. 8.16. In addition, control of turbine intake level in deep reservoirs so that deep and surface waters are mixed may minimize, if not totally mitigate, changes caused by low temperature and DO and increased nutrients. Clearly, mitigation of the primary effects on water quality and the secondary effects that result involve trade-offs which affect design and operation of hydroelectric generation. However, comprehensive consideration can lead to an optimized approach even with the information that is currently available.

Instream Flow Changes. In the United States, the term instream flow has generally been used to refer to the potential effects of dams on the throughput of waters in a drainage basin and how this affects other uses of water downstream from the dam. Although this can involve both consumptive and nonconsumptive "water rights" issues, the focus has been on commercial and recreational fisheries.

Assessment. Many methods have been developed to try to determine instream flows that will protect resources ranging from riparian vegetation (see "Flow Reduction below Dam" under "Terrestrial Impacts") to fish [63–66]. These methods can be grouped into a number of categories. The six-category classification system developed by Morhardt [63], given in Fig. 8.17, is excellent

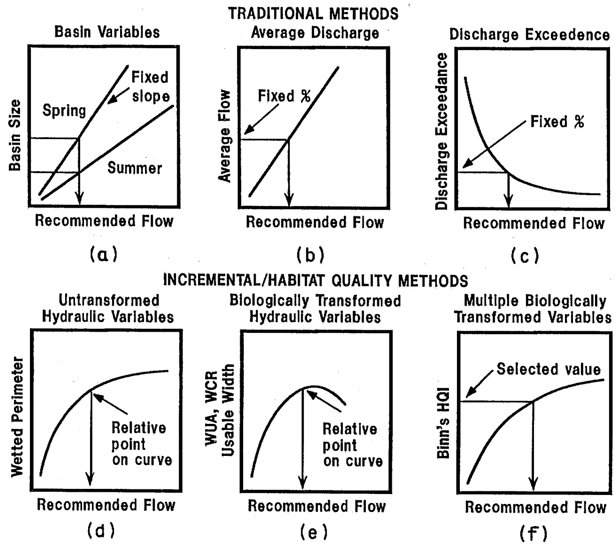


FIGURE 8.17 Categories of procedures available to determine instream flow needs below hydropower plants. *(From Morhardt [63].)*

for differentiating among methods but, from the standpoint of use for licensing, four categories seem more appropriate. Borrowing from the terminology in Loar and Sale [65], the categories are *discharge methods* (*a, b,* and *c* in Fig. 8.17), *hydraulic rating methods* (*d* in Fig. 8.17), *habitat preference methods* (*e* in Fig. 8.17), and *standard crop methods* (*f* in Fig. 8.17). This categorization includes only methods that have been used or developed to recommend appropriate instream flows.

Discharge methods utilize historical stream flow records. They are often called "office" methods because few data are required. Minimum flows are then set at a fixed percentage of mean average flow (MAF), a constant flow for a given watershed area, or a flow that is normally exceeded a fixed percent of the time. The Montana or Tennant method, for example [67, 68], involves calculation of MAF for the site based on extrapolation from data taken at existing gauging stations upstream or downstream from the site or by estimation techniques for ungauged watersheds. Based on data for temperate U.S. streams, Tennant developed a classification of "recommended base flow regimens" for summer and winter indicating effects of various percentages of MAF on biota, given in Table 8.6. Optimum conditions were suggested to occur when base flow was 60 to 100 percent of MAF and obvious degradation to occur below 30 percent of MAF. This method has been widely used and does not require explicit use of biological data.

TABLE 8.6 Instream Flow Regimes Referencing Stream Conditions to Mean Annual Flow

Narrative description of flows	Recommended base flow regimens	
	Oct.–March	April–Sept.
Flushing or maximum	200% of average flow	
Optimum range	60–100% of average flow	
Outstanding	40%	60%
Excellent	30%	50%
Good	20%	40%
Fair or degrading	10%	30%
Poor or minimum	10%	10%
Severe degradation	10% of average flow to zero flow	

Source: Tennant [68], p. 360.

Hydraulic rating methods are based on habitat-discharge relationships obtained empirically or using models. Single- or multiple-stream transects are required depending on criteria for defining habitat, but the most common habitat rating is wetted perimeter. Some biological information is inherent in the choice of habitat ratings, but no biological data are explicitly required. Minimum flows can be set as a fixed percentage loss of habitat or at an inflection in the habitat-discharge relationship (i.e., where habitat loss per unit discharge begins to increase rapidly).

Habitat preference methods utilize species-specific information on habitat selection in the process of defining minimum flow requirements. These methods can also incorporate life-stage and/or seasonal differences. Both physical and biological data are collected along representative transects and occurrence of fish with respect to characteristics such as velocity, depth, substrate, and/or cover used to define optimum habitats. Data collection for one site is shown in Fig. 8.18.

Stream characteristics are then weighted with respect to contributions to defining suitable habitat to construct habitat discharge curves for each species. Where anadromous fish–spawning habitat is of prime concern, usable width in

FIGURE 8.18 Physical hydraulic data collected along transects in representative stream reaches contribute to analyses of instream flow needs using the instream flow incremental methodology. *(Courtesy of John Homa of Ichthyological Associates, a contractor for Niagara Mohawk Power Corporation.)*

spawning areas may be chosen to define suitable habitat. Other indexes proposed as proxies for fish population response have been the Wyoming cover rating and weighted usable area. Minimum flows can then be set at a fixed percentage loss or an inflection point. Habitat preference methods are generally considered to be "state of the science" with respect to determining minimum instream flow requirements, but they involve substantially more effort and cost to accomplish than the previous two categories.

A note of explanation is necessary in including the weighted usable area index, a product of the instream flow incremental methodology (IFIM) [69, 70], as a habitat preference method. The IFIM is not strictly a habitat preference methodology. Initiation of the methodology involves deciding what limiting factor(s) controls fish population size at a particular site [70]. Theoretically, these limits could be due to fishing, availability of food, interspecies interactions, etc., as well as habitat. Where habitat is not limiting, the physical habitat simulation (PHABSIM) analyses would not be conducted. However, in practice this first part of the IFIM analysis is rarely undertaken: habitat *is* assumed to be limiting and PHABSIM *is* used. With apologies to the developers of this method, including IFIM in this category recognizes the realities rather than the theory.

Standing crop models, e.g., Binns and Eisermann [71], are empirical multiple-regression models that use various physical, chemical, and biological parameters to predict fish population size. These models appear more realistic (or theoretically more realistic) than the current crop of habitat preference models in that parameters other than physical habitat can be directly accommodated. In Binns and Eiserman's model, for example, only four of the nine variables selected for use as predictors of fish population size (22 were measured) related to stream flow. Such models show promise but have their own limitations. The regression developed by Binns and Eiserman was tested using two other sets of Wyoming trout streams, in both cases the fit was excellent. However, these are natural streams. Whether the model would accurately predict trout standing crops at reduced flows is purely speculative. In addition, it is doubtful that the same model, without changes in parameters and/or coefficients, could be used in a broader range of trout streams or in other regions of the country.

The importance of instream flow as an issue in determining design and operation of a hydropower plant may be drastically different from site to site. An off-stream plant at which water is diverted a long distance to gain head at the powerhouse has much more potential for impact via changes in instream flow than a run-of-river plant. Biological characteristics such as occurrence of spawning beds of an anadromous fish near the tailwaters at the site also can affect this potential. These differences, and the wide range of costs involved in using the three categories of methods described above, have led to proposals for a hierarchical procedure for evaluating impacts of in-stream flow changes [65, 66]. Each of the procedures involves examination of the site to determine physical, chemical, and biological characteristics and estimation of the likely severity of plant impacts on instream flow. Where these impacts appear slight, discharge methods are used. Perhaps the best of these is the Tennant method. If the method corroborates the preliminary analysis, no further evaluation is recommended. However, as expected severity increases or the discharge method indicates that the preliminary evaluation might have been incorrect, either hydraulic rating or habitat preference methods are indicated. In this way, methods supporting increased resolution, and incurring increased costs, are used only as required, e.g., where need for mitigation may make the plant economically unfeasible.

Methods do exist to evaluate instream flow requirements. It is necessary, however, to use caution in interpreting results of these methods, particularly when applying them to areas beyond where they were developed or in cases where the evaluation indicates that planned flows are near minimum required flows (see Morhardt [63] for critical analyses of all the existing methodologies). The Tennant method appears to have yielded reasonable evaluations for middle to northern areas of the United States, but will not necessarily be accurate for other regions. In addition, the IFIM [69, 72], probably the best method currently available, is dependent on several assumptions (substrate and cover are independent from depth and velocity, weighted usable area is directly proportional to fish standing stock, etc.) that have either not been evaluated or have been evaluated with conflicting results. Binns and Eisermann's multiple-regression analysis performed well in tests but probably would need modification for use in other areas.

From the above it should be clear that instream flow needs assessment is a young and evolving science. Some instream flow methodologies are arbitrary (e.g., discharge methods), but relatively simple to compare with expected plant flows. However, for the more complex methods, choice of method and input parameters, and evaluation and interpretation of results against a background of knowledge and understanding of the local ecosystem, are critical steps toward arriving at a realistic instream flow regime requirement. Let the developer beware! And obtain expert advice so that instream flow needs negotiations will not be simply capitulations.

Mitigation. In some cases effects of reduced instream flows below a powerhouse can be mitigated without increasing water releases. At the Morris power plant, operated by Newfoundland Light and Power Limited, water was diverted away from the Mobile River, resulting in lost spawning habitat for fish. This loss was mitigated by constructing a weir at the end of the tailrace and covering the bottom of the tailrace with gravel to produce substitute spawning habitat, as shown in Fig. 8.19 (personal communication, J. L. Gordon, Monenco Consultants Limited, 2045 Stanley St., Montreal, Quebec). In other cases in Washington and Virginia, enhancement of fish habitat by boulder emplacement and/or construction of channel constrictions, rock deflectors, small boulder dams, and the like were used to reduce required water flows below hydropower projects [73, 74].

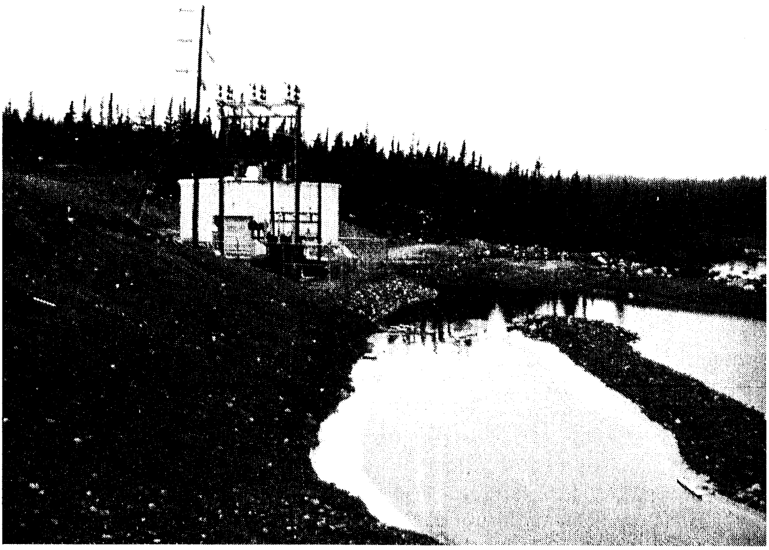


FIGURE 8.19 Spawning channels built to replace habitat lost due to operation of the Morris plant of Newfoundland Light and Power Limited. *(Courtesy of James L. Gordon, Monenco Consultants, Limited.)*

Once in-stream flow requirements have been determined there is little choice but to meet them. However, power losses can be mitigated. For example, the Tennessee Valley Authority constructed a weir below Norris Dam on the Clinch River to extend the period of downstream flows of waters released from the Norris Dam powerhouse. This reduced the need for releases of water when electricity generation was not required [75]. In addition, it is possible to incorporate a single unit, sized to the IFN requirement, or several units of different sizes to accommodate minimal in-stream flow regimes without totally losing the potential power in those releases [76].

8.8 A RECOMMENDED LICENSING STRATEGY

In order to develop a good ecological licensing strategy, it is important to understand the licensing process. This includes recognition of the advantages of promoting cooperation and reducing unnecessary conflict between the many interested parties. (This section is written specifically for nonfederal hydropower development in the United States; details are different, but generally analogous, for federal hydropower development.) The current FERC licensing process consists of submission of a preliminary permit application, then, some time later, a license application. No environmental information is explicitly required in the permit application. The primary purposes of the preliminary permit application, which is not required, are to establish priority for development of the site; to inform FERC, resource agencies, and the public of the proposed development; and to gain authorization to conduct studies. Some of these studies will be used in Exhibit E, the ecological exhibit, that documents the environmental resources at the site, the impacts of the project on those resources, and the mitigation measures to be taken to prevent or reduce those impacts.

Submission of a license application requires consultation with resource agencies at three stages. In stage 1, the developer contacts all appropriate agencies (FERC will supply a list on request), provides all available details on the project and the site, and receives from agency personnel advice on the potential impacts and studies that appear needed to evaluate these impacts. In stage 2, the developer conducts all reasonable (defined in negotiations between the developer and resource agencies) studies to evaluate potential impacts of the project. A draft of Exhibit E is then prepared and submitted with the study results to the resource agencies for comment. This is an iterative process, with modifications made, with or without further study, depending on the developer's evaluation of the severity of the comments. Finally, in stage 3, the developer files a final license application with FERC that includes all of the agency comments, recommendations, and terms and conditions, and the developers' response to this advice. Copies of the application are also sent again to the resource agencies at the same time.

The role of environmental groups or individuals who have concerns about the ecological impacts is less obvious than that of the agencies, but may be just as critical. FERC regulations do not require consultation with these groups or individuals. This may suggest that they are not important participants in the process and can be ignored. Such a conclusion can be an Achilles' heel for a project because a comment period follows submission of a license application. The required agency consultations will often have resolved many concerns. However, public comments may raise significant new issues or issues that were discussed by the developer's and agencies' staffs during stage 1 consultations and dismissed as insignificant. In either of these cases, if documentation is not included in Exhibit E, FERC staff may be forced to declare that the application is deficient and to return it for correction of inadequacies. At the very least, this correction will cost time for modification of the license application plus a further comment period. However, it is also conceivable that correction might require new studies and/or mitigation measures that would radically affect project economics (see next paragraph). A good licensing strategy will include participation of concerned environmental groups and individuals early in the license application process.

One further fact important for setting good licensing strategy involves amendment of applications. If the license is materially amended at any time in the licensing process, a new filing date is assigned. Criteria for material amendation are changes in generating units that would significantly modify flows near or through the plant; changes in the project that would modify the reach of stream affected or increase environmental impacts; or changes in the number of plant units. If a new filing date is assigned, a new adequacy review by the FERC is required before the licensing process can proceed. Each amendment can thus increase the time required to obtain a license. Timely licensing, therefore, partly depends on thorough analysis of impacts and preparation of mitigation plans. This will help ensure that design and operation do not have to be modified after the license application has been filed.

Differences between the ultimate goals and responsibilities of the groups and agencies concerned with hydropower ecological issues often establish conflict situations early in the licensing process. It is to the advantage of the developer to defuse these situations as much as possible. The FERC has delegated responsibility for evaluation of the ecological effects, recommendations on study needs, and evaluation of the sufficiency of mitigation measures to resource agencies. The public also is given a role in the licensing. Thus, these two groups, and the developer, have input to the FERC decision on whether and how the plant will be built and operated to protect the ecology.

But the groups have different ultimate goals and responsibilities. The developer's ultimate goal is generation of electricity to make a profit. Regardless of ecological philosophy, success is judged by the amount of energy and profit the developer generates. Conversely, the ultimate goal of the resource agencies and public intervenors is protection and enhancement of the environment. The two groups tend to have somewhat different focuses. Intervenors often focus on single issues, e.g., trout population effects. Protection of a single environmental use, in this example, fishing, is their goal. In contrast, resource agency personnel generally have broader views of environmental protection, although they may focus on a few target species to indicate whether that protection is assured. Regardless of their personal philosophies about development, agency personnel must protect and enhance the environment in order to successfully uphold the agency's public charge.

If each party to the licensing process adheres strictly to the above goals, it is easy to see how we-they adversarial positions can develop, with each party feeling that it is supporting the public good. However, in most cases such positions do just the opposite because when communication is stymied, effort is expended primarily toward supporting current positions rather than toward looking for alternatives that will accomplish the goals to the maximum. Under this scenario, even if the plant is eventually licensed, much time is wasted in unproductive activity and there are long delays in getting the plant on-line. Public benefits are lessened when this polarization occurs. Alternatively, if a spirit of cooperation can be encouraged, efforts can be focused on evaluating real issues and seeking mitigation alternatives that satisfy (perhaps not wholly) the developers, the agencies, and the public. Because the developer initiates the licensing process and stands to benefit most if the process goes smoothly and quickly, it is his or her responsibility to promote the cooperative atmosphere.

Unless the developer has a history of cooperative interactions with the agencies and the public involved in the licensing process, it may not be reasonable to expect to eliminate entirely the we-they attitude. However, some tactics will help. The strategy recommended below includes some tactics that have proven helpful in the past.

1. *Make sure that an ecological consultant is an integral member of the design team*. Discussions in earlier sections documented the importance of ecological issues to hydropower development and some of the ways that changes in designs and operation can prevent or mitigate ecological effects. However, a power plant must operate as an integrated system. Therefore, choices made early in the design phase may obviate, or make more expensive, alternatives that would reduce environmental impacts. An ecological expert with hydropower experience can predict the likely occurrence and magnitude of potential impacts. He or she can often predict the sensitivity of environmental agencies and groups to these impacts. As a result, more realistic cost-benefit analysis can be factored into decisions about design and operation of the project. In the design phase, a penny spent for ecological advice may represent dollars or months saved later in the licensing process.

2. *Select the ecological consultant carefully*. Three criteria are most important. Foremost, the consultant should be an expert on the ecological effects of hydropower. Hydropower generation is a technology with a unique jargon. If the consultant is not at ease with the jargon, he or she will have difficulties communicating with other members of the design team and with the resources agencies. Furthermore, ecological implications of design or operational choices or changes may not be quickly recognized. Finally, without a thorough knowledge of hydropower effects and existing procedures and technologies for mitigation, opportunities for simple engineering changes that avoid impacts on sensitive species may be missed or discovered too late to be of help.

Secondmost, the ecological consultant should be a person or firm that has interacted successfully with resource agencies (preferably those that will be involved with the current project) in the past. The word "successful" is crucial. Important questions to ask prospective consultants are: how long did it take to get the project licensed; were there disagreements with respect to ecological impacts or how to mitigate them; how were they resolved; and what was the final agreement? One way to evaluate consultants is to talk to agency personnel. But the developer should stick to factual questions such as those above and not ask for specific recommendations about who to engage. Consultants that have only worked on relatively benign projects or have simply acceded to agency suggestions may not be the best choice for a project that is controversial. The longer and more successful the track record, the more desirable the consultant. Working relationships already will have been established and we-they barriers will have been breached. Further, the consultant will be able to anticipate accurately the ecological issues that will be keys to timely licensing, the decision makers who must be convinced, and the types of arguments or mitigation that will be acceptable during negotiation.

The third criterion is that the consultant has prior knowledge of the site, the water body, or the region. Such knowledge gives the consultant an advantage in predicting project impacts even without the benefit of site studies.

If compromises must be made, the above criteria are in order of descending importance; site familiarity is not a substitute for hydropower expertise.

3. *Establish guidelines with the design team*. The design team is established to give the developer the best advice possible. Criteria for best, however, must be viewed on the basis of overall goals of the project. Each expert tends to be biased toward concerns within his or her area of expertise. These can be ecological compatibility, law, cost, efficiency, design elegance, etc. But it is the job of the project manager to integrate these inputs. The project manager will best reach

this goal by stimulating discussion among the design team, weighing suggestions according to expertise, then getting the team to reexamine a preliminary decision. Generally, it is unwise to accept an attorney's evaluation of an ecological issue over that of an ecologist or vice versa. However, discussion can often result in consideration of alternatives that benefit the project. And discussion of important criteria for project success, in the context of FERC regulations, may stimulate unique solutions to licensing questions.

4. *Contact resource agencies early*. Ecological (fisheries, water quality) impacts may be critical issues in determining whether licensing will be successful. In addition, it is very unlikely that resource agency consultations (stages 1 through 3) will consist of single meetings. The process will be iterative because it will involve negotiation. Recognizing this is only part of the task of a successful project manager. Another part is convincing resource agency personnel that the project manager and developer want to build and operate the project with minimal environmental degradation. This can only be accomplished by contacting the agencies early and often, recognizing the potential ecological problems, and discussing them forthrightly with the goal of seeking solutions that will be acceptable to all parties.

Early consultation, even before the official stage 1 (required) meetings helps to establish the developer's ecological sensitivity. Two opportunities exist for relatively nonconfrontative, and therefore nonthreatening, discussions. The first is before an ecological consultant has been chosen. If resource agency personnel have prior experience with ecological consultants being considered, their opinions can aid in selection (but see above). However, comments need to be interpreted. Total, unqualified support may mean that the consultant agreed to all agency suggestions without challenge. Completely negative responses may mean that the consultant fought on every issue whether large or small. "Fair but tough" may indicate a good negotiator. There is no right choice for all projects, but asking for agency inputs may aid in selecting a consultant and will establish initial contacts in a situation where no one has to worry about establishing precedents that will be difficult to change later.

The second opportunity is in planning of site access routes. The ecological consultant can evaluate possibilities from site maps available from the U.S. Geological Survey. A selected access can be presented to agency staff personnel for their consideration. It is not likely that access for initial studies will cause lasting impacts that will be complex to analyze. Again, this procedure demonstrates ecological concern and fosters discussion between project and agency personnel without confrontation.

5. *Conduct a preliminary analysis of potential project impacts before officially initiating stage 1 consultation*. Appropriate rules for this and any other meeting during the licensing process are: do your homework; be prepared; and document decisions. In this case, following the three rules involves making a list of all the potential impacts of the hydropower site (see, for example Table 8.7); eliminating, with reason, impacts that will be small, insignificant, or extremely unlikely; listing potential impacts that overlap with significant resources values (see Mar et al. [77] for less subjective methods to do this); and indicating at least one proposal for mitigating the impact. Take special care to document project interactions with species that are managed by the resource agencies, are subject to commercial exploitation, are of interest to hunters and anglers, or about which no clear decision can be made without further study.

This preliminary analysis has three purposes. First, it establishes the fact that

TABLE 8.7 Impacts of Small-Scale Hydropower Projects on Fish and Wildlife and Their Habitats

Impact and cause	Impact category	Importance
Air quality		
Air pollution due to open burning of construction waste	C	O
Air pollution due to open burning of maintenance refuse	OM	O
Air pollution due to forest- and brushfires	C,OM	O
Dust from fill movements (vehicle and excavation)	C	X
Vehicle exhaust, air pollution from stoves	C,OM	X
Water quality		
Thermal increases downstream (surface outlet)	OM	XX
Thermal increases downstream (inadequate flow)	OM	XX
Thermal decreases downstream (deep release)	OM	XX
Lack of seasonal temperature pattern downstream (due to deep release)	OM	X
Abrupt temperature changes from peaking	OM	X
Low DO in reservoir	F,OM	X
Diurnal DO fluctuation in reservoir	F,OM	O
Low DO below dam	F,OM	XX
Air supersaturation (spillway)	F	X
Turbidity due to dredging and sediment resuspension	C,OM	X
Fine sediment dumped in streams (construction disposal)	C	XX
Excess sediment in stream (diversions and flushing)	C,OM	X
Excess sediment in stream (dredge spoil and construction waste)	C,OM	XX
Excess sediment in stream or lake (bank and bed erosion)	C,OM	X
Sediment from land into stream	C	XX
Decreased sediment load downstream	F,OM	XX
Nutrient increase in reservoir (sediment mobilization)	C	X
Nutrient increase in reservoir (land runoff)	C	X
Nutrient release downstream (reservoir releases)	C,OM	O
Sewage pollution (workers and visitors)	C,OM	X
Toxins mobilized from sediment (in low-DO layer of reservoir)	F,OM	X
Miscellaneous chemical pollution (e.g., oils, fuels, and paint wastes)	C,OM	O
Road salt and sand in streams (maintenance)	OM	O

Herbicides in streams and lakes	OM	O
Pesticides in streams and lakes	OM	O
Aquatic weeds and algae (canal)	OM	O
Water quantity		
Flow cutoff downstream due to reservoir filling	C	X
Inadequate flows downstream from SSH facility	OM	XX
Flow cutoffs for maintenance (canal and tailrace area)	OM	XX
Dewatering reservoir for maintenance	OM	X
Discharge fluctuations		
Rapid downstream flow changes due to peaking	OM	XX
Fluctuating stream energy (from fluctuation flows)	OM	XX
Fluctuating reservoir or lake levels	OM	X
Water level fluctuating on reservoir shore due to peaking	OM	X
Habitat quality		
Stream habitat quality reduced (water shortage)	C,OM	XX
Stream habitat quality reduced (flow fluctuations)	OM	XX
Excessive scouring and substrate instability due to fluctuating flows	OM	X
Siltation downstream (loss of scouring flows)	OM	X
Turbidity injury to fish, invertebrates, and habitats	C,OM	X
Downstream eutrophication (reservoir releases)	OM	X
Aquatic plant species and aquatic habitat diversity decrease	OM	X
Eutrophication in reservoir (sediment mobilization)	C,OM	X
Eutrophication in reservoir (land runoff)	C,OM	X
Soil compaction (loss of infiltration capacity)	C	X
Soil erosion	C	XX
Terrestrial plant species loss and terrestrial habitat diversity decrease	C,OM	X
Habitat altered by transmission right-of-way	C,OM	XX
Habitat loss		
Dewatered reach (when penstock used)	F,OM	XX
Floodplain wetland losses from inadequate and fluctuating flows	C,OM	X
Floodplain habitat cutoff by channelization and/or dewatering	C,OM	X
Filling and debris dumping in wetlands and shore areas	C	X

TABLE 8.7 Impacts of Small-Scale Hydropower Projects on Fish and Wildlife and Their Habitats *(Continued)*

Impact and cause	Impact category	Importance
Inundation of terrestrial habitat above dam	C	XX
Lake shore or bottom wetland losses (inundation)	C	XX
Lake shore or wetland losses (drawdown)	OM	X
Inundation of fish spawning and rearing areas	C	XX
Inundation of flowing water habitat	C	XX
Loss of vegetable and soil cover at borrow pits	C	X
Loss of upland habitat for building and roads	C	X
Blockage of spawning runs (migratory fish)	F	XX
Organism losses		
Vehicle noise	C,OM	X
Fish kill from blasting	C	O
Fishing increase	F,OM	X
Hunting increase	OM	O
Bird and wildlife disturbance by visitors	C,OM	O
Visitor (vandalism toward wildlife)	C,OM	O
Bird mortality at transmission lines	F	XX
Blocking of terrestrial animal movement (at canals or penstock)	C,F	X
Blocking of terrestrial animal movement (reservoir or buildings)	C,F	X
Trapping terrestrial mammals in canals	F	X
Borrow pit hazard to animals	C	O
Loss of aquatic mammal burrow sites	C,F,OM	X
Impoundment predators (fish) or fish	F,OM	X
Bird predators (of fish) in forebay and tailrace	F,OM	X
Stranding of fish and invertebrates downstream due to peaking operations	OM	XX
Disruption and washout of drift species	OM	X
Loss of fish food organisms	F,OM	O
Gas bubble disease in fish	F,OM	X
Diurnal DO fluctuation resulting in fish kills	F,OM	X
Injury to fish-food organisms downstream due to low DO	F,OM	O

Thermal shock (warm or cold) due to peaking	OM	X
Fish upstream migration blockage (at dam)	F	XX
Fish delay in tailrace due to disorientation	F,OM	XX
Disease (injury) to fish due to congregation	F,OM	O
Increased current obstacle to fish movement (at diversion or during construction)	C,F,OM	X
Reservoir as low current barrier or delay factor in fish migration	F	XX
Reservoir thermal barrier to fish	F	O
Loss of reservoir species due to maintenance drawdown	OM	X
Spillway mechanical mortality (fish)	F	XX
Pressure effects on fish in penstock and other water passageways	F	X
Jet (shear) effects on fish in penstock and other water passageways	F	O
Thermal shock (cold to warm) to fish (from travel in deep release)	F,OM	X
Pressure shock (high to low) to fish (from travel in deep release)	F,OM	O
Turbine mortality (fish)	F,OM	XX
Fish injury by impingement	F,OM	XX
Interference with fish spawning (limited sites and inadequate water levels)	C,F,OM	XX
Spawning (fish, amphibians, and reptiles) reductions due to reservoir drawdowns	OM	X
Reservoir biota depletion from entrainment	F,OM	X

Impact category (aspect of an SSH project most likely related to impact):
C = construction
F = facilities (e.g., dam or turbine)
OM = operations/maintenance
Importance (in general, at many SSH plants):
O = slight importance
X = somewhat important
XX = very important
Source: Rochester et al. [11], pp. 163–170.

ecological issues are receiving concentrated attention in project design. This is an important step toward developing a cooperative licensing atmosphere with the agencies and environmental groups. Second, being able to provide the rationale for not considering an issue further helps to avoid "setting" of ideas that some impacts and/or species should be studied. Once such thoughts are expressed and not repudiated or discussed immediately, they tend to gain a momentum of their own and studies are difficult to avoid later regardless of the magnitude or chance of impact. Third, this preliminary analysis helps to demonstrate that further studies have been considered or are recommended as long as they are justified.

6. *Meet with the agencies' staffs*. This is the stage 1 consultation required by FERC. A single meeting with representatives of all of the agencies may appear most efficient at first glance. This may well be the best approach if few ecological impacts are likely or if few agency personnel are involved. However, for less-benign sites or when many agencies have concerns, a single meeting may be too cumbersome to allow everyone to attend and/or too intimidating to promote thorough discussion of issues. In fact, the other extreme, meeting with individuals may be most desirable. One-on-one meetings often allow complex and "touchy" issues to be discussed thoroughly without the pressure of an audience to finish quickly. Of course, this does require that the development team communicate regularly and often. The key concern for the developer is to learn the primary objectives of the resource agencies: are they concerned about managing a few sport or commercial species? or overall productivity? If many agencies or organizations are involved, another plan may be to arrange meetings with individual agencies and use a general meeting to iron out differences and finalize the study plans. This ensures that feedback from each agency is complete and that discussions are less threatening for being less public. After all of the issues have been discussed, a written summary of preliminary agreements as well as issues on which there is still disagreement should be prepared and circulated to the agency involved.

If the developer's initial evaluation was complete, no new issues will have come up at the meetings. However, whether they have or not, there may be issues that have not been decided to the satisfaction of all. There is still time for discussion before a combined meeting of all groups. Further consideration may bring to mind more information that sheds light on the issue. Often, further discussions will be in order. If these are conducted in an atmosphere of investigation rather than confrontation, opinions can change, especially after time has passed for reconsideration. Good ideas are not always recognized as such when they are first proposed. Acceptable alternatives may also be found once areas of concern are fully defined.

7. *Meet with environmental groups or concerned individuals*. These groups or individuals are more difficult to include because they may enter licensing proceedings on an ad hoc basis, but their concerns may be no less important for project licensing (see above). In some cases, groups or individuals may only want to make sure that their concerns are heard; others may want a more official part in the proceedings. Regardless, it is important for their views to be examined, evaluated, and dealt with in the license application. This will help to avoid declaration of deficiency by FERC staff later. Some environmental groups may have sufficient ecological support to be dealt with as resource agencies in the licensing process; others may best be accommodated via a series of local hearings. These hearings can be advertised in local newspapers and held as a (or a series of) public meeting(s). It is best to confine the discussions to fact-finding and avoid argu-

mentation. Notes should be taken of attendees and their concerns after a presentation of preliminary plans for the project. If possible, concerns should be dealt with by writing to individuals on specific issues rather than a general mailing on all topics to all attendees. Nevertheless, each issue raised should become part of the public record submitted with the license application.

8. *Hold a general meeting.* This meeting should include all agencies and some environmental groups that have participated in the licensing proceedings. In the general meeting a preliminary study plan, including any issue still in contention, should be presented so that each agency or environmental group can view the whole. The primary goal should be to obtain agreement that all the important issues have been covered to prevent new issues from being raised after the studies have been completed. Be prepared to accede on some issues that remain in dispute. The FERC process allows applicants to forgo studies that are judged to be unwarranted, but the burden of proof then falls on the applicant. It may be less expensive in the long run to conduct the study. However, it is important to consider results and what they will mean. What result will indicate that there will be no impact? What result will indicate the need for mitigation or further study? What result will indicate unacceptable impact? Once agreement is reached on the study plan, this stage of consultation is complete.

9. *Negotiate mitigation measures to be taken.* This is perhaps the most crucial aspect of licensing; mitigation costs money. One strategy at this stage is to propose little and throw the burden for recommending mitigation on the resource agencies. However, this approach seems likely to backfire. If a spirit of cooperation has been developed, a better strategy appears to be more reasonable. Concentrate efforts on mitigation in the license application on the most important impacts and the areas of most concern to agency personnel. Perhaps in less important areas choose less costly, less than optimal, mitigation measures. In addition, prepare a list of alternative measures.

If agency comments suggest further or different mitigation, arrange meetings for discussions. Be prepared to discuss the comments and your rationale for choices. [Be aware of exchanges that can be made—less costly mitigation of one type of prime habitat for some detriment to another type.] Also consider agreeing to some conditions if they can be changed later based on favorable monitoring results. The best advice here is be flexible and be prepared.

The above suggestions cover the major steps in licensing. They should help to develop a strategy honed to the site and the parties involved. However, nothing can substitute for obtaining the best expertise available, thoroughly and objectively examining the site, choosing a design that will minimize the potential for negative interaction between the plant and the environment, and providing ecological restitution where that interaction cannot be obviated.

8.9 ACKNOWLEDGMENTS

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CHAPTER 9
ECONOMIC AND FINANCIAL ANALYSIS

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9.1 PURPOSE

Inherent in any decision to proceed with the development and construction of a project is the conclusion that its benefits will outweigh its costs. Naturally, there is a desire to assign a single; standard unit of measurement to all benefits and costs, and a unit of monetary value is convenient and tangible. In general, benefits are advantages to the owner, expressed in terms of dollars. Some analyses apply the term "disbenefits" to those items that represent disadvantages to the owner. Costs are the anticipated expenditures for construction, operation, maintenance, etc. [1].

Also fundamental to most economic analyses is the comparison of mutually exclusive alternatives that each achieve the fundamental project objective. Feasibility indicators, such as benefit/cost ratio, net present worth, or internal rate of return (discussed shortly) are calculated, and unless other criteria prevail, the alternative with the highest-valued indicator is the project which should be built.

Of course, a project's benefits and costs are not all tangible, or readily assigned—or even capable of—a monetary valuation. Herein lies the distinction between economic feasibility and financial feasibility:

> *Economic feasibility*, ideally the determination of project benefits and costs from the viewpoint of society as a whole, should include both tangible and intangible benefits and costs in the analysis. However, only tangible benefits and costs can be dealt with accurately from an accounting point of view.... When measures of total benefits accruing from the project exceed the total costs incurred, the project is regarded as economically feasible.
>
> *Financial feasibility*, however, states whether the tangible value of the output

> of the project will be sufficient to amortize the project loan, pay operation and maintenance costs, and meet interest and other financial obligations. Or it can be simply construed as the cash flow situation....
>
> An economically feasible project may not be financially feasible [and vice versa]. [2, p. 11.5]

The decision to proceed with or desist from project implementation will be based not only upon criteria of financial feasibility but also upon judgments rendered in connection with the project's impact on the environment, recreational activities, and historical and cultural legacies. The regulatory process is largely established to address these latter issues.

It should be noted that the production of power may not be the only benefit derived from a hydropower facility, nor are costs always confined to those associated with the operation and maintenance of the generating plant. To serve the purpose of accurately reckoning other costs and benefits, such as water supply, flood control, recreation, and other water resources functions, a methodology of cost accounting is treated in a manual of the U.S. Army Corps of Engineers (1958) [3].

Economic analyses are distinct from financial analyses in that the former are performed to compare project alternatives or to determine if the benefits and costs directly resulting from a given alternative will warrant its development; the latter is concerned with financing structures and their ramifications for return on investment, cash flow, tax benefits, depreciation schedule, etc.

The distinction between economic and financial analyses is not always clear in application. For example, the comparison of development alternatives may necessarily entail a consideration of different tax benefits to be derived from those alternatives. Until recently, the Investment Tax Credit and the Energy Tax Credit, under provisions of the Crude Oil Windfall Profit Tax Act of 1980, were two major benefits of possible import in the economic evaluation.

An economic evaluation from Christensen et al. [4] demonstrates that rate of return (discussed shortly) may depend significantly on debt/equity ratio. In columns (1) through (6) of Table 9.1, the debt to equity ratio is zero, the capital costs of this 18-MW hydropower project are $28,700,000, and the rate of return (or internal rate of return, IRR) is 21 percent. If 75 percent of the capital costs of the project had been borrowed for a term of 15 years at a rate of 13 percent, reducing the equity investment to $7,175,000, the rate of return would be 41 percent, as seen in columns (7) through (12). In this analysis, debt leveraging (borrowing capital from outside sources) has increased the rate of return by 20 percentage points. Such an increase will occur when the IRR without debt leveraging exceeds the interest rate on borrowed capital. When the IRR without debt is less than the interest rate, the IRR will decrease with borrowed capital.

9.2 INDICATORS OF ECONOMIC FEASIBILITY

Fixed Parameters

Certain financial assumptions are required to formulate benefit and cost streams, and since an economic analysis is primarily concerned with the economic feasibility of a given alternative (or alternatives), these assumptions may be incorporated in the analysis as fixed parameters. Interest rate is an exception to this

TABLE 9.1 Economic Evaluation with and without Debt Leveraging

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
	Without debt leveraging					With debt leveraging					
Year	Capital cost	Annual O&M costs	Total annual cost	Gross annual revenue (flat rate)	Net annual benefits (costs)	Equity invest-ment	Debt service	Annual O&M costs	Total annual cost	Gross annual revenue (flat rate)	Net annual benefits (costs)
0	$28,700,000		$28,700,000		($28,700,000)	$7,175,000	$7,175,000		($7,175,000)		
1		$1,200,000	$1,200,000	$7,600,000	$6,400,000		$3,330,000	$1,200,000	$4,530,000	$7,600,000	$3,070,000
2		$1,249,000	$1,249,000	$7,600,000	$6,351,000		$3,330,000	$1,249,000	$4,579,000	$7,600,000	$3,021,000
3		$1,301,430	$1,301,430	$7,600,000	$6,298,570		$3,330,000	$1,301,430	$4,631,430	$7,600,000	$2,968,570
4		$1,357,530	$1,357,530	$7,600,000	$6,242,470		$3,330,000	$1,357,530	$4,687,530	$7,600,000	$2,912,470
5		$1,417,557	$1,417,557	$7,600,000	$6,182,443		$3,330,000	$1,417,557	$4,747,557	$7,600,000	$2,852,443
6		$1,481,786	$1,481,786	$7,600,000	$6,118,214		$3,330,000	$1,481,786	$4,811,786	$7,600,000	$2,788,214
7		$1,550,511	$1,550,511	$7,600,000	$6,049,489		$3,330,000	$1,550,511	$4,880,511	$7,600,000	$2,719,489
8		$1,624,047	$1,624,047	$7,600,000	$5,975,953		$3,330,000	$1,624,047	$4,954,047	$7,600,000	$2,645,953
9		$1,702,730	$1,702,730	$7,600,000	$5,897,270		$3,330,000	$1,702,730	$5,032,730	$7,600,000	$2,567,270
10		$1,786,921	$1,786,921	$7,600,000	$5,813,079		$3,330,000	$1,786,921	$5,116,921	$7,600,000	$2,483,079
11		$1,877,006	$1,877,006	$7,600,000	$5,722,994		$3,330,000	$1,877,006	$5,207,006	$7,600,000	$2,392,994
12		$1,973,396	$1,973,396	$7,600,000	$5,626,604		$3,330,000	$1,973,396	$5,303,396	$7,600,000	$2,296,604
13		$2,076,534	$2,076,534	$7,600,000	$5,523,466		$3,330,000	$2,076,534	$5,406,534	$7,600,000	$2,193,466
14		$2,186,892	$2,186,892	$7,600,000	$5,413,108		$3,330,000	$2,186,892	$5,516,892	$7,600,000	$2,083,108
15		$2,304,974	$2,304,974	$7,600,000	$5,295,026		$3,330,000	$2,304,974	$5,634,974	$7,600,000	$1,965,026
16		$2,431,322	$2,431,322	$7,600,000	$5,168,678	$2,431,322	$2,431,322	$7,600,000	$5,168,678		
17		$2,566,515	$2,566,515	$7,600,000	$5,033,485	$2,566,515	$2,566,515	$7,600,000	$5,033,485		
18		$2,711,171	$2,711,171	$7,600,000	$4,888,829	$2,711,171	$2,711,171	$7,600,000	$4,888,829		
19		$2,865,953	$2,865,953	$7,600,000	$4,734,047	$2,865,953	$2,865,953	$7,600,000	$4,734,047		
20		$3,031,569	$3,031,569	$7,600,000	$4,568,431	$3,031,569	$3,031,569	$7,600,000	$4,568,431		
21		$3,208,779	$3,208,779	$7,600,000	$4,391,221	$3,208,779	$3,208,779	$7,600,000	$4,391,221		
22		$3,398,394	$3,398,394	$7,600,000	$4,201,606	$3,398,394	$3,398,394	$7,600,000	$4,201,606		
23		$3,601,281	$3,601,281	$7,600,000	$3,998,719	$3,601,281	$3,601,281	$7,600,000	$3,998,719		
24		$3,818,371	$3,818,371	$7,600,000	$3,781,629	$3,818,371	$3,818,371	$7,600,000	$3,781,629		
25		$4,050,657	$4,050,657	$7,600,000	$3,549,343	$4,050,657	$4,050,657	$7,600,000	$3,549,343		
Totals:	$28,700,000	$56,774,326	$85,474,326	$190,000,000	$104,525,674	$28,700,000	$49,950,000	$56,774,326	$113,899,326	$190,000,000	$76,100,674
				IRR:	21.042%					IRR:	41.042%

since: (1) the discount rate is tied to it (usually equated with it), and (2) hydropower projects usually require an exceptionally large ratio of debt to equity [5].

Fixed parameters in an economic analysis could include the life of the project and salvage value, the term of the loan, and the terms of a power purchase contract. Of course, a financial analysis might consider these parameters to be variable, and the principles of uncertainty discussed in Sec. 9.3 could then be applied.

Life of Project and Salvage Value

The life of a project is the duration of time it is expected to be functional before major rehabilitation and replacements, or a cessation of operation, become necessary. Project life may also connote the period for which the project is licensed, or the length of time over which the developer maintains a financial interest in the project. The value of that interest at the end of the project life may be termed the "salvage value," and it represents the remaining worth of the project, including that of project lands, civil works, used equipment, and continued revenue-generating capability (if any). An asset may not be depreciated below its net salvage value.

Typical large-scale hydropower installations have a project life span in excess of 50 years, while for microhydropower facilities (capacity less than 100 kW) operating under adverse conditions found in many rural applications, the useful life can be as low as 10 years. Economic and financial feasibility will, therefore, depend not only on the period of time the developer maintains an interest in the facility but also on the useful life of the generating and auxiliary equipment and appurtenant civil works.

The U.S. Internal Revenue Service has established a 50-year composite useful life for a hydroelectric production plant, which includes most of the assets used in the hydropower production of electricity for sale, including related land improvements, such as dams, flumes, canals, and waterways. For tax purposes, the choice of a useful life which is within 20 percent of the asset guideline period will not be challenged. It should be noted that the hydroelectric plant structure is so closely interrelated with the turbine and generator in terms of use, function, and design that if any part of the unit ceases to be useful, the remaining components must be abandoned contemporaneously [5].

Terms of Power Purchase Contract

Conditions and structures of payment for capacity and energy are established in the terms of the power purchase contract, and this will bear on the financial feasibility of the project. The economic analysis may be more conveniently performed by assuming one combined and levelized payment for energy and capacity (usually expressed as mills per kilowatt-hour) for the life of the project. The determination of this amount, which will itself be a variable parameter, may be based on utility projections, the projections of regulatory agencies such as a state's public utility commission in the United States, or current rates subject to probable inflationary pressures. Alternatively, an inflationary effect may be included in the revenue stream in the event of a period of high inflation, since capital-intensive projects—which hydropower typifies—would be systematically replaced by less profitable labor-intensive projects if the revenue stream were constructed on a constant-dollar basis [6].

Benefit and Cost Streams

After determining the financing structure and the values of the parameters affecting the economic and financial feasibility of a project, a benefit and cost stream may be constructed, similar to that of Table 9.2. The stream will track the cash flow throughout the economic life of the project.

The capital costs of the project (the total of all funds required to develop the project, including interest expenses during construction) may be treated as a one-time expenditure (equity financing), a loan amortized over a period of time (debt financing), or a combination of equity and debt financing. Debt service represents the amortization of the loan over the repayment period; it is the sum of principal and interest due annually and is determined as

$$D = C_c\left[\frac{i(1+i)^m}{(1+i)^m - 1}\right] \tag{9.1}$$

where D = annual debt service
C_c = capital costs
i = interest rate
m = amortization period

Christensen et al. [4] suggest that in an economic analysis it may be preferable to treat the capital cost of the project as a short-term expenditure over the construction period, as exhibited in column (2) of Table 9.1, rather than attempting to account for debt service as an annual cost. The latter would require assumptions for rate of interest and duration of financing which, if incorrect, could significantly distort the economic evaluation of the project.

The cost stream may reflect expected escalations in the current annual operation, maintenance, and replacement (OM&R) costs as per the aging of the facility and equipment, in addition to the effects of inflation. The overall escalation rate is then applied to the current-year value so that

$$\mathrm{OM\&R}_n = \mathrm{OM\&R}_c \times (1+e)^n \tag{9.2}$$

where $\mathrm{OM\&R}_n$ = OM&R costs in year n
$\mathrm{OM\&R}_c$ = current OM&R costs
e = annual escalation rate on $\mathrm{OM\&R}_c$

A good source of information on price escalation for various components of hydropower developments is *Construction Cost Trends* of the U.S. Department of Interior, Bureau of Reclamation [7].

Actual data on costs of constructing plants and actual operation and maintenance costs to check against estimating curves may be found in the annual publication of the U.S. Department of Energy entitled *Historical Plant Cost and Annual Production Expenses for Selected Electric Plants*, formerly entitled *Hydroelectric Plant Construction Cost and Annual Production Expenses* prior to 1982 [8]. Estimating curves for OM&R are given in Ref. 9. Care should be taken in using these curves because actual OM&R costs vary widely.

The annual revenues generated by the sale of capacity and energy to a util-

TABLE 9.2 Benefit and Cost Streams of the St. Anthony Falls Minihydropower Facility

Financial assumptions

Project cost + 1st-year debt service	$231,121	Annual energy production & revenues:	
Term of loan	10 yrs	Power ouput	125.7 kW
Interest rate	8.625%	Availability of flow	91.0%
Project life	25 yrs	Hours of production	7971.6 h
Salvage value	$0	Energy production	1002030. kW
Operation, maintenance & replacement:		NSP buyback rate	0.038 $/kw h
Average annual OM&R costs	$2500	Revenue	$38,007
Escalation rate	7.250%	Escalation rate	0.0%

Benefit and cost streams*

Year	Debt service	OM&R costs	Revenues	Present worth: Benefits	Present worth: Costs	Present worth: Cash flow	Net present value
1	35,421	0	0	0	32,609	(32,609)	(32,609)
2	35,421	2,876	38,077	32,270	32,457	(186)	(32,795)
3	35,421	3,084	38,077	29,708	30,042	(334)	(33,129)
4	35,421	3,308	38,077	27,349	27,817	(468)	(33,597)
5	35,421	3,548	38,077	25,178	25,767	(589)	(34,186)
6	35,421	3,805	38,077	23,179	23,878	(699)	(34,885)
7	35,421	4,081	38,077	21,338	22,136	(798)	(35,684)
8	35,421	4,376	38,077	19,644	20,531	(888)	(36,571)
9	35,421	4,694	38,077	18,084	19,052	(968)	(37,539)
10	35,421	5,034	38,077	16,648	17,688	(1,040)	(38,579)
11	0	5,399	38,077	15,326	2,173	13,153	(25,425)
12	0	5,790	38,077	14,109	2,146	11,964	(13,462)

13	0	6,210	38,077	12,989	2,118	10,871	(2,591)
14	0	6,660	38,077	11,958	2,092	9,866	7,275
15	0	7,143	38,077	11,008	2,065	8,943	16,218
16	0	7,661	38,077	10,134	2,039	8,095	24,313
17	0	8,217	38,077	9,329	2,013	7,316	31,630
18	0	8,812	38,077	8,589	1,988	6,601	38,231
19	0	9,451	38,077	7,907	1,963	5,944	44,175
20	0	10,136	38,077	7,279	1,938	5,341	49,516
21	0	10,871	38,077	6,701	1,913	4,788	54,304
22	0	11,660	38,077	6,169	1,889	4,280	58,584
23	0	12,505	38,077	5,679	1,865	3,814	62,398
24	0	13,411	38,077	5,228	1,841	3,387	65,784
25	0	14,384	38,077	4,813	1,818	2,995	68,779
26	0	15,427	38,077	4,431	1,795	2,636	71,415

Economic Analysis:
Net present value = $71,415
Benefit cost ratio = 1.25
Internal rate of return = 17.1%

*Beginning with first year of construction. All figures in $. Present worth base year = 1987.

ity may either be fixed for the duration of the project's life, in accordance with a power purchase contract that would establish levelized rates; or they may be based upon the utility's actual avoided costs during the years of the contract (Table 9.3). In the latter event, an escalation rate would be applied to the utility's current avoided costs to construct the revenue stream. There may also exist the possibility that capacity payments will be fixed, while energy payments will accord with actual avoided energy costs. To qualify for capacity payment, energy must be determined to be dependable, that is, most always available, especially during the purchaser's peak load period. The percent availability that makes a hydroplant "dependable" is subject to interpretation, because no source of power is 100 percent dependable. Often, the definition of dependable is that which is applied to the other generating plants in the local electrical grid. In the United States, subject to the Public Utility Regulatory Policy Act (PURPA) of 1978, the Federal Energy Regulatory Commission (FERC) has issued standards to be followed by regulatory commissions and unregulated utilities in setting rates [9]. Power value determinations—the basis for the establishment of rates—are most often achieved by two common methods. These are presented by Barbour et al. [10] as the "alternative thermal plant" method and the "systems planning model" method.

Ideally, levelized rates are determined such that for the first half of the contract period they are higher than the actual rates, and for the second half of the contract period they are lower than the actual rates, with the result that the levelized rates will balance with the actual rates over the term of the contract. Of course, the establishment of levelized rates involves future projections with inherent uncertainties. A contract would likely be structured so that accumulated debits and credits, resulting from discrepancies between actual costs and the cost projections used to derive levelized rates, are accounted for as the contract nears expiration.

For the early years of the 1970s, retail electric rate increases lagged below the rising cost of living, as measured by the Bureau of Labor Statistics Consumer Price Index. However, in the last half of that same decade, electric rates—spurred by soaring fuel costs—overtook and passed the cost of living [11]. Consequently, it may be necessary in benefit and cost projections to escalate various cash inflows and outflows at different rates.

Annual revenue is the product of cost per kilowatt-hour (comprising both capacity and energy costs) and average annual kilowatt-hours generated:

$$R = U \times \text{kWh} \tag{9.3}$$

where R = annual revenue
U = utility buyback rate or value of energy, \$/kWh
kWh = annual production of energy, kilowatt-hours

For each year, benefits (i.e., revenues) and costs (debt service plus OM&R) are discounted to their present worth value using the discount rate

$$B_{n\text{PW}} = \frac{R_n}{(1 + d)^n} \tag{9.4}$$

$$C_{n\text{PW}} = \frac{D_n + \text{OM\&R}_n}{(1 + d)^n} \tag{9.5}$$

TABLE 9.3 Economic Evaluation of Avoided-Cost Power Sales Contract

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
						Discounted at 20%			Discounted at 25%			Discounted at 23.378		
Year	Capital cost, $	Annual O&M cost, $	Total annual costs, $	Gross annual revenue (avoided cost), $	Net annual benefits (costs), $	Discount factor	Annual costs, $	Annual benefits, $	Discount factor	Annual costs, $	Annual benefits, $	Discount factor	Annual costs, $	Annual benefits, $
0	28,700,000		28,700,000		(28,700,000)	1.000	28,700,000		1.000	28,700,000	0	1.000	28,700,000	0
1		1,200,000	1,200,000	5,218,400	4,018,400	0.833	1,000,000	4,348,667	0.800	960,000	4,174,720	0.811	972,621	4,229,603
2		1,249,000	1,249,000	5,653,600	4,404,600	0.694	867,361	3,926,111	0.640	799,360	3,618,304	0.657	820,516	3,714,066
3		1,301,430	1,301,430	6,258,400	4,956,970	0.579	753,142	3,621,759	0.512	666,332	3,204,301	0.532	692,959	3,332,346
4		1,357,530	1,357,530	7,371,200	6,013,670	0.482	654,673	3,554,784	0.410	556,044	3,019,244	0.432	585,866	3,181,173
5		1,417,557	1,417,557	7,960,896	6,543,339	0.402	569,684	3,199,306	0.328	464,505	2,608,626	0.350	495,852	2,784,667
6		1,481,786	1,481,786	8,756,000	7,274,214	0.335	496,247	2,932,367	0.262	388,441	2,295,333	0.284	420,106	2,482,444
7		1,550,511	1,550,511	9,084,800	7,534,289	0.279	432,719	2,535,401	0.210	325,166	1,905,221	0.230	356,296	2,087,619
8		1,624,047	1,624,047	9,573,600	7,949,553	0.233	377,701	2,226,513	0.168	272,470	1,606,184	0.186	302,480	1,783,091
9		1,702,730	1,702,730	10,596,000	8,893,270	0.194	330,001	2,053,576	0.134	228,537	1,422,171	0.151	257,043	1,599,567
10		1,786,921	1,786,921	11,324,800	9,537,879	0.162	288,598	1,829,018	0.107	191,869	1,215,991	0.122	218,639	1,385,649
11		1,877,006	1,877,006	12,157,600	10,280,594	0.135	252,622	1,636,267	0.086	161,234	1,044,330	0.099	186,145	1,205,682
12		1,973,396	1,973,396	13,151,200	11,177,804	0.112	221,330	1,474,995	0.069	135,611	903,744	0.080	158,621	1,057,092
13		2,076,534	2,076,534	14,235,200	12,158,666	0.093	194,081	1,330,477	0.055	114,159	782,588	0.065	135,285	927,413
14		2,186,892	2,186,892	15,401,600	13,214,708	0.078	170,329	1,199,578	0.044	96,181	677,370	0.053	115,478	813,275
15		2,304,974	2,304,974	16,678,400	14,373,426	0.065	149,605	1,082,519	0.035	81,099	586,819	0.043	98,651	713,820
16		2,431,322	2,431,322	18,059,200	15,627,878	0.054	131,505	976,784	0.028	68,436	508,321	0.035	84,341	626,462
17		2,566,515	2,566,515	19,164,800	16,598,285	0.045	115,681	863,820	0.023	57,793	431,553	0.028	72,161	538,844
18		2,711,171	2,711,171	21,019,200	18,308,029	0.038	101,834	789,503	0.018	48,840	378,648	0.023	61,784	479,002
19		2,865,953	2,865,953	22,688,000	19,822,047	0.031	89,707	710,154	0.014	41,303	326,969	0.018	52,936	419,063
20		3,031,569	3,031,569	24,605,600	21,574,031	0.026	79,076	641,814	0.012	34,952	283,683	0.015	45,385	368,366
21		3,208,779	3,208,779	26,355,200	23,146,421	0.022	69,748	572,875	0.009	29,596	243,084	0.012	38,936	319,797
22		3,398,394	3,398,394	28,604,800	25,206,406	0.018	61,558	518,145	0.007	25,076	211,066	0.010	33,423	281,325
23		3,601,281	3,601,281	30,484,000	26,882,719	0.015	54,361	460,154	0.006	21,258	179,946	0.008	28,707	242,999
24		3,818,371	3,818,371	33,275,200	29,456,829	0.013	48,032	418,573	0.005	18,032	157,138	0.006	24,670	214,989
25		4,050,657	4,050,657	36,668,800	32,618,143	0.010	42,461	384,384	0.004	15,303	138,531	0.005	21,212	192,023
Totals:	$28,700,000	$56,774,326	$85,474,326	$414,346,496	$328,872,170		$36,252,059	$43,287,543		$34,501,594	$31,923,883		$34,980,114	$34,980,378
						Benefit cost ratio:		1.19	Benefit cost ratio:		0.93	Benefit cost ratio:		1.00
						Net present worth:		$ 7,035,485	Net present worth:		($2,577,711)	Net present worth:	$	264

where $B_{n\mathrm{PW}}$ = present worth of year n benefits
$C_{n\mathrm{PW}}$ = present worth of year n costs
d = discount rate

This is the rate of return that could be earned by investing the capital cost of the project in a venture of similar risk or an alternative marginal project. The discount rate to use for testing economic feasibility is the opportunity cost of capital to society, which is equated with the interest rate on the tax-exempt bonds sold by public agencies to obtain construction funds.

The present-worth cash flow is equal to the present-worth benefits minus present-worth costs. The yearly net present value of the project is merely the summation, up to the year in which it is calculated, of present-worth cash flows.

The payback period of the project is the time required to fully pay for the project, and it is that year of operation in which the net present value of the project becomes positive.

From this spreadsheet format, the feasibility indicators may be determined.

Net Present Value

The net present value (NPV) of the project is simply the summation of differences between present-worth benefits and costs over the life of the project.

$$\mathrm{NPV} = \sum_{n=1}^{k} (\text{cash flow})_{n\mathrm{PW}} = \sum_{n=1}^{k} (B_{n\mathrm{PW}} - C_{n\mathrm{PW}}) \tag{9.6}$$

where NPV = project net present value
k = project life, years

A negative net present value would be unacceptable.

Net present value has the disadvantage that it yields no information about the ratio of benefits to costs.

Benefit/Cost Ratio

The benefit/cost ratio (B/C) is the present worth of accumulated project benefits divided by the present worth of accumulated project costs, the accumulation extending over the life of the project:

$$\frac{B}{C} = \frac{\sum_{n=1}^{k} B_{nPW}}{\sum_{n=1}^{k} C_{nPW}} \tag{9.7}$$

where B/C = project benefit/cost ratio

The benefit/cost ratio alone is insufficient, since the magnitudes of actual costs and benefits are not revealed. More pertinent to the sizing of plant capacity may be the incremental benefit/cost ratio associated with an increase of capacity. A

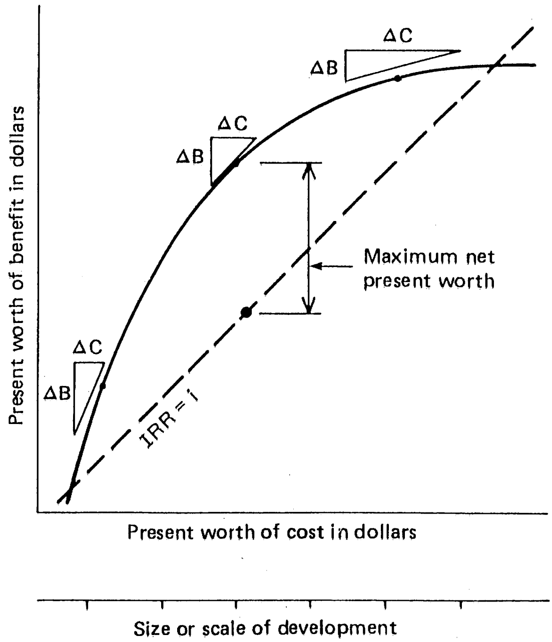

FIGURE 9.1 Graphic representation of benefits versus costs for varying size of development. (*From Warnick [13].*)

public development will normally size plant capacity to maximize net present value (net benefits) [12]. This capacity is the size for which the incremental benefit/cost ratio moves from above 1.0 to below 1.0, represented graphically in Fig. 9.1 [13]. In this representation, present-worth benefits are plotted against present-worth costs for different scales of development or different alternatives. The 45° line would signify a net present worth of zero (the case where the discount rate is set to the internal rate of return, discussed next), and the maximum net present worth is realized at that point where the slope of the plot is equal to 45°, that is, where the incremental benefit/cost ratio is unity.

Net present value and benefit/cost ratio will not often size the plant at the same capacity. This is illustrated in Fig. 9.2 [12], where the net discounted benefit, incremental benefit/cost ratio, and the benefit/cost ratio are plotted as a function of the percent exceedance at which the plant was sized for an example site. The figure shows that the maximum benefit/cost ratio occurs at the 55 percent exceedance level on the flow-duration curve, which does not maximize net discounted benefits, the latter reaching a maximum at the 39 percent exceedance level.

A private developer would normally be more concerned that a given benefit/cost ratio or internal rate of return be exceeded, sizing a plant somewhere between the maximum benefit/cost ratio and the maximum net present value. Maximizing net worth is of greater interest to public developments, since tax-exempt bond financing is likely to provide a large part or all of the capital financing required to develop most small hydropower sites [14].

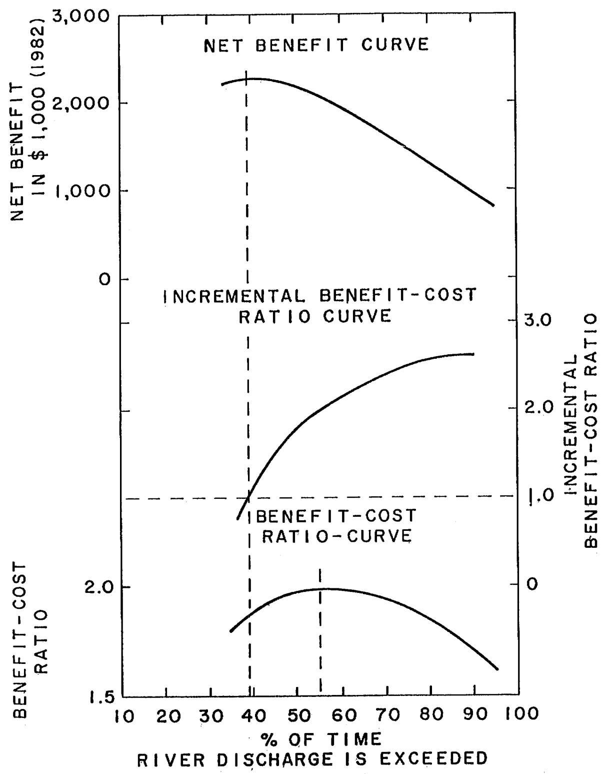

FIGURE 9.2 Example of benefit/cost ratio, incremental benefit/cost ratio, and net discounted benefit as a function of the design percent exceedance for a proposed hydropower facility.

Internal Rate of Return

The internal rate of return (IRR) is basically the return on investment. It is that discount rate at which the net present value will be equal to zero (i.e., the discount rate that equates the present value of expected cash outflows with the present value of expected inflows). Insofar as it is the rate of return, or profit, that an investment is expected to earn, the IRR determines the attractiveness of an investment opportunity.

The IRR is calculated by an iterative process in which an initial estimated discount rate is provided and then adjusted on subsequent iterations until that discount rate is found for which the summation of discounted present-worth cash flows (net present value) is zero. This discount rate is then the IRR.

Generally, a developer will establish a minimum attractive rate of return (MARR), and a project or investment opportunity whose IRR is less than the MARR would be unacceptable. Alternatively, the discount rate may be set equal to the MARR, and if the subsequent analysis yields a benefit/cost ratio less than one, that particular project is unacceptable. Table 9.3, an evaluation from Christensen et al. [4], shows a project that would be unacceptable for MARRs of more than about 23.4 percent.

One advantage of rate-of-return analysis is not having to preselect an interest rate for discounting purposes, thereby eliminating this uncertain parameter from consideration in the evaluation.

Even if NPV, *B/C*, or IRR show a project to be feasible, the project may encounter cash-flow difficulties in the early years and thus would encounter serious financing problems. Financing considerations are critical even if a project looks "feasible" from *B/C* calculations. For the St. Cloud hydropower economic analysis of Case Study 2, for example, the income due to energy sales would be significantly reduced in the event that flows reached exceedingly low values during the first year of operation. The City of St. Cloud would need to be prepared for the negative cash flow (about $250,000) resulting from a dry year in the first year of operation.

Other financing considerations will include risks associated with the construction period, including *design risks*, involving questions regarding the overall design and engineering of the project; *completion risks*, involving delays in construction resulting in late completion and startup; *force majeure risks*, or risks which are beyond the control of the developer or contractor such as floods, war, civil strife, and strikes; and *construction cost risks*, which are associated with cost contingencies and overruns. Strategies designed to address these risks for economically feasible projects are highlighted by Schaefer [15].

9.3 THE PARAMETERS OF AN ECONOMIC ANALYSIS AND THEIR ASSOCIATED UNCERTAINTIES

Economic analyses for construction of power production facilities are infamous for their lack of precision. Even if construction costs are estimated with relative accuracy, the value of energy and power, fuel costs, operation and maintenance costs, etc., must be projected 30 years or more into the future. Every analysis is therefore viewed with a natural suspicion by planners, developers, entrepreneurs, bankers, etc. The individual reviewer usually adds an ad hoc uncertainty to the economic analysis. Psychologists have established [16] that people generally underestimate the uncertainties associated with an expected event. Even experts are subject to this overconfidence factor. The value of uncertainty analysis is that most decision makers are risk adverse, or they are willing to accept a poor expected return if the risk is sufficiently reduced [17, 18]. It would therefore be worthwhile for engineers involved in an economic analysis to have a formal means of expressing the potential inaccuracies of their estimates. An appropriate concept for expressing inaccuracies is an uncertainty, where the value is provided by an uncertainty analysis. Feasibility indicators computed in an economic or financial analysis—net present value, benefit/cost ratio, and internal rate of return—all have uncertainties associated with them, and it is useful to evaluate them. These values find expression in relation to probabilities, where a feasibility indicator is reported as

$$\mathrm{FI} = \widehat{\mathrm{FI}} \pm \delta(\mathrm{FI}) \qquad (P = p\%) \tag{9.8}$$

where FI = a feasibility indicator

$\widehat{\mathrm{FI}}$ = expected value of FI

$\delta(\mathrm{FI})$ = uncertainty delineating a $p\%$ confidence interval for FI

From this we conclude that the actual FI value for the project will, with a $p\%$ probability, fall within the range delineated by Eq. (9.8). There is a $(p/2)\%$ probability that it will fall above the range and a $(p/2)\%$ probability that it will fall below.

Although it is useful to perform sensitivity studies where the impact of variations in project parameters and financial assumptions on the feasibility indicators is investigated, an uncertainty analysis attempts to go a step further. If one desires to know with what probability a certain indicator will fall within a given range of values, uncertainty analysis can provide an answer. Furthermore, uncertainty in a feasibility indicator reflects all the variable parameter uncertainties by which the indicator is computed to begin with.

The normal distribution assumed by Eq. (9.8) does not occur naturally in economic analyses. Even when the independent parameters, such as discount rate, capital costs, etc., have a normal distribution, the relationship between FI and discount rate or escalation rate is nonlinear, resulting in a skewed probability distribution of FI. In the technique described herein, the probability distribution of FI relative to each uncertain parameter will be forced into a normal curve by equating the standard deviation of the skewed distribution with that of the normal distribution, i.e., the feasibility indicators will be linearized. The effect of these errors on the uncertainties computed for the feasibility indicator depend upon the relative size of the confidence intervals for discount rate and escalation rate.

More detailed and thorough analysis of FI uncertainties would involve a number of possible probability distributions and a Monte Carlo type of simulation [19, 20, 21]. The purpose of this chapter is to present a fairly simple economic uncertainty analysis of energy projects that will illustrate the concept and the use of the results without delving too deeply into the statistical properties of these probability distributions.

Projections or predictions of an economic nature are very vulnerable to the unpredictable and unforeseen. The energy crisis in the 1970s, skyrocketing interest rates in the early part of that decade, and extremely large cost overruns in the construction of many nuclear power facilities are but a few of many economically orientated events that were not foreseen by the many people whose activities they were to greatly affect. It is therefore reasonable to compute and report uncertainties of feasibility indicators for a confidence interval of 68 percent, corresponding to one standard deviation of a normal probability distribution. Unforeseen events that would significantly alter the economic parameters would fall in the 32 percent outside the confidence interval. Thus, the feasibility indicators will have roughly a 2 out of 3 chance of falling within the range represented by their uncertainties; there is a 16 percent probability that they will fall above the range and a 16 percent probability that they will fall below the range.

This type of uncertainty analysis assumes that each of the variable parameter values is: (1) unrelated (i.e., independent of each other), and (2) from a Gaussian population (i.e., normal distribution) of possible values. The uncertainty interval quoted for each variable must also be quoted at the same "odds." The equation

$$x_i = \hat{x}_i \pm \delta x \qquad (P = 0.68) \tag{9.9}$$

means that the expected value of x_i is $\hat{x}_i$, and there is a 2 out of 3 chance that the actual value of x_i will lie within δx_i of that value.

If a result, feasibility indicator (FI), is then calculated by a function such that

$$\mathrm{FI} = f(x_1, x_2, x_3, \ldots, x_N) \tag{9.10}$$

then the uncertainty in that result, for the same confidence limits (or "odds") as the parameter uncertainties, is

$$\delta \mathrm{FI} = \left\{\left[\frac{\partial \mathrm{FI}}{\partial x_1}\,\delta x_1\right]^2 + \left[\frac{\partial \mathrm{FI}}{\partial x_2}\,\delta x_2\right]^2 + \cdots + \left[\frac{\partial \mathrm{FI}}{\partial x_N}\,\delta x_N\right]^2\right\}^{1/2} \tag{9.11}$$

The sensitivity of FI to x is normally defined as (δ FI/$\delta\, x_1$) ∂x_1. The assumption that each of the variable parameter values is unrelated may not be fulfilled if, for example, discount and escalation rates are related, i.e., when discount rates increase, escalation rates increase. Upon project initiation, however, discount rate is often set to a loan rate, and truly does become independent of future escalation rates. The normal probability requirements of the assumption that each of the variable parameter values is from a Gaussian population of possible values are not overly restricting. Given the imperfect data set used to determine the distribution of the variable parameters, a normal distribution is usually chosen. The assumption that the sensitivities have a normal probability distribution is generally not fulfilled because the nonlinearities in Eq. (9.10) will result in a skewed sensitivity distribution for many of the terms in Eq. (9.11). The uncertainties to be described in this chapter may therefore be considered "linearized." The ramifications of the assumption that the sensitivities have a normal probability distribution are discussed in Ref. 22. Equations (9.8) through (9.11) represent a first order–second moment analysis similar to that used for measurement uncertainty [23] or for a structural reliability analysis [24, 18].

In an economic analysis, the net present value (NPV), benefit/cost ratio (B/C), and internal rate of return (IRR) will generally be functions of six variable, or uncertain, parameters: capital costs (denoted C_c); discount rate and interest rate (denoted d and i), annual OM&R costs (denoted OM&R), overall escalation rate applied to OM&R (denoted e), annual energy production (denoted kWh), and a levelized utility buyback rate or energy value (denoted U). Then

$$\mathrm{NPV} = f_1\,(C_c, d = \mathrm{i}, \mathrm{OM\&R}, e, \mathrm{kWh}, U) \tag{9.12a}$$

$$B/C = f_2\,(C_c, d = i, \mathrm{OM\&R}, e, \mathrm{kWh}, U) \tag{9.12b}$$

$$\mathrm{IRR} = f_3\,(C_c, d = \mathrm{i}, \mathrm{OM\&R}, e, \mathrm{kWh}, U) \tag{9.12c}$$

Equation (9.11) for these feasibility indicators thus becomes

$$\begin{aligned}\delta(\mathrm{FI}) = \Bigg\{&\left[\frac{\partial(\mathrm{FI})}{\partial(C_c)}\,\delta\,(C_c)\right]^2 + \left[\frac{\partial(\mathrm{FI})}{\partial(d)}\,\delta(d)\right]^2 + \left[\frac{\partial(\mathrm{FI})}{\partial(\mathrm{OM\&R})}\,\delta(\mathrm{OM\&R})\right]^2 \\ &+ \left[\frac{\partial(\mathrm{FI})}{\partial(e)}\,\delta(e)\right]^2 + \left[\frac{\partial(\mathrm{FI})}{\partial(\mathrm{kWh})}\,\delta(\mathrm{kWh})\right]^2 + \left[\frac{\partial(\mathrm{FI})}{\partial(U)}\,\delta(U)\right]^2\Bigg\}^{1/2}\end{aligned} \tag{9.13}$$

Thus, the sensitivities of FI to each variable parameter are combined by the root sum of squares into an overall uncertainty for FI. Equation (9.13) is strictly true only when the probability distribution of each sensitivity is normal. The nonlinear relationship of FI to discount rate and escalation rate do not result in a normal distribution, so Eq. (9.13) is an approximation.

Equation (9.13) can also assist individuals involved in an economic analysis to allocate their efforts most efficiently. If, for example, the sensitivity due to construction costs, $\partial\mathrm{FI}/\partial C_c \delta C_c$, accounts for 20 percent of FI, then reducing δC_c by 50 percent would reduce δFI by only 5 percent. This is because the root sum of squares is used in computing δFI. Typically, in an attempt to refine an uncertainty estimate, one looks first at the variables with the largest sensitivity.

9.4 UNCERTAINTY APPLIED TO ECONOMIC ANALYSIS

Application

The expected values of the variable parameters (VP_i) and the uncertainties for $P = 0.68$ associated with them are determined and reported, according to convention, as follows:

$$C_c = \hat{C}_c \pm \delta C_c \qquad (P = 0.68)$$

$$d = \hat{d} \pm \delta \mathrm{d} \qquad (\mathrm{P} = 0.68)$$

$$\mathrm{OM\&R} = \mathrm{O\hat{M}\&R} \pm \delta \mathrm{OM\&R} \qquad (P = 0.68)$$

$$e = \hat{e} \pm \delta e \qquad (P = 0.68)$$

$$\mathrm{kWh} = \mathrm{k\hat{W}h} \pm \delta \mathrm{kWh} \qquad (P = 0.68)$$

$$U = \hat{U} \pm \delta U \qquad (P = 0.68)$$

Determination of the uncertainties of the variable parameters, e.g., $\partial(U)$, often requires considerable effort. Some techniques used to determine these uncertainties are illustrated in Sec. 9.5. When possible, the uncertainties should be based upon hard data because of the tendency for an individual to underestimate parameter uncertainty [16]. The selection of subjective probabilities is discussed in Ref. 25.

Net Present Value

For the case of a one-year construction period and debt financing with first-year interest added to capital costs, the function in Eq. (9.12*a*) by which NPV may be defined is

$$\begin{aligned}
\mathrm{NPV} &= \sum_{n=1}^{k} (B_{n\mathrm{PW}} + C_{n\mathrm{PW}}) \\
&= \sum_{n=1}^{k} \left\{ \frac{R_n - D_n - \mathrm{OM\&R}_n}{(1 + d)^n} \right\} \\
&= \sum_{n=i}^{k} \frac{(U)(\mathrm{kWh})}{(1 + d)^n} - \sum_{n=1}^{m} \frac{d(1 + d)^m C_c}{[(1 + d)^m - 1](1 + d)^n} \\
&\quad - \sum_{n=i}^{k} \frac{\mathrm{OM\&R}(1 + e)^n}{(1 + d)^n}
\end{aligned}$$

$$= \sum_{n=i}^{k} T_{R_n} - \sum_{n=i}^{k} T_{\mathrm{OM\&R}_n} - C_c \tag{9.14}$$

where i indicates the first year after construction has been completed. The uncertainty of NPV may therefore be derived in accordance with Eq. (9.13), where FI = NPV, and the partial derivatives are derived as

$$\frac{\partial(\mathrm{NPV})}{\partial(C_c)} = -1 \tag{9.15a}$$

$$\frac{\partial(\mathrm{NPV})}{\partial(d)} = \sum_{n=i}^{k} \frac{\partial(T_{R_n})}{\partial(d)} - \sum_{n=i}^{k} \frac{\partial(T_{\mathrm{OM\&R}_n})}{\partial(d)}$$

where

$$\frac{\partial(T_{R_n})}{\partial(d)} = \frac{-(U)(\mathrm{kWh})n}{(1+d)^{n+1}}$$

and

$$\frac{\partial(T_{\mathrm{OM\&R}_n})}{\partial(d)} = \frac{-\mathrm{OM\&R}_c(1+e)^n n}{(1+d)^{n+1}} \tag{9.15b}$$

$$\frac{\partial(\mathrm{NPV})}{\partial(\mathrm{OM\&R})} = -\sum_{n=2}^{k} \frac{(1+e)^n}{(1+d)^n} \tag{9.15c}$$

$$\frac{\partial(\mathrm{NPV})}{\partial(e)} = -\sum_{n=2}^{k} \frac{\mathrm{OM\&R}(1+e)^{n-1}n}{(1+d)^n} \tag{9.15d}$$

$$\frac{\partial(\mathrm{NPV})}{\partial(\mathrm{kWh})} = \sum_{n=2}^{k} \frac{U}{(1+d)^n} \tag{9.15e}$$

$$\frac{\partial(\mathrm{NPV})}{\partial(U)} = \sum_{n=2}^{k} \frac{\mathrm{kWh}}{(1+d)^n} \tag{9.15f}$$

The computed values of the partial derivatives, the products of these results and their respective variable parameter uncertainty values (which is, in effect, the NPV uncertainty with respect to the given parameter only), and the resulting overall uncertainty for NPV may be tabulated and/or plotted.

If the function by which NPV is determined is either unknown or too cumbersome for partial differentiation, the results desired in the uncertainty analysis may be computed by sequential perturbation [17]. The partial derivatives need not be known in this method, since, if one examines the definition of the partial derivative operation,

$$\frac{\partial R}{\partial x_i} = dx_i \to 0 \; \frac{(R_{x_i + dx_i} - R_{x_i})}{dx_i} \tag{9.16}$$

From the above definition, it follows that the contribution from each of the ith variables to the total uncertainty in R is

$$\left[\frac{\partial R}{\partial x_i}\,\delta x_i\right] = R_{x_i + dx_i} - R_{x_i} \tag{9.17}$$

In the application at hand, the expected value of NPV is computed, as before, with the expected values of the variable parameters, VP_i, and then computed once more for each variable parameter, with the value of that variable parameter increased (or decreased, as the case may be) by its uncertainty. All the other variable parameters are returned to their baseline (i.e., expected) values. Thus

$$\mathrm{N\hat{P}V} = f_1(\mathrm{V\hat{P}}_1, \mathrm{V\hat{P}}_2, \ldots, \mathrm{V\hat{P}}_i, \ldots, \mathrm{V\hat{P}}_N) \tag{9.18}$$

$$\mathrm{NPV}_{\delta(\mathrm{VP}_i)} = f_1(\mathrm{V\hat{P}}_1, \mathrm{V\hat{P}}_2, \ldots, \mathrm{V\hat{P}}_i + \delta(\mathrm{VP}_i), \ldots, \mathrm{V\hat{P}}_N) \tag{9.19}$$

$$\delta(\mathrm{NPV}) = \left\{\sum_{i=1}^{N} (\mathrm{NPV}_{\delta(\mathrm{VP}_i)} - \mathrm{N\hat{P}V})^2\right\}^{1/2} \tag{9.20}$$

Benefit/Cost Ratio

For the same case as above, the function by which B/C is defined is

$$\begin{aligned} B/C &= \left[\sum_{n=1}^{k} B_{n\mathrm{PW}}\right]\left[\sum_{n=1}^{k} C_{n\mathrm{PW}}\right]^{-1} \\ &= \left[\sum_{n=1}^{k} \frac{R_n}{(1+d)^n}\right]\left[\sum_{n=1}^{k} \frac{D_n + \mathrm{OM\&R}_n}{(1+d)^n}\right]^{-1} \\ &= \left[\sum_{n=2}^{k} T_{\mathrm{R}_n}\right]\left[C_c + \sum_{n=2}^{k} T_{\mathrm{OM\&R}_n}\right]^{-1} \end{aligned} \tag{9.21}$$

The uncertainty associated with B/C may therefore be derived in accordance with Eq. (9.13), where FI $= B/C$, and the partial derivatives are derived as

$$\frac{\partial(B/C)}{\partial(C_c)} = -\left[\sum_{n=1}^{k} B_{n\mathrm{PW}}\right]\left[\sum_{n=1}^{k} C_{n\mathrm{PW}}\right]^{-2}\left[-\frac{\partial(\mathrm{NPV})}{\partial(C_c)}\right] \tag{9.22a}$$

$$\begin{aligned} \frac{\partial(B/C)}{\partial(d)} &= \left[\sum_{n=i}^{k} \frac{\partial(T_{R_n})}{2(d)}\right]\left[\sum_{n=1}^{k} C_{n\mathrm{PW}}\right]^{-1} \\ &\quad - \left[\sum_{n=1}^{k} B_{n\mathrm{PW}}\right]\left[\sum_{n=1}^{k} C_{n\mathrm{PW}}\right]^{-2}\left[\sum_{n=i}^{k} \frac{\partial(T_{\mathrm{OM\&R}_i})}{\partial(d)}\right] \end{aligned} \tag{9.22b}$$

$$\frac{\partial(B/C)}{\partial(\mathrm{OM\&R})} = -\left[\sum_{n=1}^{k} B_{n\mathrm{PW}}\right]\left[\sum_{n=1}^{k} C_{n\mathrm{PW}}\right]^{-2}\left[-\frac{\partial(\mathrm{NPV})}{\partial(\mathrm{OM\&R})}\right] \tag{9.22c}$$

$$\frac{\partial(B/C)}{\partial(e)} = -\left[\sum_{n=1}^{k} B_{n\mathrm{PW}}\right]\left[\sum_{n=1}^{k} C_{n\mathrm{PW}}\right]^{-2}\left[-\frac{\partial(\mathrm{NPV})}{\partial(e)}\right] \tag{9.22d}$$

$$\frac{\partial(B/C)}{\partial(\mathrm{kWh})} = \frac{\partial(\mathrm{NPV})}{\partial(\mathrm{kWh})}\left[\sum_{n=1}^{k} C_{n\mathrm{PW}}\right]^{-1} \tag{9.22e}$$

$$\frac{\partial(B/C)}{\partial(U)} = \frac{\partial(\mathrm{NPV})}{\partial(U)}\left[\sum_{n=1}^{k} C_{n\mathrm{PW}}\right]^{-1} \tag{9.22f}$$

Internal Rate of Return

The IRR is determined iteratively, so we do not know the partial derivatives of the implicit function by which it is calculated. The method of sequential perturbation must be applied.

Ramification

The results of the uncertainty analysis may be used to gauge the risk involved in project implementation. They provide a basis for decision making, as subject to criteria, for example, that indicators of feasibility associated with a given project alternative must fall within a certain range 2 out of 3 times. Alternatives registering a greater than 1 in 6 chance of yielding an unacceptable indicator value, for example, would be rejected if an investor was highly risk-adverse.

The results may also be used to determine the value of an undetermined variable parameter below which the project alternative will not be considered. For example, if the expected values and uncertainties are established for all the variable parameters but the utility buyback rate, the uncertainty analysis can yield a range of values around an expected value of the buyback rate which would be acceptable for the alternative to be considered. The uncertainty analysis could thus serve negotiations for a power purchase contract. This concept will be illustrated further in Sec. 9.5.

9.5 CASE STUDIES

Case Study 1: The St. Anthony Falls Hydraulic Laboratory Minihydropower Facility

The Saint Anthony Falls Hydraulic Laboratory (SAFHL) is a University of Minnesota research laboratory located at the site of the only major waterfall in the Mississippi River, in the heart of Minneapolis. The laboratory is carved from the limestone ledge forming the falls and carefully planned architecturally to fulfill its mission as a center of advanced study, research, and public service in the field of water resources and fluid mechanics.

The SAFHL minihydropower facility is planned to generate up to 132 kW of power through the use of presently existing civil works appurtenant to Saint Anthony Falls and the laboratory (Figs. 9.3 to 9.5), including: (1) the horseshoe dam

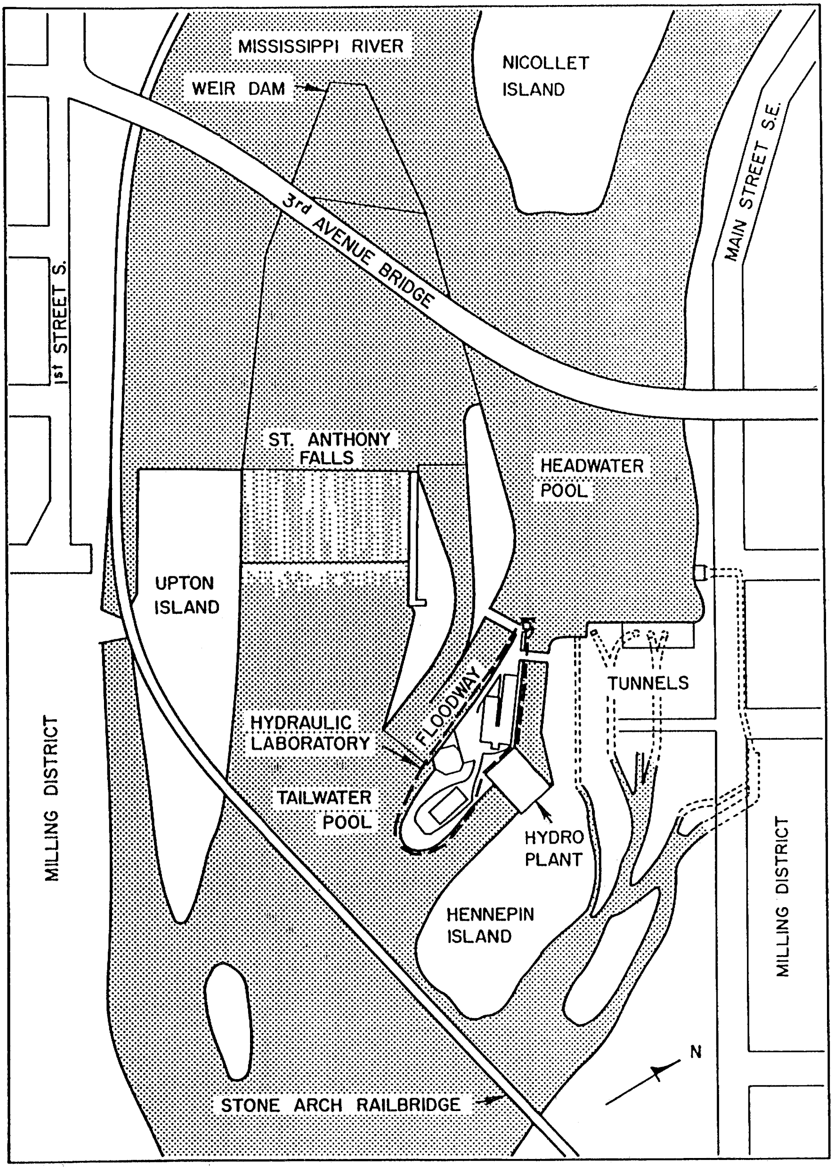

FIGURE 9.3 Vicinity of the St. Anthony Falls Hydraulic Laboratory.

of the upper falls and the lower dam of the lower falls, which together create a fairly constant gross head averaging 48.0 ft; (2) the inflow region of a power canal conveying water to a 12.4-MW hydropower facility operated by the local utility adjacent to the laboratory; (3) the laboratory's intake control house and main supply flume, which is permitted by the state to appropriate up to 50 ft^3/s, with 40

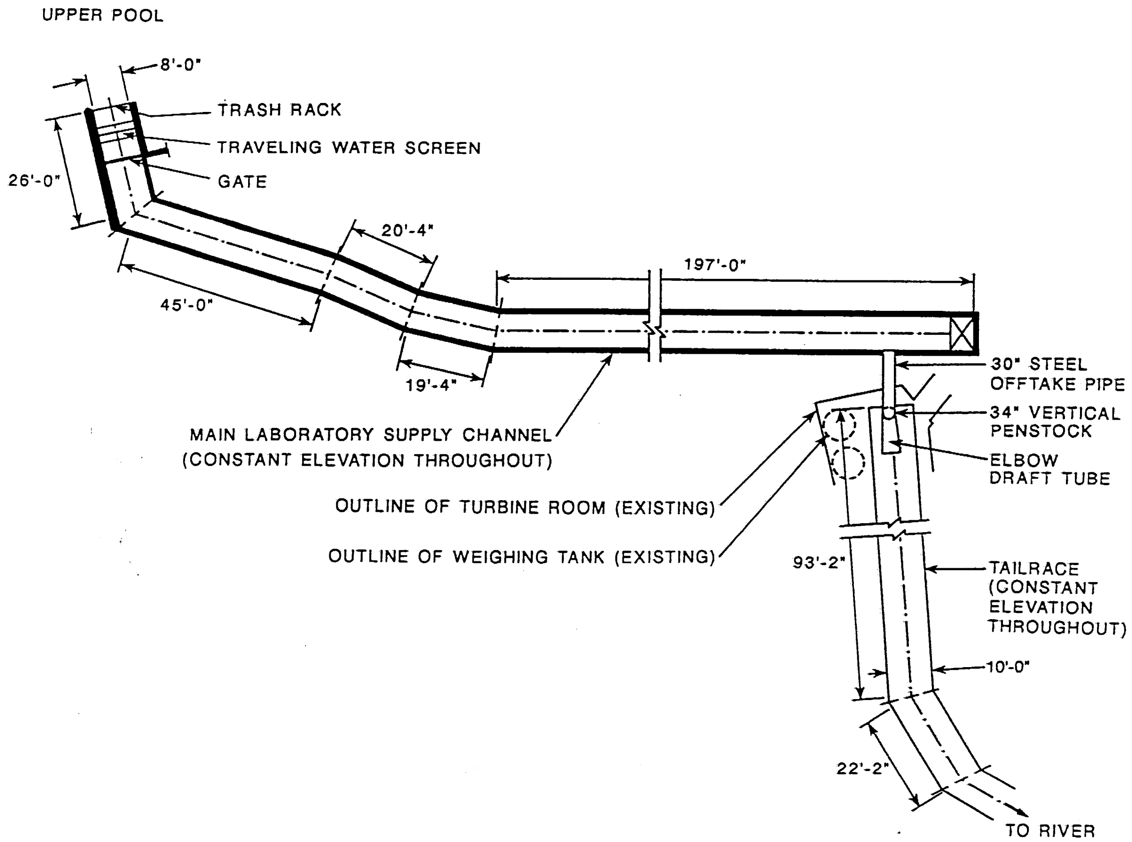

FIGURE 9.4 Route of flow for the SAFHL minihydropower facility.

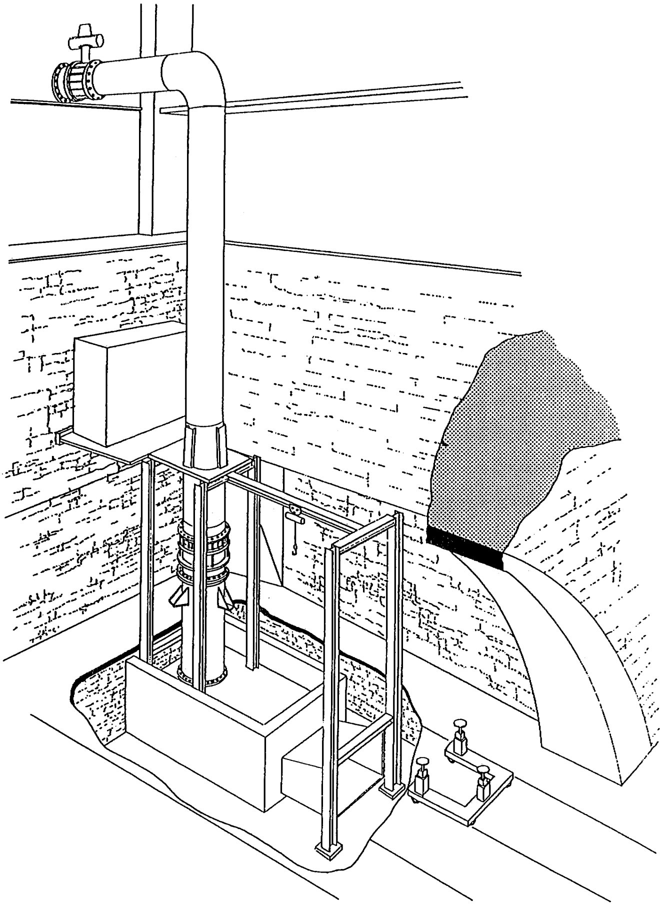

FIGURE 9.5 Artist's rendition of the SAFHL minihydropower facility.

ft^3/s of that for power generation, an amount that is perennially available given the river's substantially greater normal flows. The main supply flume would serve, in essence, as the proposed facility's power canal; and (4) the laboratory's tailrace discharge channel, which, like the main supply flume, is an integral component of the laboratory structure. River water at the level of the upper falls tailwater (the so-called intermediate pool between upper and lower Saint Anthony Falls) backs up into the tailrace channel inside the building.

An equipment supplier was selected on the basis of cost, reliability and performance record, and design concept (essentially, "workability"). Financial assumptions and the expected values and uncertainties of the variable parameters were then determined and utilized in the economic analysis. The assumptions, benefit-and-cost streams, and resultant feasibility indicators were analyzed using the spreadsheet format exhibited in Table 9.2.

Life of Project and Salvage Value. Since the first installations of commercially manufactured minihydroturbines like that selected for SAFHL are relatively recent—a standard-design, axial-flow, 132-kW, submersible turbine-generator—there was little history or experience upon which to establish their expected life span, but given the generalizations of Sec. 9.2, it seemed reasonable to assume a life span of 25 years for the laboratory facility. In addition, the purchasing utility would not offer levelized power purchase contracts for longer than 25 years.

Assuming that the facility would cease operation after 25 years (at least in its then-current condition), a salvage value of zero was assigned to it. Resale of any of the facility's components, including the turbine-generator equipment, is unlikely.

Financing. As a university research institution, the laboratory could qualify for an internal loan, funded from the proceeds of the sale of variable-rate demand bonds, to finance the first-year costs of the project (which were to include capital costs and the construction-year debt service). A loan of the magnitude desired for the facility would typically be amortized over 10 years. The term of loan for the minihydropower facility is thus fixed at 10 years.

Terms of Power Purchase Contract. The local utility offered two methods of payment for energy. The first option was a nonguaranteed rate and was based on the actual hourly system decremental cost of producing energy on its system. The *hourly system decremental cost* is defined as the costs in any given hour of fuel, operating labor, startup costs (if applicable), and normal maintenance associated with its generation of power and energy or the cost per kilowatt-hour to the utility of the next block of purchased power or energy, whichever is needed by the utility due to a seller's failure to provide energy and capacity to the utility. The second option involved levelized energy payments, wherein for all but the last 5 years of the contract up to 20 years, in lieu of the actual hourly system decremental cost, the payment rate would be a levelized value. For both options, the capacity rates are constant (or levelized) over the entire term of the contract.

For the economic analysis, the utility buyback rate was incorporated as a single, fixed rate over the life span of the project. This rate will be discussed later.

Capital Costs. Table 9.4 is a list of the capital cost component estimates. This comprises initial hardware costs, labor to install the facility, utility costs, engineering and construction management, and performance tests and report (i.e., acceptance tests). The cost estimate of the turbine-generator equipment was based on quotations from the company. Other hardware costs, including penstock components, support structure, and auxiliary equipment, were estimated from various price quotations of equipment suppliers and from laboratory experience. Miscellaneous hardware was taken at 15 percent of the cost of all other hardware components excluding the turbine-generator equipment. The cost for labor to install the facility was the most difficult to project, since the methods and schedule of construction—under laboratory direction—were primarily conjecture at this stage of analysis. It was taken to be 150 percent of the hardware costs excluding turbine-generator equipment. Labor will be supplied primarily by laboratory personnel, and the resultant cost estimate was judged reasonable based on laboratory experience. The utility cost estimates reflect the judgment of utility personnel. Costs associated with engineering and construction management and performance tests and report are set by the laboratory.

TABLE 9.4 Saint Anthony Falls Hydraulic Laboratory Minihydropower Facility Project Cost Estimates

Initial hardware costs:			
132-kW submersible hydroturbine generator, elbow draft tube, and monitoring and protection equipment and switchgear (turbine-generator control)		$58,000	
Penstock components:			
Butterfly valve with actuator	$ 4,200		
30-in steel pipe	$ 2,900		
30-in 90-degree bend	$ 1,100		
30-in × 34-in concentric reducer (diffuser)	$ 1,900		
34-in steel pipe	$ 1,700		
34-in expansion joint or dresser coupling with pipe stub	$ 5,700		
Flanges	$ 2,700		
Total, penstock components		$20,200	
Support structure:			
Concrete thrust block and access pit	$ 1,200		
Supporting steel & hardware	$ 4,000		
Total, support structure		$ 5,200	
Auxiliary equipment:			
2-ton capacity hoist	$ 1,600		
3, 1-ton capacity machine screw actuators	$ 900		
Placement/support hand truck	$ 1,000		
Total, auxiliary equipment		$ 3,500	
Miscellaneous hardware			$ 4,335
Total, initial hardware costs	$91,235		
Labor for installation of facility			$ 49,853
Utility costs:			
Control panel setup and interconnection		$10,000	
Substation accommodations		$15,000	
Total, utility costs			$ 25,000
Engineering and construction management			$ 15,000
Performance tests and report			$ 15,000
Total, project costs			$196,088

The estimated costs are given in Table 9.5 with 68 percent uncertainties, also estimated. The uncertainty in initial hardware costs was decided partly on the basis of varying price estimates or quotations for various components of the facility, and partly on experience. Then

$$C_c = \Sigma\, C_j \quad \text{(Table 9.5)}$$

$$\delta(C_c) = (\Sigma\, U_{C_j}^{\,2})^{1/2} \quad \text{(Table 9.5)}$$

$$C_c = \$195{,}700 \pm 24{,}600 \qquad (P = 0.68)$$

The uncertainty in capital costs is reasonable for the confidence limit; $\delta(C_c)$ is 12.6 percent of the expected total project cost.

TABLE 9.5 Project Cost Summary

	Expected item cost, C_j, \$	Uncertainty, U_{C_j} (P = 0.68), \$
Hardware and miscellaneous	91,100	13,500
Labor for installation of facility	49,600	16,500
Utility costs	25,000	10,000
Engineer and construction management	15,000	5,000
Performance tests and report	15,000	5,000
Expected project cost = $\Sigma_j C_j$:	195,700	
Uncertainty (P = 0.68) = $\{\Sigma_j U_{Cj}{}^2\}^{1/2}$:	24,587	

Interest (= Discount) Rate. The expected discount rate used in the analysis was an estimate (or projection) based upon yields of the previous 8 years on state and local government Aaa bonds, which most closely approximate the interest rates for which the laboratory could qualify.

Annual averages of this interest rate were tabulated for years 1945 to 1986 and plotted for the years 1965 to 1985 (Fig. 9.6) [26]. There has clearly been an upward trend in interest rates over the past four decades (before 1967, the yield on a typical Aaa state or local government bond had never climbed

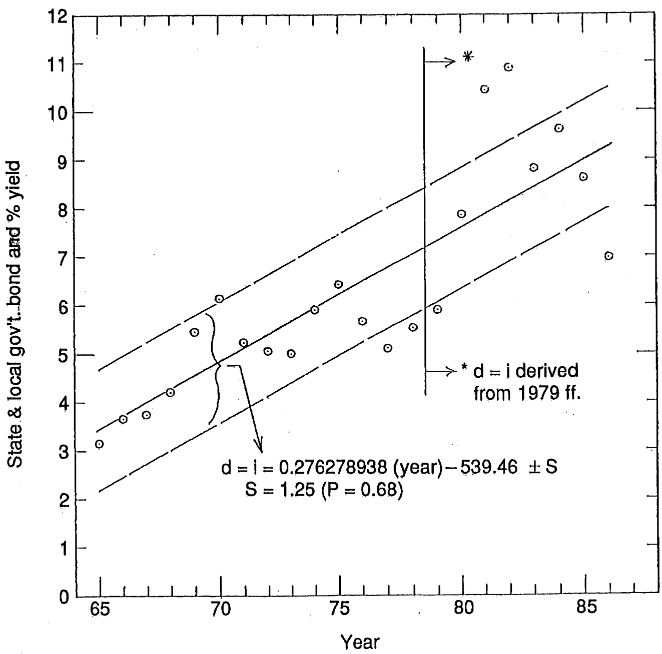

FIGURE 9.6 State and local government Aaa bond yields, 1965–1986.

above 4 percent. However, it is reasonable to presume that a continued rate of increase in the future (equal to that indicated in Fig. 9.6) would not be tolerated or sustained without major adverse impacts on the economy. It is also clear that these interest rates fluctuated wildly in the eighties, making any estimation of the discount rate to be used for the laboratory's facility difficult to make. For this analysis, the interest rates of the 8 years from 1979 to 1986 were considered (to the right of the vertical line in Fig. 9.6). They are averaged to provide the analysis with an expected interest or discount rate, and the standard deviation of this sample is multiplied by the Student t-value for the confidence interval (68 percent) at 8 degrees of freedom, to arrive at a 68 percent uncertainty in the interest or discount rate:

$$d = d \pm \delta(d) = 8\&5/8 \pm 1\&7/8\ \% \qquad (P = 0.68)$$

This was considered reasonable, and conservative, since the laboratory might obtain a loan from the university's office of finance & operations at a lower, though, variable, rate.

Annual OM&R Costs. Informal interviews to discuss OM&R were conducted with current owners of the selected supplier's turbine-generating equipment. It became apparent from these interviews that the annually averaged OM&R costs associated with the facility would total about \$2000, an estimate that was also affirmed by the supplier. Also determined from the interviews were variations from this cost, and the 2 out of 3 uncertainty was set at \$1000. One owner reported costs running between \$3000 and \$5000 annually, and another reported experience to be below \$500 a year, with at least four others affirming the \$2000 estimate as reasonable. To this was added a \$500 water appropriation permit fee to be paid annually to the state's Department of Natural Resources. The annual OM&R costs were thus determined as

$$\text{OM\&R} = \text{OM\&R} \pm \delta\text{OM\&R} = \$2500 \pm 1000 \qquad (P = 0.68)$$

Escalation Rate on OM&R. Operation, maintenance, and replacement costs may be expected to escalate due to inflationary pressures, and since the 10-year debt service period is especially critical, it was desired to determine the expected average escalation rate for this 10-year period and apply it over the life span of the project. The Consumer Price Index (CPI) was employed for this purpose, and like interest rates, the escalation rates determined from the CPI for the last 20 years are not represented well by the generally more stable, and lower, rates of the period from 1945 to 1965 (Fig. 9.7). Beginning with 1965, then, the average inflation rate for ten 10-year periods (i.e., 1965–1975, 1966–1976, . . . ,1975–1985) was computed using the CPIs. Average 10-year escalation rates beginning with 1956 are plotted in Fig. 9.8. These ten 10-year rates were themselves averaged to derive the expected escalation rate for the project's annual average OM&R costs. The standard deviation was multiplied by the Student t with 10 degrees of freedom to derive the 68 percent uncertainty. We thus have

$$e = \hat{e} \pm \delta(e) = 7\tfrac{1}{4} \pm 1\tfrac{1}{4}\% \qquad (P = 0.68)$$

Power and Energy Production; Capacity Factor. The analysis will assume energy production at an unvarying capacity of 125.7 kW (corresponding to H_T = 47.0 ft, Q = 39.3 ft^3/s). The question then becomes simply one of turbine-generator, or

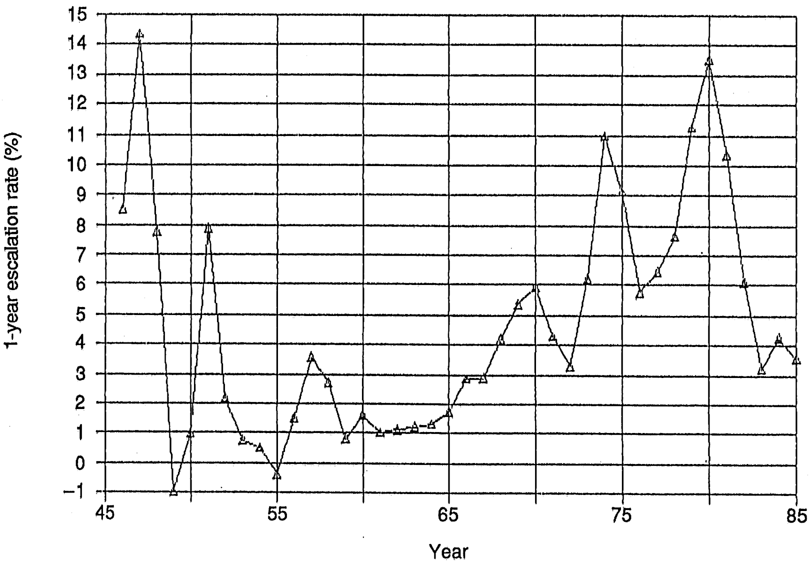

FIGURE 9.7 Yearly escalation rate; end of year.

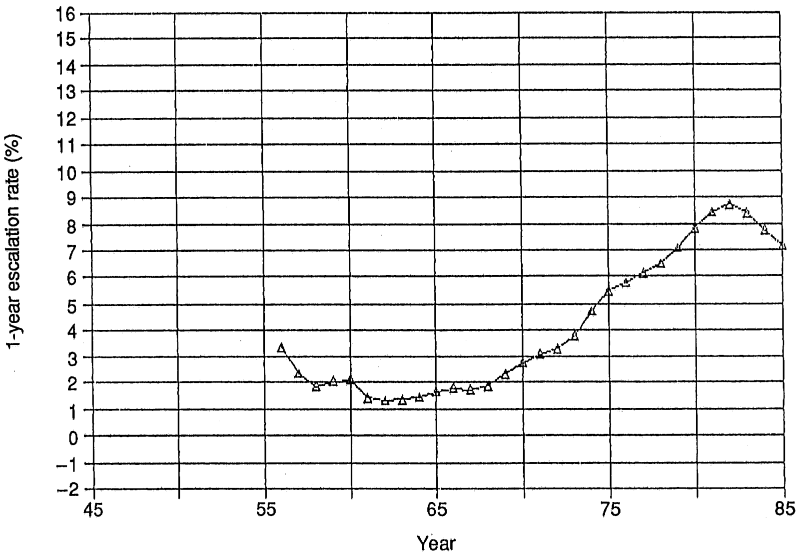

FIGURE 9.8 Ten-year average escalation rate; end of ten-year period.

flow, availability. Since the 10-year debt service period is the most critical period from a cash-flow standpoint, five series of 31 ten-year simulations of river water availability during laboratory working hours were generated by a model which accounted for the competing requirements of research and projects for the

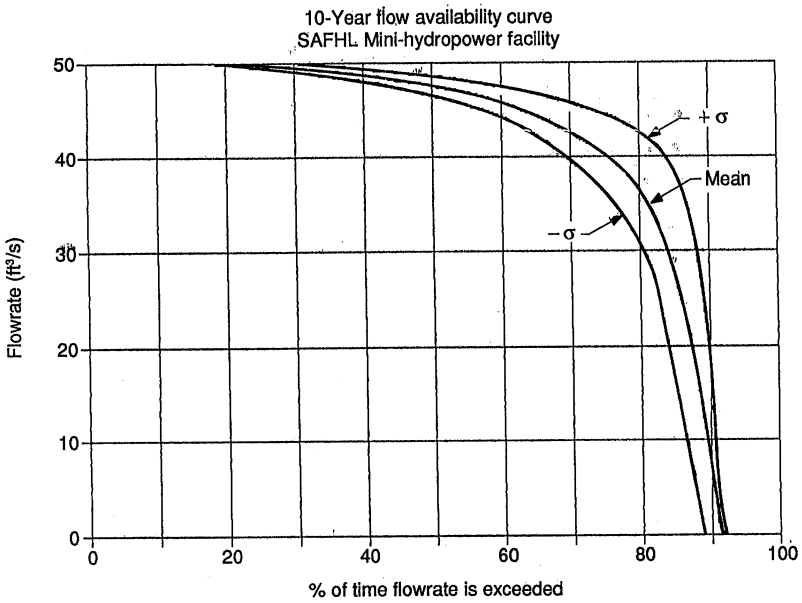

FIGURE 9.9 Ten-year flow availability curve.

laboratory's flow allotment. The mean annual flow availability curve, and two confidence intervals for the 10-year mean flow availability curve, and two curves enveloping the 68 percent (one standard deviation) confidence interval for the 10-year mean flow availability curve, are plotted in Fig. 9.9. There is negligible variation between the plots of the five series.

Flow availability during off-work hours (6672 hours annually) is virtually 100 percent when considered solely from a competing project water demand standpoint.

Since 40 ft^3/s is needed for the minihydropower facility, Fig. 9.9 reveals that during working hours (8:00 a.m. to 4:30 p.m., Monday through Friday except holidays; 2088 hours annually) the expected annual flow availability will be 75 percent.

Annually scheduled maintenance would require an outage of no more than two or three days on average. Flooding in the turbine room of the laboratory due to high tailwater levels would account for outages, on average, of a few days annually. Other than prohibiting operation at the time, flooding would not adversely affect subsequent operation of the facility. The facility is envisioned not only as a small power producer, but as a demonstration and educational facility as well; scheduled outages on the order of a few hours would be arranged for these purposes when it would not conflict with the laboratory's contractual obligations to the utility. These outages were accounted for by subtracting 3 percent from the availability percentages.

Since it was not at this point evident that the utility would pay different rates for power generated during peak and off-peak hours, the work and off-work flow availabilities were combined to give the expected annual mean flow availability with an associated uncertainty:

$$\text{Availability} = 91.0 \pm 3.1\% \qquad (P = 0.68)$$

This corresponds to a kilowatt-hour production of

$$\text{kWh} = \hat{\text{kWh}} \pm \delta(\text{kWh}) = 1{,}002{,}030 \pm 34{,}135 \text{ kWh} \qquad (P = 0.68)$$

using 125.7 kW of power production and 8760 hours in a year.

Utility Power and Energy Buyback Rates. This parameter is expressed in terms of dollars per kilowatt-hour, and it incorporates the rates to be paid by the utility for both capacity and energy. The feasibility indicators are very sensitive to this parameter. Pending changes in the way the utility determined its levelized capacity payments, and the regulatory process established to oversee and approve those changes, made an estimate of this parameter's expected value and uncertainty difficult to make. For the analysis, it sufficed to set the expected value equal to

$$U = 0.038 \text{ \$/kWh}$$

based on the last approved rate schedules in effect. The 68 percent uncertainty interval for this buyback rate could not be determined due to the aforementioned changes pending, but for the uncertainty analysis presented here

$$U = \hat{U} \pm \delta(U) = 0.038 \pm 0.003 \text{ \$/kWh} \qquad (P = 0.68)$$

represented the utility buyback and associated uncertainty with the existing buyback schedule.

Uncertainty Analysis. The computed values of the partial derivatives of Eq. (9.15), the product of these results and their respective variable parameter uncertainty values, and the resulting overall uncertainty for NPV are tabulated in Table 9.6. The sensitivity of the three feasibility indicators to each variable parameter as determined by the partial derivatives of Eq. (9.13) is given in Table 9.7. The feasibility indicators are seen to be the most sensitive to the uncertainty in discount rate.

The results of the uncertainty analysis were used in the SAFHL minihydropower project to determine the utility buyback rate below which the project would be unacceptable economically. If a utility buyback rate of 0.033

TABLE 9.6 Expected Value and Uncertainty of Parameters (P = 0.69) for the SAF Minihydropower Facility

Parameter	Expected value	± Uncertainty
Capital cost C_c, \$	195,700	± 24,600
Discount rate d, %	8.615	± 1.78
Operation, maintenance, and replacement costs OM&R, \$/yr	2,500	± 1,000
Escalation rate e, %	7.25	± 1.25
Energy production, GWh	1.00	± 0.034
Capacity and energy value, \$/kWh	0.038	± 0.003
Net present value (NPV), \$	108,000	± 67,300
Benefit-cost ratio (B/C)	1.43	± 0.28
Internal rate of return (IRR) %	14.4	± 3.7

TABLE 9.7 Sensitivity of Feasibility Indicators to the Variable Parameters for the SAF Minihydropower Facility

Variable parameter	Net present value (NPV), $	Benefit-cost ratio (B/C)	Internal rate of return (IRR), %
Project cost	24,600	0.14	1.9
Discount rate	49,700	0.18	2.4
Operation, maintenance, and replacement costs	21,000	0.09	1.2
Escalation rate	8,200	0.05	0.7
Energy production	12,100	0.04	0.5
Utility payback	28,000	0.13	1.5
FI uncertainty computed as the root sum of squares	67.200	0.28	3.7
Expected value of feasibility indicator	108,000	1.43	14.4

(± 0.0) $/kWh was used in the economic analysis, the NPV would be $61,000 ± $62,000 ($P$ = 0.68). This means that the probability that the actual NPV will be negative is approximately 16 percent. It was decided that this was an acceptable level of risk for the laboratory (approximately a 1 in 6 chance of losing money on the project) because the laboratory would act more like a small entrepreneur in energy production, rather than as a utility, municipality, or governmental agency, which are usually less risk-adverse. A power purchase contract offer yielding a buyback rate of less than 37 mills/kWh would be rejected accordingly.

Case Study 2: St. Cloud Hydropower Facility

The St. Cloud Dam, located on the Mississippi River within the City of St. Cloud, Minnesota, consists of a concrete, fixed-crest spillway 19.5 ft (5.6 m) high and 550 ft (165 m) long centered on the river. Average annual flow was established to be 4970 ft^3/s (142 m^3/s), and the river provides sufficient base flow such that a strict run-of-river operation is suited as a mode of operation for the dam. The structure will accommodate the installation of 2-ft (0.6-m) flashboards on the unregulated ogee crest to bring the rated head up to 18.5 ft (5.6 m).

A feasibility study performed in 1981 compared four alternatives for hydroelectric power generation at the site [27]. The hydrologic and hydraulic analyses are given as case studies in Chap. 2.

Project development alternatives were formulated and presented as a multiple combination of horizontal tubular units. The rated generator output for each unit is 2180 kW, corresponding to a rated head of 18.5 ft (5.6 m). Alternative A is four units, B is three, C is five, and D is three units, with allowance for expansion to four. The results of the various alternatives, including the total project cost, average annual energy production, and annual income generated by each, are summarized in Table 9.8.

TABLE 9.8 Summary of Project Development Alternatives for the St. Cloud Feasibility Study, 1981 Base Year

	Alternative			
	A	B	C	D
Description				
Total initial cost, $	9,179,000	6,649,000	11,336,000	7,752,000
Average annual energy, GWh	44.28	37.44	47.87	37.44
Annual energy income, $	611,000	517,000	661,000	517,000
Capacity credit, $	160,000	160,000	160,000	160,000
Average annual income ($: 0.0138/kWh)	771,000	677,000	821,000	677,000
Annual benefits/total initial cost	0.084	0.102	0.072	0.087
Economic analysis				
Net present value, $ million	13.30	13.06	12.53	12.03
Benefit/cost ratio	2.22	2.60	1.95	2.31
Internal rate of return	21.7	25.5	19.4	22.2

Economic Analysis. The following assumptions were incorporated into the economic analysis:

- The economic life of the project was assumed to be 50 years.
- Interest and discount rate, 11 percent.
- Annual escalation in the value of energy and power, 9 percent.
- Annual operation, maintenance, and replacement costs in 1981 dollars were determined from Ref. 9.
- A two-year construction period.
- A linear expenditure of capital during project construction.

Table 9.9 is an example of the constructed cost and benefit streams.

The benefit-cost ratio, net present value, and internal rate of return of each of the development alternatives are compared in Table 9.8. Of the indicators given herein, net present value is generally considered as the most appropriate means of comparing development alternatives for municipal developments. Use of the other economic indicators will result in a conservative choice of development alternatives (see Sec. 9.2).

Each development alternative indicates an excellent economic return. The difference in net present value between alternatives A (4 units) and B (3 units) was generally small, and the choice between the two will depend on the prevailing motive for development. If capacity or annual energy production is to be the basis for the decision, alternative A would be chosen. But alternative A requires a capital investment that is 38 percent more than alternative B, and if this extra investment is to be justified on the basis of its incremental rate of return (the rate of return on the extra investment), then alternative B might well be chosen since the incremental rate of return for alternative A is only slightly higher than the

TABLE 9.9 Cost and Benefit Streams for Development Alternative B at the St. Cloud Dam*

Year	Debt service, $	O&M costs, $	Gross income, $	Present worth, $ Benefits	Costs	Cash flow	Net present value, $
1	367,742	0	0	0	331,300	-331,300	-331,300
2	735,485	0	0	0	596,936	-596,936	-928,236
3	735,485	76,666	876,310	640,750	593,838	46,913	-881,323
4	735,485	83,566	955,178	629,205	539,534	89,671	-791,652
5	735,485	91,087	1,041,144	617,868	490,530	127,338	-664,314
6	735,485	99,284	1,134,847	606,735	446,302	160,434	-503,880
7	735,485	108,220	1,236,983	595,803	406,378	189,426	-314,454
8	735,485	117,960	1,348,311	585,068	370,332	214,736	-99,719
9	735,485	128,576	1,469,659	574,526	337,783	236,743	137,025
10	735,485	140,148	1,601,929	564,174	308,384	255,790	392,815
11	735,485	152,761	1,746,102	554,009	281,826	272,183	664,998
12	735,485	166,510	1,903,252	544,027	257,827	286,200	951,198
13	735,485	181,496	2,074,544	534,225	236,136	298,089	1,249,287
14	735,485	197,830	2,261,253	524,599	216,524	308,075	1,557,362
15	735,485	215,635	2,464,766	515,147	198,788	316,359	1,873,721
16	735,485	235,042	2,686,595	505,865	182,743	323,122	2,196,843
17	735,485	256,196	2,928,388	496,750	168,221	328,529	2,525,372
18	735,485	279,254	3,191,943	487,800	155,075	332,725	2,858,097
19	735,485	304,386	3,479,218	479,011	143,167	335,844	3,193,940
20	735,485	331,781	3,792,348	470,380	132,377	338,003	3,531,943
21	735,485	361,641	4,133,659	461,904	122,595	339,309	3,871,252
22	735,485	394,189	4,505,688	453,582	113,723	339,859	4,211,111
23	735,485	429,666	4,911,200	445,409	105,670	339,739	4,550,850
24	735,485	468,336	5,353,208	437,384	98,358	339,026	4,889,875
25	735,485	510,486	5,834,997	429,503	91,714	337,789	5,227,665
26	735,485	556,430	6,360,147	421,764	85,672	336,093	5,563,757
27	735,485	606,509	6,932,560	414,165	80,173	333,991	5,897,749
28	735,485	661,095	7,556,491	406,702	75,166	331,536	6,229,285
29	735,485	720,593	8,236,575	399,374	70,602	328,772	6,558,057
30	735,485	785,447	8,977,867	392,178	66,439	325,740	6,883,797
31	735,485	856,137	9,785,875	385,112	62,637	322,476	7,206,273
32	735,485	933,189	10,666,603	378,173	59,161	319,012	7,525,285
33	735,485	1,017,176	11,626,598	371,359	55,981	315,378	7,840,664
34	735,485	1,108,722	12,672,991	364,668	53,067	311,601	8,152,264
35	735,485	1,208,507	13,813,560	358,098	50,395	307,702	8,459,967
36	735,485	1,317,273	15,056,781	351,645	47,941	303,704	8,763,671
37	735,485	1,435,827	16,411,891	345,309	45,685	299,625	9,063,295
38	735,485	1,565,051	17,888,961	339,088	43,607	295,481	9,358,776
39	735,485	1,705,906	19,498,968	332,978	41,691	291,287	9,650,063
40	735,485	1,859,438	21,253,875	326,978	39,921	287,057	9,937,120
41	735,485	2,026,787	23,166,724	321,087	38,285	282,802	10,219,922
42	735,485	2,209,198	25,251,729	315,301	36,768	278,533	10,498,455
43	735,485	2,408,026	27,524,385	309,620	35,361	274,259	10,772,714
44	735,485	2,624,748	30,001,579	304,042	34,053	269,988	11,042,703
45	735,485	2,860,975	32,701,721	298,563	32,835	265,728	11,308,431
46	735,485	3,118,463	35,644,876	293,184	31,699	261,485	11,569,915
47	735,485	3,399,125	38,852,915	287,901	30,638	257,264	11,827,179
48	735,485	3,705,046	42,349,677	282,714	29,644	253,070	12,080,249
49	735,485	4,038,500	46,161,148	277,620	28,711	248,908	12,329,158
50	735,485	4,401,965	50,315,652	272,618	27,835	244,782	12,573,940
51	367,742	4,798,142	54,844,060	267,706	25,216	242,490	12,816,430
52	0	5,229,975	59,780,026	262,882	22,999	239,883	13,056,314

Payback period = 7 years
Present net value = $13,056,314
Benefit/cost ratio = 2.60
Internal rate of return = 31.5%

*Base year for present worth is 1981. Two-year construction period; 11 percent discount rate and 9 percent escalation rate. All figures in dollars.

discount rate; a marginal annual rate of return (MARR) much higher than this would almost certainly be required by a private developer. In addition to this latter consideration, it should be noted that negative cash flows will persist for the first two years of operation for alternative A, and if this were chosen, the owner would need to be prepared to absorb them.

Uncertainty Analysis. The expected values and the uncertainties—at 68 percent confidence intervals—for six variable parameters were estimated as follows.

Capacity and Energy Value, U. The value used in the initial alternative comparison was the purchasing utility's base energy purchase rate, but the expected value would include capacity credit amounting to $160,000 per year, or alternatively, 4.3 mills/kWh for alternative B. This is established as the value of the uncertainty in the value of capacity and energy, so that

$$U = 0.0181 \pm 0.0043 \text{ \$/kWh} \qquad (P = 0.68)$$

Capital Cost, C_c. Cost estimates in prefeasibility or feasibility studies are not as detailed as in the final design stage of the project. Unforeseen future events can alter project development costs, and cost overruns are not at all uncommon. In accordance with the accuracy that is typical for this level of study [4], the uncertainty in the capital costs estimated for alternative B is set at 20 percent of the expected capital cost:

$$C_c = \$6{,}650{,}000 \pm 1{,}330{,}000 \qquad (P = 0.68)$$

Operation, Maintenance & Replacement Costs, OM&R. Data from the U.S. Energy Information Administration [8] reveal considerable scatter in the correlation between plant capacities greater than 10 MW and annual OM&R costs (Fig. 9.10). Consequently, an uncertainty in annual OM&R amounting to 50 percent of the expected costs is not at all unreasonable. For St. Cloud alternative B,

$$\text{OM\&R} = \$59{,}200 \pm 29{,}600 \qquad (P = 0.68)$$

Discount Rate, d. The St. Cloud prefeasibility study was performed in the months just prior to the period of time in 1981 and 1982 when interest rates skyrocketed (A-rated municipal bonds reached 13½ percent in January 1982), but the trend for the year prior to August 1981 had been one of a fairly steady increase amounting to over ¼ percent a month. In consideration of market volatility, the uncertainty in the discount rate to be applied to the 50-year cash-flow stream for the St. Cloud facility was established at 3 percent, so that

$$d = 11.00 \pm 3.00\% \qquad (P = 0.68)$$

Escalation Rate, e. From the vantage point of 1981, long-term escalation rates (10 years or more) based on the Consumer Price Index were rising relentlessly since the late 1960s, the 10-year rate going from below 2 percent to above 8 percent by the beginning of 1981. Paralleling interest rates, the trend of long-term inflation rates reflected an increasingly mercurial economy. The uncertainty in the escalation rate was set at 3 percent around an expected value of 9 percent:

$$e = 9.00 \pm 3.00\% \qquad (P = 0.68)$$

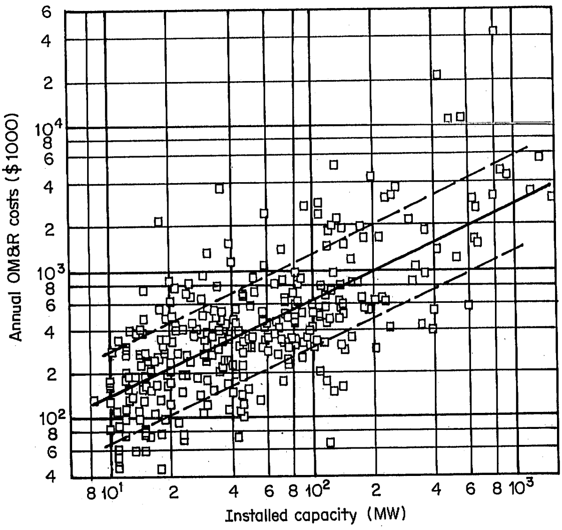

FIGURE 9.10 Annual OM&R costs versus installed capacity, 1981 base year. Dashed lines indicate one standard deviation in the logarithm of annual OM&R costs.

Energy Production, kWh. Annual energy production is calculated by the use of the flow-duration curve in combination with the corresponding net head and efficiency curves, such that

$$\text{kWh} = \int P\,dt = \int (C \times Q \times H \times e)\,dt \tag{9.23}$$

where P = instantaneous power output
Q = discharge
H = net head
e = overall generating efficiency
C = a conversion coefficient

The integral is taken over a year's time. Essentially, annual energy output, kWh, is the area beneath the annual "power-duration curve."

Though we would have little or no basis for determining "uncertainty" in the site's actual flow-duration curve as it would pertain to 50 years or more of plant operation (annual variability is only of concern in a cash-flow analysis, performed as part of a financial feasibility study), there may be a discernible error in the flow-duration curve as it is derived. This error revolves on the means by which flows in the river are measured, in this case, a staff gauge used in conjunction with a stage-discharge rating curve. The U.S. Geological Survey assigns uncertainty values, associated with a 95 percent confidence interval (two standard deviations of a normal probability distribution), to each gauge and rating curve; this value for the St. Cloud site has been determined as ±8 percent of the measured flow, and since the economic uncertainty analysis is performed for one-standard-deviation uncertainties, ±4 percent is used. For the preceding result, consideration was given to the fact that some 20 to 30 percent of the time the river's discharge will exceed plant capacity, rendering river discharge measurement inaccuracies irrelevant for those times. With respect to measurements of net head, the uncertainty is determined to be ±2 percent. Finally, an uncertainty value of ± percent is assigned to estimations of efficiency.

Since the integration necessary to perform annual energy calculations precludes the assignment of any single expected values to discharge, head, or efficiency, the nondimensional form of Eq. (9.11) must be applied:

$$\frac{\partial R}{R} = \left\{\left[\frac{\partial \ln R}{\partial \ln x_1}\frac{\partial x_1}{x_1}\right]^2 + \left[\frac{\partial \ln R}{\partial \ln x_2}\frac{\partial x_2}{x_2}\right]^2 + \left[\frac{\partial \ln R}{\partial \ln x_N}\frac{\partial x_N}{x_N}\right]^2\right\}^{1/2} \tag{9.24}$$

$$\frac{\partial \ln R}{\partial \ln x_1} = \frac{x_1}{R}\frac{\partial R}{\partial x_1} \tag{9.25}$$

It follows from Eq. (9.23) that

$$\frac{\delta(\mathrm{kWh})}{\mathrm{kWh}} = \left\{\left[\frac{Q}{\mathrm{kWh}}\frac{\partial \mathrm{kWh}}{\partial Q}\frac{\delta Q}{Q}\right]^2 + \left[\frac{H}{\mathrm{kWh}}\frac{\partial \mathrm{kWh}}{\partial H}\frac{\delta H}{H}\right]^2 + \left[\frac{e}{\mathrm{kWh}}\frac{\partial \mathrm{kWh}}{\partial e}\frac{\delta e}{e}\right]^2\right\}^{1/2} \tag{9.26}$$

which may be approximated as

$$\frac{\delta(\mathrm{kWh})}{\mathrm{kWh}} \approx \left\{\left[\frac{\delta Q}{Q}\right]^2 + \left[\frac{\delta H}{H}\right]^2 + \left[\frac{\delta e}{e}\right]^2\right\}^{1/2} \tag{9.27}$$

Since

$$\frac{\delta Q}{Q} = 0.04 \qquad \frac{\delta H}{H} = 0.02 \qquad \text{and} \qquad \frac{\delta e}{e} = 0.01 \tag{9.28}$$

it follows that

$$\frac{\delta(\mathrm{kWh})}{\mathrm{kWh}} \approx 0.046 \tag{9.29}$$

and since the expected average annual energy production is equal to 37.44 GWh, the uncertainty is computed to be ±1.72 GWh:

$$\mathrm{GWh} = 37.44 \pm 1.72\ \mathrm{GWh} \qquad (P = 0.68)$$

Results. The uncertainty in feasibility indicators was computed using the techniques developed herein and are given in Table 9.10. More uncertainty exists in the feasibility indicators for the St. Cloud Hydropower Project than for the minihydropower facility of case study 1. The analysis indicated good feasibility for the proposed development and that more detailed investigation was justified to reduce some of the uncertainty in the variable parameters.

TABLE 9.10 Uncertainty Analysis for Alternative B, 1981 Base Year

Capital cost C_c, $	6,650,000 ± 1,330,000
Discount rate d, %	11.00 ± 3.00
Operation, maintenance & replacement costs, OM&R, $/yr	59,200 ± 29,600
Escalation rate e, %	9.00 ± 3.00
Energy production, GSh	37.44 ± 1.72
Capacity and energy value, U ($/kWh)	0.0181 ± 0.0043
Net present value (NPV), $	13,060,000 ± 13,000,000
Benefit-cost ratio (B/C)	2.60 ± 1.54
Internal rate of return (IRR), %	31.5 ± 17.3

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CHAPTER 10
PLANT MAINTENANCE AND OPERATION

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10.1 BACKGROUND

There have been important changes in the character, size, and utilization of hydroelectric plants since many of the standard references were prepared. The range of sizes of individual machines has been extended to single units approaching 1-GW capacity. Several large-capacity reversible hydroelectric pumped-storage plants have been built since the late 1950s. At the same time, with relative scarcity of economical sites on the principal rivers and with ownership of generating plants by entities which are not utilities, there has been renewed construction and operation of small hydropower plants on small streams.

Low-priced digital electronic computers have increased in computing capacity so greatly that both large and small plants can be controlled automatically. With computers, some plants are being controlled by software (computer instruction programs) rather than by manual or mechanical operators. That is not universally desirable. Operation within larger systems is increasingly done with computer-aided on-line analysis. Yet many hydropower plants are still operated with preset schedules of output at different times of the day. Some new units and plants are so small or are located such that computer-aided operation is uneconomical or otherwise impractical. On the other hand, many hydropower plants are small and/or in such remote locations that it is uneconomical to have operating and maintenance people at hand. Other larger hydropower plants can be operated from remote locations and maintained periodically by roving maintenance crews.

There is such variation in hydropower plants that this chapter is not intended to cover all possible situations. Specific problems should be solved only after considering technical, economic, and organizational aspects. There is no attempt here to give guidance for maintenance where manufacturers' instruction and maintenance manuals will be more complete and more specific. Each encounter with a plant or a machine should be viewed as a new experience requiring careful investigation and thought. Environmental consequences of maintaining and operating a hydropower plant should also be considered in advance. Other chapters of this book explain environmental matters and the operation of a hydropower plant within a larger system.

Individual utilities and larger pooled systems often have regulations specifying when units can be shut down for maintenance or testing. Entire river systems are now operated for maximum reliability and maximum economy of the system rather than of the single plant. Very large organizations, such as the U.S. Army Corps of Engineers, the U.S. Bureau of Reclamation, and Ontario Hydro, have codified maintenance and operation procedures. Their publications can be obtained as references giving more detailed information than that included herein.

The "downtime" during repairs and replacement of hydropower turbine-generating units or repairs to other elements of a hydropower plant often has a greater cost associated with it than the costs for the repairs. That cost of downtime is measured either by the amount of revenue lost by a nonutility owner or by the extra cost of generating the same amount of power and energy delivered to the consumer from fuel-consuming plants elsewhere in the system of the utility owner or from neighbor systems. The equivalent amount of energy might not be generated at the same time, i.e., where storage at another hydropower plant reservoir could permit loading units at the other plant more than otherwise intended, but the water in storage would be reduced and fuel might then have to be consumed later.

The objective for maintaining the hydropower plant is keeping it able to operate efficiently, with as little downtime as is practical, and to keep it so operating for many years. If the plant and the equipment have been designed and built with sufficient care, life expectancy before replacements are required can be very long. There are units operating under the full gamut of conditions with generators not needing rewinding after 30 to 40 years. There are hydraulic turbines still operating with relatively modern efficiency after 50 years. Some turbines of older design, and therefore with lesser efficiency, are still operating reliably after 75 and more years.

10.2 ORGANIZATION

A key element of operating efficiently for a long period is planning ahead and providing the people, the tools, and the supplies for operating and maintaining the hydropower plant. Who will operate and maintain the hydropower plant and how they will do it affect the equipment purchased. In some instances, when automatic, remotely operated plants are being built, the dispatchers or substation operators must be trained and instructed, rather than plant operators. People on roving maintenance crews may be dealing with a variety of equipment, requiring differing maintenance procedures and techniques.

Large utilities having several large hydropower plants usually separate the maintenance and operating functions. Operating personnel are often directly responsible to the person in charge of dispatching, assigning responsibility for meeting a portion of the total load on the system to each of several generating plants. The dispatching also involves transmission and/or subtransmission of electricity to local area loads. The modern trend is to utilize computers to carry out detailed calculations so that loads can be met within one or more of system short-term and long-term objectives. As the systems became increasingly complex, the quick computing facility was useful in aiding the dispatchers to reach decisions. A few systems entrust the actual decisions and allocation of responsibility for meeting the load to automatic equipment, with override capability entrusted to the people.

The system operation group, including the dispatchers and the people who coordinate operation of their system with interconnected neighbor systems, must control the operation of the individual plants. System operation obtaining the most from the available resource for the least long-term disbursement of funds and wear and tear on the equipment is discussed in Chap. 11. Plants can be operated to meet a given system requirement in more than one way, however. The organization for operating the individual plant is part of the organization for operating the system.

At relatively large hydropower plants, the maintenance group is responsible for maintaining the equipment and structures of the single plant. The plant maintenance superintendent is often responsible to an engineer in the utility power production division. Organization and titles vary among utilities. Plant maintenance is often divided between electrical, mechanical, and general maintenance.

At large plants, there may also be a plant engineering person or group responsible for determining the optimum allocation of water among the units. Getting the most from the available water depends on the condition of the units, the maintenance periods available within constraints of system dispatch, the characteristics of each individual unit, the amount of water stored in the reservoir, and the headwater and tailwater levels. At some hydropower plants, additional constraints are imposed by regulations designed to protect or enhance the fishery, the quality of the water, or recreation in and near the reservoir.

The size of the organization for operating and maintaining a hydropower plant depends on these, among other, factors:

- The number of units
- The sizes of the units
- The size of the system operated by the owner
- The complexity of the power plant
- The owner's resources available outside the plant for handling purchasing, maintenance, accounting, record keeping, real estate, and legal work
- Whether the plant is part of a larger system dispatched from outside the plant
- The degree of automation and control

There has been a trend toward automating many of the details of hydropower plant operation. Status of individual units and items such as temperatures, pressures, frequency, and voltage levels can readily be sensed and reported automatically, with automatic transmittal and recording of the data. Even diagnostic information, such as shaft runouts and vibration, can be obtained from permanent sensing devices which are reliable and relatively economical. However, regardless of the degree or sophistication of the automation, provision should be made for manual override and operation and sensing by people at the plant. This is particularly important during initial startup and after each startup following maintenance on the unit.

Figure 10.1 is an outline organization chart for a large low-head hydropower plant. The size or the number of people noted on the chart are not the key matters here. The distribution of the various functions in a sizable plant having several large generating units is what is presented as an example.

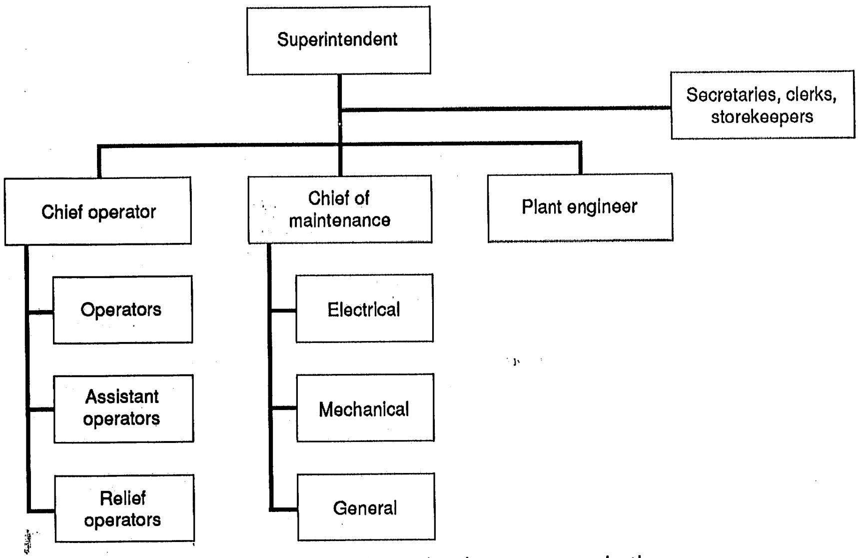

FIGURE 10.1 Example of plant operation and maintenance organization.

10.3 PLANT OPERATING AND MAINTENANCE PERSONNEL

The hydropower plant should be staffed to minimize both operating costs and downtime, when possible. The number of people assigned to the plant is, therefore, related to the design of the plant, assuming the operating needs and means for system dispatch were adequately considered by the designers. Downtime can be low only if the equipment installed at the plant is physically rugged. Providing ample mass is sometimes overlooked in the attempt to reduce investment cost. With ample mass of machinery and supporting structures, vibrations, which would otherwise fatigue and crack the materials and cause future downtime, can be absorbed without damaging the equipment. A low-mass plant is also noisy and unpleasant.

Other elements of the plant affect the number of people needed for its efficient and safe operation. A plant arranged such that trash and/or ice pile up on the trash racks requires means for removing the trash or ice. Automatic raking equipment or sluicing equipment should be provided. The continuing labor needed for clearing trash racks for long periods of time is tough on people and a costly exercise.

The locations of the equipment and the plant overall arrangement also affect how many people are needed to operate and maintain the plant. A highly automated plant needs few operating and maintenance personnel. A plant in a remote location may have a minimum or no resident staff. That choice is a matter of the importance of the plant to the owner. If the owner is a very large utility having numerous plants and if the plant is a tiny portion of the owner's generating resource, no resident staff may be provided. If there is a fire or other accident, the owner will obtain revenue required for repair by selling electricity generated with other resources or purchased from interconnected utilities. However, if the plant is the single generating resource of an owner who does not have a distribution franchise and who does not have interconnections to other utilities, outage after an accident or a fire will eliminate revenue and become a very serious financial matter. At many places in the world, the plant may be the single source of electricity for an isolated load.

Personnel requirements for a hydropower plant parallel the personnel requirements for a city. It has mechanical and electrical utility systems—potable and nonpotable water; compressed air; emergency power; oil piping; heating, ventilating, and air conditioning; communications systems; fire protection systems; and elevators (in large plants).

A hydropower plant may have tunnels and galleries, its own electrical distribution networks, cranes and hoists, yards and docks, railroad access (sometimes), lawns, painted surfaces, paving, retaining walls, alarm systems, lighting and other wiring, and washrooms and sanitary systems.

A hydropower plant may have facilities for oil purification and storage, stairways and shafts, instruments and sensing and measuring devices, need for flood protection, hazardous waste disposal problems, and warehouses and machine shops. Some large plants even have tourist facilities and museums and public relations facilities. Under environmental enhancement regulations, recreation facilities and picnic facilities may be part of a reservoir-dam-hydroelectric plant complex. All of that must be maintained and operated.

Selection and Training of Operators

The operating people are chosen from those most alert and clear thinking. The shift lead operators are often people who have previously been operators at other plants. It is common and desirable that operating assistants and floor personnel will have been working on the plant during its construction.

Operators are usually trained in basic electricity and then trained in the specifics of the transmission and distribution network and the electrical equipment in the plant. Operators must also understand the hydraulic equipment and systems in the plant. Those operators who are working when the plant and equipment are first started and tested obtain many years of equivalent experience in handling unusual situations.

Most of the time, operation is routine. There is a tendency for operation to be considered boring, routine work. The operating staff should never be allowed to devolve into that attitude. When emergencies and other special situations arise, an error in operation may have grave consequences. The value of the equipment and the potential danger to people and property are often very great. The operators should have the judgment to know when to call on assistance from their supervisors and they may have clearance to directly ask advice from plant engineering and maintenance people.

Maintenance Staff

Maintenance people must be skilled in their trade. Mechanics and electricians on the maintenance staff are often recruited from those who worked on the equipment during its initial construction and erection, with the initial supervision of the manufacturer's erecting people. They have also worked through overcoming startup errors and problems and are already familiar with the equipment. At large plants it is desirable that the master mechanic and the lead electrician be highly skilled and experienced. Future maintenance may involve the full gamut of work on many different kinds of equipment, from accurate machining, complex welding of heavy parts, working with molten silver, to balancing and alignment.

The maintenance staff usually works a single shift. It is common that the last duties on a shift are preparing thorough records and reports of what has been done during the shift. The maintenance records are important for tracing the history of operation and repairs and condition of each item of equipment in the plant. That information is essential for future decisions regarding the causes of accidents and when equipment and systems should be inspected, repaired, and replaced. While record keeping of operations is often automated, maintenance records are more a matter of unique reporting, even if the information is finally entered into computer storage.

Plant Engineering

Many decisions in operating and maintaining a hydropower plant require the knowledge and experience of trained engineers. The engineers may be at a headquarters office or at the plant. Engineering decisions are needed for forecasting water supply; for analyzing effects of operation on life of the equipment and efficiency of its operation; for deciding effects of alternatives for repairing and maintaining the equipment; for scheduling maintenance and advising the dispatchers of status of the equipment and need for changing availability; for developing tables or graphs of performance which are needed for operating the units; for allocating water among the units to produce the maximum kilowatts and kilowatt-hours from the available supply; for analyzing operating and maintenance records and comparing the most recent experience with the past history of operation and maintenance, among other items.

The plant engineers work closely with the superintendent of a large plant, who has overall responsibility for all aspects of its operation and maintenance. Work of the plant engineers straddles many fields of engineering. Some large plants and systems having a staff of engineers can enjoy the luxury of having engineers skilled in dispatch, instruments and testing, mechanical operation and maintenance, electrical equipment, and structures and welding, as well as analysis of performance and economics.

Supervision of general maintenance of the plant should not be overlooked. Polishing brass nameplates until the information is obliterated does not help. However, there are likely to be fewer accidents in a clean, orderly plant. There is also likely to be less ultimate need for costly maintenance if the regular housekeeping is adequate.

10.4 GENERAL PREVENTIVE MAINTENANCE

Preventive maintenance, as preventive medicine, may be the least costly and most effective means of prolonging the efficient, useful life of a hydropower plant. This section is divided into two parts. The first concerns scheduled inspections and routine maintenance of equipment, systems, and structures at the hydropower plant. The second concerns ongoing monitoring of status, temperatures and pressures, rates of flow, runout and vibration of shafts and equipment, and electrical characteristics of the equipment.

Inspection and Routine Maintenance

Many large organizations have detailed schedules for inspection and routine maintenance of the equipment and systems at their hydropower plants. Each structure, water conveyance facility, item of mechanical equipment, and item of electrical equipment and apparatus should be inspected and maintained at regular intervals. The intervals might be daily, weekly, monthly, annually, or longer. The intervals for inspection are usually shortest when the plant is new. As experience is gained with the plant, the intervals may be longer. Files should be kept for each structure, system, and item of apparatus, and the records must be thorough and accurate.

The plant operating and maintenance people should be aware of the manufacturer's operating and maintenance instructions. If those are violated during the guarantee and warranty periods, there may be no recourse to the manufacturer or equipment supplier.

A relatively complete series of checklists which the U.S. Bureau of Reclamation uses is included in Table 10.1 for reference. The procedures portions of the lists refer to the details of how the inspection and routine maintenance are to be carried out. The intervals are those recommended for U.S. Bureau of Reclamation plants and are not applicable to all hydropower plants.

Of course, the detailed schedules given in Table 10.1 cannot be followed for a small, remote-operated plant. At many large plants, dewatering the turbines and

Table 10.1 Inspection and Routine Maintenance Checklists

Item of inspection	Inspection interval*	
A. Power dams and canals		
Power dams and canals (in general):	D	
Concrete and masonry	D	
Earth embankments or fills		A
Canal lining and riprap	A	
Expansion joints		A
Roadways and walks		A
Railings and miscellaneous metalwork		A
Stairways and ladders	M	A
Cranes, hoists, and elevators (See sec. D below)		
Gates and valves (See sec. C below)		
Trash racks	D	
Trash-removing facilities		A
Ice prevention and removing facilities		A
Galleries and shafts	W	A
Fish ladders and screens	W	A
Conduit and fittings		A
Wiring and wiring devices		A
Lighting and power	W	
Floating boom	D	A
Reservoir or forebay		A

Table 10.1 Inspection and Routine Maintenance Checklists *(Continued)*

Item of inspection	Inspection interval*	
B. Power-plant buildings		
Components:		
Doors and windows	M	A
Stairways and ladders	M	A
Railings and miscellaneous metalwork		A
Elevators, cranes, and hoists (See sec. D below)		
Electric space heaters	D	
Ventilating fans		A
Water supply and drain piping		A
Water heaters and coolers	W	A
Wash basins, sinks, showers, toilets, etc.	W	
Water supply	W	A
Conduit and fittings		A
Wiring and wiring devices		A
Lighting	W	
Fire protection	M	A
C. Penstocks, gates, and valves		
Penstocks:		
Foundation	Q	A
Sliding supports	Q	A
Slip joints	Q	A
Rivets, welds, and bolts	Q	A
Exterior paint and surface		A
Interior paint and surface		A
Gates and valves (in general):		A
Seals and guides		NS[a]
Seat,disk, or needle ring and spider		NS[a]
Stems		A
Lubrication	NS[a]	A
Spring and cushioning mechanism		NS[a]
Journals, bearings, and bushings		NS[a]
Wheels, pins, and rollers		NS[a]
Operating cylinder or gear mechanism		NS[a]
Control piping		A
Packing glands	W	A
Operation	NS[a]	A
Gate hoist or crane (See sec. D below)		
Electric motors (See sec. G below)		
Electrical control equipment (See sec. M below)		
D. Cranes, hoists, and elevators†		
Components:		
Crane, rails, supports, and stops		A
Crane bridge and carriage		A
Hoist framework		A
Trucks		A
Bumpers		A
Trolley rails or wire and supports		A
Trolley shoes or wheels		A
Runways and catwalks		A

Table 10.1 Inspection and Routine Maintenance Checklists *(Continued)*

Item of inspection	Inspection interval*	
D. Cranes, hoists, and elevators† *(Continued)*		
Components: *(Continued)*		
Ladders and handrails		A
Cab	W	A
Driving gears, shafts, bearings, and wheels	W	A
Brakes	M	A
Cable drums and sheaves	M	A
Cables or chains	M	A
Lifting beams		A
Counterweights		A
Blocks and hooks	M	A
Electric motors and motor-generator sets (See sec. G below)		
Electrical control equipment (See sec. M below)		
Electrical wiring		A
Hydraulic-hoist cylinder, piston, and rod	NS[a]	
Hydraulic pump	M	NS[a]
Hydraulic control valves and piping	M	NS[a]
Hydraulic oil reservoir tank	M	
Guide rails and shoes	M	A
Safety devices	M	A
Operation	W	
State elevator inspection		A
E. Miscellaneous station auxiliaries		
Air compressors:		
Foundation	M	A
Frame casting		A
Belt, chain, or gear drive	D	A
Belt pulleys and idler or coupling	D	A
Crankshaft		NS[b]
Connecting rod		NS[b]
Crosshead	D	A
Piston and piston rod		NS[b]
Cylinder		NS[b]
Valves		NS[b]
Bearings		NS[b]
Packing gland	D	A
Lubricating system	D	A
Cooling system	D	A
Receiver tank	D W	A
Air intake and cleaner	D	M
Gauges	D	A
Pressure switches	M	A
Unloader	M	A
Piping and valves		A
Safety valve	M	
Guardrails or grills		A
Electrical motors (See sec. G below)		
Electrical control equipment (See sec. M below)		
Operation	W	
Cleaning		A

Table 10.1 Inspection and Routine Maintenance Checklists *(Continued)*

Item of inspection	Inspection interval	
E. Miscellaneous station auxiliaries *(Continued)*		
Water or oil pumps:		
Foundation	M	A
Frame casting or casing		A
Belt, chain, or gear drive	D	A
Belt pulleys and idler or coupling	D	A
Crank or rotor shaft		NS[b]
Connecting rod		NS[b]
Crosshead	D	A
Piston and piston rod		NS[b]
Cylinder		NS[b]
Valves		NS[b]
Impeller or rotor		NS[b]
Bearings		NS[b]
Packing gland	D	A
Lubricating system	D	A
Storage tanks	D A	NS[b]
Intake strainer	W	A
Piping and valves		A
Gauges	D	A
Pressure or float switches	M	A
Guardrails or grills		A
Electric motor (See sec. G below)		
Electrical control equipment (See sec. M below)		
Operation	W	
Storage batteries:		
Battery room and ventilation	D	A
Base or rack		A
Base pad		A
Cell jars and covers		M
Plates—Sediment		M
Separators		M
Electrolyte		M
Intercell connectors and terminals		M
Hydrometers and thermometers		A
Sink, funnels, and fillers		A
Water still		A
Distilled water storage		M
Acid storage		M
Operation	W	
Battery chargers:		
Motor and generator (See sec. G below)		
Switchboards and control equipment (See sec. M below)		
Transformer or reactor		M
Dial switch or rheostat		M
Bulb or dry rectifier element	D	M
Instruments	D	M

Table 10.1 Inspection and Routine Maintenance Checklists *(Continued)*

Item of inspection	Inspection interval*	
F. Hydraulic turbines, governors, and large pumps		
Turbines and large pumps:		
Runner or impeller		A
Seal rings	NS[a]	A
Spiral case or pump casing		A
Wicket gates	NS[a]	A
Curb plates (wearing rings)		A
Thrust collars		A
Gate linkage	D	A
Draft tube		A
Shaft and coupling	D	A
Shift ring and bearing surfaces		NS[a]
Bearings	D A	NS[a]
Servomotor cylinder, piston, and rod	D	NS[a]
Packing glands	D	NS[a]
Lubrication (See sec. E above)		
Thermometers and gauges	D	A
Governors:		
Electric control devices (See sec. M below)		
Ball head, bearings, and motor	S	A
Speeder rod and vibrator disk		A
Pilot and relay valves and strainers	W	SA A
Compensating dashpot assembly		A
Oil piping and pressure tank	D	A
PMG assembly		A
Belt and drive	D	A
Controls and indicators	D	A
Linkage and pins	D	A
Oil pump (See sec. E above)		
Safety valve		A
G. Generators, motors, and synchronous condensers		
Components:		
Foundation, base, or support	M	A
Frame		A
Laminations and pole pieces		A
Armature or rotor		A
Airgap		A
Air fans		A
Windings	M	A
Banding and lashing		A
Slot wedges		A
Commutator or collector rings	D	A
Brushes		A
Brush rigging		A
Shaft and bearings	D	A
Couplings, gears, and pulleys		A
Cooling coils and air coolers	D	A
Hydrogen cooling equipment	D	SA
CO_2 or water spray fire protection	W	SA
Temperature indicators and relays		A
Water and oil flow gauges and relays		A

Table 10.1 Inspection and Routine Maintenance Checklists *(Continued)*

Item of inspection	Inspection interval*	
H. Low-voltage switchgear, buses, and cables		
Components: *(Continued)*		
Oil and air circuit breakers (See sec. I below)		
Disconnecting switches and fuses		A
Bus bars, joints, and connections		A
Bus insulators and supports		A
Bus enclosures and barriers		A
Switchgear panels and enclosures	W	A
Locks and interlocks	W	
Warning and safety signs	W	
Current and potential transformers		A
Meters, instruments, and relays (See sec. M below)		
Control devices (See sec. M below)		
Panel wiring (See sec. M below)		
Power cables		A
Potheads		A

I. Oil and air circuit breakers

Item of inspection	Inspection interval*: Attended station		Inspection interval*: Unattended station	
Components:				
Foundation		A		A
Frame and tanks	D	A	W	A
Oil valves and plugs	D	A	W	A
Oil levels and gauges	D	A	W	A
Breathers and vents	D	A	W	A
Panels and cabinets		A		A
Bushings or insulators	W	A	W	A
Bushing current transformers and potential devices		A		A
Main terminals and ground connections	D	A	W	A
Main contacts		A		A
Contact pressure springs		A		A
Flexible shunts		A		A
Magnetic, air, or oil blowout device		A		A
Crosshead		A		A
Lift rods and guides		A		A
Operating rods, shafts, and bell cranks		A		A
Closing solenoid, air cylinder, motor, or spring		A		A
Manual operating device		A		A
Air compressor and air tank (See sec. E above)	W	A	W	A
Latch and trip mechanism	W	A	W	A
Tripping solenoid	W	A	W	A
Control and protective relays		A		A
Solenoid valves		A		A
Auxiliary switches		A		A
Operation counter	M	A	M	A
Position indicator		A		A
Dashpots or snubbers		A		A
Mechanism cabinet		A		A

Table 10.1 Inspection and Routine Maintenance Checklists *(Continued)*

I. Oil and air circuit breakers *(Continued)*

Item of inspection	Inspection interval*			
	Attended station		Unattended station	
Components: *(Continued)*				
Cabinet lights and heaters	W	A	W	A
Power supplies and wiring	W	A	W	A
Oil dielectric test		A		A
Filter oil		NS		NS
Operation		NS		NS

J. Transformers and regulators

Item of inspection	Inspection interval*							
	Large units				Small units			
	Attended		Unattended		Attended		Unattended	
Components:								
Foundation, rails, and trucks		A		A		A		A
Tanks and radiators	D	A	W	A	W	A	M	A
Oil and water piping	D	A	W	A	W	A	M	A
Valves and plugs	D	A	W	A	W	A	M	A
Oil levels, gauges, and relays	D	A	W	A	W	A	M	A
Breathers and vents	D	A	W	A	W	A	M	A
Relift diaphragm	D	A	W	A	W	A	M	A
Water-cooling coils and piping		A		A		A		A
Flow indicators and relays	D	A		A	D	A		A
Heat exchangers		A						
Oil pumps	D	A			(See sec. E above)			
Cooling fans	D	A			D	A		
Temperature indicators and relays	D	A	W	A	W	A	M	A
Inert gas tanks	D		W					
Gas regulator, gauges, and relays	D	A	W	A				
Gas piping and valves		A		A				
Gas analysis		Q		Q				
Bushings	W	A	W	A	W	A	M	D
Bushing current transformers and potential device		A		A				
Main terminal and ground connections	D	A	W	A	W	A	M	A
Core and coils		NS		NS		NS		NS
Internal inspection		A		A		A		A
Terminal board and connections		A		A		A		A
Ratio adjuster	W	A	W	A	W	A	M	A
Tap changer or regulator	D	A	W	A				
Motor and drive		A		A				
Auxiliary and limit switches		A		A				

Table 10.1 Inspection and Routine Maintenance Checklists *(Continued)*

J. Transformers and regulators *(Continued)*

Item of inspection	Inspection interval*			
	Large units		Small units	
	Attended	Unattended	Attended	Unattended
Components: *(Continued)*				
Operation counter	W A	W A		
Power and control relays			(See sec. M below)	
Operation	A	A		
Power supplies and wiring	D A	W A		
Insulation resistance	A	A	A	A
Oil dielectric	A	A	A	A
Oil acidity	5 yr	5 yr	5 yr	5 yr
Filter and reclaim oil	NS	NS	NS	NS
Fire protection	M A			

Item of inspection	Inspection interval*	
K. Disconnecting switches and fuses		
Components:		
Base and mounting		A
Insulators	W	A
Line and ground connections		A
Blades and contacts	W	A
Contact and hinge springs and shunts		A
Arcing horns		A
Blade latches and stops	W	A
Operating rods, levers, and cranks		A
Gearboxes	W	A
Operating motor and mechanism		A
Auxiliary and limit switches		A
Locks and interlocks	W	A
Switch sticks	W	A
Fuse tubes		A
Fuse links		A
Multiple-shot reclosing fuse	W	A
L. Lightning arresters		
Components:		
Base and supports	M	A
Procelain shells and insulators		A
Grading rings		A
Arrester units (internal)		NS
Gaps		A
Weather sheds and hoods		A
Line and ground connections		A
Operation indicators	W	A
Operation tests		A
M. Switchboards and control equipment		
Switchboards and control panels:		
Panels and supports	W	A

Table 10.1 Inspection and Routine Maintenance Checklists *(Continued)*

Item of inspection	Inspection interval*	
M. Switchboards and control equipment *(Continued)*		
Switchboards and control panels: *(Continued)*		
Metal boxes and cabinets	W	A
Panel wiring		A
Terminal blocks		A
Auxiliary and control relays	W	A
Control switches and pushbuttons	W	A
Indicating lamps	D W	A
Meters and instruments	D W	A
Position indicators	D W	A
Protective relays	D W	A
Test switches or blocks		A
Rheostats and resistors		A
Motor starters and controllers:		
Metal boxes and cabinets		A
Knife switches		A
Fuses and circuit breakers		A
Contacts		A
Contact springs and shunts		A
Blowout coils and arc chutes		A
Operating or holding solenoid and magnet frame		A
Operating shaft or rod		A
Mechanical and electrical interlocks		A
Latches and trip devices		A
Auxiliary switches		A
Overload trip		A
Step starter timers		A
Compensator or autotransformer		A
Miscellaneous control devices		A
Power supplies and wiring	W	A
N. Communications equipment		
Voice-frequency telephone equipment:		
Line patrol	Q	A
Line leakage resistance		SA
Line loop resistance		SA
Manual switchboard		SA
Automatic exchange	M	A
Transmitters, receivers, dials, ringers, magnetos, and switch hooks		A
Ringing generator	M	A
Batteries	M	
Fuses, heat coils, and arresters		SA
Powerline carrier equipment:		
Tubes, transistors		SA
Circuit voltages and currents	M	
Align tuned circuits in transmitter, receiver, and check line trap		A
Protective airgap in tuning unit and coupling capacitor		A

Table 10.1 Inspection and Routine Maintenance Checklists *(Continued)*

Item of inspection	Inspection interval*	
N. Communications equipment *(Continued)*		
Powerline carrier equipment: *(Continued)*		
Transmitter-receiver and control chassis		M
Lead-in insulator		A
Handsets, ringers, and magnetos		A
Equipment cabinets		A
Emergency battery	M	
Emergency motor generator	M	A
Radio-telephone equipment–fixed and mobile:		
All transmitter test (required by radio rules and regulations):		
Frequency	M	SA
Modulation percent, if AM	M	SA
Modulation frequency deviation, if FM	M	SA
Power output	M	SA
Transmitters and receivers:		
Test voltages and currents	M	
Tubes, transistors		SA
Alignment and receiver sensitivity	M	SA
Antenna and transmission line	M	SA
Ventilation and dust	M	A
Antenna tower		SA
Standby power supply	M	SA
Main power supply	M	
Mobile stations only:		
Battery, battery cable, and connections	M	
Dynamotor	A	
Generator and voltage regulator	M	
Antenna	A	
Noise level	M	
Supervisory control and telemetering equipment:		
Electronic amplifiers, signal generators, etc.:		
Test voltages and currents	M	
Tubes, transistors	SA	
Telemetering transmitters and receivers and switchboard recorders	D	AM
Relays	M	A

O. Switchyards and substations

Item of inspection	Inspection interval* — Large units: Attended	Large units: Unattended	Small units: Attended	Small units: Unattended
Components:				
Yard and fences	D	A	M	A
Buildings (See sec. B above)				
Wood structures	M	A	M	A

Table 10.1 Inspection and Routine Maintenance Checklists *(Continued)*

O. Switchyards and substations *(Continued)*

Item of inspection	Inspection interval*			
	Large units		Small units	
	Attended	Unattended	Attended	Unattended
Components: *(Continued)*				
Steel structures	M	A	M	A
Footings and guy anchors	M	A	M	A
Guys	M	A	M	A
Warning signs		A		
Ground connections	M	A	M	A
Conductors and buses	M	A	M	A
Hardware	M	A	M	A
Insulators	M	A	M	A
Transformers and regulators (See sec. J above)				
Oil and air circuit breakers (See sec. I above)				
Disconnecting switches and fuses (see sec. K above)				
Lightning arresters (See sec. L. above)				
Control equipment (See sec. M above)				
Oil storage facilities	M	A	M	A
Conduit, ducts, trenches, and tunnels	M	A	M	A
Static capacitors	D	A	M	A
Storage batteries and chargers (See sec. E above)				
Power supplies and wiring	M	A	M	A

*D = routine daily inspection; W = routine weekly inspection; M = routine monthly inspection; Q = quarterly inspection; SA = semiannual inspection; A = annual inspection; NS = not scheduled; [NS[a] = frequency as required to maintain equipment and assure its operating functions; NS[b] = frequency based on equipment operation history (1 to 5 years)].

†Always make detailed operation and maintenance checks of cranes and hoists before lifting heavy loads.

Source: Abstracted from the U.S. Bureau of Reclamation Power Operation and Maintenance Bulletin No. 19, "Maintenance Schedules and Records," rev. October 1965.

inspecting them is done at intervals of 2, 3, 4, or even 5 years. The period is determined from earlier years' experience of operating and maintaining the units.

Monitoring

Monitoring the status of mechanical, electrical, and thermal characteristics of the equipment at a hydroelectric plant has always been an element of operation and maintenance. Headwater and tailwater levels must be known for various reasons. Bearings and windings temperatures may be telltale signs of impending damage to the equipment. Noise changes may be an indication of impending failure of bearings and of impending breaks of parts. Noise may also be caused by misalignment. Changes in the electrical resistance of cables and windings may be signs of aging and impending short circuits. Increasing vibration and runout of shafts may preview mechanical damage to the bearings and changes of dynamic balance and/or alignment of the rotating parts of a machine, as well as being manifestations of cracked shafts. Changes in pressures and temperatures may advertise obstructions or leaks in piping, other pressure vessels, or strainers.

Changes in the number of kilowatts produced at a given head and percentage of maximum output may indicate damage from cavitation or an increase of leakage through turbine runner seal gaps which are increasing in opening. Seal gaps may also increase as the turbine becomes, in a sense, a boring mill opening the diameters to accommodate increasing shaft runouts caused by other factors.

Sometimes the information obtained from monitoring is immediate evidence of trouble. But there are many instances when the newly obtained information must be compared with the history of past information and carefully analyzed before the change becomes evident. It is of no value to accumulate voluminous records, even if the data can now be stored at relatively low cost, when the information is not analyzed carefully. Comparing and analyzing the data is one of the most important functions of the plant engineers.

While the sensing and frequency-control functions of modern turbine governors are accomplished with solid-state electronic equipment, the force needed for opening and closing turbine wicket gates or impulse turbine needles is still provided by hydraulic servomotors. Restoring cables and position sensing may still be accomplished by means of cables which can stretch with time. Stretching is most likely during the initial years of operation, as the cables change from their condition as manufactured to their length under constant tension. Those changes can be noticed and the cable lengths adjusted, if someone has compared the power outputs at a given head and gate opening after intervals of time.

Trash accumulations on the trash racks may not cause output to drop noticeably until there has been considerable obstruction of the racks. But someone must first be aware that there is a difference in output.

Sensing high temperatures in the oil and in the metal of bearings and in the windings of generator stators may often be too late to prevent damage to the equipment. By the time the bearing temperature rises to above the alarm level, the damage may have already occurred. In the stator winding, an increment of deterioration may have already been added to previous deterioration. However, sensing and activation of high-temperature alarms may help prevent increased damage, just as an overcurrent relay may isolate the equipment and prevent its affecting the remainder of the electrical system.

For large turbine-generator units, modern sensing equipment can be economically employed for accurately determining horizontal and vertical displacements of the shaft while it is still rotating (the runout of the shaft). This sensing equipment is based on monitoring the strength of the electromagnetic field between the rotating shaft and the sensing head. Permanently installed sensors can only be justified for large units. In their absence, dial indicator readings should be taken frequently early in the life of the equipment. When turbines were more massive per horsepower output, there was less risk of runout varying with age. Some large horizontal turbine-generating units originally with runouts the order of 0.025 mm still have the same runouts after 30 years without adjusting or dismantling the bearings. Large vertical units are more difficult to align and balance than horizontal units. The cost of equipment for sensing runout of large vertical units may be repaid initially by reducing the time needed for the original alignment and balancing.

Other modern sensing equipment utilizing electronics and having good reliability are now available at a moderate cost. Pressure-sensitive transducers are commonly employed at hydropower plants for monitoring pressures and water levels. Signal impulses from electronic equipment are readily transmitted to data analyz-

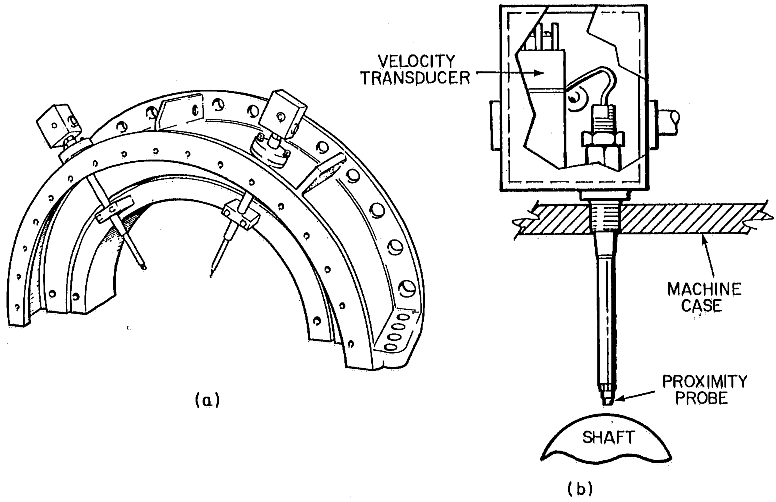

FIGURE 10.2 Dual probes for combined velocity transducer and eddy current proximity transducers for shaft absolute vibration and runout measurement. *(Courtesy Bently Nevada Corporation, Minden, Nevada.)*

ers and computers for analysis and storage. One kind of equipment available at this time is shown in Fig. 10.2

Fire Prevention

In 1987 the National Fire Protection Association (NFPA) adopted the "Recommended Practice for Fire Protection for Hydroelectric Generating Plants," (NFPA 851). The document provides guidance for those charged with the design, construction, and operation of hydropower plants. The recommendations are primarily to safeguard physical property and continuity of power production. Application of the recommendations will also enhance safety of site personnel.

Fires are not as infrequent at hydropower plants as one might expect. While developing NFPA 851, the committee collected reports of fires at many plants, ranging in extent of damage from minor nuisance to complete shutdown of a giant plant. Fire is a hazard wherever combustible materials exist. The combustibles at a hydropower plant are oil, wood, coatings of cables, and fuels. The sources of ignition are electrical faults, welding and gas-cutting torches, sparks, and high temperature.

A principal problem fighting a fire at a hydropower plant is the physical nature of many plants, where much of the volume is below ground level. Below-ground spaces must be accessible and ventilated. Smoke and products of combustion can so obscure the fire that people do not even know where the burning material is. Plant maintenance people should assure that ventilation equipment is functioning, even if the plant is small, unattended, and remote-operated.

Fire fighting and fire prevention in underground power plants and in underground galleries and tunnels of all plants require thoughtful planning. At large plants, a fire brigade should be organized and trained. The maintenance crews should assure that water-storage tanks be kept full and that piping and pumping systems for refilling the storage tanks and for distributing water to the station and yard mains are functioning adequately. At small plants, the owner may contract with local fire-fighting entities. Very isolated plants may not be accessible. Information that there has been a fire is the least that should be provided at an isolated plant.

The author has known an important plant whose carbon dioxide cylinders were empty at the time of a fire. They had been discharged while fighting a previous fire and had not been refilled. That is an example of the need for scheduled and for active concern for providing fire-fighting equipment at a hydropower plant.

Since carbon dioxide, smoke, and products of combustion are special hazards to human health and life, respirators should be available for use.

Nondestructive Examination

Nondestructive examination (NDE) of metal structural and machine parts and piping is extensively used during and after fabrication and erection. It is also a useful aid for maintaining and repairing structures and equipment and piping at hydropower plants. Large organizations will have their own staff and equipment for NDE. Smaller entities contract the work to other firms. There are standards for performing an NDE of materials and there are standards for evaluating the results of such testing. In the United States, those are the American Society of Mechanical Engineers' *Boiler and Pressure Vessel Code* [3] and the various detailed standards of the American Society for Testing and Materials (ASTM).

The skill and experience of the people doing the examination will determine the value obtained from the effort. Just as welders should be qualified by tests of their performance, people doing an NDE should also be qualified.

After a hydropower plant has been built and operating, there will be occasions when there is obvious damage or cracking or leaking. Before making the repairs, and in some instances before deciding whether repairs or replacements are needed, the material should be examined. Only then can the extent of damage and necessary repairs be determined. Often the cost of services of the skilled professionals and technicians exceeds the cost for using the test equipment and materials. But a relatively inexpensive examination may cost more than the most thorough and expensive examination, if the results are inconclusive and the equipment continues out of service. The methods available for NDE are listed below in order of likely cost:

- Visual examination
- Percussion with a hammer or sledge
- Liquid penetrant examination
- Magnetic particle examination
- Ultrasonic examination
- Radiographic examination

Visual Examination

Visual examination by an experienced person is the first NDE. There are reference photographs available showing the types of acceptable surface irregularities for castings [4]. Changes in surface smoothness and cracks can best be detected by scratching the edge of the fingernail along the surface. Light held at varying angles will sometimes disclose differences in color and texture. A surface damaged by heating will be discolored and the areal extent of that damage can be noted. Damage from electric arcing will be obvious. Pitting from cavitation can often be distinguished from erosion by solid particles.

Percussion with a Hammer or Sledge

When there is a possibility that cracks exist internally in a structure, it is often useful to percuss with a hammer or a sledge. If there are several identical structures, for example, wicket gates of a turbine or buckets of a turbine runner, the parts should all ring sharp and clear when struck. A difference in pitch between supposedly similar pieces may be a sign that there are internal voids or cracks, which may progress to breaking and failure. The piece with the cracks and/or voids will have a duller sound when struck—a thunk rather than a ping. Like pieces should be struck in like places with as nearly similar force as possible.

Liquid Penetrant

Examination with liquid penetrant is often useful for finding surface discontinuities on metal. The liquid is rubbed onto the surface, after suitable preparation, and then examined under certain lighting. Linear surface cracks are located by this means. Fluorescing dyes are often used as penetrants. The procedures for this kind of NDE are formalized in the Boiler and Pressure Vessel Code [5]. Reference standards exist for what are acceptable and unacceptable surface discontinuities.

Magnetic Particle

Examination by sprinkling metal particles on a surface and then applying a magnetic field to align the particles is useful for locating linear discontinuities, shrinkage, inclusions of foreign materials, porosity, and chills and chaplets in a casting. Surface discontinuities are easier to evaluate by this means than discontinuities deep within the metal. Standard reference photographs are available for comparison [6].

Ultrasonic

Ultrasonic examination of castings and forgings is useful for determining the condition of pieces suspected of fatigue damage or cracks, based on other indications such as change of dynamic balance and runout [7–9]. The examination should take into consideration the behavior of reflections caused by moving the search unit, the repeatability or verification of reflector indications, and the relationship between echo amplitude and indicated dimensions with more than one test frequency, beam angle, and mode. Ultrasonic testing can be successful without dismantling the shafts or the machine parts. This technique is suitable for examining old welds and welds after making repairs.

Radiographic

In the United States, radiographic (x-ray) examination of metals is conducted in accordance with the ASME Boiler and Pressure Vessel Code [10]. Standard reference radiographs are given in ASTM Standards [10] for metal up to 4½ in (11 cm) thick. Permanent records are kept in the form of developed film. Iridium-192 is the emitting material used for examining metals from ¼ to 2 in (0.6 to 5 cm) thickness. Cobalt-60 is the emitting material used for examining metal thicker than 2 in (5 cm).

Radiographic examination is the most expensive, but the most conclusive means for finding defects such as gas porosity, slag, shrinkage of all types, cracks, hot tears, and inclusions in metal. For many crucial applications, especially after repairing important pressure vessels, high-pressure piping, or rehabilitating turbine runners of large thicknesses and dimensions, radiographic examination is the only means available for assuring that the necessary quality of repair or soundness of the material exists.

10.5 TURBINE INSPECTION AND MAINTENANCE

Many different kinds of parts having different maintenance requirements make up a hydraulic turbine. Many units must be dewatered before they can be entered. Other turbines set above tailwater level may only require closing the intake gates and letting the water drain to a tailwater level lower than the rotating parts. Some parts, such as bearings and hydraulic servomotors, will usually be in the dry and accessible locations. Other parts, such as rotating parts, shaft gland seals, parts of the distributor (wicket gate positioning mechanism) may be accessible only after draining the water conductors and pumping remaining water to tailwater after closing and sealing draft tube gates or bulkheads. Impulse turbines are set above the tailwater level so that dewatering is not a problem for inspecting and maintaining the unit.

Inspections and maintenance work must be planned in advance. Dispatchers and operators release the unit from service. The equipment is "tagged" as being out of service for safety of the people who will be close to and in the machine. Large plants and systems have developed necessarily elaborate clearance routines covering safety precautions. The seemingly simplest omissions can cause great damage and increase downtime. A hatch cover left unbolted after maintenance was performed has been known to flood a major hydropower plant and put the entire plant out of service indefinitely while the generators were cleaned and coils were dried. Do not open a draft tube access door without first verifying that there is no water on the other side. The status of electrical apparatus and cables must be verified before the parts can be touched. When cleaning up after a fire in which carbon dioxide has settled into low regions in a hydropower plant, the air must first be tested to assure that the carbon dioxide has already been removed.

The machines and water passages must be prepared before inspection. At many plants, this involves steam-cleaning the walls of the turbine pit before inspection and maintenance. Since the turbine pit or water passages are below floor level, fans and piping for ventilation are required. Platforms must be built or installed so the workers inspecting and repairing the turbine parts can safely enter and work. Rope and cable rigging, ladders, hoisting devices (usually chain falls for handling small tools and materials), adequate lighting, and communications equipment must be installed in preparation for major inspections and maintenance. It may be necessary to heat the space inside the water conductors, particularly when the water might otherwise freeze. Water trapped inside hollows of wicket gates, for example, will expand on freezing after flow through the turbine ceases, and in such cases may rupture the metal of the gates.

As part of preparing for the inspection, the water passages must be cleaned and any debris should be removed. Drain cover gratings may have to be cleared so that they will allow water leaking past the closure gates and bulkheads to drain from the space. The preferred power supply for working near water is compressed air. Compressed-air chipping tools, wrenches, and hammers and the air-supply hoses must be provided. If electricity is needed, a 32-V supply is used, for safety. Weld repairs in situ are sometimes unavoidable. In those cases, only oxyacetylene or other gas welding should be permitted inside an enclosed, humid space where water may be present.

Some of the items associated with the hydraulic turbine which should be inspected are noted on the example schedule given earlier in Table 10.1. That list did not include the nozzles, needles (spears), jet deflectors, and linkages of impulse turbine units. Nor did it include items associated with the Kaplan oilhead and blade-operating servomotor and blade trunnion seals of adjustable-blade propeller turbines. Blade trunnion bearings and the servomotor mechanisms and blade-operating levers and links are not normally inspected on a regular basis. Inspection and maintenance of those items would require complete disassembly of the turbine. On some small turbines and some larger ones experiencing recurring problems with blade trunnion seals, the seals may be designed differently and the blades bolted to the movable trunnions.

Rather than using wicket gates, flow control on some turbines is accomplished with cylinder gates and, of course, that gate-operating mechanism should be inspected and maintained.

Normal maintenance of a hydraulic turbine includes the following items:

- Grease lubrication
- Adjusting and replacing packings
- Assuring that grooves and channels for lubrication are clear
- Renewing or repairing packing rings of hydraulic cylinders
- Removing, recalibrating, and reinstalling gauges and thermometers
- Flushing out and relubricating oil bearings
- Checking and adjusting all springs for proper tension
- Checking and adjusting, as necessary, restoring cables in the governor
- Checking and adjusting pointers for accurate indication
- Flushing foreign matter from strainer screens
- Adjusting backlash of linkages to make up for wear and vibration
- Refitting wicket gate stem bushings and shear pins, as necessary
- Cleaning bacterial growth and deposits from surfaces (this may be done with hoes and scrapers, rather than by sand blasting)

All oils, greases, packing materials, bushing and bearing materials, clearances, tolerances, tensions, runouts, distributor or impulse turbine operating linkage movements, air-valve motion, system pressures and pressure ranges, adjustable-blade motions, and servomotor motions should be restored to their original condition as recommended by the manufacturer, unless there is documented experience for that machine that the recommendations should be altered. Older units may have originally had materials which are no longer readily available, such as lignum vitae for blocks of adjustable, water-lubricated bearings. In such cases, other materials which have been proven applicable elsewhere should be tried. What works well at one machine with one quality of water may not work well at another machine.

A daily diary should be kept detailing the maintenance work done, the labor hours by skill, and the materials used, all with adequate comments. That information for several previous maintenance outages and inspections should be reviewed before planning the next maintenance on the turbine. Later the plant engineer may analyze the information before deciding whether repairs or replacements are preferable.

10.6 WELD REPAIRS

Steel Weld Repairs

Welded connection of pressure vessels, such as penstocks, storage tanks, turbine spiral cases, and draft tubes has become very common in the past 50 years. One reason is that fewer parts are now being made by casting. Components of hydropower plant machinery and pressure vessels which are too large to cast or to ship in one piece are often shipped as pieces prepared at the factory for field weld assembly. The technology of fusion welding and subsequent heat-treating has advanced so that even large penstock wye branches are welded in the field and stress-relieved thereafter in place. Stress-relieving is accomplished by creating an enclosure completely surrounding the work to be stress-relieved. The volume inside the enclosure is heated gradually, usually with gas flame heaters and sometimes with electric resistance heaters at confined spaces, maintained at the required temperature for the required time, and then cooled gradually.

Steel structural members of modern machinery are often designed as welded, rather than as cast, pieces. Even very large structural components of hydraulic turbine stay rings, runners, bottom rings, etc., are often weld fabrications. Since weldments cannot be unbolted, when the material is steel, cracks, gouges, tears, and other damage must be repaired by welding. Yet many old hydropower plants have cast iron or bronze turbine parts which require special techniques and materials for their repair. Those matters will be discussed in the next section of this chapter.

The ASME Boiler and Pressure Vessel Code [3] is the prime reference for information regarding welding pressure vessels. Various other standards and codes have valuable information concerning weld repairs to structural members. Regardless of the amount of welding to be done, the welder should be fully qualified by observing and testing work samples. In all cases, preparation of the work for welding is essential. Since good weld quality is most easily obtained by welding in a "down-hand" position, the weld repairs should be made in that position if possible. That is why factory welding is often done with positioning machines. All weld repairs should be carefully inspected. If the repair is to a crucial pressure vessel or structural member, nondestructive examination after welding may be required.

There are special considerations when repairing stainless steels by welding. For example, cleaning or testing materials containing chlorides, sulfur, and/or chlorine should not be used on austenitic stainless steel. There are special requirements for welding martensitic stainless steel castings. Wire brushes and grinding wheels for use on stainless steel should not have previously been used on other metals. Selecting the material composition of the weld rod used as filler material is important, since stainless steels of different compositions have widely varying properties. Preheating the base metal to which the fusion material is added and subsequent postweld heat treatment may be required.

Repairing damage from cavitation and erosion on turbine runner blades and buckets is a matter which has been extensively investigated, yet all the answers and processes are not fully understood. It is common experience that after stainless steel weld repairs are made to carbon steel members, the areal extent of the damage increases to beyond where the previous weld repair was made. Even when there has been no change in operation or operating conditions, the damaged area increases. One theory is that some of the carbon in the base metal migrates to the weld fusion zone of the stainless steel, which has a very low carbon content. The loss of carbon weakens the metal just beyond the weld.

The amount of metal removal caused by cavitation should be monitored carefully throughout the period of the manufacturer's cavitation guarantees. The plant engineers and operators presumably have limited the hours of operation at discharge and head combinations which would invalidate the guarantees. In later years, many owners have accepted a certain amount of damage and repair costs as being economical. The income from extra power and energy produced while there is some damage from cavitation may far offset the labor and materials cost of the weld repairs. Damaging a hydraulic turbine runner may ultimately cost much less than the increased downtime or the damage avoided by decreasing the number of thermal cyclings during startups and shutdowns of thermal-electric units. Weld repairs can usually be made while other maintenance is being performed on a dewatered unit and therefore may not extend the hours of downtime.

Metal lost because of cavitation must be replaced. The pitting should not be allowed to progress until it penetrates entirely through the metal. The structural weakening might then be so great that portions of a turbine runner blade or bucket might tear from the remainder. There have been instances when pieces have broken off from buckets of runners of reversible pump-turbines. The broken pieces were flung from the runner at such high peripheral velocity that they sliced through several wicket gates "as if they were made from butter." That particular failure was the consequence of cracking from vibration, rather than weakening by cavitation pitting.

The design of hydraulic turbines for reduced cavitation is discussed in Chap. 4. One factor appropriate to mention here is the surface finish. Roughened and wavy surfaces experience far more damage from cavitation than uniform, smooth surfaces. A damaged, pitted surface itself contributes toward additional damage. The final condition of the repaired area is then a key determinant of how much damage there may be thereafter. Some researchers find that the surface finish is as important a determinant of future damage as the pressure in the fluid surrounding the region of the damage. For those reasons, all weld repairs bordering the path of fluid flow should be shaped to original contours and should be ground to smooth finish, with no weld bead projections.

Repairing and Welding Metal Other than Steel

Many old and new hydraulic turbines have runners and other parts made of cast iron or bronze. A special class of cast iron is called malleable, or ductile, or nodular, or spheroidal cast iron. Those irons have carbon nodules in the matrix outside the crystals of iron or iron-eutectic. Malleable irons can be cast to tensile strength and ductility equaling that of carbon steel, stainless steel, or the high-strength bronze alloys.

Irons have two advantages over steel or stainless steel. First, iron absorbs energy of vibration much better than steel. Second, iron is more readily machineable than steel or stainless steel and requires fewer hours labor and less machining for fabrication or repair. Iron or malleable iron runners will cost less to make and to repair than steel runners. Malleable iron has a third advantage, since it is readily repaired by welding. The temperature while welding must remain in the annealing range and must not rise into the normalizing range [at or above 1455°F (760°C)]. Weld repairs of malleable iron are made with rods of either a tough nickel-manganese bronze or a nickel-manganese-aluminum bronze.

Ordinary gray cast iron is still used for nonrotating parts of turbines. For rotating parts, gray cast irons are no longer in favor because of their low tensile strength and brittleness. There is renewed interest in using malleable iron.

Many turbine runners, particularly small runners, are again being cast from the bronzes. Some bronze alloys have great strength, resistance to erosion, and resistance to attack by sea water. The bronze runners are easily repaired by welding with alloy bronze filler material. The temperatures during welding should be controlled so that the base metal outside the zone of fusion is not altered.

Old turbine parts are often of cast iron having compression strength the order of 50,000 lb/in^2 (35 MPa). It is possible to weld cast iron with nickel filler rod material, but great skill is needed to obtain a repair having good fusion with the base metal. In some instances, an iron structural member outside the water passage can be safely repaired by strapping bars across the crack or opening and riveting or bolting the straps to the base metal. The columns supporting huge punch presses, which suffer shock loading, can be repaired permanently by this means. The technique is sometimes called "metal locking" or "metal stitching." Several firms specialize in this kind of work. Cracked Francis turbine runner blades have been repaired in this manner and they have served many years thereafter.

A word of caution is appropriate regarding assessing the extent of damage by rusting. Rust occupies a volume from 10 to 30 times the volume of the metal which has oxidized. A layer of rust that is 0.2 in (5 mm) thick may be the oxidation product replacing a layer of metal only 0.006 to 0.020 in (0.15 to 0.5 mm) thick; thus the reduction of strength may be very small. The rust layer coating a cast iron surface protects the metal from further oxidation and is often thinner than the layer coating steel plates and bolts and nuts. If the rust is not causing other problems, scraping the rust away only exposes fresh metal to oxidation. New rust would form and the loss of metal thickness would be doubled.

Nonmetallic Repairs and Coatings

Several materials for coating and filling places where metal has been lost by erosion, corrosion, or cavitation have been developed during the past few decades. Many of these are epoxy-based and can be applied by plant maintenance people without their having extensive previous experience. Some of the tougher materials are complex ceramic-steel amalgams applied for rebuilding and resurfacing metal. These materials are not inexpensive, but there is little risk of further damaging the base metal by applying them, whereas there is risk if welding is not done properly. Adhesion between the coating and the base metal is very important for this kind of repair. The surface to be coated should be prepared in accordance with the recommendations of the manufacturer of the repair material.

Various compounds, each suitable for a specific repair, are marketed under trade names. The manufacturer's instructions and advice regarding selection and use of the repair material should always be obtained before making a repair.

10.7 GENERATOR INSPECTION, MAINTENANCE, AND TESTS

At appropriate intervals and at times when the generator can be taken from operation, major inspections and tests should be made for each generator. It is necessary to remove the rotor so that the rotor and the stator can be inspected. That is more quickly done on a large vertical-shaft machine than on a large horizontal-shaft machine. On a unit with the generator housed inside a bulb in the water passage, removing the generator rotor may be a major undertaking involving disconnecting much of the piping and electrical cable runs from the rest of the power plant to the generator.

The following should be done:

- Clean and inspect the rotor. Check the insulation for any mechanical damage. Check for movement or shifting of the rotor coils. Check for movement or looseness of the rotor poles. Check for tightness of the bolts or the taper keys securing the poles to the rotor rim laminations. Check the rotor pole winding connections.
- Clean and inspect the stator. Check for movement and/or looseness of the stator slot wedges. If any are missing, replace them. If wedges are loose, wedge them tight. Check the condition of the lamination compression bolts. If any are cracked, replace them. Make sure they are tight. Make sure the compression-bolt locking devices are in good condition.
- Check the condition of the insulation on the stator winding end turns. The end turns are curved beams which deflect. If epoxy-sealed insulation is used, the insulation is more rigid than the older asphaltic insulation and is more susceptible to cracking in the middle of the end turn, where the winding is unsupported after passing beyond the slots. The curved beam experiences substantial bending moment and deflection. Epoxy-coated stator coils may be loose in the slots, unless particular attention is paid to assure secure wedging, and may vibrate.
- Check for stator corona discharge. Check for slot corona discharge. Check for separation of stator winding tapes.
- If the unit has a rotating exciter, check the exciter slip ring insulation. Inspect and adjust the exciter brushes.
- Make megger tests in accordance with the manufacturer's instructions. Test for

polarization index. Test for absorption ratio. Make dc high potential tests. Make slot discharge tests. Make corona tests. Make turn-to-turn tests. Compare all the test results with the manufacturer's recommended allowable limits and with results of all previous tests.

- Measure and record rotor-to-stator air gap clearances with the rotor turned at 90° intervals. Compare the clearances with historical clearances and analyze the causes of any changes.
- If the generator has brakes, check that they release properly. Check brake shoes for wear and adjust as necessary. Check the condition and tightness of fan blades on the rotor.
- Make a complete inspection to be sure no tools, bolts, rods, or other materials have been left inside the generator, both before reinstalling the rotor and after the rotor is installed, but before watering the turbine. A loose nut or bolt striking an object at 150 mi/h rotor peripheral velocity can cause great damage.

10.8 MAINTENANCE OF TRASH RACKS

Trash racks exist at hydropower plants to prevent floating and submerged debris, logs, and ice from entering a hydraulic turbine and damaging the machine. The trash racks are designed to pass objects smaller than those which would damage the unit. If the racks are obstructed, there will be loss of head. There may also be a hydrostatic loading greater than the amount the racks and their supports can withstand. Some rivers and some plants suffer frequent buildup of accumulations of trash and/or ice. Other rivers and plants are relatively free of accumulations of materials obstructing the racks.

Maintaining the racks free from obstruction can be as simple as excavating the banks of the upstream approach so that the hydraulics are changed and less trash is brought into the plant forebay. That kind of maintenance should only be undertaken after careful engineering analysis, and possibly a hydraulic model study. At some plants, such as Safe Harbor on the Susquehanna River where there is susceptibility to ice as well as trash, a curtain wall, called a "skimmer wall" extends along the river. The wall prevents both floating and submerged materials from reaching the trash racks. A skimmer wall must completely span the approach to the power intake. If the wall does not extend the entire length, the trash and ice will first pass along the upstream face of the wall and then turn sharply around the wall, after which it will be carried to the intakes by the current of water in the approach channel.

A variation of the skimmer, which provides protection and thereby reduces maintenance clearing the racks, prevents only floating objects from reaching the racks. That is a log boom. Log booms are relatively inexpensive. They must be securely anchored if they are not to be themselves a maintenance problem. The log boom is usually a floating raft, sometimes with empty oil drums as floats. The log boom may also protect against floating ice.

Accumulation of ice and the physics of its formation under varying conditions are presently the subject of extensive research. At water temperatures slightly below (32°F) (0°C), a particular form of ice called "frazzle ice" can form in the water column, depending upon the river hydraulics. There are many instances of trash racks at hydropower plant intakes becoming completely blocked. At some plants small piping and hose systems are installed and air at low pressure is bubbled alongside the trash racks. The motion of the bubbles as they rise to the surface prevents the ice from forming and clinging to the metal surfaces of the trash racks. At other plants, electric resistance heaters are used to prevent ice from forming at the surface of the metal.

It is common that trash racks be designed for the loading resulting from 25 percent obstruction. If the inflow velocity is at or below approximately 3 ft/s (1 m/s) there is little loss of head at lower proportional obstruction. There is the danger that the racks and their supports may collapse if obstruction is much greater, so the trash racks should be kept clear. Clearing ice has been mentioned above. Clearing floating and submerged trash is a different matter.

The simplest, yet often the most unsatisfactory, means for clearing trash from racks is raking by hand. Hand raking is, of course, labor-intensive. It can also be a 24-h maintenance problem during the spring runoff on some rivers. Various mechanical screens and rakes have been used with success. Often rakes will handle floating trash but will not remove submerged logs, known as "deadheads," from the racks. On the Merced River in Yosemite Park, runoff carries leaves to the intake screens in the autumn. If the screens do not keep them out, the leaves clog the water passages in the small turbines. Traveling screens were installed, similar to those used at the intakes to steam-electric plant condensers. Traveling screens may require frequent maintenance to assure that water spray lines stripping the leaves from the screens are functioning. Since the screens enter and leave the water, they are subject to corrosion. Traveling screens are now being installed at intakes of hydropower plants as a means of preventing migrant fish from passing through the turbines. Materials relatively resistant to corrosion should be used. The screens move constantly, so gears and cogs in the power train may require frequent inspection and maintenance.

Mechanical rakes, with or without carts for holding the raked trash, exist in many designs. Older designs in sizes spanning only a few feet to 20 ft (7 m) and greater are raised and lowered with electric motor–operated hoist cable drums. The steel cables are attached to the rakes. The teeth of the rakes are raised away from the rack bars while the rake is being lowered and then rotate, as the rake is first raised, to intrude between the rack bars. An end friction clutch on the axle of the rake wheels is used to accomplish the rotation. Few mechanical rakes are sufficiently robust to hold and lift heavy submerged logs. Grapples lowered and raised from hoists traveling on rails on the intake deck or from trucks may help remove deadheads from the racks.

At new, small hydropower plants and at some larger plants, rakes are rigidly attached to a boom which is the piston rod of a hydraulic cylinder. The rake-hoist is clamped around a monorail and can move laterally, from one set of trash racks to the next. The rake can be programmed to clean one set of racks and then move on to the next automatically, at scheduled time intervals. Such rakes of two different manufacturers are depicted in Figs. 10.3 and 10.4.

Disposing of the trash and logs removed from the trash racks may be an expensive item of plant maintenance. In the United States, the EPA rules now specify that hydropower plants are a possible point source of pollution. Therefore, it is possible that at newly licensed plants, materials removed from the water may not be returned to the water downstream from the power plant. In the past, this was normally handled by placing the trash in a sluiceway and using the flowing water to return the trash to the river.

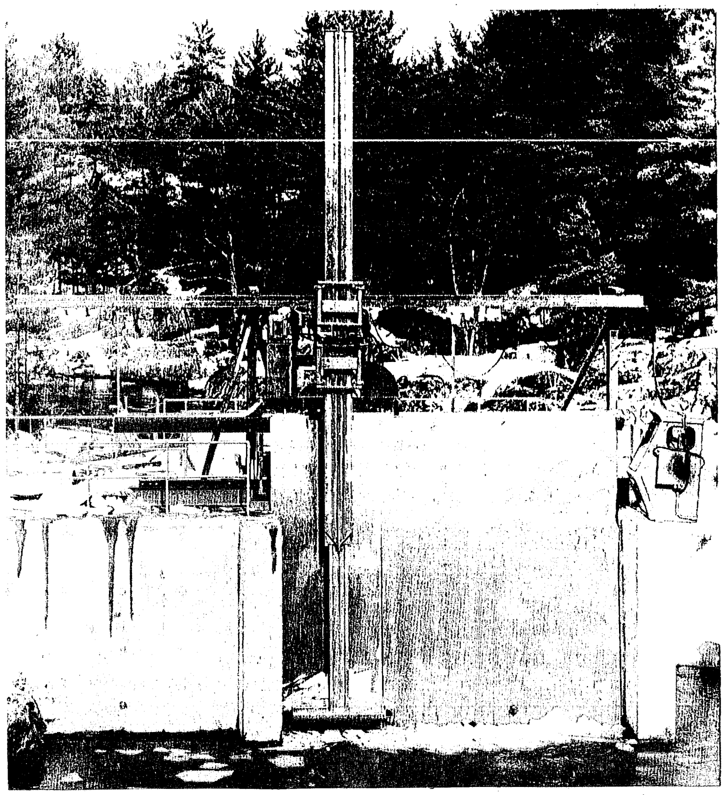

FIGURE 10.3 An automatic trash-rack rake in operation at a hydroelectric plant intake. (*Courtesy Acme Engineering Company, Cordele, Georgia.*)

10.9 REALIGNMENT AND BALANCING

Alignment of the system of generator-shafts-turbine and the balance of the system should be checked during or after each major overhaul or maintenance outage of a hydraulic turbine-generator unit. Realignment and rebalancing are essential if there has been addition or removal of metal to the rotating parts. If the parts have been removed or uncoupled for any reason, reassembly may not restore the original correct alignment and balance.

Alignment

Horizontal-shaft units are easier to align than vertical-shaft units. One problem encountered in aligning small, horizontal Francis turbines is matching the inlet venting (opening) to the turbine runner with the position of the flow passage through the stay ring and wicket gates. Axial misalignment may cause solids in the water to be forced through the upper turbine runner crown seal gaps and then erode metal from the crown and the head cover as they whirl with rotation of the

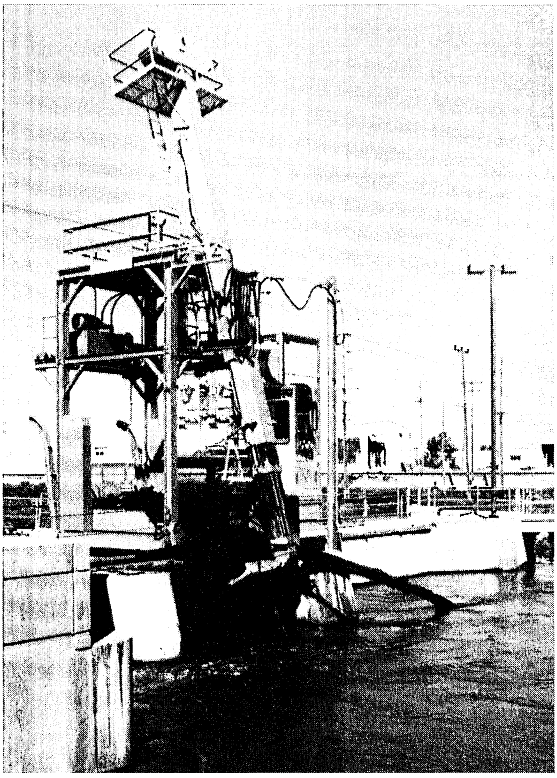

FIGURE 10.4 Automatic trash-rack rake in operation at a hydroelectric plant intake. (*Courtesy L. H. Berry, Inc., Conway, New Hampshire.*)

runner. If there are not both upper and lower seal-wearing ring gaps at the runner bottom band, axial misalignment will reduce the length of the seal gaps. Leakage of water past the runner will increase and efficiency will decrease. There have been instances of destructive vibration when the upper seals of a turbine runner bottom band opened and/or when they were not provided.

Misaligning the impulse turbine jet to the runner will cause the water to impact improperly on the bucket. Efficiency will be reduced and the turbine buckets and bearings may wear unevenly and rapidly. Axial misalignment of a propeller turbine runner set into a spherical machine throat ring will change the blade tip-to-throat clearances. There may be clearance while the turbine is assembled and not rotating. However, the centrifugal force when rotating stretches the metal. Clearance when rotating at synchronous speed will be less than when measured before the runner rotates at synchronous speed.

Some small horizontal turbine runners are cantilevered from a bearing outside the water passage, as shown in Fig. 10.5, a section through a tubular unit. If the length of the cantilever is substantial, the shaft will deflect. There have been cases of turbine runner grinding and/or galling metal from the lower portion of the

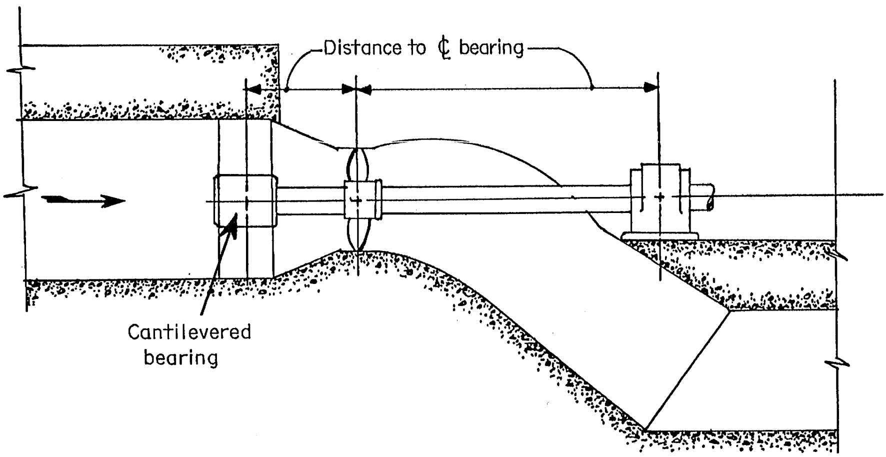

FIGURE 10.5 Turbine runner with shaft having a long cantilever from the guide bearing.

throat. Lengthening the bearing and/or stiffening the shaft will eliminate this problem.

It is sometimes said that trueness of alignment is more important than precise horizontal level or vertical plumb. That is correct in a sense. But a unit which is not level or plumb relies on pressure of the oil in the bearings to prevent damage to the bearings. If the oil does not serve the purpose, then the bearing clearances are increased either deliberately or by wear. The turbine seal clearances also increase, causing increased leakage and decreased efficiency.

Horizontal alignment is simplified by the bearings supporting the shaft and the cantilevered parts. Dial indicators supported on rigid posts or bases are often used to check runouts at various axial positions. Wires can be strung from one end of the system to the other. After the wire is taut, the runouts can be checked with an inside micrometer connected to a battery and bell or buzzer. One nub of the micrometer is pressed to the shaft and the other nub of the micrometer is moved outward, swinging in arcs to assure the shortest perpendicular distance is obtained until contact is made with the wire, completing the circuit and sounding the buzzer.

A recent innovation for aligning shafts of many machines, such as turbines, pumps, and compressors, depends on adjusting alignment until an optical laser beam strikes a target at its precise center. A small digital computer is used for analyzing the error of each trial and estimating the amount and direction the shaft should be moved before the next trial. One version of such optical aligning equipment is depicted on Fig. 10.6. That kind of equipment can be used by the maintenance people at the plant with little special training. This helps achieve alignment in a short time.

A laser beam emitter and a prism are fastened, facing each other, to shafts or coupling halves of the machines to be aligned. The beam is reflected back to a receiver opening and a position detector determines the x and y coordinates of the received beam. As the shafts are rotated, parallel dislocation of the received beam is detected as a displacement in the y direction. Angular dislocation is detected as a displacement in the x direction. Such systems can be very accurate if the laser beam is collimated precisely and the position detector is precise.

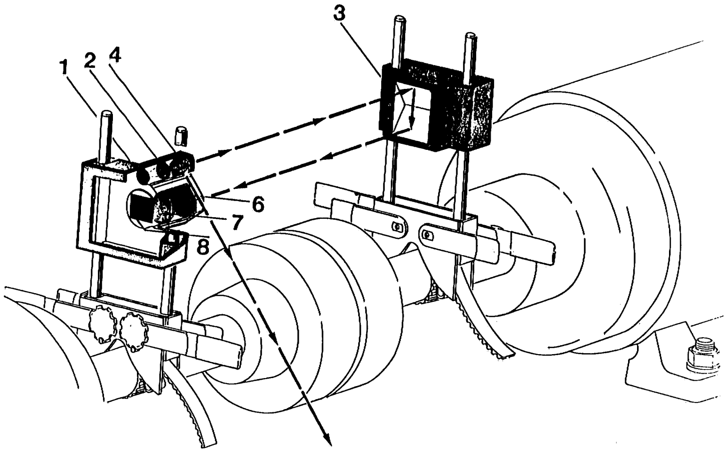

FIGURE 10.6 Alignment of horizontal shafts by laser technique. (*Courtesy Optalign "Shaft Alignment by Lasers," Pruftechnik Dieter Busch & Partner GmbH & Co., Ismaning, W. Germany.*)

Vertical units are more difficult to align than horizontal units because there is an extra degree of freedom of position. Traditionally, large vertical units are realigned the same way they were originally aligned, with plumb wires and micrometers and buzzers. Four plumb wires are usually hung from nonmoving arms of the thrust bearing support bracket. Finned weights (plumb bobs) of 20 lb (9 kg) or more are attached to the wires and the bobs are immersed in cans which are filled with oil. Such bobs quickly come to rest in the oil in the vertical plumb. An inside micrometer is secured to a rod, usually of oak or ash, stiff enough so it will not bend. The battery and wiring and buzzer are added so that an electrical circuit will be completed when a metal nub embedded in the end touching the shaft and the nub of the micrometer are both touching metal. For a large unit, the wood rod may be several feet long. It is then necessary that one person hold the metal nub to the surface of the shaft while a second person swings the other end with the micrometer to find the shortest perpendicular distance to the wire.

Differences in the extension of the inside micrometer nub from one quadrant to other quadrants around the shaft are the differences in distance, hence in centering, at the given elevation. Measurements and centering must be accomplished at several different elevations to assure the shaft system is both plumb and centered.

Alignment can be accomplished with patience and with accurately recorded data. Clear explanations of how the centering and alignment data are recorded and analyzed are given in the U.S. Bureau of Reclamation Power Operation and Maintenance Bulletin No. 2, "Alignment of Vertical Shaft Hydro Units" [11] and in [12], which is a translation from the Russian original and is available from the U.S. Department of Commerce, National Technical Information Service.

After the rotating parts of the turbine and generator have been separately centered in place, they are recoupled. Recentering and realigning the coupled turbine and generator rotating parts requires adjusting the position of the axis of the combined rotating parts within the circular openings of the generator stator, the thrust and guide bearings, the head cover and bottom and throat rings of the turbine, and the draft tube. On many old machines, there are two separate turbine runners on separate shaft pieces of the assembled unit. At other old hydropower plants, there may be three or more turbines on the same shaft assembly system. Obviously, multiple-turbine runners and additional shaft pieces complicate the problem of concentric centering of all of the rotating parts and increases the time needed for reassembling, realigning, and recentering the unit.

Recentering and realigning a hydroelectric unit can be further complicated if there has been movement, either by shifting position or by forces causing the embedded or nonrotating parts to become out-of-round. Pennsylvania Power & Light Company encountered that further complication when replacing, with its own personnel, shafts and runners of the double-runner units. The embedded generator stator support rings had become increasingly out-of-round over the 75 years since initial installation, as a consequence of alkaline aggregate reaction growth of the concrete of the powerhouse substructure. Previous attempts at re-aligning the units to decrease wear and opening of the runner seal gaps, which decreases volumetric efficiency of the turbine, without replacing Francis runner band seal rings and disassembling the entire unit so the bottom ring-wearing rings could be machined in place to round, had not been successful.

The axis of the shafts and rotor-runner system of a generating unit should be a straight line. The axis of the rotating parts should coincide with the geometric axis of the nonrotating parts of the unit. Good centering and alignment contribute toward quiet and reliable operation of a generating unit.

Absolutely perfect centering and alignment is unattainable. Though the manufacturing tolerances for the mating surfaces at coupling joints between the shafts and the generator rotor and the turbine runner are very small, there will be some misalignment between the joined parts. The axis of the shafts also will not be precisely the axis about which the rotor and runner rotate. Though the tolerances allow little error, the thrust faces of the shaft which rub on the thrust bearing will not be precisely perpendicular to the axis of the shafts. When recentering and realigning a unit, the maintenance crew should obtain final positions and alignment within close standards. In the United States, the applicable standards are those of Ref. 13.

Radial displacements, or runouts, of any part of the rotating system must not exceed the clearance gaps between the rotating and the nonrotating parts. Those radial gaps must, therefore, be measured during the centering and aligning of the unit. But gaps measured while the generating unit is not rotating are not the gaps which will exist while rotating. Allowance must also be made for deflections of the shafts system and for stretching of the parts and their coupling joints when rotating at maximum runaway speed.

Balancing

After work has been done on any element of the generator-shafts-turbine system, it is essential that runouts be checked and that the unit be rebalanced. A static (not rotating at synchronous speed) balance may not reveal failure to be in dynamic balance. Equipment is now available for detecting vibrations and accelerations in the system being balanced and analyzing what should be attempted as the next trial. Balance is achieved by adding weights to the unit. Weights may be added at various places in the generator rotor or the turbine. The balancing weights should be out of the path of fluid flow. Balance weights must be securely fastened to the member to which they are attached. Though field balancing is not an esoteric art, a useful discussion and explanation would be too lengthy for inclusion in this chapter. The reader is directed to Ref. 14 for a suitable explanation.

The rotating inertia of the unit is WR^2, or GD^2. That is, weight times radius of gyration squared or one-fourth of weight times diameter at radius of gyration squared. Units are pounds per foot squared in U.S. customary units and kilograms per meter squared in metric units. An imbalance in the unit must be offset by weight placed at radii such that the WR^2 of the net weight added will counteract the imbalance WR^2.

Generators of medium- and high-head turbine-generator units are relatively longer in the axial direction than generators of low-head units. Motor-generators of reversible Francis pump-turbines are relatively long, since the head is usually high. For such machines, balance weights added at only one elevation or axial distance may not suffice for obtaining a dynamic balance. Weights should be added at both upper and lower portions of the rotor.

Balance weights are often added to the upper surface of the crown of a Francis turbine runner and to the bottom band of the runner. Holes can be drilled into the crown or into the bottom band and parallel to the shaft axis. Weights made of lead alloy (weighing more per given volume than the metal which was removed) can be secured in the holes and the holes can then be capped with steel. The caps must be ground true.

10.10 VIBRATION

Vibration of a hydraulic turbine-generating unit will eventually damage the machine and reduce its useful life. Analyzing the frequency of the vibration together with the position of the sensing and considering the physics of the rotating system can be a powerful tool for diagnosing impending failure. Repairs can then be made, or operation can be adjusted to either delay future troubles or eliminate them.

The frequency of vibration is often a telltale of the origin of the vibration. The listing below is from a report by the author [15] analyzing vibration of a large reversible pump-turbine.

Type	Item	Cycles per revolution
Natural	Natural frequency of the system	0.16–0.25
Mechanical	Partial rub and impact against runner seals	0.48–0.67
Mechanical	Runout at the shaft-to-shaft coupling, generating	0.52–0.60
	Best gate, pumping	0.67
	In air at 360 r/min	2.7
Mechanical	Runout at the turbine guide bearing	Same as at the coupling when operating at 100 MW but not necessarily the same when operating at 50 MW
Mechanical	Underlying wave of variations of pressure under the head cover	0.50 to 0.58

Type	Item	Cycles per revolution
Mechanical	Runout above the upper generator guide bearing	1.00
Hydraulic	Time for pressure disturbance traveling to and from the upper reservoir	0.087 approx.
Hydraulic	Draft tube surges	0.30 to 0.38
Hydraulic	Rotation of water above the runner crown	0.50 approx.
Hydraulic	Resonance in the balancing lines	30–45
Hydraulic	Vortex shedding from the runner vanes	60–170
Hydraulic	Other wave reflections	20 and greater

Some of those frequencies would differ for various machines. For example, the natural frequency of the system is specific to the plant, depending on the mass and the geometry of the rotating elements. The time for a pressure disturbance to travel to and from the reservoir depends on the distance, the diameters, and the construction of the water conductors. It is evident that mechanical problems will not have vibration frequencies of many cycles per revolution.

10.11 CONCLUSIONS

Maintaining and operating a hydroelectric plant so it will operate efficiently and economically for a great many years is a complex matter. Experience and detailed knowledge are required in many technical fields. Prudence, wisdom, and diligent attention to detail are required of all people involved in making a hydroelectric plant continue doing what it was intended to do, produce electricity.

Carefully documented maintenance and operating records should be analyzed at regular intervals. The most recent condition and performance of each item of equipment and of each structure should be compared with the condition and performance at previous times. The past history should be considered before modifying, repairing, or replacing a given machine or structure. Funds spent for thorough investigations and analyses by engineers and other technical experts before deciding whether equipment or structures should be modified, repaired, or replaced, and how it should best be done, usually repay the owner severalfold in more enduring, less expensive, and greater revenue-producing maintenance and operation.

No attempt has been made in this chapter to furnish detailed information for any of the matters discussed. An entire library would not suffice. The references will lead the interested reader to further information and references. The bibliography is, necessarily, condensed. Only English-language references and bibliography items are included. An extensive literature in other languages may be accessible to engineers in other countries.

The engineer should not hesitate to discuss technical problems with people in other organizations. After all, as the overworked manager of a tin mine in Bolivia once explained his willingness to go out of his way to help me, "All over the world we are members of the same fraternity of people who are trying to build and maintain, to ease the lot of human beings."

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CHAPTER 11
SYSTEM PLANNING AND OPERATION

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11.1 SYSTEMS APPROACH

Hydropower units often operate as part of a larger system. In a multiunit hydroplant, they share the same gross head. In a river basin, they interact with each other by their storage and release activities and in a power system they contribute, together with other types of units, to meet the common electrical system load. The systems approach is the analysis of individual units or projects in the context of the system of which they are a part. It is based on a mathematical description of all relationships between the variables of the system. The solution of the resulting equation set for specified variables provides information on the performance of the system under the described conditions. Experimenting with such a mathematical model of the system is called simulation. The information on the most likely system performance gained by such simulation could never be obtained in the field, not even over the entire lifetime of the real system. It can capture the synergistic effects and possibly reveal new options for system management. Therefore, the systems approach is an invaluable tool for a realistic analysis of contemplated operational and planning decisions.

In a system with many power units, there are usually many operational options. Also, an operational decision at one plant usually has consequences for many other plants, in the time frame considered as well as in the future. No human operator is able to tackle the multivariate, time-sequenced problems that must be solved to arrive at an optimal schedule. Therefore, the application opportunities for the systems approach range from large and complex hydrosystems, such as the Columbia River system shown in Fig. 11.1 [1], over smaller systems, down to individual multiunit plants. Even modest efficiency improvements can pay off investments in methodology and hardware within a very short time many times over, especially when considering the investment cost for capacity saved.

Optimization

The simulation of processes that are operator-driven, such as power system operation, requires the inclusion of a decision mechanism that steers the process in

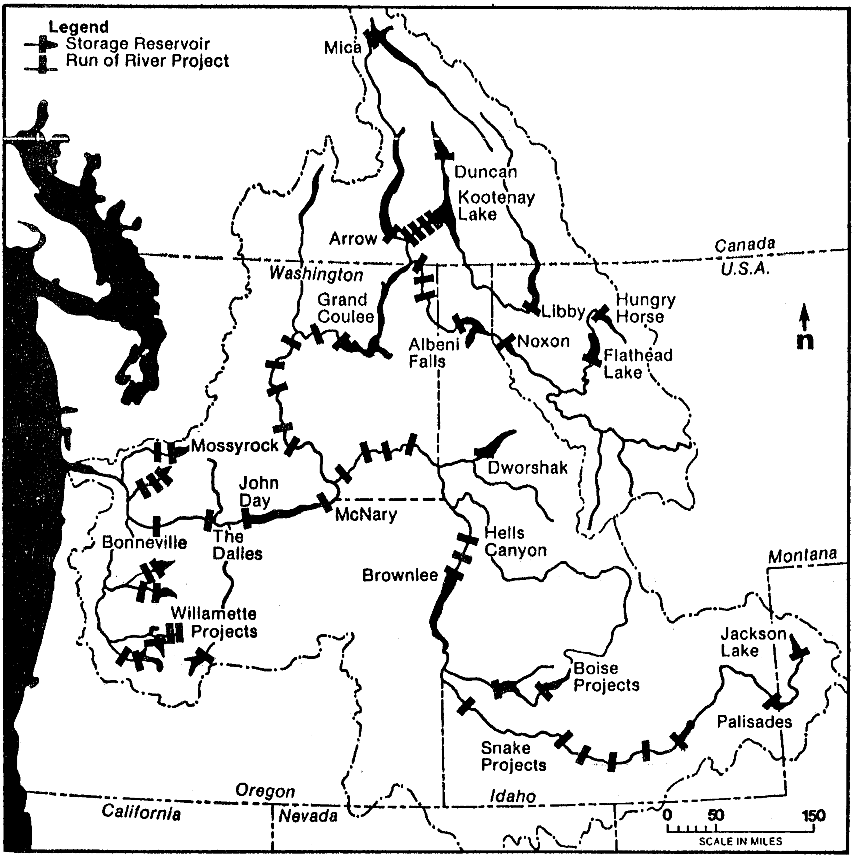


FIGURE 11.1 Hydropower installations in the Columbia River Basin. *(From Green et al. [1].)*

the desired direction, e.g., toward feasibility and preferred solutions, such as minimum cost schedules. This mechanism must be based on as complete a description as possible of all constraints, rules and objectives, and provisions for resolving conflicts among them. The solution of such problems is accomplished by a class of models referred to as prescriptive models, in contrast to pure physical process simulations, which are descriptive [2].

A typical problem encountered with hydropower scheduling is finding the best allocation of the limited power resource to present and future use, which minimizes power production cost over the long run. This trade-off situation is illustrated in Fig. 11.2. For a given state of the system, the total production cost is the sum of a present and a long-range future cost. The present cost is a function of the resource units used and the future cost is a function of the resource units available in the future. If the entire resource is used in the present time step, the present cost is minimized but the future cost will be highest. If no resource is allocated to present use, the present cost will be highest but the future cost will lowest. None of these two options produces the preferred solution, i.e., minimum

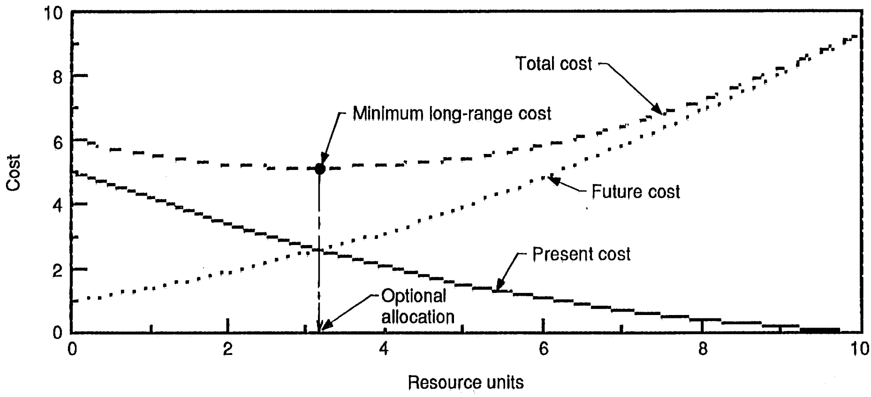


FIGURE 11.2 Optimal allocation of a limited resource.

total cost. Obviously, there are many other possible allocations between these two extremes which also do not qualify. Special search algorithms exist that find this preferred solution. Such methods are referred to as optimization methods.

The application of optimization methods, such as the calculus of variations and the method of Lagrange, to power system planning and operation problems dates back to the 1950s [3, 4]. In the 1960s and 1970s, through the epochal work of Dantzig [5], Bellman [6], and Kuhn and Tucker [7], mathematical programming methods and generalized Lagrangian methods have been increasingly used because of their capability of including inequality constraints. Computer technology has made it possible to implement these usually laborious optimization techniques. Overviews of methods related to power systems and reservoir scheduling problems have been given by Sasson [8] and Yeh [9].

Limitations of Hydropower

The amount of energy and capacity that can be obtained from hydropower is determined by a number of factors, such as stream flow, head, water storage, and location. Competing land and water uses put a rather severe limit on development scale. An important element of hydropower developments is temporary water storage. Storage raises the value of hydropower considerably, because it allows targeting the use of hydropower when it is at its highest value. But storage is difficult to obtain and is a limiting factor for the degree of hydropower development that is attainable. In the Columbia River basin (Fig. 11.1), useful storage is about 30 percent of the 280,000 ft^3/s (8000 m^3/s) annual discharge at the mouth. This is comparable to the Tennessee River system, where useful storage is about 25 percent of the 66,000 ft^3/s (1900 m^3/s) annual discharge at the mouth. While systems may be considered highly developed in terms of river miles dammed, this is not necessarily the case in terms of storage provided. Also, storage is typically limited to the headwaters and must usually be shared with other water management objectives, such as flood control and flow augmentation. Upstream storage is often crucial for downstream hydroplants because it provides them with a reliable streamflow. The limits on storage and stream flow make hydropower a fuel-limited resource. This means that only a limited capacity can be installed usually to be used about 50 to 60 percent of time.

Hydropower as a Complement to Thermal Power

In contrast to the fuel limitations of hydropower units, the fuel supply of thermal units is largely under the control of the system operator. Therefore, thermal units can be considered fuel-unlimited, even though careful fuel management may be required to achieve this goal [10]. However, the steam power apparatus is complicated and, consequently, subject to high wear and tear. This results in frequent outages and high labor and maintenance costs. Even with periodic maintenance, unplanned (or forced) outages may range from a few percent to 20 percent and more of scheduled operation time. The energy conversion efficiency of any steam-powered unit is in the neighborhood of 34 percent, which is low compared to a hydropower unit efficiency of about 85 percent. In order to reduce wear and tear and to increase unit availability, service life, and overall energy conversion efficiency, thermal units are preferably operated under a steady, uniform load. Their siting is, in principle, not geographically limited, and installed capacity can be as large as demand requires.

The different properties of hydropower and thermal units make them ideal partners in meeting the usually fluctuating electrical demand. Steam units are best at carrying more or less constant base loads, while hydropower units are well suited to cover the fluctuating, short-duration portions of the load and to flatten the up and down ramps of the load for the steam units. The relatively simple hydropower apparatus is operationally very flexible, has quick startup and shutdown capability, very low forced-outage rates, low operation and maintenance costs, and no fuel costs. Hydropower operation cost at about \$1/MWh can thus replace steam power operation cost (including fuel cost) at \$18/MWh and alternative peaking capacity of oil- and gas-fired units at about \$80/MWh.

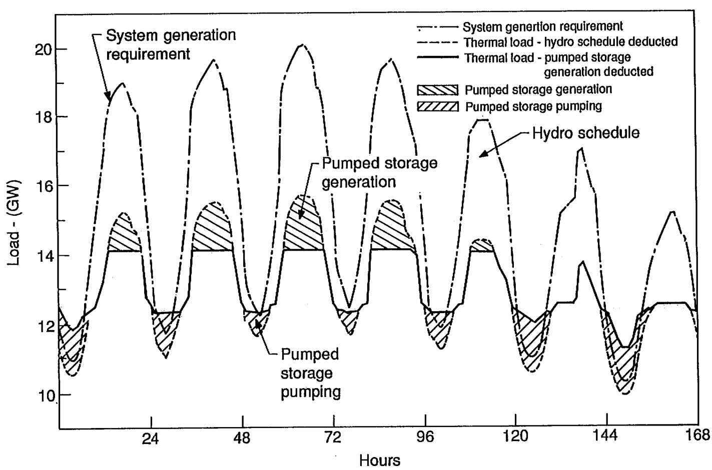


FIGURE 11.3 Hydropower-thermal load sharing to meet system generation requirement over a weekly period. *(From Giles and Wunderlich [18].)*

A typical example of load sharing between thermal and hydropower units is shown in Fig. 11.3. Load peaks are carried by hydropower to the extent that storage allows deployment of hydropower at the load peaks. Run-of-river hydropower is seen to further deepen the load valleys during low load periods, thus further aggravating the load fluctuations for the thermal units. Pumped-storage, if available, performs double duty in smoothing the thermal load by filling in the load valleys with pumping load and flattening the load peaks with generation.

In a hydropower-thermal system, hydropower is valued by the amount of thermal generation replaced. As thermal incremental cost increases with load level, hydropower can replace those portions of the load that are most expensive for the thermal units to meet. This is illustrated in Fig. 11.4*a*. It shows the cost of generation as a function of the load. By replacing the highest-cost block, $\Delta C_1 = 100$ MW, of thermal power by hydropower, thermal generation cost is reduced by ΔV_1. Hence, the cost decrement ΔV_1, in dollars, is the replacement value of the hydropower capacity ΔC_1, in megawatts. The incremental replacement value of hydropower for this first block is $\Delta V_1/\Delta C_1$, in \$/MW or, if sustained

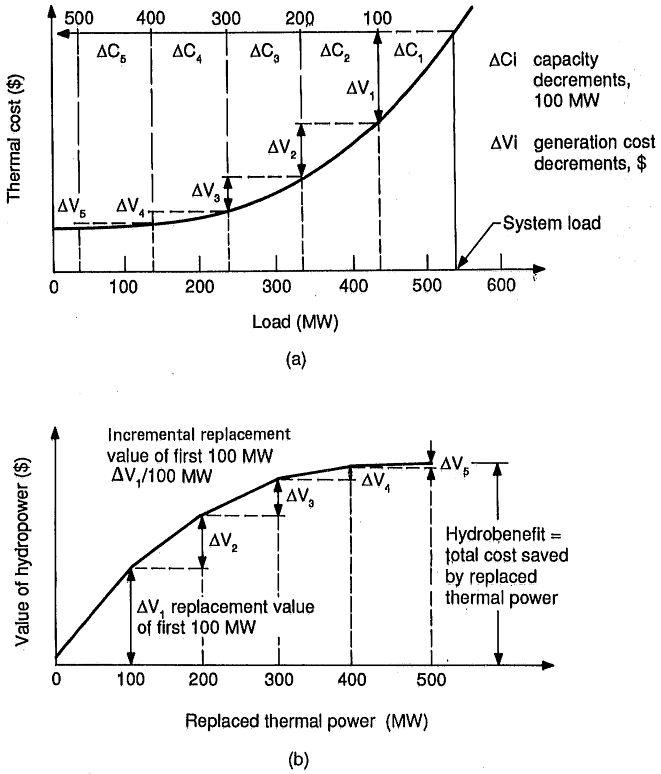


FIGURE 11.4 Value of hydropower replacement for thermal generation [2]. (*a*) Thermal cost as a function of load. (*b*) Hydropower value as cost saved by replacing thermal generation.

for an hour, in $/MWh. This incremental replacement value is also the incremental thermal generation cost, but with opposite sign. As more hydropower is used, less costly thermal power is replaced, which diminishes the incremental replacement value of hydropower, as illustrated in Fig. 11.4*b*. The integral of the incremental replacement value over the total thermal generation replaced is the total hydropower benefit for a particular time step.

In a power system without natural hydropower resources, or in addition to the limited hydropower resources, other means must be used to meet short-time fluctuating power needs. Options are flexible steam units, oil- and gas-fired combustion turbines, pumped storage, purchase, load curtailment (peak shaving), and load management. Economic, feasibility, and environmental studies must be made to determine the best mix of power resources for a particular system to meet its generation requirement.

Chapter Overview

In the following sections, various elements of hydropower operations and planning are discussed. Section 11.2 deals with the calculation of hydropower output for various types of operation, for individual hydropower units and entire hydropower plants, of plant head, plant discharge, and spill. This is followed by methods for the calculation of a probabilistic thermal system generation cost for the load that must be carried by the thermal system after the hydropower contribution has been deducted. Section 11.3 deals with the probabilistic aspects of stream flows and loads. Section 11.4 gives an overview of various optimization methods that have been used in power system simulation, with emphasis on hydropower. Section 11.5 deals with application examples of these methods to operational planning and analysis of hydropower systems and Section 11.6 deals with the use of optimization methods for the sizing and time sequencing of new projects.

11.2 HYDROPOWER–THERMAL SYSTEM CHARACTERISTICS

Hydropower System Representation

Various mathematical models have been used for the simulation of hydropower systems [4, 11]. The approach presented here constructs the plant characteristics from individual unit characteristics and provides an approximate, but concise and physically sound, approach to hydropower simulation.

Turbine Discharge Coefficient. Hydropower units, such as that shown in Fig. 11.5, can operate at many different load levels which can be controlled by regulating the flow through the turbine with wicket gates or turbine vanes. Two of these load levels are particularly important: maximum capacity and best efficiency. Maximum capacity is determined either by the maximum rating of the generator or by fully open wicket gate, whichever comes first. The best efficiency power level occurs at around 70 to 80 percent gate opening. At this load level, the ratio of power extracted to the input power is a maximum, but the power output is 20 to 30 percent below maximum capacity.

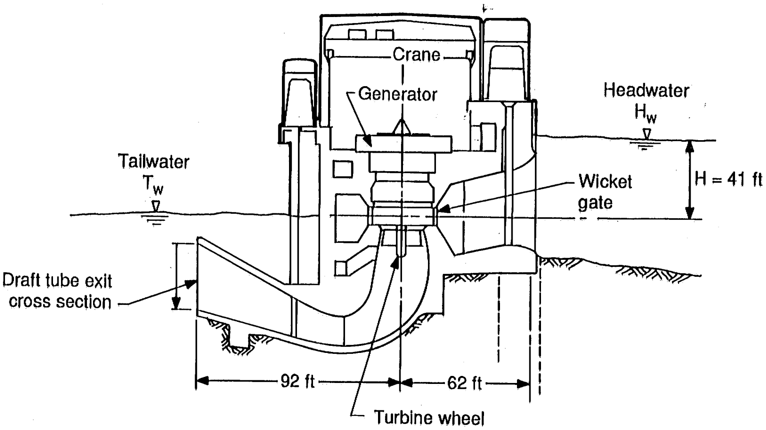


FIGURE 11.5 Example of low-head hydropower unit installation.

The flow through a hydroturbine can be visualized as being similar to the flow through a venturi tube, with the turbine penstock as the entrance portion, the turbine wheel channel as the throttle, and the draft tube as the exit portion. The energy extracted by the turbine wheel is related to the difference in energy heads over a headwater cross section for which the measured headwater level is a representative piezometric head, and over the draft tube exit section, for which the measured tailwater level is a representative piezometric head. By applying the Bernoulli equation to these two sections, the discharge can be expressed by

$$Q = k A_d \sqrt{2gH} \tag{11.1a}$$

with k an empirical discharge coefficient of the system; A_d the draft tube exit cross section; and H the gross head defined as the difference between headwater and tailwater level. A dimensionless discharge coefficient can be defined as

$$k = \frac{Q}{A_d \sqrt{2gH}} \tag{11.1b}$$

For fixed turbine wheels, the amount of flow through the turbine and, thus, the power output are controlled by the wicket gate. A typical turbine characteristics chart is shown in Fig. 11.6. It depicts the family of curves $P = P(Q)$ for constant H. Also shown are the lines of constant gate opening and efficiencies for constant H. Based on these data, functions $k = k(s)$ can be calculated, with s being the fraction of full gate. Data for a low-head fixed-blade Kaplan turbine and a high-head Francis turbine are shown in Fig. 11.7a and b, respectively. Gate openings range from 100 percent ($s = 1$, full gate) to 50 percent, and heads range from 8 to 12.5 m (26 to 41 ft) for the low head, and from 260 to 320 m (850 to 1050 ft) for the high-head unit. As a first approximation, the k values can be considered independent of head.

When full-gate power would exceed the generator limit (which can be a generator rating or the generator capacity, Fig. 11.6), P_c, for any higher head, the flow through the turbine must be controlled by partially closing the gate. At the transition of flow control from the turbine to the generator, the output is $P = P_c$, and the discharge is

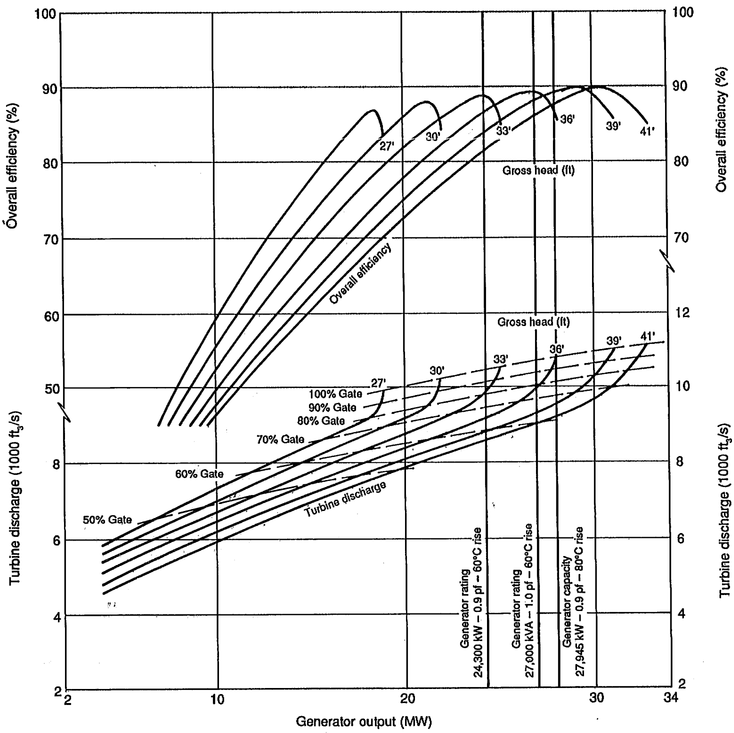


FIGURE 11.6 Hydropower characteristics of a low-head fixed-blade Kaplan unit.

$$Q_c = \frac{P_c}{\rho g \, \eta_c \, H_c} \tag{11.2}$$

with η_c the efficiency for this operation, and H_c the critical head at which the transition occurs. P_c is a given generator rating and Q_c, η_c, and H_c depend on the choice of P_c.

Power Coefficient. Substituting the discharge relationship, Eq. (11.1), into the power formula ($P = \rho g \eta Q H$) produces

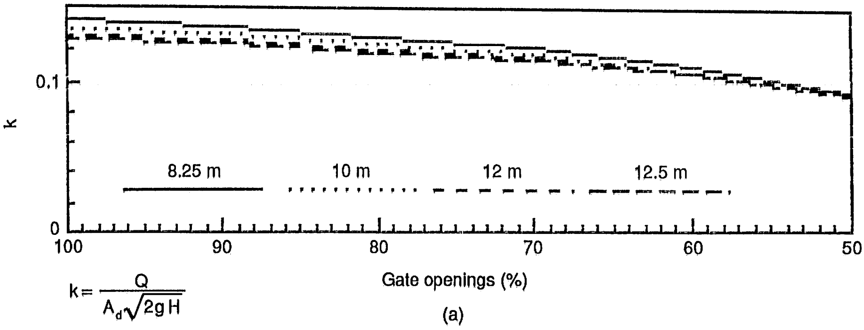
$$P = \rho g \, p \, A_d \sqrt{2g} \, H^{1.5} \tag{11.3}$$

with $p = \eta \, k$ and

$$p = \frac{P}{\rho g \, A_d \sqrt{2g} \, H^{1.5}} \tag{11.4}$$

where p is the power coefficient. Examples of power coefficients, as a function of gate opening s for the same low-head and high-head units as before, are shown in Fig. 11.8a and b, respectively.

By the above definition, the efficiency is equal to the dimensionless power/discharge ratio



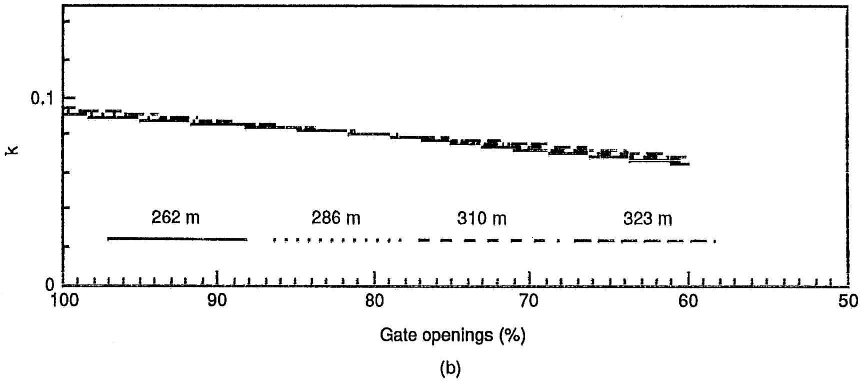


FIGURE 11.7 Discharge coefficient versus gate opening. (*a*) Fixed-blade Kaplan turbine; (*b*) Francis pump turbine—turbine mode.

$$\eta = \frac{p}{k} \tag{11.5a}$$

The efficiency can also be expressed by

$$\eta = \frac{P}{Q}\frac{1}{\rho g H} \tag{11.5b}$$

The ratio, P/Q, is also known as "power constant." Since it is actually not a constant, it will be referred to here as "power/discharge ratio." The dimensionless power/discharge ratio p/k is shown in Figure 11.9*a* and *b* as a function of gate opening for the low- and high-head unit respectively. For large gate openings, the influence of head on η is seen to be small.

Best efficiency gate can be obtained by setting the derivative of the efficiency versus gate opening to zero,

$$\frac{d\,\eta(s)}{ds} = \frac{d(p/k)}{ds} = 0$$

The resulting expression,

$$p'k - pk' = 0 \tag{11.6}$$

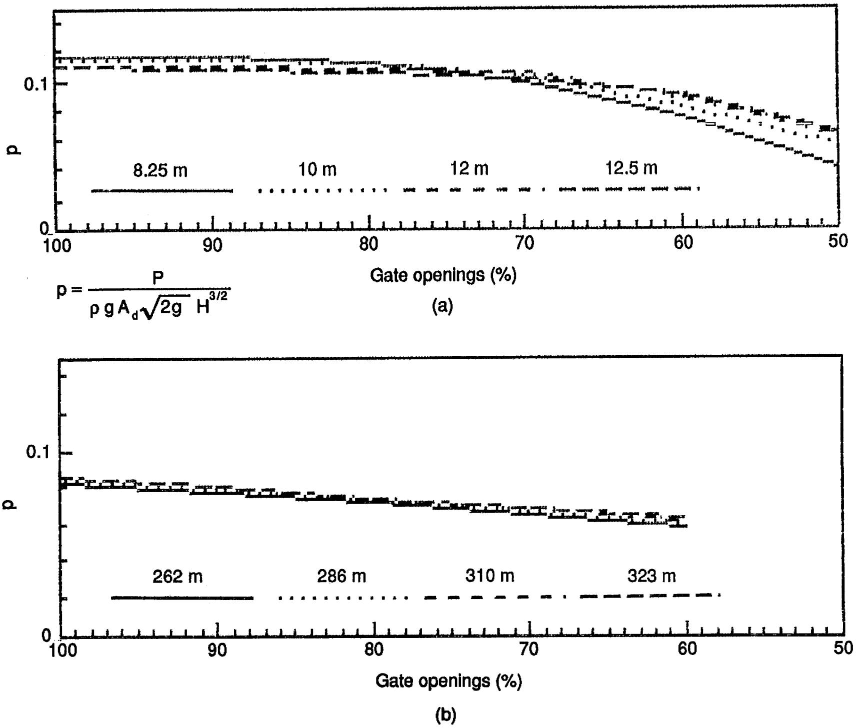


FIGURE 11.8 Power coefficient versus gate opening. (*a*) Fixed-blade Kaplan turbine; (*b*) Francis pump turbine—turbine mode.

is then solved for the best efficiency gate s_b.

The secondary influence of H on k and p can be included by cross interpolation between curves for constant H. But the small effect of H on $k(s)$ and $p(s)$ over the entire practical range of heads in the range of practical gate openings may not require this for planning applications. An approach which includes the variability with H can be found in Wunderlich [12].

The critical head at which transition of control from turbine to generator occurs is obtained by solving Eq. (11.3) for the given generator rating P_c

$$H_c = \left[\frac{P_c}{p_1\, \rho g\, A_d \sqrt{2g}}\right]^{2/3} \tag{11.7a}$$

with P_c the given generator rating and p_1 the power coefficient for full gate. H_c is determined once and for all, as it is an invariable characteristic of the unit, solely dependent on the choice of P_c.

For $H > H_c$, the discharge is calculated by

$$Q = \frac{P_c}{\rho g\, \eta_m H} \tag{11.7b}$$

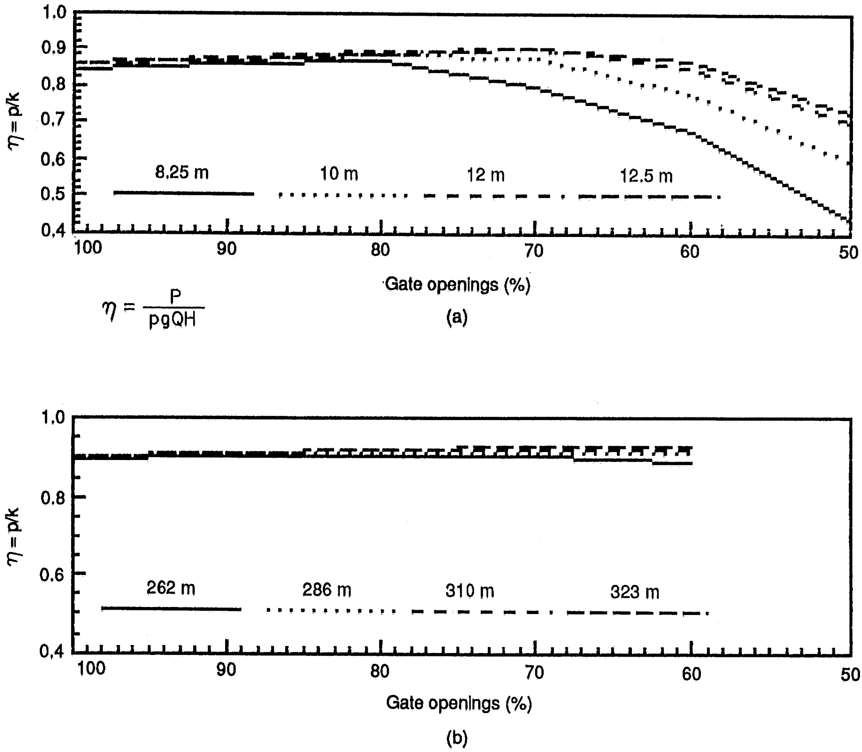


FIGURE 11.9 Efficiency versus gate opening. (*a*) Fixed-blade Kaplan turbine; (*b*) Francis pump turbine—turbine mode.

with P_c remaining constant for all $H > H_c$; $\eta_m = p_m/k_m$ is some average efficiency between full gate and best gate. As shown in Fig. 11.9, efficiencies vary only little with head in this range of s. Q is seen to be a hyperbolic function of H in this range.

EXAMPLE 11.1 *Calculate best efficiency gate, discharge and power; calculate full gate discharge and power, and calculate the critical head where shift of control occurs from turbine to generator control.*

A sample of hydro characteristics of a low-head hydro unit is given in Table 11.1. These data can be expressed by

$$k = -0.0267 + 0.3141\, s - 0.1545\, s^2$$

$$p = -0.1758 + 0.6433\, s - 0.3554\, s^2$$

The fitted curves are valid in the range $0.5 \le s \le 1$. The derivatives versus s are

$$k' = 0.3141 - 0.3090\, s$$

$$p' = 0.6433 - 0.7108\, s$$

TABLE 11.1 Hydropower Characteristics—Low-Head Fixed-Blade Kaplan Turbine

Head H, m	Gate s, %	Power P, MW	Discharge Q, m³/s	Power/discharge P/Q, MW/(m³/s)	Discharge coefficient k	Efficiency ef	Power coefficient
8.23	100.00	18.90	280.34	0.0674	0.1404	0.837	0.1175
8.23	90.00	18.80	271.84	0.0692	0.1362	0.858	0.1169
8.23	80.00	18.20	260.51	0.0699	0.1305	0.867	0.1131
8.23	70.00	15.80	246.36	0.0641	0.1234	0.796	0.0982
8.23	60.00	12.00	220.87	0.0543	0.1106	0.674	0.0746
8.23	50.00	6.60	184.06	0.0359	0.0922	0.445	0.0410
9.14	100.00	22.00	288.83	0.0762	0.1372	0.851	0.1168
9.14	90.00	21.80	280.34	0.0778	0.1332	0.869	0.1157
9.14	80.00	21.20	269.01	0.0788	0.1278	0.880	0.1125
9.14	70.00	19.20	254.85	0.0753	0.1211	0.842	0.1019
9.14	60.00	14.80	226.53	0.0653	0.1076	0.730	0.0786
9.14	50.00	9.40	195.39	0.0481	0.0928	0.537	0.0499
10.06	100.00	25.00	297.33	0.0841	0.1347	0.854	0.1150
10.06	90.00	24.80	288.83	0.0859	0.1309	0.872	0.1141
10.06	80.00	24.20	277.50	0.0872	0.1257	0.886	0.1113
10.06	70.00	22.60	263.35	0.0858	0.1193	0.872	0.1040
10.06	60.00	18.00	235.03	0.0766	0.1065	0.778	0.0828
10.06	50.00	12.00	203.88	0.0589	0.0924	0.598	0.0552
10.97	100.00	28.00	305.82	0.0916	0.1327	0.852	0.1131
10.97	90.00	27.80	297.33	0.0935	0.1290	0.870	0.1123
10.97	80.00	27.20	283.17	0.0961	0.1228	0.894	0.1098
10.97	70.00	26.00	271.84	0.0956	0.1179	0.890	0.1050
10.97	60.00	21.40	243.52	0.0879	0.1056	0.818	0.0864
10.97	50.00	14.60	209.54	0.0697	0.0909	0.649	0.0590
11.89	100.00	31.00	311.48	0.0995	0.1298	0.855	0.1110
11.89	90.00	30.80	302.99	0.1017	0.1263	0.873	0.1103
11.89	80.00	30.20	291.66	0.1035	0.1216	0.890	0.1081
11.89	70.00	29.20	277.50	0.1052	0.1157	0.904	0.1046
11.89	60.00	24.80	252.02	0.0984	0.1050	0.846	0.0888
11.89	50.00	17.60	215.21	0.0818	0.0897	0.703	0.0630
12.50	100.00	32.80	314.32	0.1044	0.1278	0.853	0.1090
12.50	90.00	32.40	305.82	0.1059	0.1243	0.866	0.1076
12.50	80.00	31.80	294.49	0.1080	0.1197	0.883	0.1056
12.50	70.00	30.80	280.34	0.1099	0.1139	0.898	0.1023
12.50	60.00	27.20	257.68	0.1056	0.1047	0.863	0.0904
12.50	50.00	19.60	220.87	0.0887	0.0898	0.725	0.0651

$\rho g = 9.79 \times 10^{-3}$ MW/[(m³/s)m].
$A_d = 157.2$ m².
$g = 9.80$ m/s².

The best efficiency gate s_b is found by substituting the above formulas into Eq. (11.6) which produces the form

$$(0.6433 - 0.7108\,s)(-0.0267 + 0.3141\,s - 0.1545\,s^2) - (0.3141 - 0.3090\,s)(-0.1758 + 0.6433\,s - 0.3554\,s^2) = 0$$

Solving for s produces the best efficiency gate, $s_b = 0.83$. This means that best gate occurs at 83 percent full gate and is the same for all heads. This is not exactly true but reasonably close to the data. Substituting s_b into the above formulas for k and p produces $k_b = 0.1278$ and $p_b = 0.1136$, from which best efficiency is obtained as $\eta_b = p_b/k_b = 0.8889$. Q_b and P_b can be obtained from Eqs. (11.1*a*) and (11.3), respectively, for a known head. The simultaneous calculation of Q, H, and P will be described below (see also Example 11.2).

For full gate discharge and power, substituting $s = 1$ into the formulas for k and p produces $k_1 = 0.1329$, $p_1 = 0.1121$, and $\eta_1 = 0.1121/0.1329 = 0.8435$. Substituting k_1 and p_1 into the formulas for Q and P gives the full gate turbine discharge and power output, if H is known.

The critical head above which power output is generator controlled is computed by Eq. (11.7*a*). For $P_c = 28$ MW, $p_1 = 0.1121$, $\rho g = 0.00979$ MW/[(m^3/s)m], $A_d = 152.7$ m^2, g $= 9.8$ m/s^2, Eq. (11.7) gives

$$H_c = 11.25 \text{ m}$$

The critical discharge at transition is

$$Q_c = k_1 A_d \sqrt{2gH_c} = 310 \text{ m}^3/\text{s}$$

H_c and Q_c are computed once and for all. They do not vary with operational conditions, but depend only on the choice of P_c. Graphical presentations of discharge functions for 1, 2, 3, and 4 units are given in Fig. 11.10. The parabolic

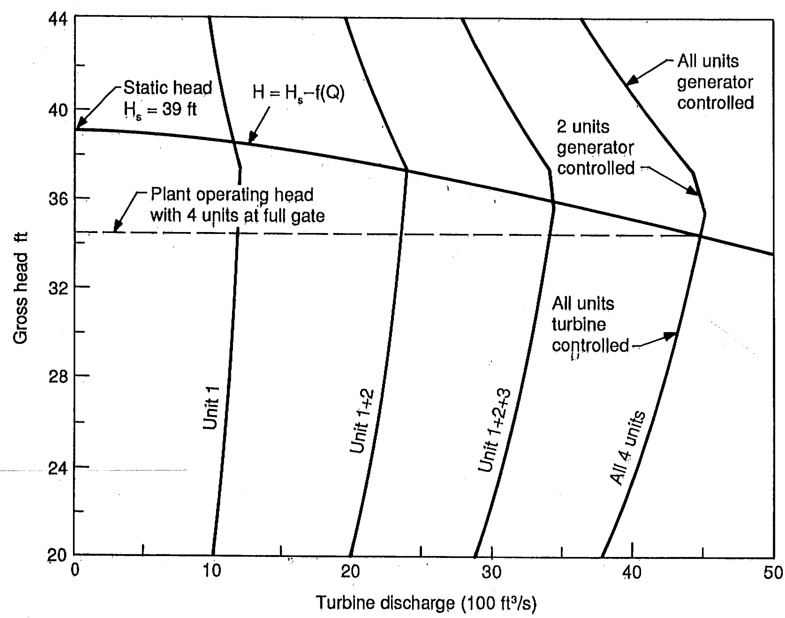


FIGURE 11.10 Discharge functions for 1 to 4 units operating at full gate and gross head function for given static head.

lower portion along which Q increases with head is the turbine controlled range of the discharge function, and the hyperbolic upper portion along which Q recedes with head is the generator controlled range. The figure shows discharge functions for two pairs of turbines with different critical heads which causes the function for more than two units to have two break points.

Gross Head. The tailwater elevation of a hydroplant usually varies as a function of project discharge. The tailwater can be kept low if downstream flow is unobstructed, as in a free-flowing tailwater. By plant design, the tailwater rise should be kept as small as possible. For its calculation, see Chapter 2.

If a downstream dam impounds the tailwater, as is the case in a cascade of projects providing a navigable channel, the tailwater elevation becomes a function of discharge and the headwater at the downstream dam. For large project discharges, the downstream headwater effect disappears, and the flow below the dam assumes a free-river-flow regime. The lower the headwater at the downstream dam, the earlier the free-river-flow regime takes control of the tailwater rise. The free-river regime is the lower bound on the tailwater rise. A downstream impoundment can only raise the tailwater above this lower bound.

The gross head H is defined as

$$H = H_w - T_w \tag{11.8}$$

with H_w and T_w the headwater and tailwater elevations, respectively, of the project under consideration. For zero discharge (assuming that there is no substantial local inflow into the tailwater), the downstream pool is flat and the gross head becomes equal to the difference between the headwaters of the two neighboring projects,

$$H_s = H_w - H_d \tag{11.9}$$

with H_s being called the static head, and H_d the downstream headwater elevation. The tailwater rise can be expressed by

$$T_w = H_d + f(\mathrm{QP}) \tag{11.10}$$

with $f(\mathrm{QP})$ being the tailwater function and QP a representative downstream flow that is responsible for the tailwater rise. Combining Eqs. (11.8), (11.9), and (11.10) leads to the gross head function

$$H = H_s - f(\mathrm{QP}) \tag{11.11}$$

with

$$f(\mathrm{QP}) = \sum_{i=0}^{3} \mathrm{t_i}\ \mathrm{QP}^i \tag{11.12}$$

$$t_i = z_{i0} + y(z_{i1} + yz_{i2}) \quad i = 0, 1, 2, 3 \tag{11.13}$$

$$y = H_d - H_r$$

The coefficients t_i are functions of a representative depth y of the downstream reservoir, and H_r is a reference elevation, for which one may use the elevation of the river channel bottom in the tailwater [13]. The empirical coefficients z_{ij} are valid for specified ranges of QP. An example of a tailwater function is given in Table 11.2.

TABLE 11.2 Tailwater Function*

t_i	z_{i0}	z_{i1}	z_{i2}
$0 < \text{QP} \leq 40$			
0	0.010	0	0
1	-0.77367×10^{-2}	0.70209×10^{-2}	-0.40792×10^{-3}
2	0.85672×10^{-2}	-0.22348×10^{-2}	0.19007×10^{-3}
3	-0.10759×10^{-3}	0.36749×10^{-4}	-0.36529×10^{-5}
$40 < \text{QP} \leq 120$			
0	-1.948	0	0
1	0.25769	-0.40271×10^{-1}	0.29321×10^{-2}
2	-0.13123×10^{-2}	0.52630×10^{-3}	-0.54516×10^{-4}
3	0.38370×10^{-5}	-0.21339×10^{-5}	0.25459×10^{-6}
$\text{QP} > 120$			
0	-0.845	0	0
1	0.18816	-0.14246×10^{-1}	0.12946×10^{-3}
2	-0.40319×10^{-3}	0.66144×10^{-4}	-0.87541×10^{-6}
3	0.43226×10^{-6}	-0.95415×10^{-7}	0.14982×10^{-8}

*
$$\text{TW} = H_d + \sum_{i=0}^{3} t_i\, QP^i$$

with TW in ft m.s.l.; H_d in ft m.s.l.; QP in 1000 ft³/s; $t_i = z_{i0} + y(z_{i1} + yz_{i2})$; $y = H_d - H_r$; $H_r = 590$ ft; 1 ft = 0.3048 m; 1 ft³/s = 0.0283 m³/s.

Source: After Garanflo [13].

The flow QP can usually be calculated as the total project discharge plus any major tributary inflow into the tailwater by

$$\text{QP} = Q_p + S + L \tag{11.14}$$

where Q_p is the plant discharge, S is the spillway discharge, and L is a local inflow that enters into the plant's tailwater or its vicinity. An example of a gross head function is shown in Fig. 11.11. The two curves shown approximately bound this projects's annual gross head range.

The plant discharge function is the sum of individual unit discharge functions

$$Q_p = \sum_{i=1}^{N} Q_i \tag{11.15}$$

with N the units included in the operation schedule. The Q_i of Eq. (11.15) are all functions of the same H. Q_p and H are obtained by simultaneously solving Eqs. (11.11) and (11.15), as illustrated in Fig. 11.10. The overall plant efficiency is obtained by

$$\eta_p = \frac{\sum_{i=1}^{N} \eta_i Q_i}{\sum_{i=1}^{N} Q_i} \tag{11.16}$$

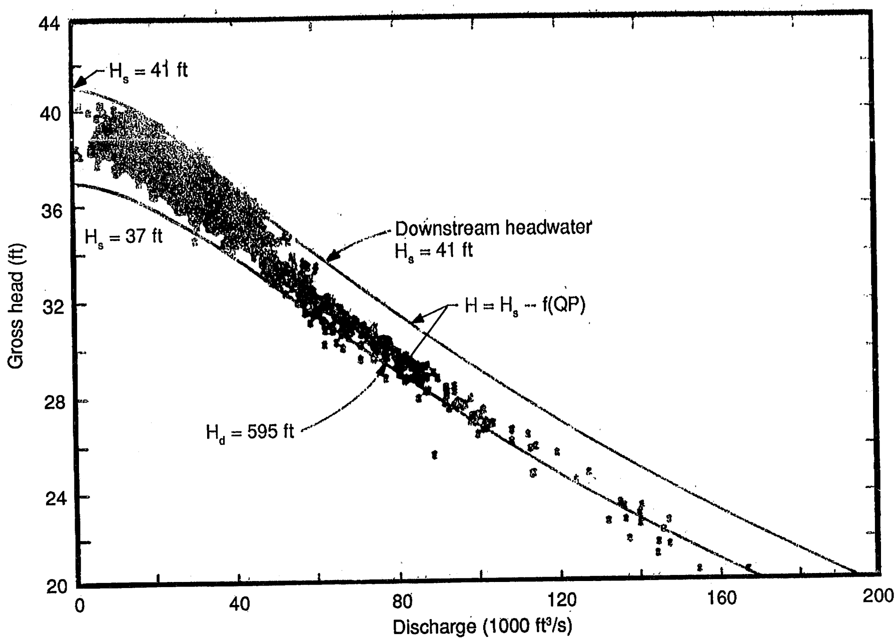


FIGURE 11.11 Gross head function for maximum and minimum downstream headwater and given static heads—with observed data from high-flow days.

with the sums being taken over all operating units. More details can be found in Example 11.2.

EXAMPLE 11.2 *Calculation of plant head and plant power output.*

Assume there are four identical units with the above discharge coefficient k and power coefficient p. Table 11.2 is in English units. Therefore, the calculations will be made in English units. The critical head was obtained in Example 11.1 as H_c = 36.9 ft (11.25 m). Since the units are assumed to be identical, there is only one breakpoint in the discharge function. The discharge function at full gate is

$$Q_p = 4\, k_1 A_d \sqrt{2gH}$$

for $H \leq 36.9$ ft (11.25 m), with A_d = 1692 ft², g = 32.15 ft/s, and H in ft, and under generator control,

$$Q_p = \frac{4\, P_c}{\rho g\, \eta_m\, H}$$

for $H \geq 36.9$ ft (11.25 m), with $\eta_m = (p_1 + p_b)/(k_1 + k_b) = 0.865$; P_c = 28 MW; ρg = 0.0000844 MW/[(ft³/s)ft]; and H in ft.

The static head is H_s = 39.37 ft (12 m). The reference elevation is assumed to be 589.41 ft (179.60 m) above m.s.l., and the headwater elevation of the downstream project is assumed at 594 ft (181 m); $y = 4.59$ ft. The equations to calculate the gross head and power discharge are

$$H = H_s - t_0 - t_1(Q_p + L) - t_2(Q_p + L)^2 - t_3(Q_p + L)^3 \qquad (a)$$

$$Q_p = 4 \cdot 0.1329 \cdot 1692 \cdot (2 \cdot 32.15 \cdot H)^{0.5}/1000 \qquad \text{if } H \leq H_c \qquad (b)$$

or

$$Q_p = 4 \cdot 28/(0.0000844 \cdot 0.865 \cdot H)/1000 \qquad \text{if } H > H_c$$

Both discharge functions are divided by 1000 because Table 11.2 is set up for discharges in 1000 ft³/s. The coefficients t_i are computed by:

$$t_i = z_{i0} + 4.59\,(z_{i1} + 4.59 z_{i2}) \qquad i = 0, 1, 2, 3 \qquad (c)$$

For the estimated flow range, $40 < QP \leq 120$, in 1000 ft³/s, the coefficients read from Table 11.2 are:

$z_{00} = -1.948$	$z_{01} = 0$	$z_{02} = 0$
$z_{10} = 0.25769$	$z_{11} = -0.040271;$	$z_{12} = 0.29321 \cdot 10^{-2}$
$z_{20} = -0.13123 \cdot 10^{-2}$	$z_{21} = 0.52630 \cdot 10^{-3}$	$z_{22} = -0.54516 \cdot 10^{-4}$
$z_{30} = 0.38370 \cdot 10^{-5}$	$z_{31} = -0.21339 \cdot 10^{-5}$	$z_{32} = 0.25459 \cdot 10^{-6}$

The following cases are discussed:

1. Normal flow and no local inflow: The solution of Eqs. (a) and (b) together with the four equations represented by Eq. (c) by a nonlinear solution technique produces $H = 35.74$ ft (10.89 m), which is less than $H_c = 36.9$ ft (11.25 m), as anticipated, and $Q = 42{,}310$ ft³/s (1198 m³/s), which is also in the assumed range. The full gate power output for each of the four units is

$$P = 0.0000844 \cdot 0.1121 \cdot 1692 \cdot (2 \cdot 32.2)^{0.5} \cdot 35.74^{1.5} = 27.45 \text{ MW}$$

2. A tributary empties 35,315 ft³/s (1000 m³/s) into the tailwater: In the gross head equation, $L = 35.315$ is used instead of $L = 0$ in case (a). The resulting plant head is $H = 31.7$ ft (9.66 m) $< H_c$, and the maximum power discharge is $Q_p = 39{,}850$ ft³/s (1128 m³/s). The power output is 22.8 MW per unit.
3. Plant operation at a static head $H_s = 41$ ft (12.5 m): The resulting plant gross head is 37.65 ft (11.45 m) and the plant discharge is 40,840 ft³/s (1156 m³/s). In this case, $H > H_c = 36.9$ ft (11.25 m) and the plant discharge and head are obtained by intersecting Eq. (a) with Eq. (b) for $H > H_c$. The plant operates at the generator rating of $4 \cdot 28 = 112$ MW.
4. The project is required to discharge 124,000 ft³/s (3500 m³/s): H is directly calculated from Eq. (a), because the discharge is determined by other considerations and also exceeds the maximum plant discharge. For an $H_s = 40$ ft, an approximate value $H = 26$ ft (7.92 m) is read from the gross head function in Fig. 11.11. For this head, the full gate plant discharge is $Q_p = 4 \cdot 0.1329 \cdot 1692 \cdot (64.4 \cdot 26)^{0.5} = 36{,}806$ ft³/s (1042 m³/s) and the plant power output is $P = 4 \cdot 0.0000844 \cdot 0.1121 \cdot 1692 \cdot (64.4)^{0.5} \cdot 26^{1.5} = 68.2$ MW. For

computing the gross head, another set of coefficients valid for the range QP > 120 in Table 11.2 must be used.

Use of Computed Hydropower Functions. Examples of computed and observed power output are shown as a function of head in Fig. 11.12 and of discharge in Fig. 11.13. The ascending branch on the left of Fig. 11.13 shows the full plant output that is possible under the given static head. The branches to the right with a flat top indicate that generator capacity was reached and maintained over a range of increasing flows (and rising tailwater), until full gate was reached again, whereupon power output starts to decline for continued flow increase resulting from rising tailwater. The curves branching off downward, such as the curve for all four units operating, indicate that generator capacity was not reached or maintained over a range of discharges for this static head.

The power versus discharge function of Fig. 11.13 is valid only for one particular pair of H_w and H_d. If H_w and H_d are variables, the function can be precomputed for selected (H_w, H_d) and the power output for any head combination and discharge can be found by interpolating between neighboring functions or the above-described calculation can be carried out in each case.

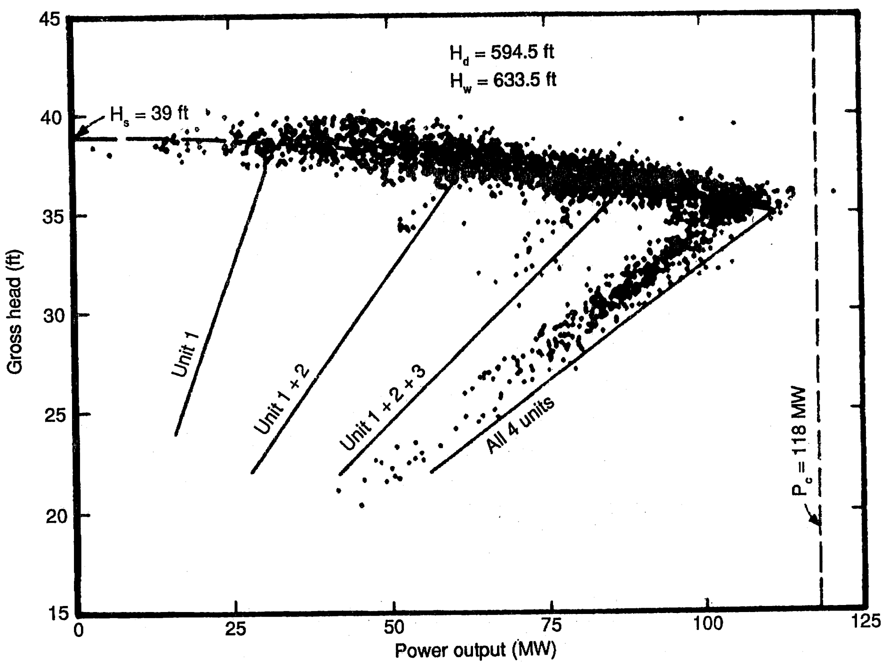


FIGURE 11.12 Calculated gross head and plant power output—with observed data from high-flow days.

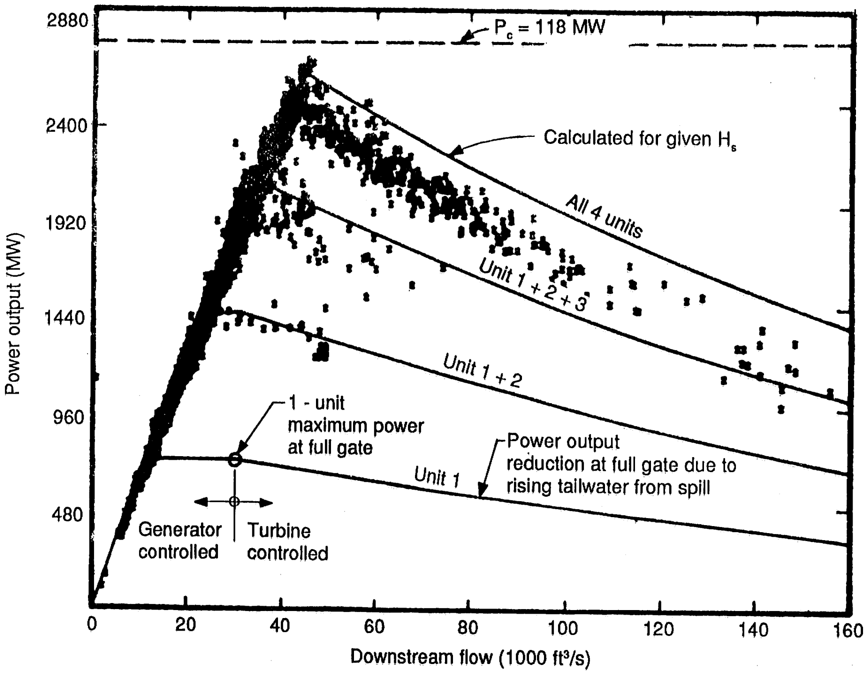


FIGURE 11.13 Calculated plant power output versus project discharge—with observed daily data.

Thermal System Representation

Hydropower Benefit. The hydropower benefit (HB) is defined as the difference between the cost of meeting the entire system load without hydropower and the cost of meeting it after a part of the thermal production has been replaced by hydropower. This can be expressed by

$$HB = CS - CH \tag{11.17}$$

where CS is the cost of meeting the load without hydropower, and CH is the cost of meeting the load after thermal replacement by hydropower has occurred, all in a \$/time period, the time period being the time step of the schedule, such as a week.

For calculating CS, alternative power resources have to be substituted for hydropower to meet peak and other highly fluctuating loads. Such alternative power resources are gas- and oil-fired combustion turbines, pumped storage, purchases from neighboring systems, or peak shaving. This makes the computation of CS rather hypothetical. Usually, CS is not required for hydropower–thermal power scheduling. Since CS does not vary with the amount of hydropower used, it is treated as a constant when forming the necessary condition for maximizing the hydropower benefit. This necessary condition is

$$dHB = 0 = d(-CH) \tag{11.18}$$

Maximizing HB is seen to be equivalent to maximizing (−CH), which is also equivalent to minimizing CH. Figure 11.4 illustrates the hydropower benefit as a cost reduction by replacing thermal generation by hydropower. In Fig. 11.4, it is assumed that hydropower can take over the most expensive portions of the load. This is only true if hydropower storage is available so that hydropower generation can be timed for use during load peaks. The hydropower benefit varies with the hour of the day. In high-load hours, more expensive thermal generation can be replaced so that the benefit is higher than in low-load hours, as illustrated in Fig. 11.14. The optimal hydropower use strategy, over the long run, aims at using the right amount of hydropower in each time step (hour), so that the benefit from the available hydropower resource is maximized.

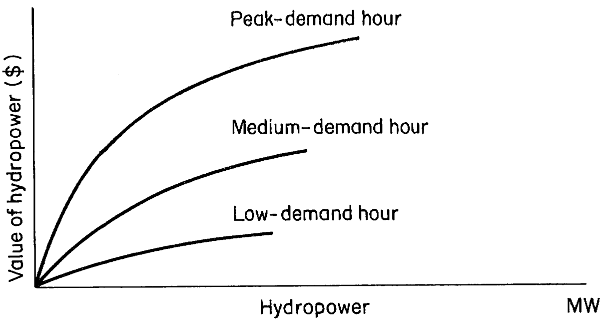


FIGURE 11.14 Hourly hydropower benefit functions for high, medium, and low loads.

Thermal System Elements. In order to quantify the hydropower benefit and to arrive at an optimal hydropower schedule, a realistic calculation of the thermal generation cost is required. Models which emphasize the thermal system aspects usually use a lumped hydropower system representation [11, 14, 15]. Here, emphasis is on the hydropower system schedule. Such a schedule is guided by the overall thermal replacement cost but not by details of thermal system operation. Therefore, a lumped thermal system representation can be used [16].

The hydropower system scheduling procedure begins with an initial feasible hydropower schedule. After the thermal cost CH associated with this schedule has been found, a search procedure is used to find a new hydropower schedule with a lesser cost. This iterative process is continued until no further cost reduction is possible. Search procedures for this purpose are discussed in Secs. 11.4 and 11.5. Six elements needed for the calculation of the cost CH will be briefly discussed.

Thermal Unit Heat Rates. The efficiency of a thermal unit is related to the incremental heat rate, which is the heat energy rate, in MW or Btu/h, required per megawatt of electrical output. Typically, heat rates are of the order of 6700 to 10,200 Btu/kWh [17]. This corresponds to 2.0 to 3.0 MW_t/MW_e (1 Btu = $2.9302 \cdot 10^{-4}$ kWh), with the indices referring to thermal and electrical energy rates. The inverse of the heat rate is the thermal conversion efficiency.

For the above heat rates, the thermal efficiencies range from 0.33 to 0.51, in contrast to hydropower efficiencies of around 0.85.

The heat rates generally vary with the power level. They are relatively low at low power levels and increase at a higher than linear rate with the power level. An example is given in Fig. 11.15. The heat rate increase indicates decreasing efficiency with increasing power level.

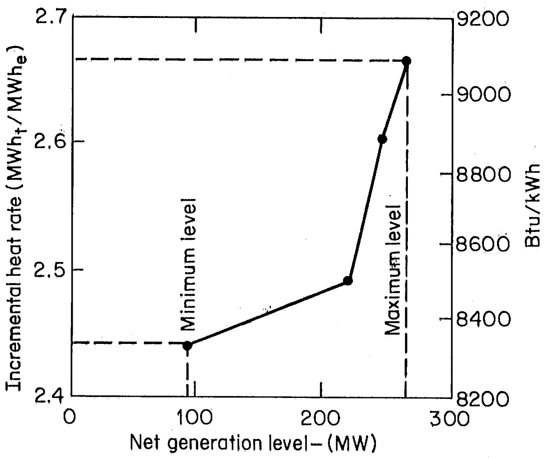


FIGURE 11.15 Incremental heat rates of thermal unit. *(From Fan and Giles [22].)*

Thermal Unit Fuel Costs, Operating Costs, and Maintenance Costs. The power production cost as a function of power level is obtained as the sum of heat energy costs plus operation and maintenance costs. The heat energy costs are obtained by multiplying the incremental heat rates with the fuel costs. The fuel costs, in $/t, and the fuel-to-heat conversion rate, in t/MWh_t, vary with the fuel used. Some examples of heat content values are 10,600 to 12,100 Btu/lb (6.8 to 7.8 kWh/kg). An average fuel cost for a southeastern U.S. utility in 1984 was $37/ton (the English short ton, or 2000 lb). By dividing this fuel cost by the heat content, 10,600 Btu/lb, one obtains an average heat energy cost of $1.75/MBtu ($6/MWh_t) at the plant. This cost represents the cost of coal burned, consisting of purchase price, transportation and purchasing expense, strip-mining reclamation cost, ash disposal cost, etc. [17].

The costs of heat energy are subject to rather drastic changes. They have increased more than fivefold over a 10-year period, from the mid-1970s to the mid-1980s, from about $0.3/MBtu to about $1.72/MBtu [17]. Up-to-date data must be used in studies. With a heat energy conversion efficiency of about 0.33, the cost of 1 MWh_e is approximately three times the heat energy cost, or $18/$MWh_e$.

Usually there are many thermal units in the system. Incremental cost functions must be made available for each unit. Optimal load allocation to the units requires that they be loaded at equal incremental cost. The total power output that can be achieved at each incremental cost level is obtained by summing the power output of all committed units at each incremental cost level [18]. In a weekly simulation, there is a different load duration curve and usually also a different maintenance schedule for each week. This requires assembling an incremental system cost for each weekly unit commitment.

System Load and Capacity Requirements. The load is the chronological sequence of instantaneous power levels over a period of time, such as a week, that the power system must meet. Two examples of weekly load patterns are given in Fig. 11.16. Usually hourly load data are used. While there is a solid base load of about 10,000 MW in the summer week and 13,000 MW in the winter week, a fluctuating load of about 9000 MW must be met in both weeks above this base load. This fluctuating load is correlated with meteorological factors, in particular air temperature. The characteristically cyclic pattern of the daily summer loads is caused by air conditioning, while the rather irregular winter load fluctuations reflect the more irregular winter air temperature fluctuations.

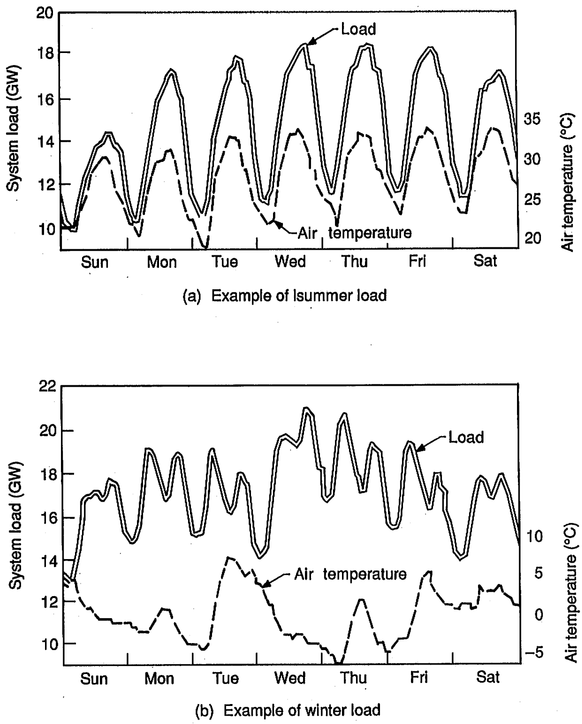


FIGURE 11.16 Samples of weekly loads for a southeastern U.S. utility. *(From Boston [160].)*

The allocation of the power resources to meeting the load is made either by a chronological method [19, 20] or a load-duration curve method [16, 21]. The chronological method is more precise but also more laborious. It includes the load gradients with time and steam unit ramping in the cost calculation. The load-duration curve method is a shortcut that uses a simplified load representation that retains only the load levels and their total duration. The relationship between a chronological load and the corresponding load-duration curve is illustrated in Fig. 11.17. Only the load-duration curve method will be briefly discussed.

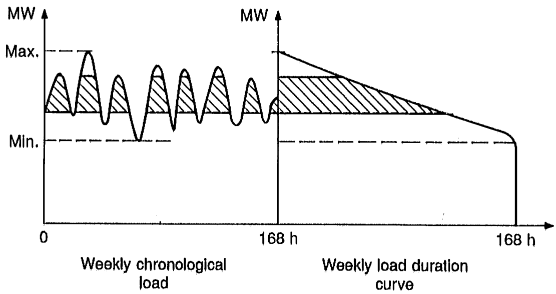


FIGURE 11.17 Chronological load and load duration curve.

A weekly load-duration curve is obtained by sorting the 168 sequential hourly loads, more precisely, generation requirements, into a monotonically decreasing sequence of hourly loads, with the highest on the left and the lowest on the right. The resulting load-duration curve is a compact but simplistic representation of the capacity and energy requirements of the week. The generation requirement is the net sum of system load, transmission losses, and contractual exchanges with neighboring power systems. The load-duration curve, at any power level, shows the fraction of time that a power level is equaled or exceeded, but it gives no indication of the frequency at which this occurs. This omission of load fluctuations tends to produce a nonconservative estimate of the generation cost, which is the primary weakness of the load-duration curve method.

The load-duration curve is usually averaged into a reduced number of subperiods, such as 21 eight-hour periods. However, the highest instantaneous load is retained. This load plus the spinning reserve requirement, the latter usually being the size of the largest operating thermal unit, is the highest instantaneous load that the system must meet, also referred to as designated load. Load projections used in planning studies are usually normalized loads for normal air temperatures. Before the load is used for a particular hydrologic year, the ordinates of the load-duration curve are adjusted to the air temperature departures from normal for that particular year [22].

Thermal System Uncertainties. Major sources of uncertainty in thermal generation cost calculations are system loads and thermal unit availability. The load uncertainty is an external input uncertainty that is taken into account by analyzing several load scenarios. The uncertainty about unit availability is a system uncertainty that is dealt with in the generation cost function.

The rather complicated power apparatus of the steam thermal units is responsible for relatively frequent, unexpected outages of thermal units. These outages have a nonnegligible impact on production cost because the load that is dropped by a forced outage must be picked up by reserve units with higher operation costs.

Units selected from the set of system units to carry the load in a scheduling period are "committed" units. These are units available to pick up load as needed. They are arranged in some order of use, for example, generation cost at maximum capacity, in what is called a unit stack. Lowest-cost units are at the bottom of the stack and used with probability 1 if available. The highest-cost units are at the top of the stack with a low probability of use, even if they are available. Pictorially speaking, units in the stack above the (thermal) generation requirement must sink below that level by outages of cheaper units in the lower portions of the unit stack before they are called on to contribute to the load. Units on scheduled maintenance are not included in the unit stack. The maintenance schedule is treated as an input to the model, but it could be made a variable if optimization of the maintenance schedule were called for.

UNIT AVAILABILITY AND CONTRIBUTION PROBABILITY: A unit not on maintenance is available if it is not on forced outage. The forced outage rate of unit i is $f_i = 1 - a_i$, with a_i being the unit's availability. Various techniques have been developed to assess the effect of unit availability on cost. Examples are the convolution method [14, 23, 24] and the normal distribution method [25].

Both techniques treat the thermal units as Bernoulli random variables, i.e., unit i is out of service with probability f_i and in service with probability a_i. In the method of Baleriaux et al. [23], the forced outage of units is represented by transforming the load-duration curve into an "equivalent load distribution." The expected system operation cost is then obtained by evaluating this equivalent load distribution. Details of the method can be found in the references. A literature survey on the method was given by Wood and Wollenberg [11]. A sample of thermal unit availabilities is given in Table 11.3.

TABLE 11.3 Sample of Thermal Unit Capacities and Availabilities

Unit number	Capacity, MW	Availability a
1–3	262	0.887
4	875	0.778
5–8	185	0.898
9	495	0.855
10	1261	0.815
11	1261	0.600
12	179	0.922
13	228	0.922
14, 15	270	0.883
16–19	190	0.961
20–25	113	0.957
26–29	149	0.891
30–	136	0.918

The normal distribution method [22, 25] does not change the load. The stack capacities are construed as sums of random variables which are themselves random variables. According to the central limit theorem, the means of sufficiently large sums of random variables are nearly normally distributed, regardless of the distribution of the original random variables. This property is used to calculate the probability b_j of unit j being used by determining the probability of the stack capability up to and including unit j being less than the load, using the normal distribution of stack capabilities. The probability of unit j contributing to meeting the load is then obtained as the product of unit j being available times the probability of unit j being used or, $c_j = a_j b_j$. The cost of meeting the load is then obtained as the sum of the probability-weighted costs, using this probability of contribution c_j.

LOSS-OF-LOAD PROBABILITY: The central limit theorem is also used for estimating the loss-of-load probability [25]. This is the probability by which the unit stack will fail to meet the allocated load requirement. Whereas in the previous case the probability of a certain stack level being exceeded was of interest, now the probability of the entire stack capacity being exceeded is determined. The stack capacities are again sums of random variables, normally distributed for sufficiently large N, according to the central limit theorem. Calculated loss-of-load probabilities are shown in Fig. 11.18 for 20 and 30 units. It is seen that increasing the stack capacity from 20 to 30 units reduces the loss-of-load probability. The figure also shows a comparison of the loss-of-load probabilities calculated by the normal distribution with those obtained by a Monte Carlo simulation using 10,000 trials [25]. It demonstrates the applicability of the described approach, even for a relatively small number of units.

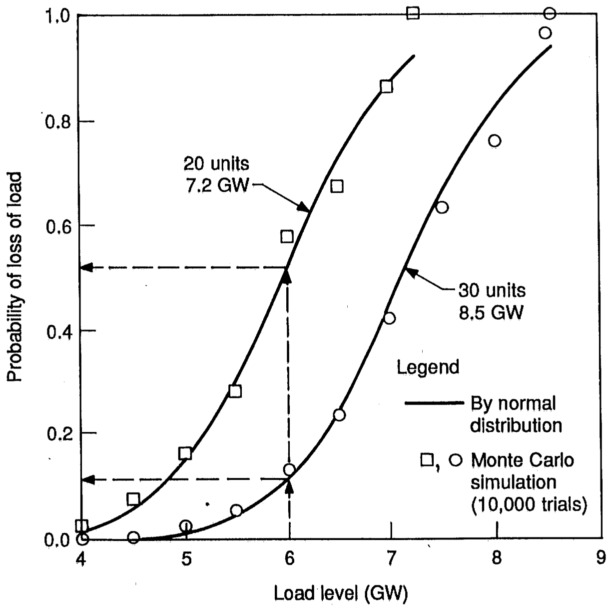


FIGURE 11.18 Loss-of-load probability for two different-sized unit stacks. *(From Giles et al. [25].)*

Thermal Unit Load Allocation. Steam thermal generation is best carried out on a continuous basis. The conditions for such an operation can be provided by flattening the load that is allocated to the steam units. In other words, hydropower is allocated to the more intermittent generation in the upper portion of the load-duration curve while steam thermal generation takes over the more continuous loads in the lower portion. In the lumped system representation used here, if a thermal unit is scheduled to contribute to the peak, it is kept on-line during the remaining period at least at minimum power level.

HYDROPOWER LOAD-DURATION CURVE: The hydropower allocation is accomplished by subtracting the hydropower load-duration curve from the system generation requirement. A hydropower load-duration curve is constructed by converting a feasible hydropower schedule for each plant into a block of generation, usually at maximum capacity, for a duration that can be sustained at that capacity by the available release, and summing these blocks of generation over all plants of the system. Subtracting this hydropower load-duration curve from the system generation requirement produces the residual thermal generation requirement, as illustrated in Fig. 11.19. This residual load-duration curve may not be monotone decreasing from left to right, as shown in the figure. The load is then leveled by shifting hydropower from the left to the right, as illustrated in Fig. 11.20, until a monotone decreasing residual thermal load-duration curve is restored, while at the same time maintaining hydropower feasibility. Shifting hydropower from left to right in the load-duration curve means shifting hydropower from peak to off-peak without exceeding maximum hydropower capability.

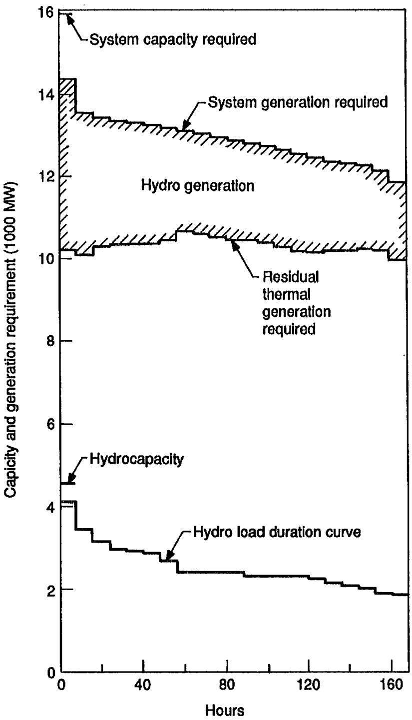


FIGURE 11.19 Typical weekly system generation requirement, hydro load duration curve and residual thermal generation requirement.

Pumped-Storage Use. Pumped-storage generation is placed in the thermal unit stack at the level of its generation cost. This generation cost is based on the energy cost for pumping, but the recuperated energy is diminished by the losses of the pumped-storage cycle. These losses include losses due to conversion of pumping energy to stored energy, evaporation and seepage losses from the storage reservoir, and generating losses. The input energy by pumping is $E_p = \rho g V_p H/\eta_p$, and the output energy by generating is $E_g = \rho g \eta_g V_g H$, with V_p and V_g the volumes of water pumped and generated for a constant head H, respectively. If evaporation and seepage losses are negligible, $V_p = V_g$, the efficiency of the pumped-storage process, defined as the ratio of energy out to energy in, is

$$\frac{E_g}{E_p} = \eta_g \eta_p = \eta_s \tag{11.19}$$

If one assumes 0.85 for both pumping and generating efficiencies, then the efficiency of the pumped-storage cycle is $\eta_s = 0.72$.

The optimal pumped-storage use during a planning period is the one that minimizes the total cost of meeting the residual system load. Similar to the definition

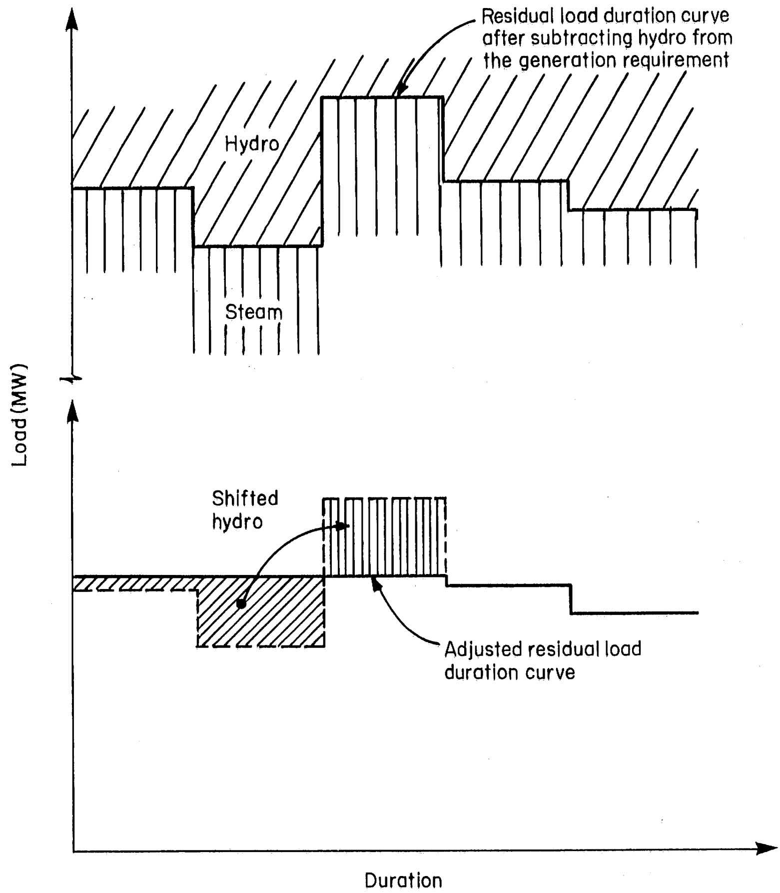


FIGURE 11.20 Adjusting the residual load duration curve.

of the hydropower benefit by Eq. (11.17), the pumped-storage benefit can be defined by

$$B_s = \mathrm{CS}_r - \mathrm{CH}_r$$

with CS_r and CH_r being the cost of meeting the residual thermal load without and with pumped storage, respectively, and B_s being the pumped storage benefit, all expressed in \$/week. B_s is the difference between the replacement value of the pumped-storage generation and the pumping cost,

$$B_s = F_g(\mathrm{PG}) - F_p(\mathrm{PP})$$

where F_g(PG) is the integral over the generation cost function, $f_g(P)$, from an as yet unknown optimal residual generation level PG to the top of the residual load PX; F_p(PP) is the integral over the pumping cost function, $f_p(P)$, from the minimum off-peak power level PN to an as yet unknown thermal off-peak load level PP that includes pumping; and P is the power level, in MW. The integration

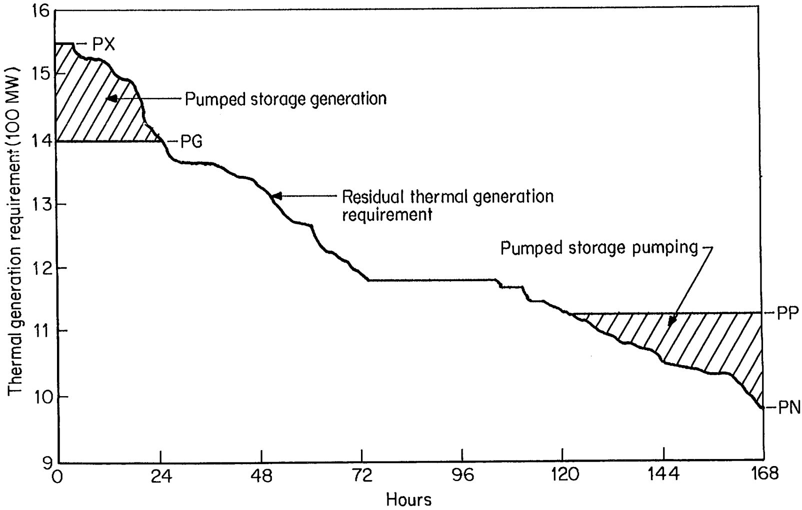


FIGURE 11.21 Generating and pumping levels of pumped storage contribution. *(From Giles [28].)*

bounds are illustrated in Fig. 11.21. The figure also shows a typical flattening of the residual load by generating on-peak and pumping off-peak.

The necessary maximum condition for B_s is $dB_s = 0$, which leads to

$$\frac{\partial B_s}{\partial \text{PG}} d\text{PG} = \frac{\partial B_s}{\partial \text{PP}} d\text{PP} \tag{11.20}$$

The partial derivatives of B_s with respect to PG and PP are expressed by the differentials $dF_g(\text{PG})/d\text{PG}$ and $dF_p(\text{PP})/d\text{PP}$. According to the fundamental theorem of calculus, they can be replaced by $f_g(\text{PG})$ and $f_p(\text{PP})$, respectively, which are the incremental thermal production costs at power levels PG and PP over the fraction of the period indicated by the load-duration curve, in \$/(MW · week). An increment of pumped-storage generation can be expressed by $d\text{PG}\ dt = \eta_s\ d\text{PP}\ dt$, with dt a unit of time. Thus, the optimality condition becomes

$$\eta_s f_g(\text{PG}) = f_p(\text{PP}) \tag{11.21}$$

According to Eq. (11.21), the maximum pumped-storage benefit is obtained when the incremental replacement cost at peak level PG (after pumped-storage generation has been deducted), multiplied by the pumped-storage cycle efficiency (about 0.72), is equal to the incremental cost at the highest pumping level PP (after pumped-storage pumping has been included). In other words, generation can replace residual thermal generation down to an incremental cost that is 1/0.72 = 1.38 times the incremental pumping cost. Figure 11.22 illustrates the optimal pumped-storage benefit. As long as the right side of Eq. (11.20) is larger than the left, the benefit is growing and more pumping is desirable. When the right side becomes less than the left, the benefit is decreasing, but can still be substantial before equality of replacement value and pumping cost is reached. The benefit turns negative for any further pumped-storage contribution.

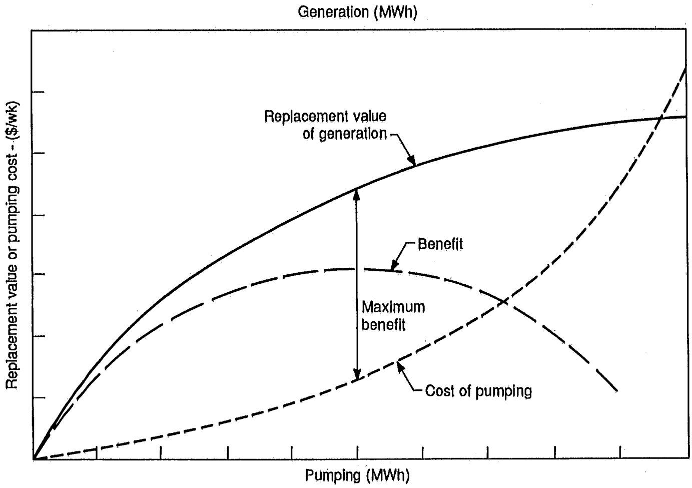


FIGURE 11.22 Maximum pumped-storage benefit.

An additional incentive for pumped-storage use, besides flattening the thermal load, are rapid response to load fluctuations and frequency control [26]. Shielding the thermal system from load fluctuations should result in reduced wear and tear on thermal units, reduced thermal ramping, and increased thermal unit availability in the long run. Therefore, pumped-storage benefits must not only be assessed by short-range power economy, but should include dynamic operating benefits and long-range overall system benefits, such as increased reliability increases as well [27–29].

System Cost Calculation. A basic assumption of the cost calculation is that steam-based thermal power resource carries the base load and slowly varying loads, while hydropower, as a fuel-limited resource, carries base load only to the extent that this is necessary, e.g., in order to meet minimum release requirements (because of lack of storage or other constraints), and deploys all other hydropower at times when its replacement value is highest. All hydropower projects that have temporary storage can contribute to meeting the rapidly varying loads, such as peak and shoulder loads, to spinning reserve and frequency control. An example of the allocation of available power resources to the generation requirement for a weekly period so as to maximally flatten residual thermal generation is shown in Fig. 11.23. In the right-hand portion of the load-duration curve, off-peak hydropower is seen to further reduce off-peak loads, as is also shown in Fig. 11.3. The deepened original load valleys are filled in by pumped-storage pumping.

The method of evaluating a hydropower schedule is summarized as follows:

1. Determine a hydropower schedule as a feasible reservoir level and release schedule (see also Secs. 11.4 and 11.5).
2. Form a hydropower system load-duration curve and subtract it from the total system load-duration curve (generation requirement) and smooth the residual thermal load-duration curve to make it monotonically decreasing.

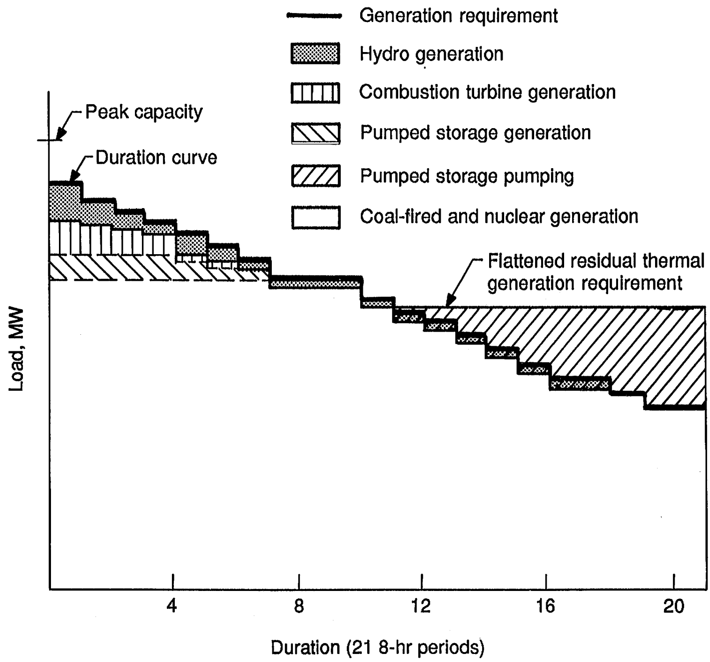


FIGURE 11.23 Allocation of available power resources to meet the system generation requirement for a weekly period.

3. Add the reserve requirement to the highest instantaneous load to obtain the "designated" load used for determining the unit contribution probability and the loss-of-load probability.
4. Compute the expected incremental cost function of the thermal system by summing for each incremental cost level the capacities of the units in the unit stack weighted by their contribution probability.
5. Compute the cost for each of the (21 eight-hour) time increments of the residual load-duration curve and sum them up to obtain the total power generation cost for the (weekly) time step.
6. Determine pumped-storage use by balancing the supplementary thermal generation cost for pumping on the right side of the load-duration curve with the replacement cost of thermal generation on the left. By repeatedly calculating the thermal system cost for pumped-storage generation and pumping, find the optimal pumped-storage contribution that satisfies all pumped-storage system constraints, such as installed capacity and water storage.

Finding an optimal reservoir system schedule requires repetitious applications of the hydropower schedule evaluation procedures to many hydropower schedules until the one that minimizes the system generation cost over the long run is found. Methods used for this search will be discussed in Secs. 11.4 and 11.5.

11.3 INPUT MODELS AND ROUTING MODELS

Hydropower planning and operations depend heavily on stream flow and load data. The input models that provide this information are used externally or interactively with system scheduling models. Flow-routing models for reservoirs and rivers that connect reservoirs are required for short-time-step models when they become a substantial part of the scheduling model. An overview of input and routing models will be given.

Input Models

Future system inputs and demands will never be known precisely in advance, but certainly nothing is gained by compounding uncertainty about the future with poor data acquisition, analysis, and forecasting techniques. Short-term and long-term prediction models should be based on state-of-the-art data acquisition and analysis programs. Advanced hydrologic data acquisition, collection, and analysis technology is available in the form of electronic and remote sensing, automated data collection, data transmission by means of terrestrial and satellite communications, and computerized data processing [30, 31].

Reliable input data prediction is water management's last frontier. Lack of reliable predictions thwarts reaping the benefits of modern system analysis techniques by not allowing sufficiently reliable scheduling, and, consequently, opportunities for more efficient operations are usually forgone. In existing systems, wasteful storage use often compensates for poor or no prediction efforts. Major stream-flow prediction errors are due to the usually sparse station network (100 and more square miles per station) and the resulting inaccurate areal rainfall estimates, and poor rainfall and runoff simulation methods. Remote-sensing techniques by radar and satellites, while achieving better areal coverage, are still subject to a wide variety of technical errors. While modest investments in better methods are often shunned, increased benefits and user satisfaction that could be derived from existing systems are forgone.

The prediction method used should make full use of available knowledge of the underlying process. Processes that are complex and not understood can only be modeled by so-called black box models which are based on statistical correlations between input and output data. These are the simplest models, which also produce the poorest results. Processes whose causal relationships are understood should be modeled by simulating the underlying physical process. Such models are also called "conceptual" models. A major disadvantage of black box models is their reliance on past records. Changes in the underlying process cannot be captured by these models. Also, they should be strictly used within their calibration range and for average conditions for which they are derived. Conceptual models, in contrast, have a better chance of producing valid predictions for unusual conditions and outside their calibration range. A major problem for any forecasting model is the heterogeneity of the process to which they are applied, such as the meteorologic differences, rainfall variations, and runoff variations across basins and, in the case of power loads, the complex interdependence among data and the great variety of factors that determine future electrical consumption.

Use of Historic Data. The use of historic data is attractive. Without additional modeling effort, it overcomes many of the just-mentioned problems but is not universally applicable over the long range. It is limited to planning for sites with records, if the underlying process can be considered stationary to the extent that it will repeat itself over the planning period. The preservation of time and space correlations among the same data types, as well as among time series of different types, has been an incentive for the use of historic data. In multireservoir systems, the time-sequential and spatial correlations of the stream flows are preserved by using historic data. Also, the electrical system load is correlated with air temperatures and air temperature is assumed to be correlated to stream flows. Therefore, if simulations are carried out for a stream-flow sequence, the associated air temperature in the basin is used to create a hydrologically adjusted power demand.

If there is a reasonable record length, as is the case for planning studies for existing systems, the use of existing data is an acceptable expedient if the record is not seriously biased toward extreme hydrologic regimes. One should keep in mind that multiyear persistent high- or low-flow regimes may be hidden in the record. Serious errors can result from such bias. The practice becomes questionable if unique events cause major changes in the underlying process, such as changes in climate or in the basin runoff conditions. Much discussed now are the potential changes in basin runoff through deforestation, irrigation, land use changes, etc., as well as the climate changes through the greenhouse effect due to atmospheric pollution, especially SO_2. The greenhouse effect may lead to changes in areal insolation and, subsequently, to changes in weather patterns [32, 33]. Studies to discern such events in existing records have been reported by Lettenmaier and Burgess [34]. An argument against the use of only the historic record is that it represents just one realization of many possible combinations of the historic data, so that only partial use is made of the available information [35].

In project studies for sites where no prior records exist, methods that can construct records from neighboring basin records or from usually more frequently available rainfall records and other hydrologic information must be used. Such methods are reviewed below.

Electric energy demand is an example of a process that is highly unstationary and strongly influenced by virtually unpredictable events. Simplistic use of such highly unstationary data as a basis for long-range project planning can have disastrous consequences in the form of inadequate designs.

Markovian Models. Markovian models are a class of statistical models that use past and present system states as a basis for predicting the future. A one-lag Markov model can generate an artificial record of arbitrary length given the present state of the system. However, such models also usually assume a stationary stochastic process and depend on the data sample as being representative of the true population.

A well-known model of this type is the autoregressive, one-lag, single-site model [36, 37]. This model is based on the assumption that the stream flow at a site consists of a deterministic and a random component. The deterministic component is correlated to the previous flow and the random component is a serially independent random fraction of the standard deviation:

$$x_i = m + \rho(x_{i-1} - m) + t_i\,\sigma(1 - \rho^2)^{0.5} \tag{11.22}$$

with x_i the predicted flow; m the data mean; ρ the lag-one serial correlation between x_i and x_{i-1}; σ the standard deviation of the data; and t_i a serially independent random variable, normally distributed with zero mean and standard deviation 1, also denoted as $N(0, 1)$. The lag-one serial correlation coefficient is an indicator of persistence between the given system state and the next, $\rho = 1$ meaning it will be the same, and $\rho = 0$ meaning a random transition.

The model represented by Eq. (11.22) applies to periods that have identical distributions, means, standard deviations, and serial correlation, such as annual periods. For shorter periods, such as months, these parameters may be different. An example of annual and quarterly average flow distributions for the Tennessee River at Chattanooga is shown in Fig. 11.24. While the distributions of the annual flows and the first-quarter average flows are approximately bell-shaped, the third- and fourth-quarter flows are distinctly biased toward low flows. Annual flows which can be construed as sums of four random quarterly flows again approach bell-shaped or normal distribution, regardless of the distributions of their respective populations (a manifestation of the central limit theorem, Sec. 11.2).

To account for seasonal parameter changes, the model can be modified and written in nondimensional form as

$$y_{ij} = \rho_j y_{ij-1} + t_{ij}\,\sigma_j(1 - \rho_j^2)^{0.5} \tag{11.23}$$

where $y_{i,j} = (x_{i,j} - m_j)/\sigma_j$, the normalized variable; $x_{i,j}$ is a variable within the season j; and ρ_j is the expectation of the correlation between seasonal flows.

This model has been extensively used for stream-flow data generation. It is only applicable to processes which are satisfactorily described by a single time lag correlation. The first term on the right-hand side is again the deterministic basis of the model. The random perturbation of this basis, as expressed by the second term on the right-hand side, may not be meaningful for too closely related periods because closely sequenced flows in large basins may not exhibit such random behavior. Burgess [35] suggests that the model could be used for forecasting monthly flows. Shorter time step flows can be determined by disaggregation [38–40]. Another requirement is that the flows must be jointly normally distributed. If they are not, data transformations, such as the logarithms of the data, must be applied to meet this requirement [35, 36].

Expansions of this model to multisite problems have been developed [35–37]. An n-site model can be written in vector form as

$$\mathbf{y}_{j+1} = \mathbf{A}\mathbf{y}_j + \mathbf{B}\mathbf{e}_{j+1} \tag{11.24}$$

where $\mathbf{y} = (x - m)/\sigma$ is the reduced stream-flow variable forming a column vector $n \times 1$, $\mathbf{A}$ and $\mathbf{B}$ are $n \times n$ matrices, and $\mathbf{e}$ is a column vector $n \times 1$ of mutually independent random numbers taken from a normal distribution; the **e**'s are assumed to be also independent of the **y**'s.

An application to six sites for weekly stream-flow generation was reported by Giles [41]. The weekly flows, which were originally skewed, were transformed as suggested by Burgess [42]. After substituting transformed stream-flow data for the **y**'s and generated random numbers for the **e**'s, the matrices

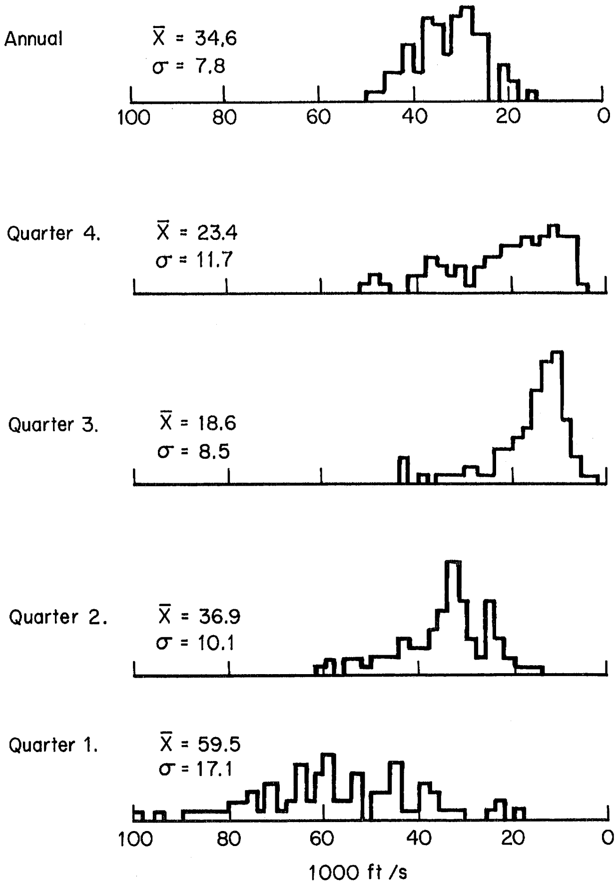


FIGURE 11.24 Annual and quarterly adjusted average flows—Tennessee River at Chattanooga (1903–1981; 79 years).

A and B can estimated. Based on a 70-year record, 200 years of synthetic weekly flows were generated for each site. The generated flows for the first year at two sites in neighboring basins are shown in Fig. 11.25. The weekly lag-one serial correlations for the first site, with a drainage basin of 703 square miles, are shown in Fig. 11.26. The weekly lag-0 spatial correlations between this basin and a neighboring basin (4541 mi^2) under a similar climate (southeastern Appalachians, about 90 mi apart) are shown in Fig. 11.27. These and all other statistical parameters were satisfactorily preserved in the generated time series. The coefficient of variation, the ratio of standard deviation to mean, was found to be about 0.7 in the first 20 weeks and about 1.0 thereafter.

Box-Jenkins Models. These models are generalized correlation models [43]. Typically, these models include multiple lags and multiple random terms in the form

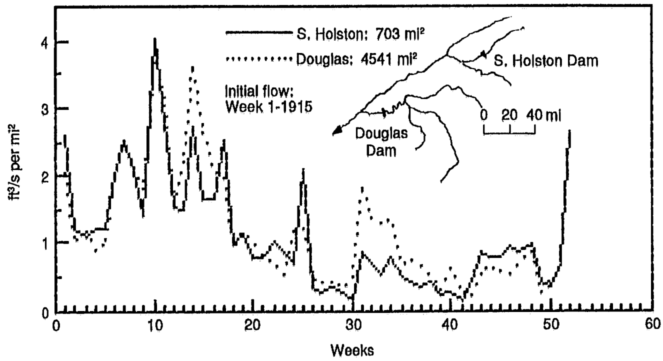


FIGURE 11.25 Synthetic runoff per unit area for neighboring drainage basins. *(From data by Giles [41].)*

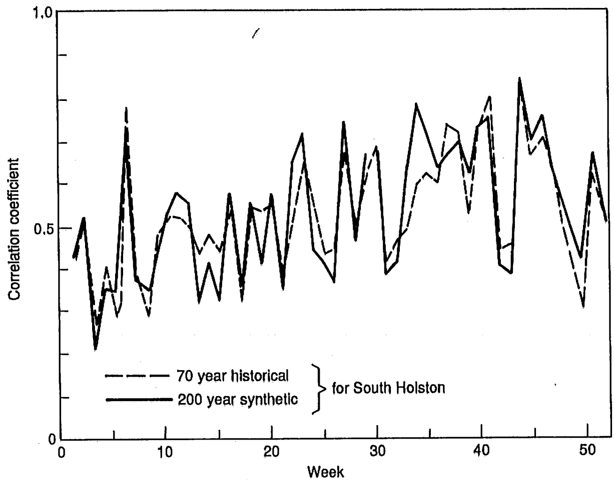


FIGURE 11.26 Synthetic and historical lag-one serial correlations. *(From Giles [41].)*

$$x_t - b_1 x_{t-1} - b_2 x_{t-2} - \cdots - b_p x_{t-p} = f_0 + a_t - f_1 a_{t-1} - f_2 a_{t-2} - \cdots - f_q a_{t-q} \quad (11.25)$$

On the left-hand side of Eq. (11.25) are the "autoregressive" terms and on the right-hand side are the "moving average" terms. A backshift operator B is defined as

$$Bx_t = x_{t-1} \qquad \text{and} \qquad B^j x_t = x_{t-j}$$

so that the model can be written as

$$(1 - b_1 B - b_2 B^2 - \cdots - b_p B^p) x_t = f_0 + (1 - f_1 B - f_2 B^2 - \cdots - f_q B^q) a_t$$

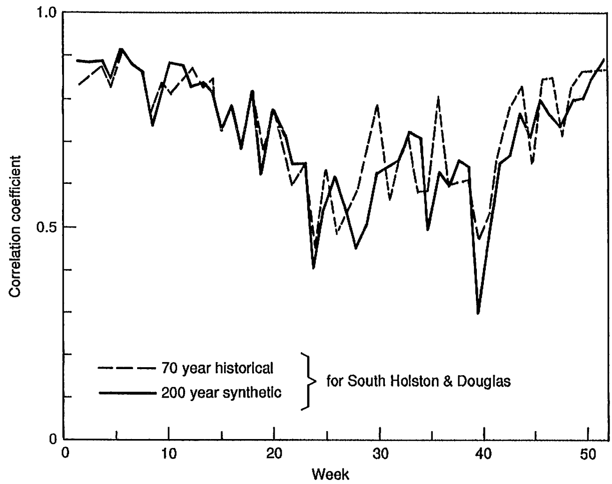


FIGURE 11.27 Synthetic and historical lag-0 spatial correlations. *(From Giles [41].)*

Such a model is called a mixed autoregressive–moving average process, or ARMA(p, q) process, with p being the highest order of the autoregressive backshift operator and q the highest order of the moving average backshift operator. In short notation this model can be written as

$$P_p(B)x_t = D_q(B)a_t$$

If the time series is nonstationary (due to trends), the series is differenced by computing a series of first, second, and higher differences, such as

$$V = x_t - x_{t-1}$$

and

$$V^2 = x_t - x_{t-1} - (x_{t-1} - x_{t-2}), \text{ etc.}$$

until stationarity is induced. The backshift operator $P(B)$ is then applied to the first or higher differences of x_t, depending upon the difference level at which stationarity is achieved. The resulting models are called autoregressive integrated moving average, or ARIMA(p, d, q) models, which are expressed by

$$P_p(B)V^d x_t = D_q(B)a_t \tag{11.26}$$

with d being the order of the differences of the original data.

Another model of interest is the transfer function (TFN) model, which relates two or more time series to each other,

$$N_1(B)y_t = N_2(B)x_t + N_3(B)a_t \tag{11.27}$$

where N_1 and N_2 are the time series of variables y_t and x_t, respectively, and N_3 is a white noise function.

Box-Jenkins models have also been extended to multivariate time series [44] and intervention analysis, where an unusual event causes an impact on the time series [45]. For a state-of-the-art review of Box-Jenkins modeling, especially extensions to multivariate (multisite) problems, see Hipel [46]. These extensions are especially important for the operational use of Box-Jenkins models, where additional knowledge about the near future may be available in the form of leading indicators, such as air temperature for load adjustment or rainfall for runoff prediction. Also, in many real world situations, more than one time series needs to be predicted simultaneously. Further extensions to modeling of nonstationary time series are possible, including nonlinear terms [47]. To facilitate the choice of the model, Hill and Woodworth [48] propose a pattern recognition procedure.

The potential pitfalls of these (black box) models must not be overlooked. These pitfalls are a result of the simplifications inherent in modeling complex physical processes exclusively by a sum of autoregressive and moving average components based on historic data. New developments in multi-time-series modeling which allow the inclusion of leading indicators may overcome some present limitations for real time use [46].

The comparison of a periodic autoregressive model, a transfer function model, and a conceptual model for quarter-monthly reservoir inflow forecasting has been described by McLeod et al. [49]. Good performance was reported of the transfer function model. It predicts basin runoff as a linear combination of a rainfall and a snowmelt component.

Statistical Rainfall and Runoff Models. Conventional graphical or numerical multivariate regression has been used to model the rainfall and runoff process as a basis for stream-flow forecasting. A major difficulty with modeling this process is the heterogeneity of the input and of the rainfall to runoff conversion throughout the basin. This complex process is modeled as a black box, which operates on present rainfall, past rainfall, and time of the year to produce runoff. It is applied to subbasins, assuming homogeneity throughout the subbasin. This approach is exemplified by the API model [50–52]. The API (antecedent precipitation index) is a weighted sum of past rainfalls. It is used to determine a runoff index for a particular time of the year from a family of week-number curves established for the basin or subbasin. The runoff index is then used to convert observed or predicted precipitation into runoff. This model has also been extended to include parameters such as snowpack water equivalent, areal snow cover, and time of the year. It also includes a groundwater component. Another example of such a model is the Soil Conservation Service model [53].

These models lack adaptability to special runoff conditions. This shortcoming is compensated for to some extent by skilled users through subjective parameter choice, such as using a week number for a week that actually has another number. The problem with these models is that they can represent only average system behavior and fail whenever major departures from such average conditions occur. This usually happens in extraordinary situations [54], when reliable stream-flow prediction is especially important, such as in floods or droughts. Nevertheless, they are still widely used in operational stream-flow prediction [55].

Conceptual Rainfall and Runoff Models. The conceptual approach to stream-flow forecasting attempts to model the various components of the rainfall-runoff process, such as surface runoff, infiltration, subsurface runoff, groundwater storage, evaporation, and evapotranspiration extraction from various groundwater storage zones. This approach became possible with computer technology. The

Stanford model was the first of this kind of runoff model [52, 56]. In principle, this approach is more desirable than the black box approach because the model can keep track of important hydrologic parameters such as soil moisture that are important for basin water management including groundwater.

These models are suitable for planning and operational use. In operational applications, the runoff models, calibrated for the usually numerous subbasins of a large basin, receive their input from point rainfall observations (or forecasts) which are transformed into areal rainfall. In planning applications, this process can be simulated by substituting the real-time rainfall forecast by some kind of rainfall generator [57].

Many variations of the Stanford model have been developed over the years. The Kentucky model is an adaptation of the Stanford model to eastern U.S. hydrologic conditions [52]. A widely used adaptation is known as the Sacramento model [58]. It is part of the National Weather Service River Forecast System (NWSRFS) and has been applied to watersheds in eastern and western basins of the United States [54]. A disadvantage of these models is the need for calibrating a large number of parameters, for example, as many as 20 in the Sacramento model. But basin calibrations can be obtained from the NWS, and adaptations to neighboring basins are possible. A recent addition to this family of models is the nine-parameter Large Basin Runoff Model (LBRM) by Croley and Hartmann [59].

The conceptual models have not demonstrated clear superiority over the multivariate correlation models [60]. While they are more consistent in their performance, users prefer to hang on to the simpler methods [54, 55, 57]. As long as rainfall input into the model remains a source of major prediction error, the full advantages of advanced models cannot be reaped. A review of runoff modeling methods can be found in [61].

Load Forecasting. Load forecasting must be broadly based and should be carried out using submodels that are linked to broader national and regional socioeconomic models that analyze trends in many areas, such as population growth, consumer habits, per capita income, industrial and residential uses, human behavior, technological advances, other energy resource developments, energy price, and foreign politics [62]. This problem is of a probabilistic nature and forecasts must produce a distribution of energy consumption and peak loads. The treatment of this complex subject goes beyond the scope of this chapter. Further information can be found in Munasinghe [63] and Sullivan [64]. The models used for relating the various factors with energy consumption are of the multivariate black box type.

The evolution of system generation and required peak load (not necessarily to be met by the system) for a southeastern U.S. utility from 1955 to 1985 is shown in Fig. 11.28. The growth trend of more than 20 years was interrupted in the late 1970s to give way to no growth at all or even a decline. Planning major long-range investments based on simple electrical growth trends may be an invitation to disastrous mistakes. Load management and consumer guidance through education and pricing mechanisms can create completely new load shapes within a few years. An example is the elimination of the diurnal load cycle over a period of 5 years in the city of Hamburg, as illustrated in Fig. 11.29 [65]. Long-range forecasts of the electrical demand over the next 30 years remains an unabashed challenge. Short-term load forecasts for a year are straightforward, usually based on historic data and short-term trends and adjustments for weather.

An application of an ARIMA model to short-term operational load forecasting

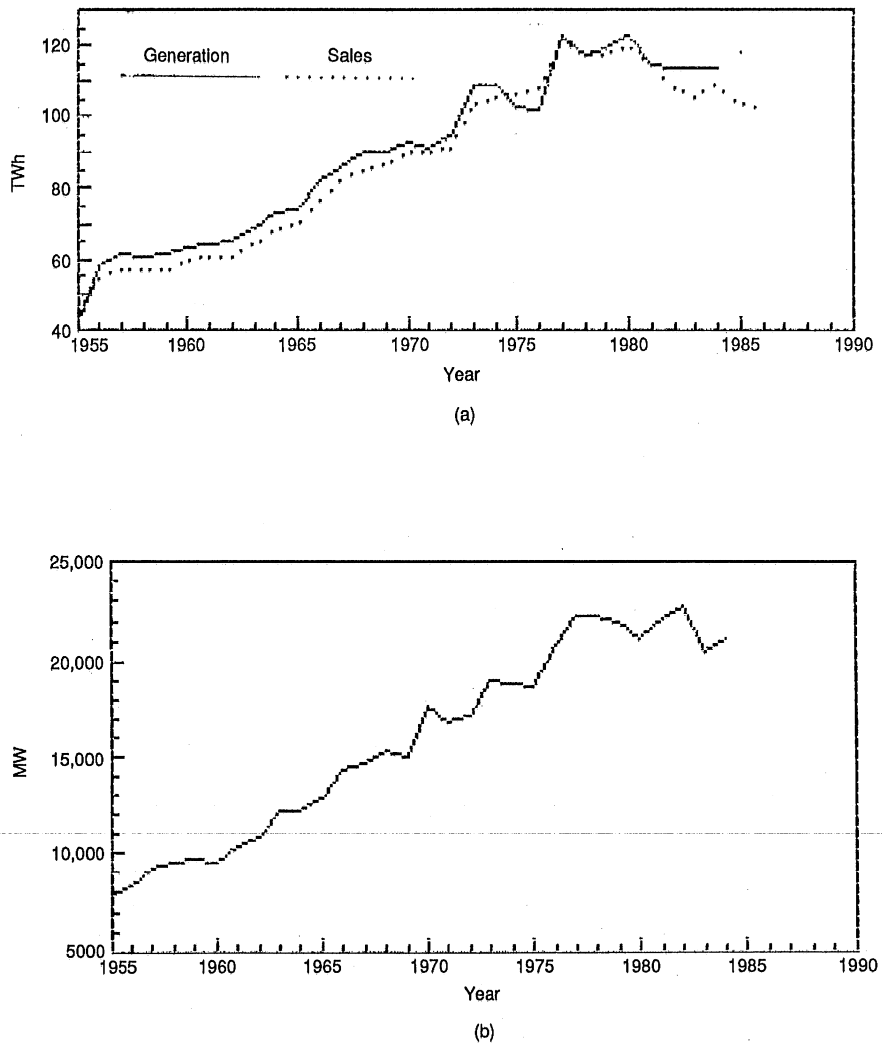


FIGURE 11.28 Evolution of system generation and peak requirement of a Southeastern U.S. utility over a 30-year period. (*a*) Total annual system generation and sales. (*b*) Net system peak requirement.

for the Electricité de France power system [66] is illustrated in Fig. 11.30. The load series is transformed into a stationary series. The optimal model is based on only a few terms of past loads and on a few terms of past prediction errors. The model parameter optimization is based on minimizing the sum of squared prediction errors of the chosen model. An application to half-hourly load forecasting over a 24-h planning period for two days with different temperatures is shown in the figure. The authors claim a reduction in the standard error of the prediction

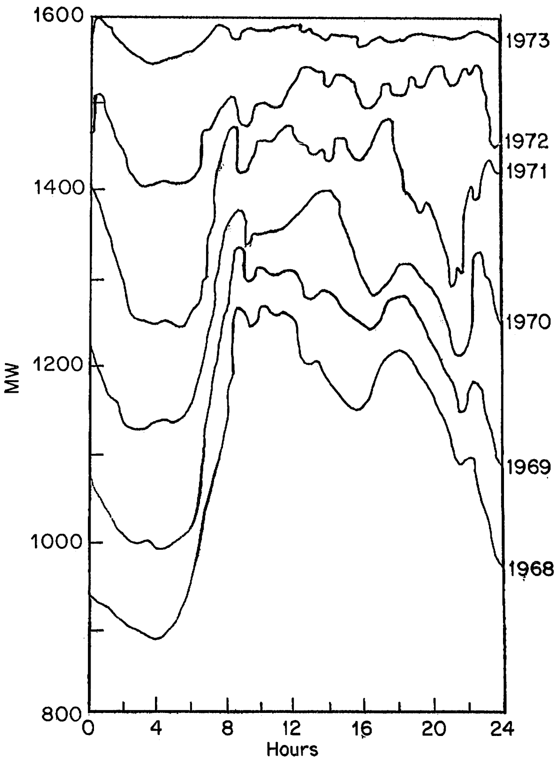


FIGURE 11.29 Elimination of daily load cycle over a 5-year period through load management—City of Hamburg. *(From Troescher [65].)*

by 15 percent through the use of the model. Only one temperature is used for adjusting the load in the entire load supply area (the whole of France). The standard error reduction corresponds to 2 percent of the daily load. This amounts to hundreds of megawatt-hours that would otherwise have been assigned to reserve units, requiring more units to operate at inefficient power levels. Further improvements may be possible by regionalized forecasting.

Flow Routing

The usual on/off operation of hydropower plants to meet fluctuating loads causes rapidly varying flows in downstream rivers and reservoirs. The power output calculations of Sec. 11.2 assume that the tailwater elevation always corresponds to the discharge. Actually, when turbine discharge changes, the tailwater responds over a finite time period and not instantly to the new discharge. Unsteady flow observations and calculations have shown that the tailwater response is quite fast, but at a declining rate, so that a steady-state level may actually not be reached during the entire multihour operation period that follows.

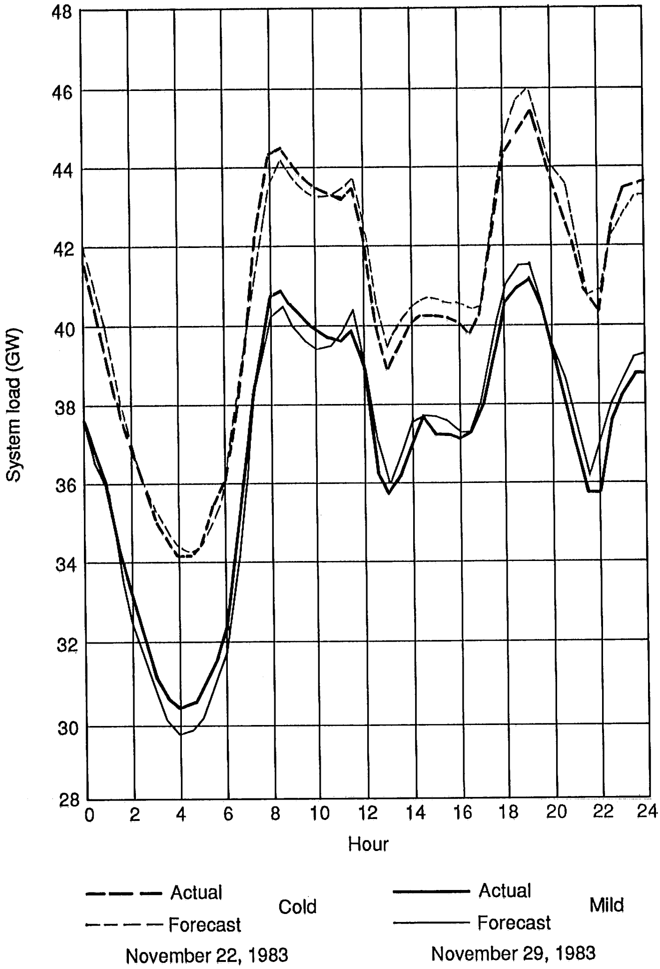


FIGURE 11.30 Daily load adjustment to air temperature—Electricité de France System. *(From Garrigues et al. [66], p. 17.)*

An example of a tailwater response to discharge pulses is given in Fig. 11.31. The upper portion of the figure shows the time-varying inflows and outflows of a reservoir over several days that are produced by power operations at both ends of the reservoir. The water-level responses in the project's tailwater and at the next dam 44 miles downstream are also shown. For discharge variation between 15,000 and 44,000 ft^3/s, the tailwater fluctuation is about 3 ft. It can be seen that most of the tailwater rise occurs rather immediately following the discharge change.

Considerable work has been done to develop methods for accurate stage and discharges predictions along rivers and reservoirs under unsteady flow regimes. The most accurate descriptions of unsteady flow are the Saint-Venant

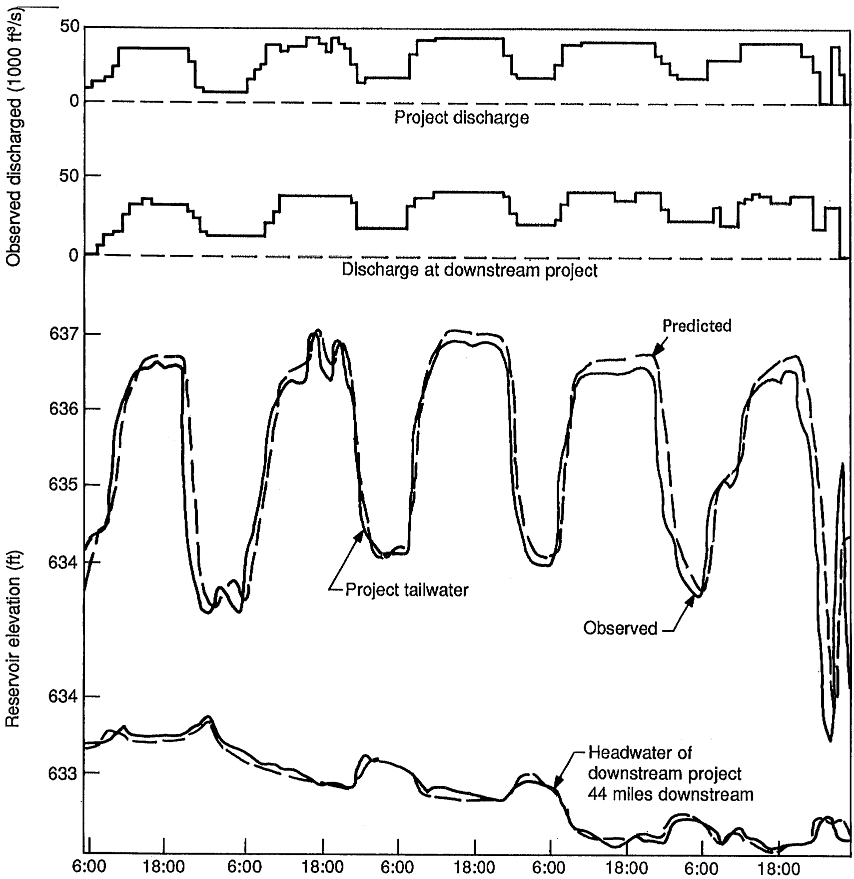


FIGURE 11.31 Calculated reservoir levels using Saint Venant equations and comparison with observations. *(From Goranflo [70].)*

equations [67]. Computer technology has made it possible to simulate unsteady flow by solving these equations for extended distances and time periods. A simulation over some 300 reservoir miles and a 168-h planning period within a few minutes on a minicomputer was reported by Goranflo [30]. A summary of work on unsteady flow in open channels can be found in Mahmood and Jevjevich [68].

Saint-Venant Equations. Numerous mathematical models exist that take various computational short cuts to flow routing. But the state-of-the-art approach to unsteady flow simulation is the solution of the Saint-Venant equations. The Saint-Venant equations express conservation of mass and equilibrium of forces for a control volume in the form of two partial differential equations. For river flow routing in connection with hydropower operations, one-dimensional models are satisfactory which simulate flow and water level over time and distance. Usually reservoir cross sections are irregular and the equations are written in terms of changes of discharge Q and cross section area A, as shown below.

conservation of mass (or volume):

$$\frac{\partial Q}{\partial x} + q + \frac{\partial(A + A_0)}{\partial t} = 0 \tag{11.28}$$

equilibrium of forces (in flow direction):

$$\frac{\partial Q}{\partial t} + \left[\frac{\partial Q}{\partial x}\right]\frac{dx}{dt} = gAS - gAS_f - gA\frac{\partial h}{\partial x} + qv_x \tag{11.29}$$

Equation (11.28) states that the change in flow passing through the control volume plus the change in volume must be zero; the first term represents the change in river flow across the reach, the second is the lateral inflow, and the third is the volume change of the reach. Equation (11.29) states that the change in momentum of the moving mass, in time and space, expressed by the left side, is equal to the sum of the external forces acting on the water mass of the reach in or opposite to the direction of the flow; the forces on the right side of Eq. (11.29) are the weight component in flow direction, the friction resistance, the pressure difference, and the momentum contribution of the lateral inflow; Q is the flow rate; q is the lateral inflow and v_x its velocity component in the x direction; A is the flow cross section and A_0 is a lateral storage cross section which has zero flow velocity; S is the channel slope and S_f is the friction slope, calculated by Manning's formula with Q^2 replaced by $Q|Q|$, the product of Q and the absolute value $|Q|$, to account for the possible action of the resistance force in both upstream and downstream directions by flow reversal; h is the piezometric pressure head or water surface elevation in the cross section. Typical n-values for river channels can be found in Chow [67].

For the special case of steady nonuniform flow, the time differentials are zero and Eq. (11.29) reduces to an ordinary differential equation in x. With $(dQ/dx)(dx/dt) = (Q/A)(dQ/dx) = 0.5\, d(Q^2/A)/dx$, and zero lateral inflow, one obtains

$$d(v^2/2g) - (S - S_f)dx + dh = 0 \tag{11.30}$$

and the steady-state form of the conservation of mass equation,

$$d(vA) = 0 \tag{11.31}$$

with $v = Q/A$, the flow velocity.

Equation (11.30) is the formula for steady nonuniform flow. The results of steady nonuniform reservoir routing can be represented in the form of wedge storage plots. An example of such a plot is shown in Fig. 11.32. For flat pool, the upstream project's tailwater elevation (Watts Bar) is equal to the project's (Chickamauga) headwater elevation, and upstream inflow is zero. As upstream inflow increases, the upstream tailwater elevation rises above the project's headwater elevation and wedge storage is built up under a sloping water surface. For example, for a headwater elevation of 680 ft (227 m) and an upstream discharge of 40,000 ft^3/s (1100 m^3/s), the intersection of headwater elevation and inflow curve gives a tailwater elevation of 683 ft (228 m) at Watts Bar on the ordinate and a reservoir storage of 276,000 day-ft^3/s (7900 day-m^3/s) on the abscissa. These plots are the basis for the tailwater functions f(QP) in Sec. 11.2. For rapidly varying flows, the quasi-steady volume estimation using this diagram is, however, only

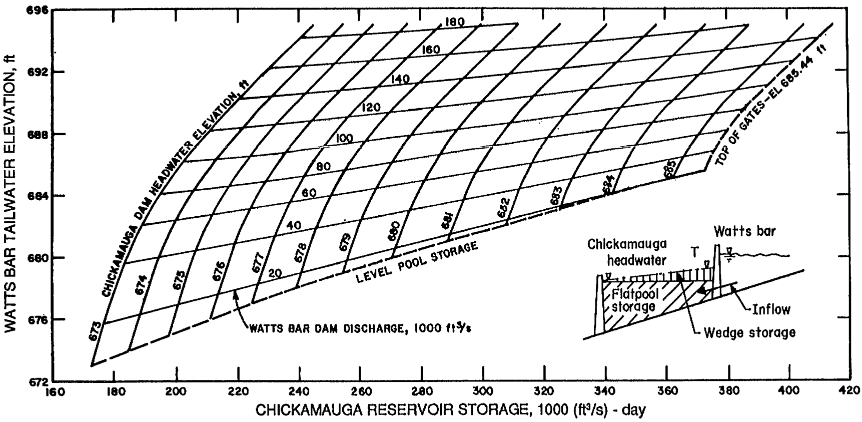


FIGURE 11.32 Storage as function of headwater and inflow for steady nonuniform flow.

approximate. Accurate storage accounting for scheduling requires integration of the cross-sectional areas under the instantaneous water levels along the reservoir, computed by the Saint Venant equations.

Numerical Solutions. The numerical solution of the Saint Venant equations requires that the partial differentials be replaced by finite differences. The x-t plane is subdivided by grid lines parallel to the t axis, which represent cross sections, and parallel to the x axis, which represent time instants. Gradients in the x direction are replaced by differences between grid-point values in the x direction divided by the grid-point distances, and gradients in the t direction are replaced by differences between grid-point values in the t direction divided by the time interval. The river stretch to be routed is divided into n reaches, which produces $n + 1$ grid points for each time step. Usually the values of the variables at all (x, t) grid points for $x = 1$ and $x = n + 1$ are referred to as boundary conditions, and those for $t = 0$ are referred to as initial conditions. Various methods have been described for computing the variables at the grid points [30, 69–73].

Muskingum Method. An approximate routing method is the Muskingum method. This method uses the continuity equation and an empirical relationship between flow and river reach volume to calculate the discharge and storage change of the reach [51, 52, 67, 74].

Many applications of this method can be found in the literature. A well known one is the SSARR model for the Columbia River system [75]. Cunge [76, 77] has given a critical review of the method. If correctly applied, the Muskingum method can produce satisfactory results, particularly for planning use. This applies to routing of free-flowing rivers. It cannot be expected to give reliable results in impounded rivers because it is a "downstream" method.

Flow Lagging. Flow lagging is a way of accounting for the travel time of water between reservoirs or dams when time delays are large compared to the time step used in scheduling. In the case of daily flow routing, the inflow into reservoir i on day j, I_{ij}, can be represented by

$$I_{ij} = L_{ij} + \sum_{k=1}^{K} (a_k U_{kj} + (1 - a_k) U_{kj-1}) \tag{11.32}$$

with L_{ij} being the local flow that arrives in reservoir i on day j; U_{kj} is the release from an upstream reservoir k on day j; a_k is the fraction of day j's release and $1 - a_k$ is the fraction of the previous day's release arriving at reservoir i on day j. The sum is taken over all K upstream neighbors of reservoir i. By summing Eq. (11.32) over several time steps, it is seen that the error in volume transfer is the difference of the delayed flows of the last day of the period and the day before the beginning of the period. For volume transfer in weekly time step models, this difference is usually small compared to the weekly flow and no flow lagging is usually required.

11.4 OPTIMAL USE OF HYDROPOWER RESOURCES

Optimization Objectives

The interest in the optimal use of hydropower is a longstanding one. Single-unit best-efficiency operation has been discussed in Sec. 11.2. Interest in system optimization has steadily increased over the past 30 years. Creager and Justin [78] state that interconnecting major generating plants has emphasized the need for operating the plants in a system context instead of as individual plants. A simulation model that seeks a weekly multireservoir schedule using a search based on balancing current incremental thermal replacement value and long-range incremental operation cost was described by Brudenell and Gilbreath [79]. A dynamic programming approach was described by Bernholtz and Graham [80]. Historic reviews have been given by El-Hawary and Christensen [4], Wood and Wollenberg [11], and Yeh [9]. Reports on development and use of hydropower system models can be found in periodic publications, for example the biennial *Water Power* symposium volumes [81, 82].

Hydropower may serve as primary supplier of energy in a hydroelectric system but more often it is an intermittent peak power and energy supplier in a hydropower-thermal system. Correspondingly, the operational objective is

1. Maximizing total energy output by operating at best-efficiency power level, or
2. Maximizing the hydropower benefit by concentrating hydropower use in periods when its thermal generation replacement value is highest.

The first objective attempts to meet the load with minimum expense of raw energy. This is equivalent to minimizing water use and maintaining as high as possible energy-in-storage levels in the reservoirs. This objective is also equivalent to minimizing unserved energy (see Sec. 11.6). The second objective maximizes the value of replaced thermal power and energy and is equivalent to residual thermal generation cost minimization in the context of a hydropower-thermal system. Optimizing both objectives at the same time is usually not possible, as best-efficiency operation does not occur at maximum power output (see Sec. 11.2).

Classical Optimization Methods

Unconstrained Optimization. Optimization is a technique that finds variables x^* that extremize a stated (objective) function $f(x)$. Classical optimization deals with finding the optima (extreme points) of unconstrained functions or of equality-constrained functions. Equality constraints are side conditions that further describe the underlying problem. Usually they must be satisfied for optimality to be meaningful. The classical optimization methods are all special cases of the more general modern optimization methods that can also handle inequality constraints [83].

Usually the functions dealt with are multivariable functions or vector functions, $f(\mathbf{x}) = f(x_1,x_2,\ldots,x_n)$, with $x_1,x_2,\ldots,x_n$ being the components of the variable in form of a vector **x**. The solution **x*** is also a vector, expressed here as the transpose of a row vector,

$$\mathbf{x}^* = (x_1{}^*,x_2{}^*,\ldots,x_n{}^*)' \qquad i = 1,\ldots,n$$

Transposing means rearranging rows into columns and columns into rows for the purposes of vector calculus. The function may or may not have an extremum in the given range of the vector $\mathbf{x} = (x_1, x_2,\ldots, x_n)'$. For example, if a minimum exists at **x***, then for all **x**, at least in a small vicinity of **x***,

$$f(\mathbf{x}^*) < f(\mathbf{x}) \tag{11.33}$$

In this case, at least a local minimum exists. If Eq. (11.33) holds for the entire range of **x**, then $f(\mathbf{x}^*)$ is also a global minimum. Similar terminology applies for a maximum.

The necessary condition for an extremum is that in **x*** the gradient of $f(\mathbf{x}^*)$ vanishes or,

$$\nabla f(\mathbf{x}^*) = 0 \tag{11.34}$$

with $\nabla f(\mathbf{x})$ being the column vector of the first (partial) derivatives of $f(\mathbf{x})$, expressed as the transpose of the row vector of the partial derivatives,

$$\nabla f(\mathbf{x}) = \left(\frac{\partial f(\mathbf{x})}{\partial x_1},\ldots,\frac{\partial f(\mathbf{x})}{\partial x_n}\right)' \tag{11.35}$$

In order to also determine if the extremum is a maximum or a minimum, the sufficient condition must be met, which for a minimum requires that the second derivative be greater than zero. For a multivariate function, this is a somewhat involved procedure. Suffice it here to say that the sufficient condition for a minimum is

$$\mathbf{z}'\mathbf{H}\mathbf{z} > 0 \tag{11.36}$$

with **z′** and **z** being row and column vectors, respectively, of small increments of **x**, and **H** is the matrix of second-order partial derivatives, known as the Hessian.

One of the best-known practical applications of classical optimization is the least-square fit. Its objective function is the sum of all squared errors. Equality constraints in the form of the function to be fitted are written for each observation and substituted into the objective function. This function is then minimized as an unconstrained function. An important lesson about optimization can be learned from this example. Optimization does not necessarily provide the best solution for every subproblem, i.e., it does not provide the best fit for each point, only overall is the objective optimized.

Method of Lagrange. Only in a simple case can side conditions be substituted into the objective function. When this is not possible, the method of Lagrange can be used to append the side conditions to the objective function. Let

$$\min f(\mathbf{x}) \tag{11.37}$$

subject to $g_i(\mathbf{x}) = 0, i = 1,\ldots,m$, where $g_i(\mathbf{x}) = 0$ are the equality constraints and $\mathbf{x}$ is a column vector of n variables, with $m < n$. The method of Lagrange includes the side conditions in an expanded objective function and then solves this so-called Lagrangian as if it were an unconstrained function [84]. The Lagrangian for the above problem is

$$L(\mathbf{x},\boldsymbol{\lambda}) = f(\mathbf{x}) + \sum_{i=1}^{m} (\lambda_i \; g_i(\mathbf{x})) \qquad i = 1,\ldots,m \tag{11.38}$$

with λ_i and $i = 1,\ldots,m$, the Lagrange multipliers. The necessary extremum conditions are

$$\nabla L(\mathbf{x}^*,\boldsymbol{\lambda}^*) = 0, \tag{11.39a}$$

from which the $\mathbf{x}^*$ and the $\boldsymbol{\lambda}^*$ can be calculated. For the sufficient conditions for a local minimum, see Avriel [83].

The Lagrangian method is the basis of many modern extensions, specifically in the area of nonlinear programming, with nonlinear objective functions and with linear and nonlinear inequality constraints, as well as in hierarchical optimization where large system models are decomposed into submodels and coördinated by the use of Lagrange multipliers [84].

EXAMPLE 11.3 *Multiunit Hydropower Plant Optimization.*

Several hydropower units operate in a plant under a constant head H to meet the generation requirement P. The objective is to minimize the discharge while meeting P [after 85]. The objective function is

$$\min \sum_{i=1}^{N} Q_i$$

subject to

$$\sum_{i=1}^{N} P_i = P$$

where N is the number of units; Q_i and P_i are the discharge and power output of the individual units, and P is the plant generation requirement. Minimizing the discharge while meeting a load requirement is equivalent to maximizing efficiency. Assuming a three-unit plant ($N = 3$), the Lagrangian is

$$L = Q_1 + Q_2 + Q_3 - \lambda(P_1 + P_2 + P_3 - P)$$

The necessary minimum conditions are the derivatives of L versus Q_i and λ, set to zero

$$\begin{aligned} 1 - \lambda \frac{\partial P_1}{\partial Q_1} &= 0 \\ 1 - \lambda \frac{\partial P_2}{\partial Q_2} &= 0 \\ 1 - \lambda \frac{\partial P_3}{\partial Q_3} &= 0 \\ P_1 + P_2 + P_3 &= P \end{aligned} \tag{11.39b}$$

For H = constant, an empirical polynomial can be fitted to the P versus Q curves, similar to the ones shown in the lower portion of Fig. 11.6:

$$P_i = a_{0i} + a_{1i}Q_i + a_{2i}Q_i^2 + a_{3i}Q_i^3 \qquad i = 1, 2, 3$$

The derivatives of the Lagrangian then become

$$1 - \lambda(a_{1i} + 2a_{2i}Q_i + 3a_{3i}\, Q_i^2) = 0 \qquad i = 1, 2, 3$$

and

$$\sum_{i=1}^{N} (a_{0i} + a_{1i}Q_i + a_{2i}Q_i^2 + a_{3i}Q_i^3) = P$$

The last two expressions represent four equations in the three Q_i's and in λ, which are solved for the three unknowns Q_i and the Lagrange multiplier λ. Note that the necessary optimality conditions, expressed by Eq. (11.39*b*), require the incremental power outputs, $\partial P/\partial Q$, to be the same for all units, i.e., $\partial P_i/\partial Q_i = 1/\lambda$.

Calculus of Variations. The calculus of variations seeks the function $y = y(x)$ that, when integrated between two points, $y_1 = y(x_1)$ and $y_2 = y(x_2)$, extremizes the integral over this range. Therefore, $y = y(x)$ is also called the extremizing function [86]. This is exactly the problem encountered in power system operations planning where the cost function over the planning period is sought whose integral is the minimum cost. An application of the variational principle to power system optimization is given by Kirchmayer [3].

Modern Optimization Methods

Constraints and Objective Functions. In contrast to classical optimization, modern optimization can deal effectively with constraints and, especially, inequality constraints. The optimization of a linear function, which is the purpose of linear programming, would be actually meaningless without constraints, because a straight line can only have an extremum (or optimum) at a lower or upper limit, and an inclined plane can have an optimum only along a boundary or in a corner point, where two or more boundaries intersect. An example of such a linear op-

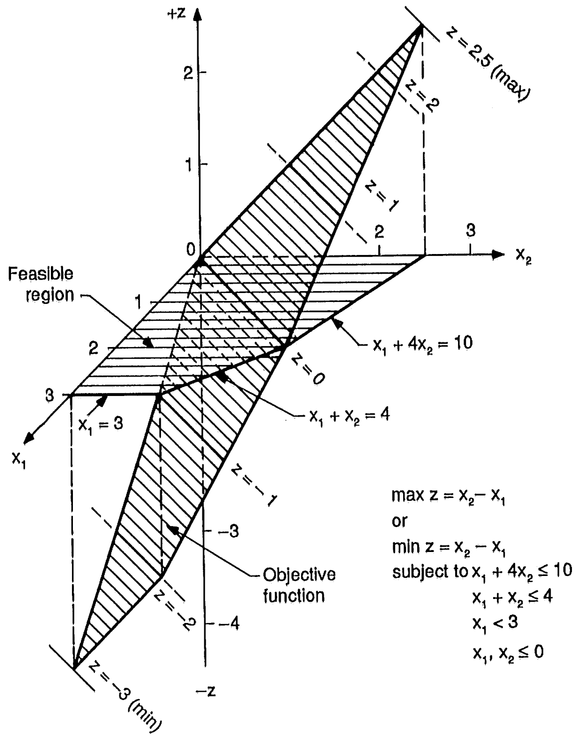


FIGURE 11.33 Feasible region and objective function values for maximization or minimization by linear programming.

timization problem is shown in Fig. 11.33. It shows a linear objective function in the form of a plane as it assumes maximum or minimum values above or below the confines of the feasible area delineated by the constraints in the x_1,x_2 plane.

Linear Programming. Linear programming (LP) epitomizes modern optimization methods [5, 87–90]. It is a very well developed algorithm that can handle very large sets of equality and inequality constraints. When selecting a method, it is always advisable to consider linearization of the problem so as to make use of this algorithm. The standard LP formulation is

$$\begin{aligned} &\text{Maximize } z = \mathbf{c}'\,\mathbf{x} \\ &\text{Subject to } \mathbf{Ax} = \mathbf{b} \qquad \mathbf{x} \geq 0 \end{aligned} \tag{11.40}$$

where **c** and **x** are column vectors having n components; **b** is a column vector having m components, and **A** is a matrix with m rows and n columns. The objective function $z(\mathbf{x})$, whose extremum is being sought, as well as the set of constraints $Ax = b$, are linear in the variables $x = x_1, \ldots, x_n$. As mentioned above and as is illustrated in Fig. 11.33, it is meaningful to speak of an extremum only if the linear objective function is bounded. Therefore, in all LP problems, the extrema are located on the boundaries and the search for the extremum can be limited to a search over the intersections of the constraints (vertices of the feasible region), which indicates a limitation of the method.

To solve Eq. (11.40), the matrix **A** is partitioned into matrices **B** and **N**,

$$\mathbf{A} = (\mathbf{B}, \mathbf{N})$$

where **B** is an $m \times m$ (square) matrix. Correspondingly, the vector **x** is partitioned into

$$\mathbf{x} = (\mathbf{x}_B, \mathbf{x}_N)$$

where $\mathbf{x}_B$ and $\mathbf{x}_N$ are the two vectors corresponding to the partitioned matrix **A**. Hence, the constraint set becomes

$$\mathbf{B}\mathbf{x}_B + \mathbf{N}\mathbf{x}_N = \mathbf{b}$$

This equation can be solved for x_B by

$$x_B = \mathbf{B}^{-1}\mathbf{b} - \mathbf{B}^{-1}\mathbf{N}\mathbf{x}_N \tag{11.41}$$

where $\mathbf{B}^{-1}$ denotes the inverse of matrix **B**. The vector $\mathbf{x}_B$ is called the vector of basic variables, while $\mathbf{x}_N$ is called the vector of nonbasic variables. The basis is a matrix that has as many rows as it has columns and, thus, allows a unique solution for the basic variables $\mathbf{x}_B$ except for special cases. The solution method, also known as the simplex method, works exclusively with solutions to the basis while moving variables into and out of the basis until the desired solution is found.

A basic solution is obtained by setting all nonbasic variables equal to their bounds; here $x_N = 0$. This produces

$$\mathbf{x}_B = \mathbf{B}^{-1}\mathbf{b}$$

With this solution, the objective function is tested. For this purpose, the column vector **c** is partitioned to match the partitioning of **x**. This produces

$$\mathbf{c}'\mathbf{x} = (\mathbf{c}_B', \mathbf{c}_N')(\mathbf{x}_B, \mathbf{x}_N) = \mathbf{c}_B'\,\mathbf{x}_B + \mathbf{c}_N'\,\mathbf{x}_N$$

Replacing $\mathbf{x}_B$ by Eq. (11.42) leads to

$$\mathbf{c}'\mathbf{x} = \mathbf{c}_B'(\mathbf{B}^{-1}\mathbf{b} - \mathbf{B}^{-1}\mathbf{N}\mathbf{x}_N) + \mathbf{c}_N'\,\mathbf{x}_N$$

and, after multiplying out,

$$\mathbf{c}'\mathbf{x} = \mathbf{c}_B'\mathbf{B}^{-1}\mathbf{b} - (\mathbf{c}_B'\,\mathbf{B}^{-1}\mathbf{N} - \mathbf{c}_N')\,\mathbf{x}_N \tag{11.42}$$

Since the nonbasic variables are $\mathbf{x}_N = 0$, the value of the objective function is

$$\mathbf{c}'\mathbf{x} = \mathbf{c}_B'\mathbf{B}^{-1}\mathbf{b} \tag{11.43}$$

The second term on the right-hand side of Eq. (11.42) is an indicator of where to move next, along the constraints, as an element of $\mathbf{x}_N$ is changed. As we want to maximize $\mathbf{c}'\mathbf{x}$, if

$$\mathbf{d}_N' = (\mathbf{c}_B'\,\mathbf{B}^{-1}\mathbf{N} - \mathbf{c}_N') > 0 \tag{11.44}$$

for a component of $\mathbf{x}_N$, then this component is a candidate for moving into the basis, while a current basic variable must leave the basis. Equation (11.41) can be used to determine which basic variable becomes zero first as a component of $\mathbf{x}_N$ increases above its lower bound, here zero. This procedure is repeated, changing the basis whenever d_N' becomes negative. When d_N' becomes nonnegative, the optimal solution of Eq. (11.40) has been found and the basis updating is discontinued.

It is useful to know the incremental values of the objective function. Under the nondegeneracy assumption [91], it can be assumed that a small change of b does not change the basis $\mathbf{B}$ in Eq. (11.43). Then, the derivative of the objective function, Eq. (11.43), with respect to the right-hand sides of the constraints is

$$\frac{\partial \mathbf{z}}{\partial \mathbf{b}} = \mathbf{c}_B'\mathbf{B}^{-1} = \mathbf{p}' \tag{11.45}$$

and, with

$$\mathbf{z} = \mathbf{p}'\mathbf{b} = \sum_i p_i\, b_i \tag{11.46}$$

the partial derivatives of the objective function are

$$\frac{\partial z}{\partial b_i} = p_i$$

The value p_i is called the "shadow price" of resource b_i. It indicates the change in the objective function caused by a small change in the right-hand side of the ith constraint.

The Dual of Linear Programming. Solving an LP problem by the method of Lagrange sheds some more light on LP as well as on the Lagrange method. Forming the Lagrangian of the LP described by Eq. (11.40) yields

$$\max L(\mathbf{x},\boldsymbol{\lambda}) = \mathbf{c}'\mathbf{x} - \boldsymbol{\lambda}'(\mathbf{A}\mathbf{x} - \mathbf{b}) \tag{11.47}$$

The necessary extremum condition $\nabla L(\mathbf{x}^*,\lambda^*) = 0$ requires

$$\nabla_x L = \mathbf{c}' - \lambda^{*\prime}\mathbf{A} = 0 \qquad \text{and} \qquad \nabla_\lambda L = \mathbf{A}\mathbf{x}^* - \mathbf{b} = 0$$

Substituting $\mathbf{c}' = \lambda^{*\prime}\mathbf{A}$ into Eq. (11.47) yields

$$L(\mathbf{x}^*,\boldsymbol{\lambda}^*) = \boldsymbol{\lambda}^{*\prime}\mathbf{A}\mathbf{x}^* - \boldsymbol{\lambda}^{*\prime}\mathbf{A}\mathbf{x}^* + \boldsymbol{\lambda}^{*\prime}\mathbf{b} = \boldsymbol{\lambda}^{*\prime}\mathbf{b} \tag{11.48a}$$

Also, the derivative of L versus $\boldsymbol{\lambda}$ at optimality, $\mathbf{A}\mathbf{x}^* - \mathbf{b} = 0$, substituted into Eq. (11.47) yields

$$L(\mathbf{x}^*,\boldsymbol{\lambda}^*) = \mathbf{c}'\mathbf{x}^* - \boldsymbol{\lambda}^{*\prime}(\mathbf{A}\mathbf{x}^* - \mathbf{b}) = \mathbf{c}'\mathbf{x}^* \tag{11.48b}$$

which shows that the Lagrangian at optimality is equal to the extremized objective function of the original (primal) linear problem. Hence, for each primal problem, there exists an equivalent problem, the dual, which for the above primal can be stated as

$$\text{minimize } \mathbf{y}_0 = \boldsymbol{\lambda}'\mathbf{b} \tag{11.49}$$

subject to $\mathbf{A}'\boldsymbol{\lambda} = \mathbf{c}$, with $\mathbf{y}_0$ being the dual objective function and $\mathbf{A}'$ the transpose of $\mathbf{A}$. If the Lagrangian of Eq. (11.49) is formed by using $\mathbf{x}$ as the column vector of Lagrange multipliers, the primal can be reconstructed by observing that $\mathbf{x}'\mathbf{c} = \mathbf{c}'\mathbf{x}$ and $\mathbf{x}'\mathbf{A}'\boldsymbol{\lambda} = \boldsymbol{\lambda}'\mathbf{A}\mathbf{x}$, as

$$L = \boldsymbol{\lambda}'\,\mathbf{b} - \mathbf{x}'\,(\mathbf{A}'\,\boldsymbol{\lambda} - \mathbf{c}) = \mathbf{c}'\mathbf{x} - \boldsymbol{\lambda}'\,(\mathbf{A}\mathbf{x} - \mathbf{b})$$

with the final expression on the right-hand side being Eq. (11.47). From Eqs. (11.46), (11.48*a*), and (11.48*b*) one can see that at optimality

$$\mathbf{z}^* = \mathbf{c}'\mathbf{x}^* = \mathbf{p}^{*\prime}\mathbf{b} = \boldsymbol{\lambda}^{*\prime}\mathbf{b},$$

so that

$$\lambda_i^* = p_i^* \qquad \text{for all } i$$

This shows that, at optimality, the Lagrange multipliers are the shadow prices of the resources. One unit change in resource i affects the optimized objective function by the amount λ_i^*. This is also the marginal (incremental) value of the resource. The objective of the dual, Eq. (11.49), can be interpreted as minimizing the cost of the resources.

Goal Programming. Goal programming is a special LP approach that satisfies constraints sequentially [92–94]. If all constraints cannot be met simultaneously, then, in order to find a solution, the most important ones are met first at the expense of violating lower-ranked ones. In case of violations, minimum adjustments are forced onto constraints of lesser importance until all constraints have been satisfied or adjusted in order to achieve feasibility. Assume a single constraint or a set of constraints has the general form

$$\mathbf{A}\mathbf{x} \le \mathbf{b} \tag{11.50a}$$

To satisfy this constraint, it is rewritten in the form

$$\mathbf{A}\mathbf{x} + \mathbf{d} = \mathbf{b} \tag{11.50b}$$

If the constraint is to be met, the deviation variable $\mathbf{d}$ must be greater than or equal to zero. To test the constraint, the following LP is solved:

$$\min \mathbf{d}$$

$$\text{subject to } \mathbf{A}\mathbf{x} + \mathbf{d} = \mathbf{b}, \qquad \mathbf{A}_1\mathbf{y} \le \mathbf{b}_1$$

where $\mathbf{A}_1\mathbf{y} \le \mathbf{b}_1$ are constraints that have been already satisfied and which are not subject to further adjustment. Any necessary constraint adjustment in order to overcome infeasibility must come from relaxing the right-hand side $\mathbf{b}$ of the constraint equation or equations being examined. If a solution

$$\min \mathbf{d} \ge 0$$

is found, the constraint $\mathbf{A}\mathbf{x} \le \mathbf{b}$ is feasible and will, without adjustment, be included in the set $\mathbf{A}_1\mathbf{y} \le \mathbf{b}_1$.

If $\min \mathbf{d} < 0$, then the constraint cannot be satisfied. The right-hand side of constraint, Eq. (11.50*a*), is then adjusted to

$$\mathbf{A}\mathbf{x} = \mathbf{b} - \min \mathbf{d} \tag{11.51}$$

This means that the right-hand side, $\mathbf{b}$, is increased by the amount $\mathbf{d}$ so that the constraint can be met. Then, the next constraint or set of constraints in the priority listing of constraints is considered in a similar way, with the previously adjusted constraint incorporated into the set $\mathbf{A}_1\mathbf{y} \le \mathbf{b}_1$. The constraints treated in this fashion can be single constraints or a whole set of constraints if they have the same priority.

If constraints are satisfied in this way, without giving simultaneous consideration to adjustments of other constraints, the constraints are said to be preemptively satisfied. This procedure is appealing because it allows some control over constraint violations if a feasible solution is otherwise not obtainable. It has, however, weaknesses. For example, it may satisfy an arbitrarily higher-ranked constraint at the expense of lower-ranked constraints. Or, while fully satisfying a higher-ranked constraint, it may cause a major violation of a lower-ranked constraint [94]. Experimenting with the approach is necessary to find a ranking that is acceptable [95].

Nonlinear Programming. Hydropower problems are typically nonlinear in the objective function and also in the constraints. Among other things, this is due to the fact that head, which is representative of the state of the system (dependent variable), and discharge, which is the decision variable (independent variable), appear in product form whenever capacity or generation occurs.

The search for the extremum of a nonlinear, unconstrained function can be facilitated by the use of Newton's method. Any function $f(\mathbf{x})$, in the vicinity of a point x_0, can be approximated by a Taylor series expansion

$$f(\mathbf{x}) = f(x_0) + f'(x_0)(x - x_0) + 0.5f''(x_0)(x - x_0)^2 \tag{11.52}$$

with x_0 being a point in the vicinity of the extremum; $f'(x_0)$ and $f''(x_0)$ are the first and second derivatives of the function in x_0, i.e., they are constant coefficients. The necessary extremum condition for Eq. (11.52) is $f'(\mathbf{x}) = 0$, which produces

$$f'(x_0) + f''(x_0)(x - x_0) = 0$$

The improved location of the extremum is found by solving for $x = x_1$. Expanding this concept to multivariate problems is described in [83].

A general nonlinear programming (NLP) problem that is nonlinear in the constraints as well as in the objective can be formulated according to Murtagh and Saunders [96] as

$$\min\ (F(\mathbf{x}) + \mathbf{c}'\mathbf{x} + \mathbf{d}'\mathbf{y}) \tag{11.53}$$

$$\text{subject to } f(\mathbf{x}) + \mathbf{A}_1\mathbf{y} = \mathbf{b}_1, \qquad \mathbf{A}_2\mathbf{x} + \mathbf{A}_3\mathbf{y} = \mathbf{b}_2 \qquad \boldsymbol{\ell} \le \begin{pmatrix}\mathbf{x}\\ \mathbf{y}\end{pmatrix} \le \mathbf{u}$$

where $\mathbf{x}$ is a column vector of "nonlinear variables" with n_1 components; $\mathbf{y}$ is a column vector of "linear variables" with n_2 components; $\mathbf{c}$, $\mathbf{d}$, $\mathbf{b}_1$, $\mathbf{b}_2$, $\boldsymbol{\ell}$, and $\mathbf{u}$ are column vectors of objective function coefficients, right-hand sides, and lower and upper bounds; $\mathbf{A}_1$, $\mathbf{A}_2$, and $\mathbf{A}_3$ are matrices of the linear portions of the constraint sets; $F(\mathbf{x})$ is a smooth nonlinear portion of the objective function, and $f(\mathbf{x})$ is a vector of smooth nonlinear constraint functions. If $F(\mathbf{x})$ and $f(\mathbf{x})$ are absent, the problem reduces to an LP.

In the absence of $f(\mathbf{x})$, the problem is nonlinear in the objective function only. In this case, the solution proceeds by partitioning the constraints into

$$\mathbf{B}\ \mathrm{x}_\mathrm{B} + \mathbf{S}\ \mathrm{x}_\mathrm{S} + \mathbf{N}\ \mathrm{x}_\mathrm{N} = 0 \tag{11.54}$$

with x_B the basic variables, x_S the superbasic variables, and x_N the nonbasic variables. The latter ones are at their bounds at any solution point, while the basic and superbasic variables are somewhere in their range at a solution point. The number of superbasic variables determines how nonlinear the problem is. They are free to move to improve the objective function or to reduce infeasibility.

In a general-purpose implementation of NLP, Murtagh and Saunders [96] use a projected augmented Lagrangian algorithm based on a method of Robinson [97] (see also Avriel [83]). The augmented Lagrangian is the vector of functions $f(x)$ and its matrix of first derivatives, the Jacobian matrix, are evaluated at the starting point x_0 but rarely thereafter, so that the nonlinear constraints may be violated during the solution process. A similar solution technique has been described by Chuang et al. [98]. An application of NLP to a multiunit hydroplant optimization is described in Sec. 11.5.

$$L = F(\mathbf{x}) + \mathbf{c}'\mathbf{x} + \mathbf{d}'\mathbf{y} - \lambda' (\mathbf{f} - \mathbf{f}_l) + r\,(\mathbf{f} - \mathbf{f}_l)'\,(\mathbf{f} - \mathbf{f}_l) \tag{11.55}$$

with $\mathbf{f}_1$ being the vector of linearized constraint functions and r a coefficient on a quadratic penally term.

Kuhn-Tucker Condition. The optimality conditions for optimizing constrained functions, specifically inequality-constrained functions, can be related to the concept of the Lagrangian. While there are other methods for transforming inequalities into equalities, e.g., by the classical method of introducing slack variables [11, 84], such methods may cause weakening of certain results [83].

Kuhn and Tucker [7] presented necessary conditions for inequality-constrained mathematical programs which restrict the constraint functions by what are called constraint qualifications, which assure the existence of positive Lagrange multipliers at the solution.

Given the general formulation,

$$\min f(\mathbf{x}) \tag{11.56}$$

$$\text{subject to } g_i(\mathbf{x}) \geq 0 \qquad i = 1,\ldots,m$$

$$h_j(\mathbf{x}) = 0 \qquad j = 1,\ldots,p \tag{11.57a}$$

the Lagrangian can be written as

$$L = f(\mathbf{x}) - \sum_{i=1}^{m} \lambda_i\, g_i(\mathbf{x}) - \sum_{j=1}^{p} \mu_i h_j(\mathbf{x})$$

and, at optimality,

$$\partial f(\mathbf{x}^*) - \sum_{i=1}^{m} \lambda_i^*\ \partial g_i(\mathbf{x}^*) - \sum_{j=1}^{p} \mu_i^*\ \partial h_j(\mathbf{x}^*) = 0 \tag{11.57b}$$

$$\lambda_i^*\, g_i(\mathbf{x}^*) = 0 \qquad \lambda_i^* \geq 0$$

Equation (11.57*b*) represents the Kuhn-Tucker conditions. The product λ_i^* $g_i(x^*) = 0$ at optimality requires that either λ_i^*, $g_i(x^*)$, or both must be zero. If the constraint is exactly met, then $\lambda_i^* \geq 0$, otherwise $\lambda_i^* = 0$. The Kuhn-Tucker conditions provide only necessary extremum conditions. A solution that satisfies all conditions must be found by testing the various possibilities. Examples are given in [11] and [84].

EXAMPLE 11.4 *Thermal power generation cost minimization.*

Assume that for meeting the residual load of a hydrothermal system, three coal-fired units are available. They must generate at least the expected demand P plus the network losses, PL [after 11]. The Lagrangian of the problem is

$$\min \sum_{i=1}^{N} C_i$$

$$\text{subject to} \sum_{i=1}^{N} P_i = P + \text{PL}$$

The incremental heat rates of the three units can be expressed by

$$H_1 = 510 + 7.20P_1 + 0.00142P_1^2$$

$$H_2 = 310 + 7.85P_2 + 0.00194P_2^2$$

$$H_3 = 78 + 7.97P_3 + 0.00482P_3^2$$

where H is in MBtu/h (1 MBtu/h = 0.293 MW), and P is the output (capacity) level of the unit, in MW. Assuming a production cost of $18/MWh ($5.27/MBtu), the heat rate functions are converted into cost functions by multiplying all three equations by $5.27/MBtu,

$$C_1(P_1) = 5.27(510 + 7.20\,P_1 + 0.00142P_1^2)$$

$$C_2(P_2) = 5.27(310 + 7.85\,P_2 + 0.00194P_2^2)$$

$$C_3(P_3) = 5.27(78 + 7.97\,P_3 + 0.00482P_3^2)$$

with all costs, $C_i(P_i)$, in $/h. The load that the units must meet is the generation requirement, P plus the system losses,

$$P + \text{PL} = P_1 + P_2 + P_3$$

with $P = 850$ MW. The unit capacities are constrained by upper and lower bounds:

$$150 < P_1 < 600 \text{ MW},\ 100 < P_2 < 400 \text{ MW},\ 50 < P_3 < 200 \text{ MW}$$

The system line losses are represented by the sum of line losses of all units,

$$\text{PL} = 0.00003P_1^2 + 0.00009P_2^2 + 0.00012P_3^2$$

with each additive term being the loss, PL_i, incurred by unit i. The Lagrangian of the problem is

$$L = C_1(P_1) + C_2(P_2) + C_3(P_3) - \lambda(P_1 + P_2 + P_3 - P - \text{PL})$$

The necessary minimum conditions are

$$\frac{\partial L}{\partial P_1} = 5.27(7.20 + 2 \cdot 0.00142\,P_1) - \lambda(1 - 2 \cdot 0.00003\,P_1) = 0$$

$$\frac{\partial L}{\partial P_2} = 5.27(7.85 + 2 \cdot 0.00194\, P_2) - \lambda(1 - 2 \cdot 0.00009\, P_2) = 0$$

$$\frac{\partial L}{\partial P_3} = 5.27(7.97 + 2 \cdot 0.00482\, P_3) - \lambda(1 - 2 \cdot 0.00012\, P_3) = 0$$

$$\frac{\partial L}{\partial \lambda} = P_1 + P_2 + P_3 - 850 - (0.00003P_1^2 + 0.00009P_2^2 + 0.00012P_3^2) = 0$$

This system of four equations needs to be solved for the load levels P_i of the units within their maximum and minimum capacity constraints and the Lagrange multiplier λ. The generation requirement must be met while minimizing total cost. With an initial feasible solution for P_1, P_2, and P_3 (e.g., their sum equals P), the set of nonlinear equations can be solved by a Newton iteration method. The solution obtained is

$$P_1 = 551.3 \text{ MW} \qquad P_2 = 220.5 \text{ MW} \qquad P_3 = 92.7 \text{ MW}$$

$$\text{PL} = 14.5 \text{ MW} \qquad P + \text{PL} = 864.5 \text{ MW} \qquad \lambda = 47.8 \text{ \$/MW}$$

This solution is within the feasible ranges of the units' capacities. If P_1 has an upper limit of 500 MW, P_1 is tentatively set to that limit and the solution recalculated without the derivative $\partial L/\partial P_1$, treating P_1 as a constant. The general approach is to introduce an additional constraint $P_1 \leq 500$, and append the term $\mu_1(P_1 - 500)$ to the Lagrangian with an additional Lagrange multiplier and satisfy the Kuhn-Tucker condition $\mu_1(P_1 - 500) = 0$. The new solution is

$$P_1 = 500 \text{ MW} \qquad P_2 = 255.4 \text{ MW} \qquad P_3 = 109.4 \text{ MW}$$

$$\text{PL} = 14.8 \text{ MW} \qquad \lambda = 48.8 \text{ \$/MWh}$$

The three units produce 864.8 MW, 850 MW for meeting the system load, and 14.8 MW for the transmission losses. The Lagrange multiplier with the constraint on unit 1 binding is greater than the Lagrange multiplier for the nonbinding constraint. Without the restriction, the cheaper unit 1 was loaded to 551 MW. The constraint on unit 1 increases the marginal cost of meeting the load from \$47.8/MW to \$48.8/MW.

Network Methods. Reservoir systems can be construed as networks with sparse linkages between nodes (reservoirs or channel junctions) in space to downstream neighbors and in time between storage nodes. The three-dimensional network of a dynamic system consists of a repetition of the spatial system for each time step. Figure 11.35 illustrates such a network. Releases are variables within a time step and storages are variables of water transfer to the next time step; run-of-river reservoirs or river junctions do not transfer water in time. The set of continuity equations for such a space-time system is characterized by a sparse matrix.

Network algorithms, such as the out-of-kilter algorithm (OKA) [99], that deal efficiently with sparse matrices have been applied to reservoir system scheduling [100–102]. Applications to hydropower scheduling over multiple time periods are only of interest if the problem of foresight can be properly dealt with (see Sec. 11.5). In such cases, the nonlinearities pose problems [101].

Dynamic Programming

Dynamic programming (DP) is particularly suited for the optimization of sequential processes. Such processes allocate a finite resource over time or to several available units within a time step. A typical example of a time-sequential process is the allocation of a limited hydropower resource to the current week and future weeks, so as to maximize its contribution to meeting the load over the long run, as illustrated by Fig. 11.2.

Principle of Optimality. The numerical integration of an a priori unknown function over a planning period, such that the integral is extremized (the problem of the calculus of variations), is made computationally feasible by Bellman's principle of optimality [6, 103] which states, "An optimal policy has the property that whatever the initial state and the initial decision are, the remaining decisions must constitute an optimal policy with regard to the state resulting from the first decision."

Based on this principle, the integral over a planning period can be split into two parts, the integral over the first time step and the integral over the remaining time steps. According to the principle of optimality, if the integral over the remaining time steps is already optimized with respect to its starting state, then all that remains to be done is to find the optimal sum of the current time step's return plus the already optimized return for the remaining time steps. Thus, the multi-time-step decision problem that basically has a rapidly expanding tree structure is reduced to a series of single-time-step decision problems.

Deterministic Backward DP. The usual computational implementation of the principle of optimality uses backward DP. This approach moves from the end of the planning period backward in time toward the beginning of the planning period. As it moves backward in time, it builds up the optimized returns for the remaining decisions. Counting backward in time, beginning with $n = 1$ as the last time step in the planning period, the optimal value of the state level i at the beginning of stage n is defined as [104]

$$f(n, i) = \max_{k \text{ in } K} [r(n,i,k) + f(n - 1, j)] \tag{11.58}$$

where (n, i) is the level i of the state variable at the beginning of stage n; $k = 1,\ldots,K$ are the actions that transform the state variable level i into j at the end of stage n; $r(n,i,k)$ is the transition return associated with the transition from state (n,i) to $(n - 1, j)$ under action k; and $f(n - 1, j)$ is the optimal value associated with the state level j at the end of stage n, as determined by a previous optimization. For reservoir operations, the transition function $j = g(n,i,k)$ is the continuity equation which determines a new level j for a given starting level i, an inflow during stage n, and a release decision k. The maximization is performed by evaluating, for each starting state, all actions k and retaining the one that produces the maximum return.

For the last stage, which is the first stage of the backward computation, Eq. (11.58) is

$$f(1,i) = \max_{k \text{ in } K} [r(1,i,k) + f(0,j)]$$

usually with $f(0,j) = 0$, for lack of better knowledge. This is also equivalent to assuming that $f(0,j)$ has the same value for all ending state levels, with no discrimination between state levels. As the procedure moves backward in time, a few "warm-up" stages are usually required to build up a representative end-of-stage function, $f(n - 1, j)$.

Deterministic Forward DP. Equation (11.58) can also be used in a forward procedure. There, the counting proceeds forward in time, with $f(n, i)$ being associated with the ending state level of the current stage, and $f(n - 1, j)$ with the starting state level. The forward procedure has the advantage that one works forward in the direction of time. In the deterministic case, forward and backward methods produce the same result.

DP is based on the assumption that only the present decision influences future decisions, in other words, that the process is memoryless. It is important that the time step length be selected in such a way that this requirement is not violated. For example, water released at an upstream project must be available at the downstream project within the time step, otherwise releases one or more time steps back in time would influence the current decision.

DP with Discounting. If DP is used over long planning periods, discounting may become necessary. In the backward DP formulation, a discount factor is applied to the end-of-stage return,

$$f(n,i) = \max_{k \text{ in } K} [r(n,i,k) + qf(n - 1,j)] \tag{11.59}$$

The discount factor, $q = 1/(1 + d)$, discounts the return incurred at the end of the stage to the beginning of the stage. By discounting the end-of-stage returns in each step of the backward procedure, numerical discounting is accomplished over the entire planning period, with only the current stage remaining undiscounted.

In forward DP, the discount factor is applied to the stage return, while the return at the beginning of the stage, $f(n - 1,j)$, represents present worth,

$$f(n,i) = \max_{k \text{ in } K} [q^{n-1} r(n,i,k) + f(n - 1,j)] \tag{11.60}$$

As the time count starts with $n = 1$, the first stage return and the return at the beginning of the first stage, $f(0,j)$, (usually assumed 0) are not discounted.

The discounting formulas show that discounting reduces the value of future benefits or the cost of future actions. This leads to the tendency of scheduling for maximum present returns at the expense of future returns. Such policies that favor present returns at the expense of future returns are called "greedy." In the real world, such policies are risky. If forecasts see plenty of resources in the future, present resources may be used wastefully, as no need is seen for being frugal. If the forecast turns out wrong, high costs for the future will result (see also "Foresight and Shortsight in Scheduling" in Sec. 11.5). This also holds for project planning applications, where high discount rates diminish future returns and make projects with large present returns economically preferable to projects that have high future returns.

Probabilistic Dynamic Programming. For the solution of probabilistic problems, it is important to determine where the probabilistic characteristics of the system reside [89]. If a probabilistic problem can be reduced to a deterministic one, it can be more easily solved. This is especially important in multivariate problems, such as a multiproject hydropower system. In the case discussed here, the probabilistic aspects reside partly in the system itself and partly in the inputs and demands. This situation is typical for a hydropower-thermal system. In a pure hydropower system, the system uncertainty is practically zero and uncertainty is due only to future water supply and demand. In a mixed hydropower-thermal system, the probabilistic aspects residing in the system are due to the thermal unit forced outages. The system uncertainty is included in the probabilistic power cost calculation. The input and demand uncertainty is included by a deterministic sequential evaluation of many stream-flow and demand scenarios, with foresight limited to one stage at a time. Such a computational scheme is described by the following Markov decision process formulation [104].

$$f(n,i) = \sum_{h=1}^{H} \{p(n,h)(\max_{k \text{ in } K} [r(n,i,k,h) + f(n-1,j)])\} \tag{11.61}$$

where (n, i) is the level i of the state variable at the start of stage n [or a predetermined combination of state variable levels in a multistate variable (reservoir) problem]; k is the action or path from level i of the state variable at the beginning of stage n to a new level j of the state variable at the end of stage n (or a set of paths in a multivariate problem from a combination of beginning levels to a combination of ending levels); there is assumed to be a total of K such actions; $j = g(n,i,k)$ is the transition function from state (n,i) to state $(n-1,j)$ under action k; $r(n,i,k,h)$ is the return associated with the transition from state (n,i) to state $(n-1,j)$ under action k for the flow realization h; $f(n-1,j)$ is the value associated with state $(n-1,j)$ that has been determined by a previous computation step; $p(n,h)$ is the probability associated with the probabilistic input h (stream flow) in stage n. In case a probability distribution of flows is not known, equal likelihood can be given to each input used, which amounts to replacing $p(n,h)$ by $1/H$, with H being the number of hydrologic input sequences (flow years) used in the evaluation of Eq. (11.61).

The calculation starts with the last week of the planning period and proceeds backward in time toward the beginning of the planning period. For the levels of the state variable at the beginning of the last week of the planning period, which is the first stage, $n = 1$, several actions are evaluated, such as staying at that same level, or moving to levels $j = 1$, 2, or 3. The feasibility of these moves depends on the available inflow. Feasible actions may produce a zero or positive release. The resulting releases and ending levels are used to calculate the stage returns for all possible actions under the given inflows. The expected returns that are associated with the starting state levels of stage $n = 1$ are calculated as the probability-weighted sums of the maximum returns for each of the possible inflows, as expressed by Eq. (11.61). The values associated with each state level produce the return function $f(n-1,j)$ for the next stage. This procedure is repeated backward in time for each stage of the planning period.

The probabilistic end-of-stage return function cumulates the expected return (or cost) over the planning period as a function of the state variable used. The gradient versus the state variable of this function, for example, in terms of $/MWh (hydropower energy in storage), represents the expected incremental return (or cost) associated with the ending-state variable level. An example of using a similar procedure in an LP and DP system scheduling model is described in Sec. 11.5.

Some salient features of the above-described probabilistic DP procedure are pointed out. The probabilistic end-of-stage return functions can only be obtained for a global or lumped state. When applied to multivariate problems, some disaggregation of this lumped state must be embedded in the procedure to assure that the guide function obtained is meaningful. A one-step forward scheduling procedure depends on the accuracy of this function. The assumption of $f(0,j) = 0$ at the end of the last week causes an invalid policy for that week, recommending emptying the reservoirs as there is no incentive to keep water in storage. This quirk is rapidly overcome after the procedure has moved backward in time through a few warm-up stages. While total returns (or costs) continue to mount as the procedure moves through the stages, the cost differences between state levels show a rather stable behavior. It is these incremental values that provide the guiding function in the one-step forward applications, while the base cost is constant for all state levels and has no effect on the calculated policy.

Multiple-Objective Optimization. If several objectives compete for resource allocation, the problem can be formulated in the most general way as a vector optimization problem in which the objective function is a vector of objective functions. The components of the objective function vector, $f_i(x_i)$, are linear or nonlinear functions of the decision variable vector x. The functions are not necessarily in the same units. Power generation cost is measured in dollars, system reliability may be measured in probability of loss of load or energy unserved, etc. Deciding on the best possible multipurpose operation requires value judgments in some form, such as a priority ranking of objectives, weighing of objectives, or expressing them in a specific form, such as goals. A large body of literature is available on the subject [9, 84, 94, 105–115].

11.5 APPLICATIONS OF OPTIMIZATION METHODS

Hierarchical Analysis of Systems

Hierarchical analysis of systems is an approach to modeling very large systems. The idea is to decompose a large system into subsystems and to solve each of the subsystems independently but coordinated with each other through coordination variables. This approach is also called the decomposition-coordination method. By iteratively varying the coordination variables and solving the subproblems, the optimal solution for the total system is found. This approach has been described by Hall et al. [111] and Haimes [84] as decomposition and multilevel optimization. An application to a large-power-system dispatch problem has been described by Sandrin [116] and Merlin et al. [117]. Monti et al. [118] described an application to pumped-storage planning.

The submodels typically arise from constraint sets that are coordinated with a system objective function by Lagrange multipliers which serve as coordination variables. An important feature of the decomposition-coordination approach is that it allows coordination of several submodels without requiring homogeneous treatment of the entire problem. In other words, each subproblem can be dealt with independently with its simulation and optimization methods selected to best suit the problem. This makes hierarchical analysis a well-suited approach for modeling real-world power systems, which typically consist of a thermal subsystem, a hydropower subsystem, an electrical network subsystem, a pumped-storage subsystem and environmental effects subsystems.

Optimal Multiunit Plant Loading

When a multiunit hydropower plant is called upon to meet a certain load, the operator is faced with two actions: (1) to select the units to be used in the operation, and (2) to load the selected units so that the plant efficiency is maximized. The first action is called unit commitment and the second economic dispatch. In the application here, it is assumed that the unit commitment problem has been solved and the set of units to carry the load has been identified. What remains to be done is to economically load (dispatch) the units. Here, the objective is to maximize plant efficiency so that the required load is met with a minimum expenditure of water. Thus, more water is available for additional power production. Actually, it may not be feasible to operate all committed units at best efficiency. Some units may have to be operated at higher and some at lower than best efficiency power levels.

The problem is solved in two steps: a simulation step in which the user makes up a feasible solution, and a search method which successively improves this feasible solution within the constraints until the optimal unit loading is found. A nonlinear programming method is used to carry out this search for a 21-unit hydropower plant [119]. A similar problem for a constant tailwater was dealt with in Example 11.3. Here, for a given downstream headwater elevation, the tailwater varies as a function of the project discharge. The method of calculating the gross head for variable tailwater was described in Sec. 11.2. In the simulation step, based on the initial guess of the load on each unit, the *P-Q* curves for each operating unit are used to calculate the discharge for the static head H_s. The total discharge QP is used to recalculate H. The newly calculated H is used to select new *P-Q* curves, whereupon QP and H are recalculated, and so on, until QP and H have converged. The solution thus obtained is the simulation solution for the assumed unit loadings. A typical result is shown in Table 11.4.

The second step of the method searches for an improved solution by finding unit loadings that maximize plant efficiency. This problem is formulated as Eq. (11.62).

TABLE 11.4 Multiunit Plant Optimization—Simulation and Optimization Step

	Simulation			Optimization		
Unit	Power, MW	Discharge ft^3/s	Efficiency 1	Power, MW	Discharge ft^3/s	Efficiency 1
1	16	2417	0.841	10.9	1808	0.767
2	16	2417	0.841	10.9	1808	0.767
3	16	2417	0.841	10.9	1808	0.767
4	16	2417	0.841	10.9	1808	0.767
5	20	3090	0.823	17.4	2766	0.800
6	20	3090	0.823	17.4	2766	0.800
11	20	2884	0.882	20.7	2965	0.886
14	20	2921	0.870	22.1	3163	0.886
19	30	5204	0.733	52.8	7422	0.903
Total	174	26,857	0.824	174	26,315	0.840

H_s = 95 ft (given)
H = 93.22 ft (computed)

$$\max \left\{ \frac{P}{\rho g \; QP \; H} \right\}$$

$$\text{subject to } P = \sum_{i=1}^{N} P_i \qquad QP = \sum_{i=1}^{N} Q_i \qquad H = H_s - f(QP) \qquad (11.62)$$

$$QN_i \le Q_i \le QX_i \qquad PN_i \le P_i \le PX_i$$

where $i = 1,\ldots,N$ is the number of committed units; QN and PN are lower bounds, and QX and PX are the upper bounds on discharge and power, respectively; P is the load requirement of the plant; and QP is the project discharge, here the sum of the discharges of the committed units. The relationship $P_i = f(Q_i)$ for H = constant is based on empirical fits of the P-Q curves. Cubic splines were used in the example. The lower and upper bounds on Q_i and P_i are given or calculated. If there is more than one constraint, such as generator rating and full gate discharge, the one that is reached first from inside the feasible range is binding.

The search for an extremum of the objective function $f(\mathbf{x})$ begins with a starting value (vector) $\mathbf{x}_0$ which does not necessarily optimize $f(\mathbf{x})$ but satisfies the constraints, such as the assumed load of the units for the simulation step. In order to find a new $\mathbf{x}$, called $\mathbf{x}_1$, that is nearer to or at the optimum, the negative gradient of f at $\mathbf{x}_0$ is used that gives the direction of steepest descent. A parameter α determines the length of the step along the gradient,

$$\mathbf{x}_1 = \mathbf{x}_0 + \alpha(-\nabla f(\mathbf{x}_0)) \qquad (11.63)$$

with $\nabla f(\mathbf{x}_0)$ the gradient of $f(\mathbf{x})$ at $\mathbf{x}_0$, represented by the vector of the partial derivatives for all variables $\mathbf{x}_i$. The improvement of the solution is determined by a one-dimensional search over the scalar α, in other words, the same α is used for moving in all partial gradient directions. In a constrained problem, the $\mathbf{x}_1$ thus found may not satisfy the constraints. Therefore, a "reduced gradient" that keeps the constraints satisfied is used. The reduced-gradient method [90, 120] consists of dividing the $\mathbf{x}$ vector into three subvectors:

$\mathbf{x_B}$ the basic subvector
$\mathbf{x_S}$ the superbasic subvector
$\mathbf{x_N}$ the nonbasic subvector

Accordingly, the original matrix of the constraint set, $\mathbf{Ax} = \mathbf{b}$, is divided into three parts,

$$\mathbf{Bx_B} + \mathbf{Sx_S} + \mathbf{Nx_N} = \mathbf{b} \qquad (11.64a)$$

The elements of $\mathbf{x_N}$ must lie on one of their bounds, the upper or lower one, while the elements of $\mathbf{x_B}$ and $\mathbf{x_S}$ can vary between their bounds. Only the subvectors $\mathbf{x_B}$ and $\mathbf{x_S}$ are changed by moving in the reduced-gradient direction. The idea of the reduced-gradient method is to eliminate $\mathbf{x_B}$ by expressing it as a function of $\mathbf{x}_S$ *and* $\mathbf{x}_N$ which produces

$$\mathbf{x_B} = \mathbf{B}^{-1}\mathbf{b} - \mathbf{B}^{-1}\mathbf{Sx_S} - \mathbf{B}^{-1}\mathbf{Nx_N} \qquad (11.64b)$$

where $\mathbf{B}^{-1}$ is the inverse of $\mathbf{B}$. The reduced gradient of $f(\mathbf{x_B}(\mathbf{x_S}),\mathbf{x_S},\mathbf{x_N})$ is then obtained [83] as

$$r(\mathbf{x}_S) = \nabla_S f(\cdot) - \nabla_B f(\cdot)\mathbf{B}^{-1}\mathbf{S} \tag{11.65}$$

where the first right-hand-side term is the derivative of $f(\cdot) = f(\mathbf{x}_B(\mathbf{x}_S),\mathbf{x}_S,\mathbf{x}_N)$ versus $\mathbf{x}_S$ and the second is the derivative of $f(\cdot)$ versus $\mathbf{x}_S$ via $\mathbf{x}_B$ by the chain rule. The nonbasic variables $\mathbf{x}_N$ and the right-hand sides $\mathbf{b}$ are treated as constants.

Introducing a change in the vector of $\mathbf{x}$ with no change in the nonbasic variables and with the constraints remaining satisfied can be expressed by differencing Eq. (11.64*b*),

$$\partial\mathbf{x}_B = -\mathbf{B}^{-1}\mathbf{S}\partial\mathbf{x}_S \tag{11.66}$$

Since $-r(\mathbf{x}_S)$ is a descent direction for $\mathbf{x}_S$, the direction for the basic variables, $\partial\mathbf{x}_S$, can be set equal to $-r(\mathbf{x}_S)$. Hence, the improved solution $\mathbf{x}_1$ obtained by the reduced search direction becomes

$$\mathbf{x}_1 = \mathbf{x}_0 + \alpha(\mathbf{B}^{-1}\mathbf{S}\, r(\mathbf{x}_S),\ -r(\mathbf{x}_S),\ 0)' \tag{11.67}$$

The search for the scalar α is restricted by the bounds on the basic and superbasic variables. Once the best α is found, a basic or superbasic variable may have reached one of its bounds. It is then necessary to make such a variable nonbasic and select one of the superbasic variables as a new basic variable.

Whenever the search progress turns out to be "small" or becomes bogged down, a test is made to determine if any nonbasic variables should become superbasic, in other words, if moving a nonbasic variable away from its bound will improve the minimization of f. This is done by computing

$$\lambda = \nabla_N f(\cdot) - \nabla_B f(\cdot)\,\mathbf{B}^{-1}\mathbf{N} \tag{11.68}$$

$\boldsymbol{\lambda}$ is the direction in which to move the nonbasic variables away from their bounds in order to decrease the objective function. Only if gradients are positive for variables at their lower bounds and negative for those at their upper bounds can the variables be considered for inclusion in the superbasic set. Otherwise the solution would not improve by moving variables away from their bounds. This procedure is known as estimating the Lagrange multipliers and gives the decrease in the objective function for a small step away from a variable's bound.

The result of a typical optimization run is shown in Table 11.4. The first column shows the nine units selected by the user for carrying the load of 174 MW (total plant capacity is about 600 MW). Column 2 shows the loads for each unit chosen by the user. They are within the units' bounds and meet the plant load requirement. They represent the starting values for the optimization. Columns 3 and 4 show the calculated discharges and efficiencies for the chosen loads obtained by the simulation step. Columns 5 through 7 show the results of the optimization. The improvement of the plant's efficiency over the efficiency based on the guessed loading is 1.9 percent.

Heuristic Scheduling. Keeping track of the fuel supply of hydropower plants through storage scheduling is fundamental for all hydropower operations with storage reservoirs. The storage equation describes the transition from one storage level to another during a scheduling time step,

$$S_{i,t+1} - S_{i,t} = \sum_{k=1}^{K} U_{k,t} + L_{i,t} - R_{i,t} - E_{i,t} \tag{11.69}$$

with $S_{i,t+1}$ being the storage of reservoir i at the end of time step t; $S_{i,t}$ is the storage of reservoir i at the beginning of time step t; $U_{k,t}$ are the releases from K upstream neighbors of reservoir i which reach reservoir i during the time step t; $R_{i,t}$ is the release from reservoir i during time step t; $E_{i,t}$ represents net water gains or losses of the reservoir, which includes evaporation from the reservoir surface and seepage losses; and $L_{i,t}$ is the inflow from the local drainage area of reservoir i, including the reservoir surface area, during time step t. In Eq. (11.69), all terms are expressed in average flow rates per time step. This means that reservoir volumes are divided by the time step length. For example, if a weekly time step is used, dividing a volume in cubic meters by the seconds of the week (604,800 s/wk) converts it to an average flow rate in (m^3/s) · week or 1 (ft^3/s) · week (also "week-second-feet" in American lingo). If flow travel times are nonnegligible, flow lagging or routing must be used for the upstream inflows in Eq. (11.69) (see Sec. 11.3). The evaporation flow rate $E_{i,t} = e_{i,t}A_{i,t}$ can be related to storage volume by

$$E_{i,t} = e_{i,t}[k_0 + 0.5k_1 (S_{i,t+1} + S_{i,t})] \tag{11.70}$$

with k_0 and k_1 being empirical coefficients in compatible units of an empirical surface area–storage relationship, $A = k_0 + k_1S$; $e_{i,t}$ is the local evaporation flux, a flow rate per unit area.

The heuristic approach to reservoir scheduling is based solely on the solution of Eq. (11.69). The operator chooses one of the two unknowns in the storage equation for each reservoir, ending elevation or release, and computes the other. For multiple time steps, rules can be made up of how discharge and storage should vary over the period, so that it suffices giving only the ending level or total release of the period. A chosen release may represent some externally estimated hydropower requirement and the chosen reservoir level may represent some desired reservoir state. The calculated storages and releases are not necessarily feasible or optimal. They may violate the storage bounds of reservoirs or discharge limits of channels. The selection is then repeated until a feasible schedule is found [30].

When the dynamics of water movement are no longer negligible, such as in reservoirs with nonnegligible velocities, wedge storage is built up so that storage change becomes a function of reservoir level and inflow (Sec. 11.3). When quasi-steady flow routing also is inadequate, the unsteady flow equations, Eqs. (11.28) and (11.29), must be used.

Reservoir Scheduling Using DP

A hydropower system is basically a deterministic system, as the system transitions follow physical laws, such as the storage equation or the flow-routing equations. Also, hydropower system capacity and performance have a very high reliability. Only the inputs and demands are subject to uncertainty. The deterministic approach to hydropower operations scheduling assumes that a forecast of inflows and power loads is available for a number of time steps ahead. Usually only a limited number of time steps is allowed in order to limit biasing the schedule by foresight (see "Foresight and Shortsight in Scheduling," in this section). The least-cost schedule for the next few time steps is then computed based on these input data. This computation can be repeated for several likely forecasts and some average or expected result can be implemented for the first time step.

DP is very suitable for hydropower scheduling, as it makes few requirements on the form of the objective function and the constraints. They can be nonlinear, continuous, or discontinuous; their derivatives can be continuous, discontinuous, or nonexistent, e.g., in the form of discrete step functions. The only requirement is separability, which means that the functions must contain only variables of the current stage. They cannot depend on variables of neighboring stages. Also, since DP is an enumeration procedure, it is sensitive to the number of state variables considered simultaneously. Therefore, only one or very few state variables (reservoirs) can be handled simultaneously.

Decomposition. The number of state variables encountered in practical problems usually exceeds the very few variables that can be handled by DP. The application example discussed here has about 18 state variables. Decomposition is the only way DP can handle such a problem. Straightforward DP is based on the principle of optimality, which is already a decomposition method. But still DP is plagued by the "curse of dimensionality" when it comes to dealing with more than one or a few state variables (reservoirs) simultaneously. The number of evaluations grows exponentially with the number of reservoirs because all possible paths for one reservoir have to be combined with all possible paths of the other reservoirs for each stage. Let there be i levels for each state variable with k pathways leading from each level, then the number of evaluations is $(ik)^r$. For only 2 reservoirs and 10 state variable levels with 10 pathways out of each, $(10 \cdot 10)^2 = 10{,}000$ evaluations would be required per stage.

For practical applications, decomposition methods have been developed that keep i and r low. Examples of such methods are state increment dynamic programming [120], discrete differential dynamic programming (DDDP) [121], and dynamic programming by successive approximations (DPSA) [9, 122, 123]. An application of DPSA to a 18-reservoir system by operating on only one reservoir at a time [109] will be briefly discussed.

Feasible Initial Policy. A feasible initial policy or trial policy is required for each reservoir. This policy, which specifies reservoir storages and releases over the planning period, assures that the variables in each single reservoir equation do not violate any constraints in any of the stages. Feasibility is achieved by a constraint satisfaction method. A simple example is forcing all violations of bounds on water levels and releases to zero by minimizing the squares of all deviations. If the resulting objective value is nonzero, the policy is infeasible. An adjustment must then be made to the violated bounds and the procedure repeated until a zero objective value is obtained. A more sophisticated approach to finding a feasible solution is via goal programming (see Sec. 11.4). In the subsequent optimization procedure, feasibility is maintained by excluding paths that would violate constraints.

Optimization Procedure. The procedure to find an optimal policy is as follows: (1) select reservoir iteration sequence, corridor sizes, tolerance limits, and number of loops; (2) set up corridor for the reservoir selected for iteration; (3) apply DP within corridor, with storages of other reservoirs held at previously determined prescribed storages; (4) if improvement in the objective function exceeds the tolerance limit, set up new corridor around new trial policy of same size and repeat DP, otherwise, reduce corridor size and repeat DP; (5) repeat steps 2 through 4 until the finest corridor size has been used; and (6) repeat steps 2 through 5 for the next reservoir and terminate after reaching the specified number of loops over all reservoirs. The number of such iterations depends on the quality of the selected trial policy. Convergence may be achieved after a few iterations over all reservoirs. The DPSA procedure is illustrated in Fig. 11.34. The hydropower system shown in Fig. 11.35 with up to 18 storage variables (open triangles)

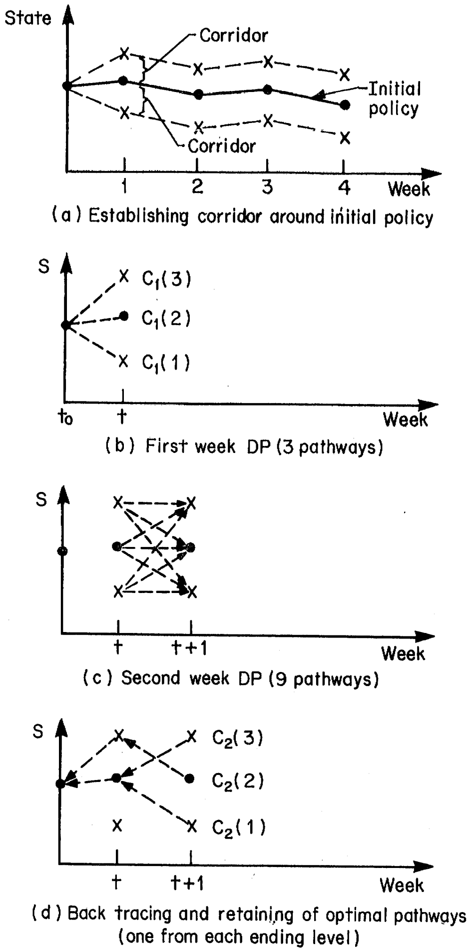


FIGURE 11.34 Steps of the DPSA algorithm.

was successfully scheduled using weekly time steps over planning periods of up to 52 weeks by this method.

The objective function is the cost function for the residual thermal generation requirement. Each change in the hydropower schedule by the DP method requires a cost evaluation. Streamlined cost functions that relate hydropower load to thermal generation cost are prepared for each stage (week) of the planning period [16, 22] based on the probabilistic unit stack of each week (see Sec. 11.2).

The method was found to be very robust. A comparison of scheduling algorithms was made for a subsystem of six reservoirs marked in Fig. 11.35 [101]. The DPSA algorithm converged monotonously to an optimal solution that could not be matched in terms of speed and level of convergence by several other methods tested, which included a network method (out-of-kilter algorithm) and a reduced-gradient method [102, 124]. The possibility that an algorithm of this type termi-

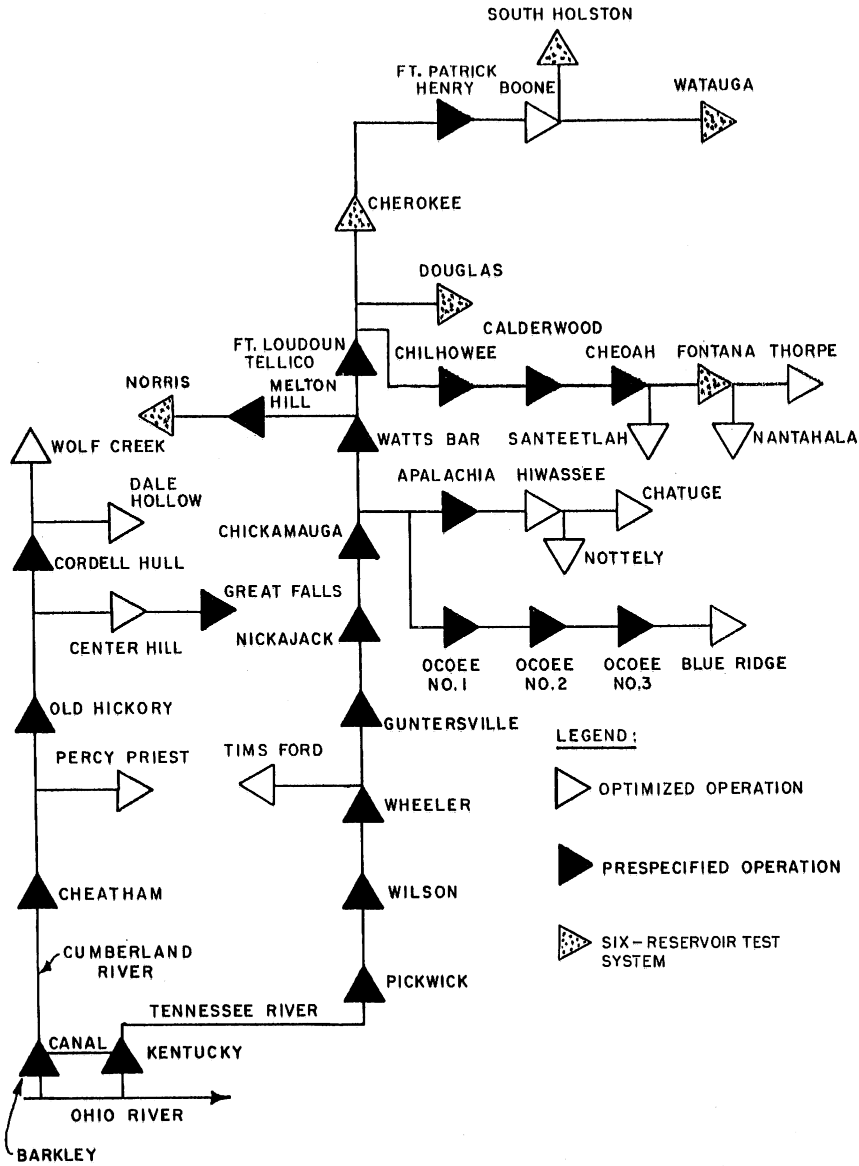


FIGURE 11.35 Schematic diagram of multireservoir system for which DPSA and LP–DP models were developed [101],[109].

nates at a local optimum is a concern. A practical means is to start at different initial policies. An example of several initial policies that converged to the same final policy for the six-reservoir test case is shown in Fig. 11.36.

An application example of the method in an investigation of new operation poli-

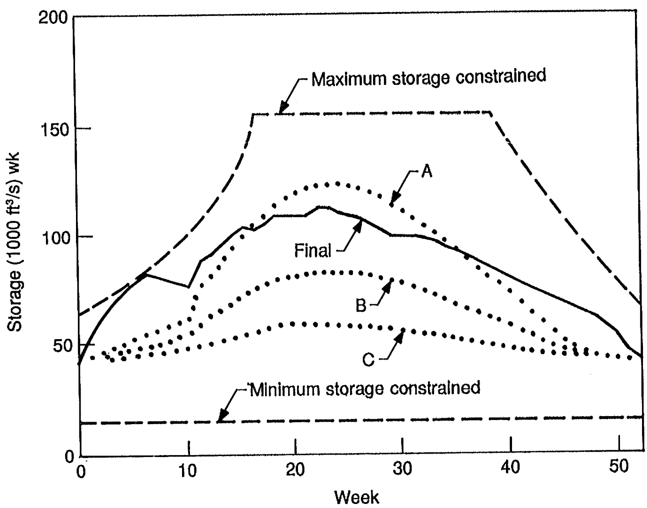


FIGURE 11.36 Improvement of initial feasible policies (A, B, C) by DPSA. All converge to the same final policy. *(From Tennessee Valley Authority [101].)*

cies is shown in Fig. 11.37. In this case a power and flood-control reservoir was constrained to operate more like a natural lake with reduced annual level fluctuations. The calculated policies for the original and restricted operation space are shown. As flood control and energy storage were given up, the other reservoirs of the system compensated to the extent possible. The noncompensated loss showed in the form of expected flood damage cost and power cost increases.

The extreme decomposition used in DPSA does not allow dealing effectively with constraints that include more than one state variable, such as meeting a downstream flow requirement from several upstream reservoirs. In such a case, less decomposition could be attempted to accommodate several reservoirs simultaneously, such as the method proposed by Heidari et al. [121]. Possibly, a combination of various degrees of decomposition can be used to overcome the problems associated with too much or too little decomposition.

Reservoir Scheduling Using LP and Probabilistic DP

An important aspect of actual operations is the correct simulation of the knowledge of the input information that the decision maker has when the decision needs to be made. For most of the prevailing hydrologic regimes, it is not realistic to assume knowledge over an extended future period and to use this knowledge for scheduling the current stage. One can cope with this problem by reducing the number of future stages (time steps) included in scheduling. A reliable forecast is assumed to be available for the first week and the system is scheduled forward one week at a time. Appropriate consideration must be given to the future, which is done by a long-range guide attached to the end of the scheduled week. This model was developed for the same system shown in Fig. 11.35 [95, 125, 126].

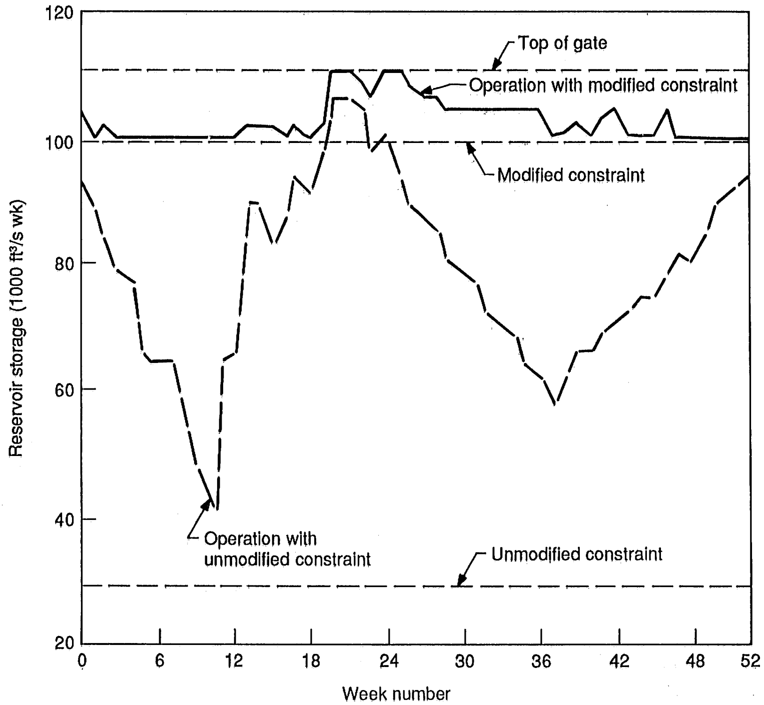


FIGURE 11.37 Reservoir operation as computed for normal and restricted range of storage.

Special emphasis is on simulating as closely as possible ongoing hydropower operations in a hydropower-thermal system.

Feasible Schedule. A feasible schedule is found by a constraint satisfaction procedure based on goal programming. Linear preemptive goal programming is used that satisfies one constraint or a group of constraints at a time (see Sec. 11.4). Besides the physical limits that need to be met, some 400 other constraints or goals are considered. A list of ranked constraints is given in Table 11.5.

Some special hydropower objectives are met in the form of constraints. A constraint on reservoir release equal to full-capacity turbine discharge is observed as long as it is not overridden by higher-priority constraints that would require a higher discharge in order to stay within maximum reservoir bounds. This avoids unnecessary spill. Hydropower use can be optionally constrained by a lower bound on total hydropower system energy in storage. This minimum hydropower energy in storage is required to meet the system load over a low-flow period with all other power resources committed. Also, a constraint that maintains an instantaneous capacity requirement can be imposed. Another constraint provides for balanced peaking and concomitant water withdrawal from all storage reservoirs aimed at meeting the higher-priority end-of-year flood control drawdown requirement.

Hydropower Optimization. A hydropower schedule is found by either minimizing the departure from median reservoir levels or by a schedule that minimizes long-

TABLE 11.5 Priority Listing of Constraints and Guides

Priority* number	Constraint (C) guide (G)	Constraints & guides
1	C	Avoid if possible or otherwise minimize flood damage at specified sites
2	C	Avoid if possible or otherwise minimize violation of flood easements
3	C	Avoid if possible or otherwise minimize flood damage at specified locations
4	C	Avoid if possible or otherwise minimize exceedance of top of power pool elevation in specified reservoirs
5	C	Avoid if possible or otherwise minimize exceedance of allowable channel capacities below dams
	G	Avoid if possible or otherwise minimize exceedance of normal full pool levels
6	C	Avoid if possible or otherwise minimize violation of minimum allowable water levels for navigation
	C	Avoid if possible or otherwise minimize violation of minimum allowable discharges for navigation
7	C	Avoid if possible reservoir drawdown below water supply intake levels
	C	Avoid if possible reservoir releases smaller than required for downstream water supplies
8	C	Avoid if possible reservoir drawdown below minimum acceptable levels for power generation
9	G	Avoid if possible or otherwise minimize exceedance of flood guide levels (or temporary flood storage levels if needed to avoid spill)
10	C	Avoid if possible or otherwise minimize violation of water quality discharge constraints
11	G	Avoid if possible operating levels that could cause power peaking capacity or generation requirement shortfall at some time in the future
12	G	Avoid if possible operating levels that could lead to violation of winter flood guide levels more than 50% of the time
13	C	Avoid if possible or otherwise minimize exceedance of maximum allowable reservoir level drawdown rates
14	G	Avoid if possible or otherwise minimize drawdown below normal minimum levels
15	C	Avoid if possible or otherwise minimize violation of summertime recreation levels
		Minimize expected future power cost

*All reservoir elevations are maintained within specified upper and lower limits. Consequently these constraints are always given highest priority. The ranking of other constraints and guides is reflected by their priority number. Highest priority is 1.

range generation cost. The uncertainties associated with the thermal power system, its loads, and the availability of thermal units are treated as described in Sec. 11.2. For hydropower schedule optimization, a trade-off is made between current and future hydropower use. The cost for the current week is calculated by the residual thermal generation cost function, as explained in Sec. 11.2. The long-range cost beyond the first week is provided in the form of a function that relates long-range operation cost to system hydropower energy in storage. This function is also called economy guide. It is computed externally for an extended period, such as a year or several months ahead. A probabilistic DP procedure is used with system energy in storage as the single state variable [95]. The weekly scheduling model is embedded in this procedure to keep track of feasibility of individual reservoir operations. The derivative of the long-range expected cost function versus energy in storage is the expected incremental thermal generation cost reduction (with increasing storage) or the expected incremental hydropower replacement value. A typical result of this calculation are the hydropower energy-in-storage value isopleths shown in Fig. 11.38.

While the current week's generation cost decreases with incremental hydropower use, the expected future cost increases. The optimal amount of hydropower used in the current week is determined by balancing the incremental hydropower replacement value with the expected incremental generation cost represented by the economy guide. In other words, using less than the optimal amount of hydropower in the current week would make the current generation cost too high, and using more would make the expected generation cost too high. The optimal balance between current and future resource use is illustrated in Fig. 11.2. The computation of the balanced amounts starts with a schedule designed to achieve balanced peaking among the reservoirs. A nonlinear search procedure is used to increase or decrease current hydropower use until total power generation cost is minimized. Hydropower adjustments are made by increasing or decreasing weekly plant peaking hours, whereupon the current week's thermal generation cost is recomputed.

An application of the model to probabilistic forecasting of energy-in-storage levels is shown in Fig. 11.39. Starting at a time t of the year, here at the end of January,

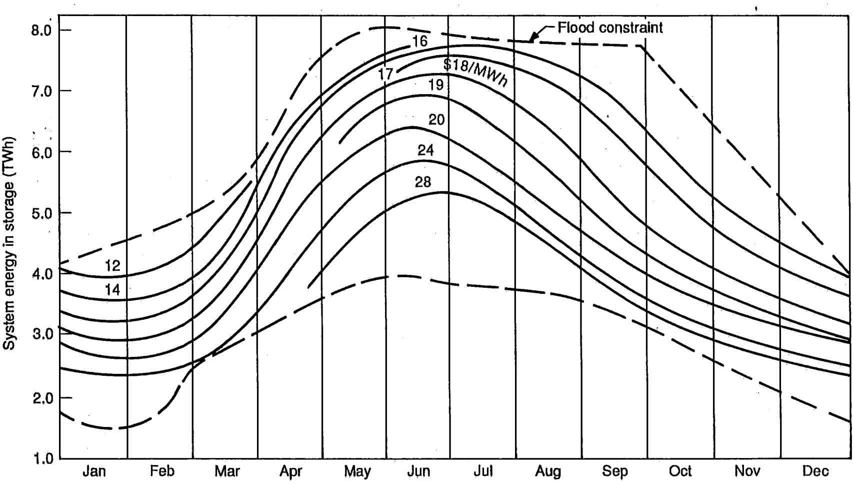


FIGURE 11.38 System economy guides—incremental values of energy in storage. *(From Shane and Boston [126].)*

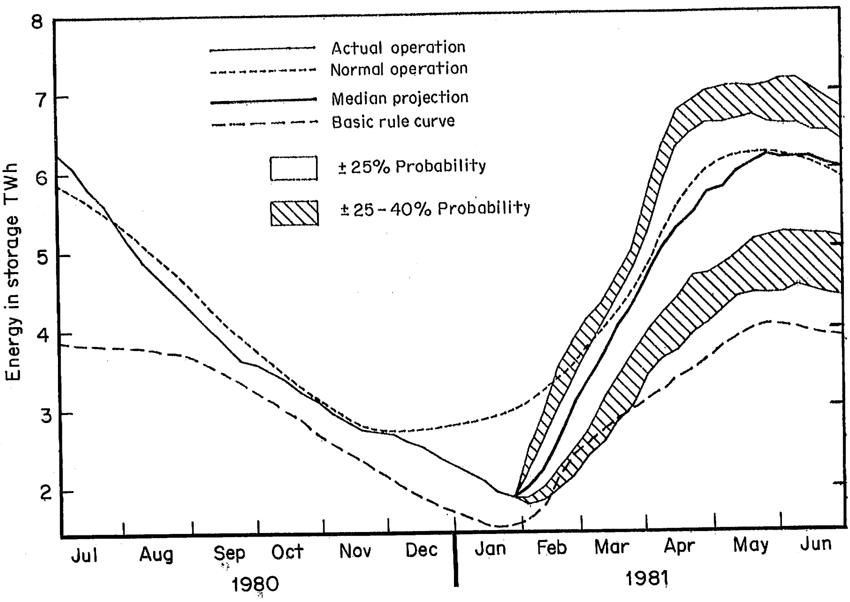


FIGURE 11.39 Probabilistic system hydroenergy in storage forecast. *(From Shane and Boston [126].)*

the model is applied over the following five months to forecast probable energy-in-storage levels for each week. A probability distribution is produced for each week-end of this period by running the model one week at a time, over the entire historic flow record or a selected sample of it, and extracting probabilistic information from the results. In another example, in Fig. 11.40, the average annual power output of a hydropower subsystem is simulated over a historic 24-year period and compared with the actual output. After testing and calibrating the model on these 24 years, a simulation can be conducted for the entire stream-flow record of about 80 years to obtain more reliable information on the productivity of this subsystem.

Foresight and Shortsight in Scheduling

Use of Input Information. An important aspect of realistic scheduling is the modeling of available information. The use of modern mathematical programming methods, such as LP, NLP, and DP, has in many instances neglected this fact because of the relative ease with which multi-time-step problems can be modeled. This has created dangerous pitfalls for the use of these methods in the form of overestimation of the productivity of systems or by underdesigning systems. Wagner [89] distinguishes four different ways in which information is used to make decisions:

1. Perfect information: All inputs and demands are known prior to the decision; this means that a perfect optimal schedule can be produced with the highest possible return. Models which implement this approach produce the upper limit of possible returns, which are usually not obtainable in the real world because such foresight over inputs is not available to the operator. Also, the

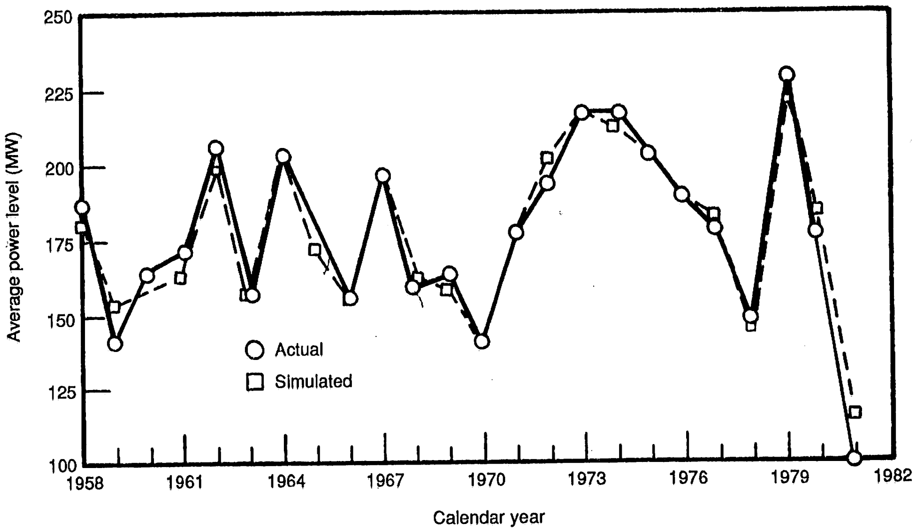


FIGURE 11.40 Simulation of hydrologic period for which power data are available. (*From Shane and Boston [126].*)

results could indicate that a relatively small-sized design can accomplish what is required, but in reality it may turn out to be underdesigned.

2. Learn demand, then produce: A good forecast is assumed to be available for the upcoming period and the decision is based on this forecast and the existing system state. This case approximates the real-world situation, as reasonably good forecasts of system capacity, load, and stream flows are available for the near future. This approach should produce a reasonably realistic schedule for the upcoming (current) period.
3. Produce, then learn demand: The forecast is not available when the decision needs to be made, but becomes available later so that corrective action can be taken, as necessary. This case occurs when lead time is required for implementing a decision, e.g., a preflood drawdown or the up- and downramping of thermal units.
4. Full commitment: All decisions are made independent of inputs and demands that normally guide system operations, e.g., a fixed requirement is met and no attention needs to be paid to what could be accomplished. This approach most likely produces the most inefficient resource allocation.

According to Wagner [89], progressively less information is available for scheduling in the above four cases. Consequently, the optimal value of the objective function will successively decrease. The difference in outcomes between the perfect foresight schedule and those which use less information is the worth of information.

Foresight or Shortsight—A Dilemma. If the model has more information than the operator can be expected to have, it is "foresighted" or "anticipative." If the model has less information than the operator can be expected to have, it is "shortsighted" or "underanticipative." The effect of these model biases on the results is not predictable, as the direction in which the available information deviates from the true information is not known.

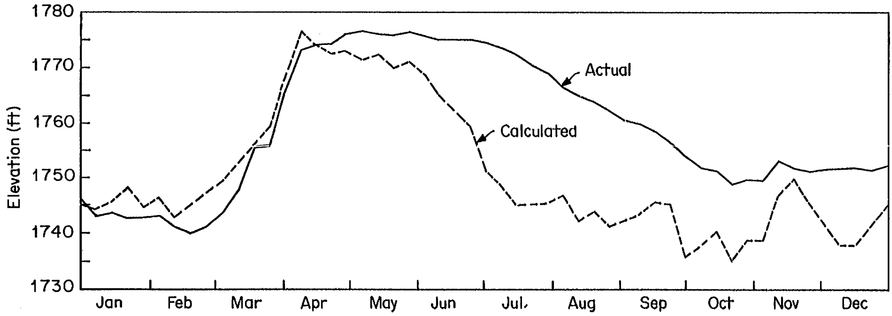


FIGURE 11.41 Example of anticipative drawdown calculated with foresighted model.

A typical example of the effect of using foresight is given in Fig. 11.41. It shows the storage in a reservoir as it was actually observed and as it was calculated with perfect foresight. While the actual drawdown occurred over a period of four months, the foresighted model drew the reservoir down rapidly by using knowledge of relatively high inflows later in the year. Thus, the stored water could be used for summer power peaking at high replacement value. In actual operation, no prior knowledge of high inflows later in the year was available, so that the drawdown was spread over a longer period.

It is important to realize that for each particular problem, an approach must be selected that best simulates the use of available information. The LP model discussed above implements case 2, "learn demand, then produce" [95]. A perfect forecast is assumed only for the first time step. The remainder of the planning period is represented by a probabilistic guide. But, if the probabilistic guide represents an average long-range future, a nonconservative projection will result in case of a flood or drought.

The DPSA model was designed to implement perfect foresight over multiple time steps ranging from perfect foresight over the entire planning period to foresight over only a few periods [109]. This capability not only allows the model to assess the worth of information but also to simulate a real-time operational approach. An example of using a four-week schedule of which the first week is implemented each time is shown in Fig. 11.42. This approach combines long-range probabilistic guide information at the end of the four-week period and explicit information for the near future weeks. An analysis of various amounts of foresight on hydropower and flood control operations can be found in Georgakakos [127].

Basically, the planner or operator is faced with a dilemma. There is no unbiased approach to dealing with an uncertain future. Avoiding both shortsight and foresight would require a perfect forecast, which is not available. An operational strategy to cope with this dilemma in real-time planning is tracking the system closely by repetitively scheduling at short time intervals so as to minimize the negative effects of forecast and decision errors. For project planning applications, the model must be designed to closely simulate this real-time approach. A decisive improvement of this situation can only come through forecast improvements by global meteorological data collection by satellites and large-scale weather prediction models [128].

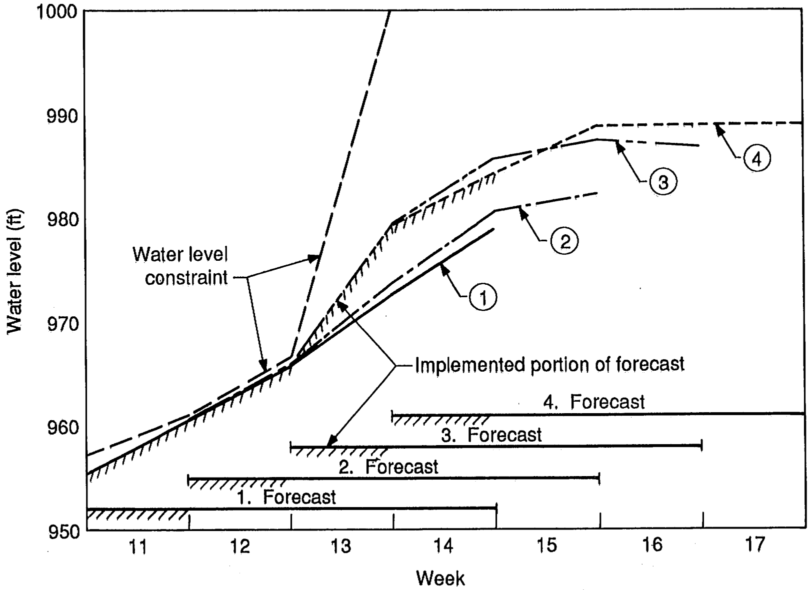


FIGURE 11.42 Weekly schedules based on staggered four-week planning periods.

11.6 SYSTEM EXPANSION PLANNING

Project Interdependence

Planning the expansion of an existing project or planning of a new project that is part of a system should use methods that can take into account the interactions of the additions or new project with the system. Due to the usually dispersed nature of hydropower resources in a river basin, one project by itself may not be economical, but in combination with a second or several complementary projects the resulting system may be economically viable.

Planning Approach

In project planning, the variables include the project characteristics, such as head, storage, number of units, and individual unit capacities. The planning process determines values for these variables by testing alternative choices of them in simulated operational settings in which these variables are given. Thus, project planning is but an extension of operational planning with an extended set of variables. Project planning must also cope with predicting demands (loads) and supplies (inflows) over the lifetime of the project of some 50 years.

The project planning approach consists of two steps:

1. Screening for feasible alternatives.

2. Simulation of selected feasible alternatives with the objective of identifying the preferred scheme.

Screening models are used to find the most effective project sizes and combinations. Once alternatives have been identified that warrant in-depth study, more detailed system simulation is carried out by using models of the type described in Sec. 11.5. If there are only a few alternatives, heuristic screening rules can be used to identify the schemes that warrant further study.

Screening models are often based on considerable simplifications of the problem [129]. Care must be taken that these simplifications do not interfere with the result of the screening process by discarding potentially good schemes. The screening and the simulation approach may have to be used iteratively until a preferred scheme has been identified.

Optimal System Expansion

Capital investment decisions are usually based on benefit/cost analysis [130, 131]. The relationship between the conventional benefit/cost analysis and the total cost minimization approach is illustrated in Fig. 11.43. The investment cost is the capitalized investment stream over the investment period, a function of the amount of capacity added. The generation cost is the capitalized annual excess generation cost over the lifetime of the added capacity. Both costs are discounted to present worth. The excess generation cost is a maximum if no capacity is added and decreases with capacity added. The decision on the optimal capacity addition can be made as the one for which the total cost is a minimum. This point is characterized by the necessary minimum conditions, which require that incremental generation cost reduction due to capacity addition is equal to the incremental investment cost increase. In terms of benefit/cost analysis, at this point the discrepancy between benefits and costs reaches its maximum and begins to decline [130]. In the case of a flat minimum, the decision criterion may need to be sharpened. This can be accomplished by giving consideration to other objectives that may not be fully expressed by the costs considered, such as reliability or environmental externalities of system expansion. Both investment and generation

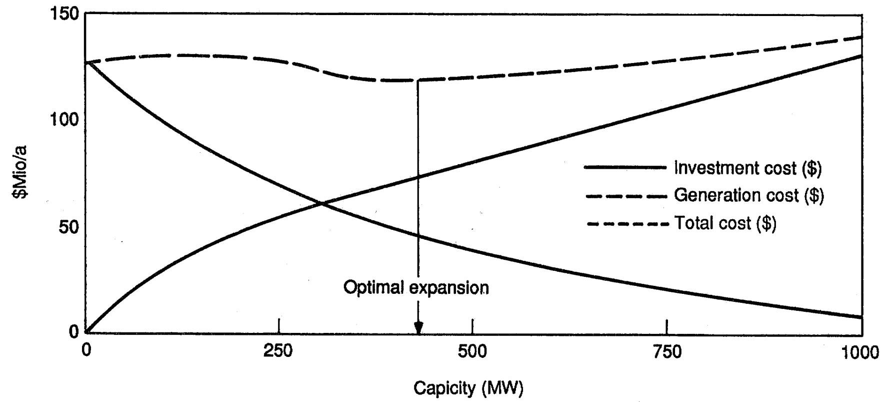


FIGURE 11.43 Expansion cost versus capacity.

costs may be affected by including these additional considerations, thus creating a new total cost function shape with a new and possibly sharpened optimality criterion.

It is possible that the total cost function is a monotonic increasing function of capacity addition and no minimum exists. In this case, a zero capacity addition is the optimal choice unless, as mentioned before, other considerations than the ones considered justify an expansion.

Investment Staging

In order to minimize capital outlays, system growth should match the demand growth as closely as possible. The matching of a demand forecast over a planning period of 30 years by capacity expansions is shown in Fig. 11.44 [134]. For

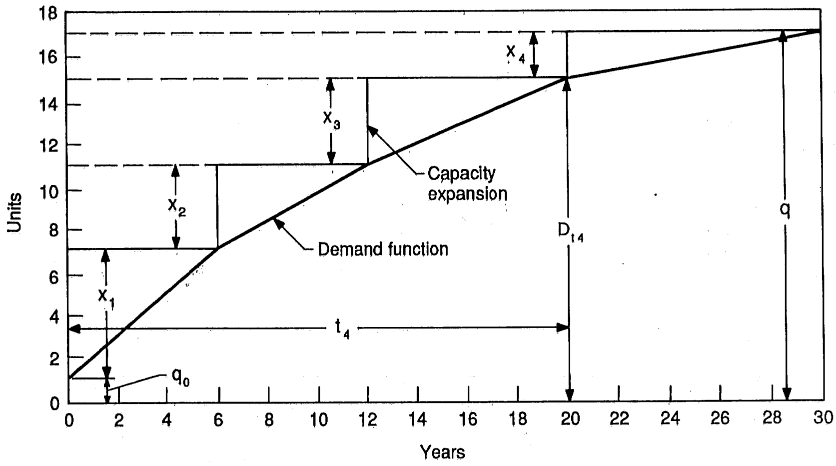


FIGURE 11.44 Demand growth and optimal supply planning. *(From Butcher et al. [134].)*

capital-intensive projects, such as hydropower projects, with no fuel costs and relatively small production, labor, and maintenance costs, the reduction of capital investment by staging can be very effective for reducing the price of the energy produced. If all capacity were built at the beginning of the planning period at a cost C, a part of it would sit idle almost throughout the planning period, while the charges for the expended capital would accrue at rates that could have been obtained elsewhere in the market. In order to reduce this investment cost, assume that the expansion is staged such that half the investment is made at the start of the planning period and half at the midpoint, i.e., $j = n/2$. The compounded cost of this "staged" investment is $C' = 0.5C(1 + r)^n + 0.5C(1 + r)^{n/2}$. In terms of present cost, expressed as a fraction of the full investment cost C, the staged investment cost is

$$\frac{C'}{C} = 0.5 + 0.5(1 + r)^{-n/2} \tag{11.71}$$

For an interest rate $r = 0.08$ (8 percent) and $n = 30$ years, $C'/C = 0.66$. In other words, the assumed investment staging reduces the investment cost without staging by about 34 percent. Hence, the basic rule of capacity expansion is to minimize excess capacity at installation time.

Optimal Allocation of Funds

If there are several competing projects, the question arises as to how the available funds should be distributed so as to maximize returns. Dynamic programming has been used for optimal allocation of funds [84, 111, 132, 133, 134]. The optimal fund allocation problem is stated as

$$f(n, q) = \max_{\substack{0 < x_n \leq q \\ 0 \leq q \leq Q}} \{r(n,x_n) + f(n - 1,q - x_n) \tag{11.72}$$

where $f(n,q)$ is the optimal return when q funds remain to be allocated to n projects; $r(n,x_n)$ is the return from allocating decision x_n concerning project n; n is the number of projects remaining; (n, q) is the state in which the funds q remain to be allocated to n projects; x_n is the funds to be allocated at the next stage; the total funds available is $q \leq Q$.

The method starts with determining the best allocation to one project remaining, which can be arbitrarily selected. The return function, $r(n,x_n)$ must be available for each project. In the second stage, the total funds are split among x_1 and x_2. According to the principle of optimality [103] (see Sec. 11.4), only the optimal outcomes of the previous allocation $f(1,q)$ need to be considered. This is expressed by

$$f(2,q) = \max_{\substack{0 \leq x_2 \leq q \\ 0 \leq q \leq Q}} \{r(2,x_2) + f(1,q - x_2)\} \tag{11.73}$$

Equation (11.73) requires for each level of q the evaluation of all feasible levels x_2; $f(1,q - x_2)$ is found from the optimal values $f(1q)^*$ retained from the previous stage. This recursive computation is continued through all remaining stages.

EXAMPLE 11.5 *A capital investment of 140 units is to be allocated to a four-project scheme. The return functions for each project are given in Table 11.6, which also gives the solution of the problem.*

In stage 1, part a of Table 11.6, there is no prior allocation (stage). Therefore, $f(0,q - x_1) = 0$. The result for each funding level is given by $f(1,q) = r(1,x_1)$. Since there is only one value per level q, the optimal values retained in the last column under $f(1,q)^*$ are again the $r(1,x_1)$. In the second stage, there is a previous allocation, $x_1 = q - x_2$, so the resource q must be shared by x_2 and $q - x_2$, as expressed by Eq. (11.73). The results are shown in part b of Table 11.6. According to the principle of optimality, the function $f(1,q - x_2)$ is taken from the last column of the previous stage, $f(1,q)^*$. The results for the stages 3 and 4 are given in parts c and d of Table 11.6, respectively.

The optimal allocation of available funds is found by tracing back from the maximum return obtained for $f(4,q)$, when all projects have been considered for all levels of $Q = 140$. The maximum total return for $q = 140$ is $f(4,q)^* = 35$, which is obtained by allocating $x_4 = 100$ (see last section of part d of Table 11.6). This leaves $q - x_4 = 140 - 100 = 40$ to be allocated at stage 3. For $f(3, 40)$, the

TABLE 11.6 Example of Optimal Allocation of Funds to Four Different Projects

q	x_n	$q - x_n$	$f(n-1, q-x_n)$	$r(n, x_n)$	$f(n, q)$	$f(n, q)^*$
a. Stage $n = 1$						
20	20	0	0	−5	−5	−5
40	40	0	0	0	0	0
60	60	0	0	5	5	5
80	80	0	0	8	8	8
100	100	0	0	12	12	12
120	120	0	0	17	17	17
140	140	0	0	21	21	21
b. Stage $n = 2$						
20	20	0	0	2	2	2
20	0	20	−5	0	−5	
40	40	0	0	4	4	4
40	20	20	−5	2	−3	
40	0	40	0	0	0	
60	60	0	0	6	6	6
60	40	20	−5	4	−1	
60	20	40	0	2	2	
60	0	60	5	0	5	
80	80	0	0	8	8	8
80	60	20	−5	6	1	
80	40	40	0	4	4	
80	20	60	5	2	7	
80	0	80	8	0	8	
100	100	0	0	10	10	12
100	80	20	−5	8	3	
100	60	40	0	6	6	
100	40	60	5	4	9	
100	20	80	8	2	10	
100	0	100	12	0	12	
120	120	0	0	12	12	17
120	100	20	−5	10	5	
120	80	40	0	8	8	
120	60	60	5	6	11	
120	40	80	8	4	12	
120	20	100	12	2	14	
120	0	120	17	0	17	
140	140	0	0	14	14	21
140	120	20	−5	12	7	
140	100	40	0	10	10	
140	80	60	5	8	13	
140	60	80	8	6	14	
140	40	100	12	4	16	
140	20	120	17	2	19	
140	0	140	21	0	21	

TABLE 11.6 Example of Optimal Allocation of Funds to Four Different Projects *(Continued)*

q	x_n	$q - x_n$	$f(n-1, q-x_n)$	$r(n, x_n)$	$f(n, q)$	$f(n, q)^*$
c. Stage $n = 3$						
20	20	0	0	5	5	5
20	0	20	2	0	2	
40	40	0	0	10	10	10
40	20	20	2	5	7	
40	0	40	4	0	4	
60	60	0	0	15	15	15
60	40	20	2	10	12	
60	20	40	4	5	9	
60	0	60	6	0	6	
80	80	0	0	20	20	20
80	60	20	2	15	17	
80	40	40	4	10	14	
80	20	60	6	5	11	
80	0	80	8	0	8	
100	100	0	0	22	22	22
100	80	20	2	20	22	
100	60	40	4	15	19	
100	40	60	6	10	16	
100	20	80	8	5	13	
100	0	100	12	0	12	
120	120	0	0	18	18	24
120	100	20	2	22	24	
120	80	40	4	20	24	
120	60	60	6	15	21	
120	40	80	8	10	18	
120	20	100	12	5	17	
120	0	120	17	0	17	
140	140	0	0	15	15	26
140	120	20	2	18	20	
140	100	40	4	22	26	
140	80	60	6	20	26	
140	60	80	8	15	23	
140	40	100	12	10	22	
140	20	120	17	5	22	
140	0	140	21	0	21	
d. Stage $n = 4$						
20	20	0	0	1	1	5
20	0	20	5	0	5	
40	40	0	0	2	2	10
40	20	20	5	1	6	
40	0	40	10	0	10	

TABLE 11.6 Example of Optimal Allocation of Funds to Four Different Projects *(Continued)*

d. Stage $n = 4$

q	x_n	$q - x_n$	$f(n-1, q-x_n)$	$r(n, x_n)$	$f(n, q)$	$f(n, q)^*$
60	60	0	0	5	5	15
60	40	20	5	2	7	
60	20	40	10	1	11	
60	0	60	15	0	15	
80	80	0	0	15	15	20
80	60	20	5	5	10	
80	40	40	10	2	12	
80	20	60	15	1	16	
80	0	80	20	0	20	
100	100	0	0	25	25	25
100	80	20	5	15	20	
100	60	40	10	5	15	
100	40	60	15	2	17	
100	20	80	20	1	21	
100	0	100	22	0	22	
120	120	0	0	27	27	30
120	100	20	5	25	30	
120	80	40	10	15	25	
120	60	60	15	5	20	
120	40	80	20	2	22	
120	20	100	22	1	23	
120	0	120	24	0	24	
140	140	0	0	23	23	35
140	120	20	5	27	32	
140	100	40	10	25	35	
140	80	60	15	15	30	
140	60	80	20	5	25	
140	40	100	22	2	24	
140	20	120	24	1	25	
140	0	140	26	0	26	

Source: Adapted from Gupta [133].

optimal allocation is $x_3 = 40$. This leaves $q = 0$ for stage 2. Thus the optimal allocation leaves no funds for projects 1 and 2. The optimal allocation policy in this case is $(x_1^*, x_2^*, x_3^*, x_4^*) = (0, 0, 40, 100)$. Now, suppose the funds were cut to $q = 80$. For $f(4, 80)$, the best return is obtained for $x_4 = 0$ (part *d*, Table 11.6). This leaves $q = 80$ for other stages (projects). Entering part *c* of Table 11.6 with $q = 80$, the optimal allocation is $x_3 = 80$, which leaves no funds for the remaining projects. Hence, in this case, the optimum allocation policy is (0, 0, 80, 0).

Time Sequencing of Projects. Dynamic programming can also be used for time sequencing of projects. A project-staging formulation for minimizing investment cost, based on backward DP with discounting, was given by Butcher et al. [134]. The time t_n at which a new investment occurs is a function of the amount of capacity added as well as of the demand growth. Therefore, the time when new capacity is needed can be expressed in terms of the capacity addition, so that t_n can be eliminated as a variable.

A method with reduced computational requirements was described by Morin and Esogbue [135] and Morin [136]. It uses the demand curve to determine the demands at given points in time, say every 5 years in a 50-year planning period. Then, at these time instances, project combinations are selected that can match these demands. This reduces the number of states that need to be examined.

Time Sequencing of Interdependent Projects. In hydropower system expansions, the interdependence of projects must be considered. Erlenkotter [137] described such an expansion problem and used a method closely related to the DP methods that are used for staging independent projects. The method deals with sets of projects instead of individual projects. Assume that a set X of projects with capacity $z(X)$ already exists. Then, adding a project i will increase the capacity of the set so that $z(X,i) \geq z(X)$, with $z(X,i)$ being the union of old and new capacity. The cost of establishing project i within the set of existing projects is $c_i(X)$. It is assumed that the demand at the start of the planning period D_0 is met exactly by existing projects. A first addition is assumed to occur at the start of the planning period. This addition can meet the demand over a number of years that depends on the size of the addition and the demand growth function, as illustrated in Fig. 11.44. The expansion timing function is related to the demand growth D_t. It gives the time $t(X)$ at which an expansion from capacity level z is required. The DP formulation used by Erlenkotter [137] is

$$C(X) = \min_{i \text{ in } X} \{c_i(X) \ \exp[-rt(X)] + C(X,i)\} \tag{11.74}$$

where $C(X)$ is the optimal cost for all expansions undertaken for set X, discounted to time 0; exp(.) is the continuous discount function; r is the discount rate; $t\ (X)$ is the period from $t = 0$ to the time when the capacity addition occurs; $C(X,i)$ is the discounted cost of the union of old and new projects; and $c_i(X)$ is the expansion cost.

Erlenkotter [137] used an example based on data from the Columbia River system. Only capacity was used as a project parameter. Two types of projects were distinguished, run-of-river (RR) and storage (S) projects. The data and results are summarized in Table 11.7. The optimal sequence of projects is shown in column 1. The added capacity that becomes available from previously sequenced projects (1,2,3,5,6, and 8) as storage projects (4,5,7, and 9) are added is shown in columns 7 through 11. These capacity additions illustrate project interdependence. The unit costs of the base capacity are given in column 6 and those of the combined capacity, as it results from project additions, in the last column of the table. Both unit costs show that the projects are sequenced approximately according to rising unit costs. A graphical presentation of the results is given in Fig. 11.45.

The location of the reservoirs in the basin and their optimal sequence is shown in Fig. 11.46. The optimal sequence alternates between run-of-river projects and storage projects. The first project, no. 2, a run-of-river project, is scheduled at time $t = 0$ because of its low unit cost. It is followed by storage project no. 4. Scheduling the run-of-river project no. 8 before the storage project no. 9 is not justified because of the high base unit cost of no. 8. After no. 9 has been scheduled, no. 8 becomes economical with a low unit cost.

TABLE 11.7 Optimal Hydropower Expansion Schedule [137]

Project code	Project type	Base capacity, MW	Added capacity, MW	Base investment $Mio	Unit cost, Mio$/MW
1	2	3	4	5	6
2	RR	260.00	260.00	70.00	0.27
4	S	270.00		80.00	0.30
1	RR	200.00	200.00	60.00	0.30
9	S	220.00		100.00	0.45
8	RR	80.00	120.00	36.00	0.45
3	RR	180.00	180.00	80.00	0.44
5	S	315.00	105.00	131.60	0.42
6	RR	140.00	210.00	108.00	0.77
7	S	120.00		96.00	0.80

Fraction of total additional capacity becoming available at projects 1, 2, 3, 5, 6, and 8 (for total additional capacity see column 4).

Possible upstream projects*	Fraction of total additional capacity for projects added			
	One	Two	Three	Four $
7	8	9	10	11
4, 5, 7, 9	0.4	0.75	0.9	1.0
—	—	—	—	—
4, 5, 7, 9	0.4	0.75	0.9	1.0
—	—	—	—	—
9	1.0	—	—	—
4, 5, 7, 9	0.4	0.75	0.9	1.0
7, 9	0.7	1.0	—	—
7, 9	0.75	1.0	—	—
—	—	—	—	—

Combined capacity, MW	Cumulative capacity, MW	Scheduled on-line time, years	Combined unit cost, Mio$/MW
12	13	14	15
260.00	760.00	0.00	0.27
374.00	1134.00	5.98	0.24
280.00	1414.00	11.70	0.24
381.00	1795.00	14.85	0.30
200.00	1995.00	18.26	0.23
315.00	2310.00	19.77	0.29
484.50	2794.50	21.86	0.30
297.50	3092.00	24.58	0.41
268.00	3360.00	26.03	0.40

*See Fig. 11.46a; $ added fraction is assumed the same for whatever storage project is added next.
Source: Adapted from Erlenkotter [137].

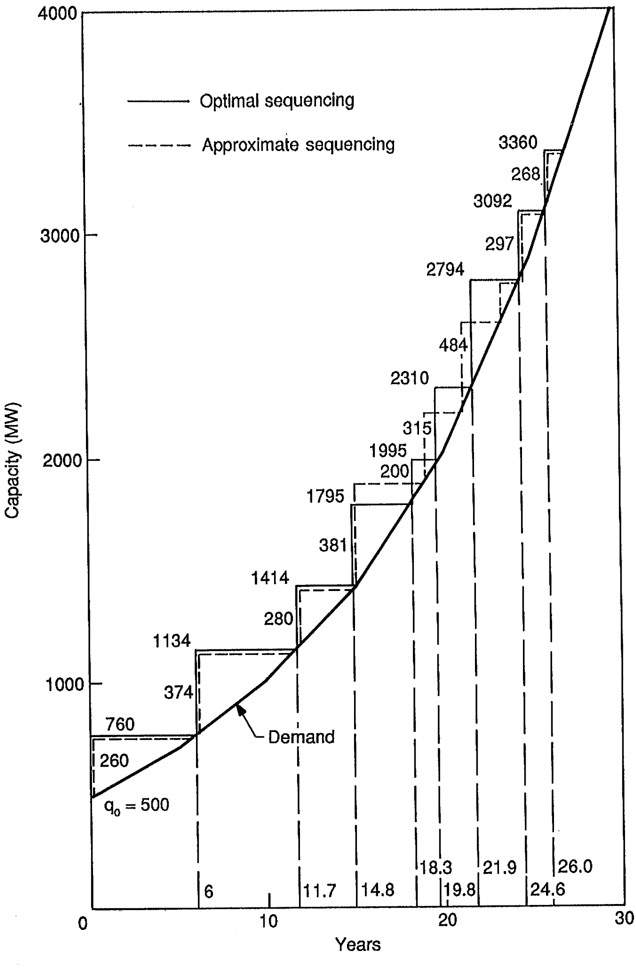


FIGURE 11.45 Optimal and approximate sequencing of interdependent hydropower projects. *(From Erlenkotter [137].)*

Capacity Sizing and Sequencing by Combined DP and LP Methods. A capacity expansion problem for two state variables, firm water and firm power, was described by Becker and Yeh [138]. The planning period of 50 years was subdivided into five equal stages. Precomputed pairs of firm water and firm power values were used as state variable levels. Forward dynamic programming was used to evaluate the set of firm water and power options. At each stage an LP technique was used to search for the least storages and generating plant capacities that can accommodate the projected water and power demands. The system was simulated by monthly time steps over a critical hydrologic period.

Another application of simultaneously sizing and sequencing projects for the Lancang River development in China was reported by Shih [139]. This

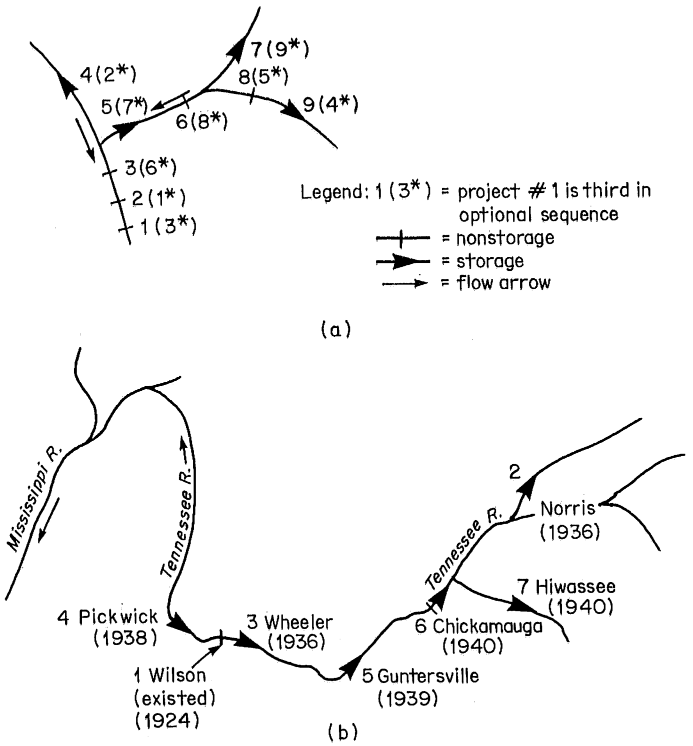


FIGURE 11.46 Examples of hydroproject sequences. (*a*) Reservoir system used in the example by Erlenkotter [137]. (*b*) Hydroproject sequence of first seven projects in the Tennessee River system [17].

project consists of a cascade of six projects with a combined installed capacity of 11,250 MW (see Fig. 11.47). The projects and some of their characteristics are given in Table 11.8. For a projected load over the planning period, a DP model was used to determine the optimal sequence as well as the project parameters. The objective was to find minimum total cost, including present investment and operation cost for a hydropower-thermal system. The thermal units were lumped into a single unit. Hydropower plant interaction was simulated by an LP model. The optimal sequence for four out of six Lancang River projects [139] is shown in Table 11.8. Project no. 3 (counted from the top of the cascade), with the lowest unit cost, is scheduled first. Next, no. 1, another small project, is scheduled that has the highest unit cost of the four. Thereafter, nos. 2 and 6 are scheduled.

Capacity Sizing and Sequencing by LP and NLP Methods. Investment problems typically contain discrete and continuous variables. Discrete variables are associated with project selection, in other words with functions that are activated or not. Accordingly, these variables assume only values of 1 or 0. Project output is represented by continuous variables, such as power and energy. Such problems

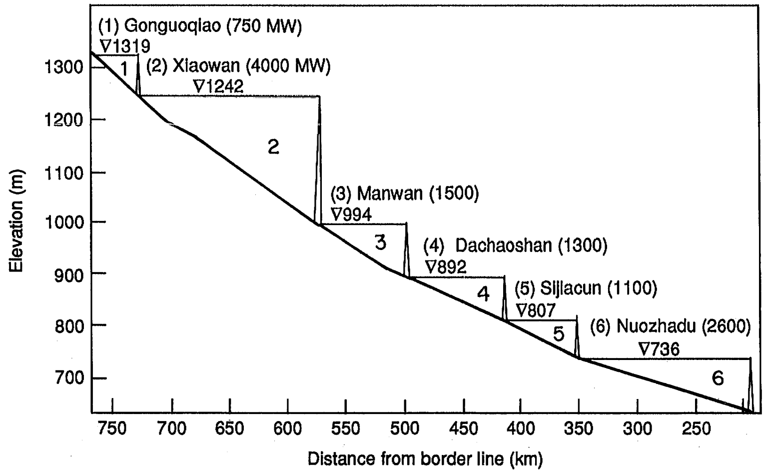


FIGURE 11.47 Profile of Lancang River in Yunnan Province, China. *(From Shih Jiayang [139], p. 616.)*

are called mixed-variable problems or, in the case here, mixed-integer problems. A typical formulation of such a problem is [91]:

$$\min\,(\mathbf{c}'\mathbf{x} + f(\mathbf{y})) \tag{11.75}$$

$$\text{subject to } \mathbf{A}\mathbf{x} + F(\mathbf{y}) = \mathbf{b},\ \mathbf{x} \geq 0$$

where $\mathbf{x}$ is the vector of continuous variables with n components, $\mathbf{A}$ is an $m \times n$ matrix of constraints; $\mathbf{c}'$ is a row vector of n coefficients; $\mathbf{y}$ is a vector of p components; $f(\mathbf{y})$ is a scalar function of $\mathbf{y}$ and $F(\mathbf{y})$ is a vector of functions (a set of constraints) of $\mathbf{y}$. In a more general case, the coefficients $\mathbf{c}$ and the matrix $\mathbf{A}$ may also be functions of $\mathbf{y}$, if $\mathbf{y}$ is construed as a discrete system variable. In a first step, one may fix $\mathbf{y}$, whereupon the functions of $\mathbf{y}$ become constants and the problem is reduced to an LP problem and easily solved. Then, in a second step, in an integer linear program, the $\mathbf{y}$ variables are changed and the process is repeated [91]. The objective is to select and size a set of projects that minimizes the objective function while meeting all system constraints and the load requirement. A recent application of this approach to hydropower system design has been described by Desrochers et al. [140].

Desrochers et al. [140] use a partitioning procedure proposed by Benders, known as Benders' decomposition method [91]. Partitioning procedures divide a problem into a master problem and a subproblem. For fixed design variables, a linear subproblem is solved to obtain the output of the system. Then, the master program, a linear integer program, is solved to obtain new design variables, including a new project set. This new design is then used to solve the subproblem again, and so on, until the minimum of the objective function is obtained. This approach analyzes projects in the system context and not individually, and avoids eliminating potentially favorable sites one by one.

A hydropower capacity expansion problem is basically nonlinear. Typical nonlinearities are inherent in the head-area and -volume relationships; the power formula, which contains the product of head and discharge; the discharge-head

TABLE 11.8 Result of Optimization of Lancang River Cascade

Project no.*	Project name	Expected completion year	Normal pool level (m)	Total storage, 10^6 m^3
3	Manwan	1990	993	913
1	Gonguoqiao	1993	1319	515
2	Xiaowan	1998	1240	15,340
6	Nuozhadu	1999	738	7474

Project no. *	Dead storage, 10^6 m^3	Installed capacity, MW	Discounted investment, Ch$Mio	Average unit cost, Ch$Mio/MW
3	700	1450	2480	1.71
1	390	900	5600	6.22
2	5050	3100	9940	3.21
6	4400	2200	11,750	5.34

*From top of cascade.
Source: Shih Jiayang [139], p. 620.

relationship; the power-discharge relationship, etc. LP algorithms can only be applied after all nonlinear functions have been linearized, usually by replacing nonlinear functions by piecewise linear functions. This linearization enlarges the problem considerably. However, as long as the constraint set is convex (in two dimensions, any straight line connecting two points on the boundaries lies in its entirety within the feasible region), any local minimum of a convex programming problem is a global minimum and LP can cope with very large problems. This also holds for a concave objective function, for which a global maximum is obtained [91]. But, if the constraint functions are nonconvex, unless special precautions are taken, no global extremum can be guaranteed. The linearized (separable) program may get trapped in local optima, far from the true optimum. Turgeon et al. [141] suggest a branch-and-bound routine which guarantees a global optimum of the resulting linearized problem.

A parametric mixed-integer LP method with a branch-and-bound algorithm for selecting the sites and evaluating project sizes has been described by Turgeon [142]. The method finds the optimum development for a river scheme consisting of a set of projects. The problem formulation is

$$\min \mathbf{c}'\mathbf{x} \tag{11.76}$$

$$\text{subject to } \mathbf{Ax} \geq \mathbf{b} + d\,\mathbf{h},\ \mathbf{x} \geq 0;\ \bar{\mathbf{x}} = 0 \text{ or } 1$$

where $\mathbf{c}'$ is the row vector of the objective function coefficients; $\mathbf{x}$ is a vector of size n; $\bar{\mathbf{x}}$ is a subvector of $\mathbf{x}$ consisting of integer variables which take only values of 0 or 1; $\mathbf{A}$ is an $m \times n$ matrix of linearized constraints; $\mathbf{b}$ is the vector of size m of right-hand sides of linearized constraints and contains input data; $\mathbf{h}$ is a change vector of dimension m; and d is a continuous scalar parameter.

The minimization of the objective function is subject to the linearized constraints on dam height, power-plant head and discharge, installed capacity, operation range, energy and capacity requirements, and reservoir elevation-volume relationships. Turgeon [142] gives an example that includes four sites. The most upstream one can accommodate only a reservoir, the other three can accommodate a reservoir and a power plant. A 14-year flow period is used by monthly time steps in the subproblem, which uses deterministic LP over all time steps. Both the monthly time steps and the foresight of the model can be expected to overestimate production, as the author acknowledges. This is a reminder that care must be taken in the choice of models and time steps in order to avoid nonconservative designs.

An application of Benders' decomposition that includes probabilistic reliability constraints was described by Bloom [143]. A hydropower expansion planning model including transmission planning using Lagrangian relaxation [144, 145] was described by Fahlbusch et al. [146].

Past Planning Experience and Heuristic Scheduling Rules

Several of the large hydropower systems in existence today were scheduled without the benefit of formal optimal sequencing methods. Many started out with a run-of-river downstream project, followed by an upstream storage project. The Columbia River system (Fig. 11.1) started with Bonneville on the lower mainstem (actually Rock Island, an older run-of-river project was first), followed by Grand Coulee, a major upstream storage reservoir [1]. The Tennessee River system started with Wilson, a run-of-river reservoir [17, 147], followed by Norris, a large upstream storage reservoir, which in turn was followed by four mainstem projects, Wheeler, Pickwick, Guntersville and Chickamauga.

From the examples given, some general rules for project scheduling can be summarized:

1. Start with the project that has the lowest cost/capacity ratio; continue by adding capacity and select the project that provides the lowest cost/capacity ratio for combined base and added capacity. Erlenkotter [137] found that such a heuristic approach resulted in a schedule very close to the one found by DP, as was shown in Fig. 11.45.
2. A downstream run-of-river project is a reasonable first step for starting a development. This step is favored by low-risk project financing and by the likelihood of opening up an early source of income. Also, run-of-river or limited-storage projects are usually less risky and less hazardous for people downstream. They allow gaining design and construction experience and usually cause fewer disruptions and environmental impacts than a high dam project. Alternating upstream storage projects with downstream run-of-river projects is a logical continuation of a river development.
3. The capacity added by each expansion step should be kept closely related to the demand growth by stepwise installation of equipment in order to minimize idle capacity during the period in which demand catches up with the added capacity. Large blocks should be scheduled when rapid demand growth is predicted and vice versa.
4. Projects that trigger additional capacity or energy production (upstream storage) should be scheduled early, as they may significantly lower the combined system unit capacity cost. The most disruptive and hazardous projects (with respect to dislocation of population, inundation of land, dam break hazard, and environmental impacts) should be delayed, so that they can be thoroughly

reviewed at subsequent stages (see also "Planning under Uncertainty," below) in the light of different information. Project costs and benefits should include all known externalities.

5. Careful model construction, specifically with respect to modeling the use of available information, is necessary. Biased models used in the sizing of structures may result in over- or underdesign, loss of reliability, misinvestment of capital, and failure to meet expectations (see "Planning under Uncertainty," below).

Power demands and system requirements are hard to predict for a 30- to 50-year planning period (see also Sec. 11.3). A ground rule is to aim at minimum capital investment now while maintaining flexibility for expansion later. Several equally likely or probability-weighted power demand scenarios should be projected and analyzed with the sequencing methods. By comparing the resulting construction sequences, the planner can see to which extent the first implementation period is affected. It may be found that the first period expansion is not sensitive to the predicted demand growth [134]. If this is the case, the implementation decision for that stage can be made with reasonable confidence.

One should not expect that a planned time sequence of projects will be implemented over the entire multiyear planning period. Similar to operations planning in the face of uncertainty, only the first-stage expansion is a candidate for implementation. A new assessment based on updated information can then be prepared when the time for the next investment approaches, say 10 years hence. At that time, again only the first stage is a candidate for implementation. As the first-stage decision is the one that is cast in concrete, additional criteria, such as reliability, flexibility for future corrections and expansions, and minimum externalities should be given serious consideration.

Planning under Uncertainty

Measures of Reliability. Reliability of a system is defined as the probability of performing as expected within a given period of time. Its complement is the probability of failure that is the probability of the system failing to perform as expected, e.g., of not meeting the load requirement.

The reliability of a system can be measured or specified in various ways. Possible measures are

1. Reliability constraints which specify an acceptable probability of meeting or exceeding a specified energy amount or capacity level
2. Reliability level found by balancing the incremental costs of additional safety (e.g., capacity addition) against the incremental reduction of the cost of failure (e.g., cost of loss of load). This search for a preferred reliability level is also called reliability optimization
3. Load-carrying capability (LCC) or dependable capacity of the system
4. Loss-of-load probability (LOLP) in hydrothermal systems
5. Expected unserved energy (EUE)
6. Frequency, duration, and magnitude of capacity deficiencies

An overview of probabilistic methods used in power system analysis can be found in IEEE [148] and Billington [149]. Methods with applicability to power system reliability can be found in Yen [150, 151] and Chow et al. [152].

Load-Carrying Capability and Dependable Capacity. The traditional definition of dependable capacity is "the load-carrying capability under the most adverse combination of system loads, hydrologic conditions, and plant capabilities" [153]. It represents a capacity that would be available 100 percent of the time. The capacity that can be so qualified represents usually only a fraction of the capacity that is available during a large percentage of the time. Therefore, dependable capacity, according to this definition, does not give proper credit to the actual capacity value of installed hydropower capability and other, probabilistic methods have been examined to measure LCC.

An extended definition of dependable capacity is "that capacity which can be relied upon to serve the system load with an acceptable degree of assurance, taking into account the characteristics of the resource, the probability of various hydrologic conditions, the characteristics of the power system of which the hydrosystem is a part, headwater and utility operating policies, and the characteristics of the load to be served" [154]. This definition is in line with the load-carrying capability (LCC) used in the utility industry [155, 156]. According to this definition, the LCC of a resource is the increase in total system load that can be served by the generation system at equal system reliability with the added resource.

Calculation of LCC. A system simulation model such as those described in Sec. 11.5 can be used to calculate system reliability (nonfailure probability) for successively increasing loads, first without and then with the additional capacity. The difference in capacity between the resulting two functions of reliability versus load at the desired reliability level is the LCC. The approach was proposed by Kuliasha [155]. As a measure of reliability, he suggested to use expected unserved energy (see below). Figure 11.48 gives reliability as a function of load for two sets of thermal units. The complement of the loss-of-load probability of Fig. 11.18 is used as a reliability measure. For an assumed reliability level of 95 percent, the 20-unit system with an installed capacity of 7.2 GW (1 GW = 1000 MW) can carry 4.4 GW and the 30-unit system with 8.5 GW can carry 5.5 GW. Thus, the actual capacity increase of 1.3 GW provides a 1.1-GW capacity increase at the 95 percent reliability level. A similar analysis can be carried out for hydropower capacity.

The procedure for calculating the reliability of hydropower capacity must be especially geared to the characteristics of hydropower resources. The basic philosophy of approach follows the hydropower-thermal generation cost calculation described in Sec. 11.2. First, a feasible hydropower schedule is found that satisfies all hydropower system constraints for the time step considered. This schedule is converted into a hydropower load-duration curve. In addition, an instantaneous maximum capacity level for the week is determined. The hydro schedule is then subtracted from the system generation requirement and the cost of the residual thermal generation is determined. This procedure is repeated until the proper hydrothermal schedule has been found. Reliability of the system is found by calculating the complement to the loss of load probability. The calculation of probabilities requires the evaluation of the hydrologic record or a representative sample of it and the evaluation of various generation requirements. For example, in order to evaluate a hydro capacity addition, the computation must evaluate the initial and augmented ca-

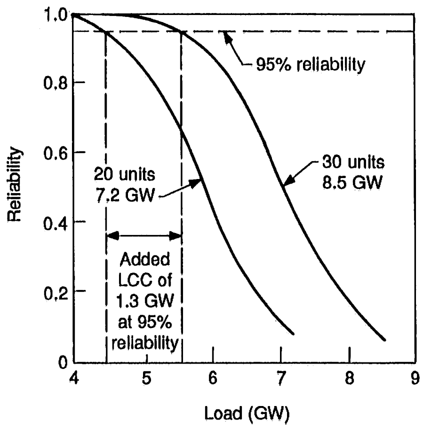


FIGURE 11.48 Load-carrying capability as a function of reliability.

pacities, the projected system generation requirements (loads), and all weeks of the hydrologic record in a series of nested loops:

Select system capacity (and capacity additions)

  Select system generation requirement (high, middle, low)

    Select hydrologic sequence (one year at a time)

      Select time step (one week at a time)

        Select hydro schedule (feasible)

          Evaluate residual thermal generation
            (for the thermal unit stack of the week)

        Next hydro schedule (until optimal)

      Next time step

    Next hydrologic sequence

  Next system generation requirement

Next system capacity

From the information generated, the two reliability versus load curves in Fig. 11.48 can be established. The LCC is the capacity between the two curves for the selected reliability level. Many mathematical models have been developed for power system expansion planning and probabilistic system performance assessment. An overview is given in [157]. The approaches discussed in Secs. 11.2 and 11.5 of this chapter emphasize the hydropower component. Quantifying uncertainty in hydropower planning is discussed in [158].

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APPENDIX A
UNIT CONVERSION

	Linear measure
1 inch (in)	= 0.02540 meters (m)
	= 2.540 centimeters (cm)
1 foot (ft)	= 0.3048 meters (m)
	= 30.48 centimeters (cm)
1 yard (yd)	= 0.91444 meters (m)
	= 91.44 centimeters (cm)
1 E. statute mile (sta. mi)	= 1.609 kilometers (km)
= 1760 yards (yd)	= 1609 meters (m)
= 5280 feet (ft)	
1 nautical mile (naut. mi)	= 1.853 kilometers (km)
= 6080 feet (ft)	= 1853 meters (m)
1 meter	= 39.37 inches
= 100 centimeters	= 3.281 feet
= 1000 millimeters	= 1.094 yards
1 kilometer	= 0.6214 statute miles
= 1000 meters	= 0.5396 nautical miles
	Square measure
1 square inch (in^2)	= 0.0006452 square meters (m^2)
	= 6.452 square centimeters (cm^2)
1 square foot (ft^2)	= 0.09290 square meters (m^2)
	= 929.0 square centimeters (cm^2)
1 square yard (yd^2)	= 0.8361 square meters (m^2)
	= 8361.0 square centimeters (cm^2)
1 acre (ac)	= 4047.0 square meters (m^2)
	= 0.4047 hectares (ha)
1 hectare	= 2.471 acres
= 10,000 square meters	
	Cubic measure
1 cubic inch (in^3)	= 16.39 cubic centimeters (cm^3)
1 cubic foot (ft^3)	= 0.02832 cubic meters (m^3)
1 cubic yard (yd^3)	= 0.7646 cubic meters (m^3)
1 acre foot (ft^3)	= 1233.0 cubic meters (m^3)
1 cubic centimeter	= 0.06102 cubic inch
1 cubic meter	= 35.31 cubic feet
= 1,000,000 cubic centimeters	= 1.308 cubic yards
	= 0.0008110 acre-feet

Liquid measure	
1 quarter (qr) [British]	= 290.8 liters (L)
1 gallon (gal) [British]	= 4.546 liters (L)
1 gallon (gal) [U.S.]	= 3.785 liters (L)
1 liter	= 0.2200 gallons [British]
	= 0.2642 gallons [U.S.]

Mass and weight measure	
1 kilogram (kg)	= 0.0685 slugs
1 slug	= 14.59 kilograms (kg)
1 long ton (l.t) [British]	= 1.016 tons (metric) (t)
1 short ton (sh.t) [British]	= 0.9072 tons (metric) (t).
1 pound (lb)	= 4.448 newtons (N)
1 ton (metric)	= 0.9842 long tons [British]
= 1000 kilograms	= 1102 short tons [British]
1 newton (N)	= 0.2248 pounds (lb)

Pressure measure	
1 bar	= 1 × 10^5 Pascal (Pa)
1 Pascal (Pa)	= 1 N/m^2
1 foot of water (39.2°F)	= 2.98 Pascal (Pa)
1 atmosphere (atm)	= 1.013 bar
1 inch of mercury (39.2°F)	= 3386 Pascal (Pa)
1 millimeter of mercury (0°C)	= 133.3 Pascal (Pa)

Velocity	
1 foot per second (ft/s)	= 0.3048 meters per second (m/s)
1 statute mile per hour (mi/h)	= 1.609 kilometers per hour (km/h)
1 meter per second	= 3.281 feet per second
1 kilometer per hour	= 0.6214 miles per hour

Discharge	
1 cubic foot per second (ft^3/sec)	= 0.02832 cubic meters per second (m^3/s)
1 gallon per minute	= 28.32 liters per second (L/s)
	= 2.16 × 10^{-3} cubic feet per second (ft^3/s)
1 cubic meter per second	= 35.32 cubic feet per second (ft^3/s)
1 liter per second	= 0.03532 cubic feet per second (ft^3/s)

Work, energy and moment	
1 Caloric (Cal)	= 0.00163 kilowatt-hour (kWh)
1 British thermal unit (Btu)	= 0.252 Calorie (Cal)
	= 0.000293 kilowatt-hour (kWh)
1 kilowatt-hour (kWh)	= 2,646,800 foot-pounds
	= 860 Calories
	= 3414 British thermal units
1 Calorie	= 3.97 British thermal units

Power	
1 foot-pound per second (ft-lb/s)	= 0.00135 kilowatt (kW)
	= 0.00182 horsepower (hp)
1 horsepower (hp)	= 76.05 meter-kilograms per second (mkg/s)
1 kilowatt (kW)	= 1.342 horsepower (hp)
	= 860 Calories per hour (Cal/h)
	= 0.239 Calories per second (Cal/s)
1 joule per second	= 1 watt (W)
1 horsepower	= 0.745 kilowatts
	= 550 foot-pounds per second
1 kilowatt	= 738 foot-pounds per second
1 horsepower (metric)	= 0.735 kilowatts (kW)
Heat capacity	
1 British thermal unit per cubic foot (Btu/ft^3)	= 8.92 Calories per cubic meter (Cal/m^3)
1 Caloric per cubic meter	= 0.112 British thermal units per cubic foot (Btu/ft^3)

APPENDIX B
GLOSSARY

Alternating current An electric current changing regularly from one direction to the opposite direction.

Ampere The common unit of measure of electrical current.

Anchor block A concrete block on a hillside for supporting and fixing the penstock.

Assembly floor (erection bay) A separated area inside or next to the powerhouse, having a platform of adequate load-bearing capacity for mounting one, exceptionally two machine units.

Axial-flow turbine A collective term for turbines with axial flow through the runner blades axially to the turbine shaft. Both propeller turbines and Kaplan turbines are axial-flow turbines.

Baseload Typically, the minimum load over a given period of time.

Blade servomotor The hydraulic cylinder actuated by governor oil pressure which supplies the force necessary to adjust the runner blades.

Brake jet The water jet that provides the counterrotational force used to decelerate an impulse runner.

Bulb The streamlined watertight housing for the bulb turbine generators.

Butterfly valve A disk-shape closing body rotating around a shaft perpendicular to the penstock axis.

Capacity The greatest load which a piece of equipment can safely serve.

Clearance loss The discharge which escapes through the gaps between the casing and the runner and through the sealings of the runner of the turbine, and therefore is lost for power production. Also known as leakage.

Conduit/penstock valve *See* Control valve.

Connecting rods The elements connecting the servomotor piston rod to the gate operating ring.

Control valve (conduit/penstock valve) The valve installed between the surge tank or the headpond and the penstock.

Dam A massive wall or structure built across a valley or river for storing water.

Design head The head at which the turbine is designed to operate at maximum efficiency.

Direct current The electric current going in one direction only.

Discharge ring The structural member on a Francis turbine that surrounds the runner band. On a propeller turbine, it surrounds the blades and forms a guide for the water. It may be integral with the bottom ring. The draft tube liner is attached to the downstream end of the discharge ring.

Diversion weir; diversion dam A weir or dam created for the sole purpose of diverting flow from the water course into a power conduit.

Draft tube The diffuser which regains the residual velocity energy of the water leaving the turbine runner.

Draft tube liner The steel lining used in the draft tube to protect the concrete from the high velocity of the water.

Elbow draft tube A diffuser in the form of an elbow.

Energy The power of doing work, for a given period. Usually measured in kilowatt-hours.

Erection bay *See* Assembly floor.

Fixed-blade propeller-type turbine An axial-flow reaction turbine with blades keyed to the hub, unlike those of the Kaplan turbine.

Follower-type gate valve A prolonged sliding plate with an opening on its lower part, of an area equal to the cross section of the penstock. In flow-through position this opening faces the pipe cross section, in closed position the upper part of the plate blocks the cross section.

Forebay The upstream part of the baylike extension of the river for the location of a powerhouse.

Francis turbine A radial-inflow reaction turbine, where the flow through the runner is radial to the shaft.

Gate chamber The part of the power conduit where the gate is accommodated.

Gate house Located above the gate chamber, contains the hoist and pertaining equipment for operating the gate.

Gate linkage Any linkage connecting the gate operating ring and the wicket gates.

Gate operating ring The ring rotated by the servomotors which distributes the force from the servomotors to the individual wicket gate linkages to provide simultaneous movement of all wicket gates.

Gate servomotors The hydraulic cylinders actuated by oil pressure which supply the force necessary to operate the wicket gates through the gate operating ring.

Gate valve A leaflike closing gate, sliding in a plane perpendicular to the penstock axis.

Generator A machine powered by a turbine which produces electric current.

Generator brake A device for stopping the revolving part of the generating unit.

Gross head The difference between headwater level and tailwater level at the powerhouse.

Guide vanes The streamlined movable blades regulating inflow to the turbine runner.

Head cover The axisymmetric structural member in vertical machines that spans the top of the distributor, provides the separation between the watered runner chamber and the dry turbine pit, and supports the main shaft packing box and the main bearing.

Head gate Built in the intake portion of the entrance flume of low-head and medium-head powerhouses, or in the headpond of a high-head power plant, to control inflow into the penstock.

Headrace That portion of the power canal which extends from the intake works to the powerhouse.

Headwater The water upstream from the powerhouse, or generally, the water upstream from any hydraulic structure creating a head.

Headwater elevation; headwater level The height of the headwater in the reservoir.

Headworks *See* Intake.

Housing The enclosure, surrounding an impulse runner, which forms the aerated chamber in which the runner operates.

Hydraulic efficiency An efficiency component of the turbine, expressing exclusively the power decrement due to hydraulic losses (friction, separation, impact), including the losses in the scroll case and the draft tube.

Hydroelectric power The electric current produced from water power.

Hydroelectric powerplant A building in which turbines are operated, to drive generators, by the energy of natural or artificial waterfalls.

Hydropower plant (hydropower development) The comprehensive term for all structures (one powerhouse and pertaining installations) necessary for utilizing a selected power site.

Hydropower station A term sometimes equivalent to the powerhouse, sometimes including the structures situated nearby.

Hydropower system Two or more power plants (and therefore two or more powerhouses) which are cooperating electrically through a common network.

Impeller *See* Runner.

Impeller vanes *See* Runner buckets.

Inlet valve (turbine valve) The valve installed immediately ahead of the turbine, i.e., at the bottom of the penstock or the pressure shaft.

Intake (intake works, headworks) A hydraulic structure built at the upstream end of the diversion canal (or tunnel) for controlling the discharge and preventing silt, debris, and ice from entering the diversion.

Intake tower A pressure tunnel intake erected separately in the reservoir for housing the flow control valves or gates.

Jet nozzle *See* Nozzle.

Kaplan turbine An axial-flow reaction turbine with adjustable runner blades and adjustable guide vanes.

Kinetic energy Energy which a moving body has because of its motion, dependent on its mass and the rate at which it is moving.

Leakage *See* Clearance loss.

Load The amount of electric energy delivered at a given point.

Load demand A sudden electrical load upon the generating units, inducing the rapid opening of the turbines.

Load factor The ratio of the annually produced kilowatt-hours and of the energy theoretically producible at installed capacity during the whole year.

Load rejection A sudden cessation of electrical load on the generating units, inducing the rapid closure of the turbines.

Main guide bearing The bearing located nearest the runner.

Main shaft The rotating element that transmits torque developed by the turbine runner to the generator rotor or transmits torque developed by the motor to the pump impeller.

Manifold (header) The lowest portion of the penstock from which the unit penstocks bifurcate.

Mechanical efficiency An efficiency component of the turbine, expressing exclusively the power losses of the revolving parts, due to mechanical friction.

Mixed-flow turbine *See* Radial-inflow turbine.

Needle valve A streamlined regulating body moving like a piston in the enlarged housing of the valve.

Net head That part of the gross head which is directly available for the turbines.

Nozzle (jet nozzle) A curved steel pipe supplied with a discharge-regulating device to direct the jet onto the buckets in impulse runners.

Peakload The greatest amount of power given out or taken in by a machine or power distribution system in a given time.

Pelton turbine The main type of turbine used under high heads.

Penstock (pressure pipe) A pressurized pipeline conveying the water in high-head developments from the headpond or the surge tank to the powerhouse.

Penstock valve *See* Control valve.

Pier The structural member used to support the upper surface of the horizontal portions of water passages such as the draft tube and the spiral case inlet.

Pier nose The steel lining used at the upstream ends of a pier.

Pit liner The plate steel lining in the turbine pit. It serves as an internal form and as a protective liner for the surrounding concrete.

Plant discharge (plant discharge capacity) The maximum discharge that can be utilized by all turbines of the power plant with full gateage, i.e., the entire discharging capacity of the turbines.

Plug valve *See* Spherical valve.

Pondage That rate of storage in run-of-river developments which can cover daily peaks only.

Power The rate at which work is done by an electric current or mechanical force, generally measured in watts or horsepower.

Powerhouse The main structure of a water power plant, housing the generating units and the pertaining installations.

Pressure pipe *See* Penstock.

Propeller-type turbine The collective term for axial-flow reaction turbines. In this terminology it denotes two types: fixed-blade propeller turbines and adjustable-blade propeller turbines, i.e., Kaplan turbines. (This is the original English terminology, whereas in continental practice the fixed-blade type is also termed briefly propeller-type turbine.)

Pumped-storage development A combined pumping and generating plant; hence it is not a primary producer of electrical power but, by means of a dual conversion, stores the superfluous power of the network and returns it in peak load periods as would a battery.

Radial-inflow turbine (mixed-flow turbine) A collective term for turbines in which the water enters radially into the runner and leaves it axially. Francis turbines and some similar but obsolete types are radial-inflow turbines.

Reaction turbine A collective term for turbines in which the water jet enters the runner under a pressure exceeding the atmospheric value. The water flowing to the runner still has potential energy, in the form of pressure, which is converted into mechanical power along the runner blades.

Reservoir An artificial lake into which water flows and is stored for future use.

Reversible pump-turbine A hydraulic machine used in pumped-storage developments, suitable for operating both as a pump and as a turbine.

Runaway speed The maximum rotational speed attained by a turbine with no generator load. It is attained in load rejection.

Run-of-river plant A development with little or no pondage regulation such that the power output varies with the fluctuations in the stream flow.

Runner The rotating element of the turbine which converts hydraulic energy into mechanical energy. For reversible pump-turbines, the element is called an *impeller* and converts mechanical energy into hydraulic energy for the pump mode.

Runner band The lower axisymmetric portion (outer shroud) of the runner to which the lower or outer ends of the runner buckets attach.

Runner band seal The close running clearance between the rotating runner band and the stationary bottom or discharge ring. The close clearance restricts the flow of water between the high-pressure zone and the low-pressure zone of the runner.

Runner blades The contoured components of a propeller runner that radiate from the hub, deflect the flowing water and transfer the energy to the runner hub. The blades may be angularly adjustable or rigidly fixed in the hub.

Runner buckets (impeller vanes) The contoured components of Francis and impulse runners that deflect the flowing water and transfer the energy to the runner crown or disk when operating as a turbine.

Runner cone The extension of the runner crown, or runner hub, that guides the water as it leaves the runner.

Runner crown The upper axisymmetric portion (inner shroud) of the runner which provides a mechanical attachment to the main shaft and to which the top or inner ends of the runner buckets attach.

Runner crown seal The close running clearance between the rotating runner crown and the stationary head cover. The close clearance restricts the flow of water into the chamber between the top of the runner and the bottom of the head cover.

Runner hub The axisymmetric portion of a propeller runner which provides the attachment to the main shaft and to which the inner ends of the runner blades attach.

Scroll case (spiral case) A spiral-shaped steel intake guiding the flow into the wicket gates of the reaction turbine.

Semi-scroll case (spiral case) A concrete intake directing flow to the upstream portion of the turbine with a spiral case surrounding the downstream portion of the turbine to provide uniform water distribution.

Setting The vertical distance between the tailwater level and the center of a turbine runner.

Specific speed A universal number that indicates the machine design, i.e., impulse, Francis, or axial.

Speed increaser The geared drive unit which increases turbine shaft speed to drive the generator at an economic speed for power generation.

Spherical valve (plug valve) A cylindrical closing body, encased in a spherical housing, and rotating around a shaft perpendicular to the penstock axis.

Spilling surge tank Different types of surge tanks whose riser shaft or upper chamber (if any) has an overflow berm discharging the excess water into a wasteway in order to limit upsurges.

Spiral case *See* Scroll case; Semi-scroll case.

Stay ring The structural member surrounding the wicket gates having two annular rings connected by a number of fixed stay vanes in the water passages. Its function is to provide support and structural continuity between the upper and lower portions of the turbine distributor, while guiding the water as it enters or leaves the spiral case.

Stay vane One of the streamlined steel or cast-iron supports built at the cylindrical discharging surface of the scroll case. The stay vane serves mainly structural purposes, though it may have a hydraulic function as well.

Stay-vane ring The stay vanes and the lower and upper speed ring holding them.

Surge tank; surge chamber A hydraulic structure erected in the power conduit of high-head developments between the pressure tunnel and the penstock (or pressure shaft) to protect the pressure tunnel from water-hammer effects, to diminish overpressures due to water hammer in the penstock itself, and to store water for sudden load demand. A surge tank can also be located between the draft-tube port and the tailwater tunnel.

Tailrace That portion of the power canal which extends from the powerhouse to the recipient watercourse.

Tailwater The water downstream from the powerhouse. In general, the water downstream from any hydraulic structure creating a head.

Tidal power plant; tidal power station A power station that utilizes the potential hydraulic power originating from the tidal cycles of the sea.

Turbine A device which produces power by diverting water through blades of a rotating wheel which turns a shaft to drive generators. *See also* specific types of turbines.

Turbine discharge capacity The maximum flow that can be discharged by a single turbine at full gateage.

Turbine efficiency The entire efficiency of the turbine, i.e., the product of hydraulic mechanical and volumetric efficiencies.

Turbine valve *See* Inlet valve.

Underground power station A development where at least the machine hall is located in an excavated cavern.

Volumetric efficiency An efficiency component of the turbine, expressing exclusively the power losses due to leakage (clearance losses, etc.).

Volt (V) The unit of electromotive force or potential difference that will cause a current of 1 ampere to flow through a conductor with a resistance of 1 ohm.

Wearing rings Replaceable rotating rings fastened to the runner or adjacent stationary rings fastened to the head cover and the bottom ring (or discharge ring), thus forming removable seals with small clearances.

Water power A general term used for characterizing both power (kW) and energy (kWh) of watercourses, lakes, reservoirs, and seas.

Wicket gates The angularly adjustable streamlined elements which control the flow of water to the turbine or discharge from the pump.

APPENDIX C
PHYSICAL PROPERTIES OF WATER AND GRAVITATIONAL ACCELERATION

TABLE C.1 Physical Properties of Water in SI Units

Temp., °C	Specific weight γ, N/m^3	Density ρ, kg/m^3	Viscosity $\mu \times 10^3$, $N \cdot s/m^2$	Kinematic viscosity $\nu \times 10^6$, m^2/s	Surface tension $\sigma \times 10^2$ N/m	Vapor pressure head p_v/γ,* m	Bulk modulus of elasticity $K \times 10^{-7}$, N/m^2
0	9806	999.9	1.792	1.792	7.62	0.06	204
5	9807	1000.0	1.519	1.519	7.54	0.09	206
10	9804	999.7	1.308	1.308	7.48	0.12	211
15	9798	999.1	1.140	1.141	7.41	0.17	214
20	9789	998.2	1.005	1.007	7.36	0.25	220
25	9778	997.1	0.894	0.897	7.26	0.33	222
30	9764	995.7	0.801	0.804	7.18	0.44	223
35	9749	994.1	0.723	0.727	7.10	0.58	224
40	9730	992.2	0.656	0.661	7.01	0.76	227
45	9711	990.2	0.599	0.605	6.92	0.98	229
50	9690	988.1	0.549	0.556	6.82	1.26	230
55	9666	985.7	0.506	0.513	6.74	1.61	231
60	9642	983.2	0.469	0.477	6.68	2.03	228
65	9616	980.6	0.436	0.444	6.58	2.56	226
70	9589	977.8	0.406	0.415	6.50	3.20	225
75	9560	974.9	0.380	0.390	6.40	3.96	223
80	9530	971.8	0.357	0.367	6.30	4.86	221
85	9499	968.6	0.336	0.347	6.20	5.93	217
90	9466	965.3	0.317	0.328	6.12	7.18	216
95	9433	961.9	0.299	0.311	6.02	8.62	211
100	9399	958.4	0.284	0.296	5.94	10.33	207

*γ = 9806 N/m^3.

TABLE C.2 Physical Properties of Water in USC Units

Temp., °F	Specific weight γ, lb/ft^3	Density ρ, slugs/ft^3	Viscosity $\mu \times 10^5$, lb · s/ft^2	Kinematic viscosity $\nu \times 10^5$, ft^2/s	Surface tension $\sigma \times 10^2$ lb/ft	Vapor pressure head p_v/γ,* ft	Bulk modulus of elasticity $K \times 10^{-3}$, lb/in^2
32	62.42	1.940	3.746	1.931	0.518	0.20	293
40	62.42	1.940	3.229	1.664	0.514	0.28	294
50	62.42	1.940	2.735	1.410	0.509	0.41	305
60	62.35	1.938	2.359	1.217	0.504	0.59	311
70	62.29	1.936	2.050	1.059	0.500	0.84	320
80	62.22	1.934	1.799	0.930	0.492	1.17	322
90	62.13	1.931	1.595	0.826	0.486	1.61	323
100	62.00	1.927	1.424	0.739	0.480	2.19	327
110	61.87	1.923	1.284	0.667	0.473	2.95	331
120	61.71	1.918	1.168	0.609	0.465	3.91	333
130	61.55	1.913	1.069	0.558	0.460	5.13	334
140	61.39	1.908	0.981	0.514	0.454	6.67	330
150	61.19	1.902	0.905	0.476	0.447	8.58	328
160	61.00	1.896	0.838	0.442	0.441	10.95	326
170	60.81	1.890	0.780	0.413	0.433	13.83	322
180	60.58	1.883	0.726	0.385	0.426	17.33	313
190	60.36	1.876	0.678	0.362	0.419	21.55	313
200	60.10	1.868	0.637	0.341	0.412	26.59	308
212	59.84	1.860	0.593	0.319	0.404	33.90	300

*γ = 62.4 lb/ft^3.

TABLE C.3 Acceleration g (in m/s^2) due to Gravity*

Latitude ϕ, degrees	Height z in meters above mean sea level: 0	1000	2000	4000
0	9.780	9.777	9.774	9.768
10	9.782	9.779	9.776	9.770
20	9.786	9.783	9.780	9.774
30	9.793	9.790	9.787	9.781
40	9.802	9.799	9.796	9.789
50	9.811	9.808	9.804	9.798
60	9.819	9.816	9.813	9.807
70	9.826	9.823	9.820	9.814

*The above table is derived from the formula: $g = 9.806\,17(1 - 2.64 \times 10^{-3} \cos 2\phi + 7 \times 10^{-6} \cos^2 2\phi) - 3.086 \times 10^{-6} z$. The international standard value of g is 9.806 65 m/s^2.

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John S. Gulliver is associate professor at the St. Anthony Falls Hydraulic Laboratory of the Department of Civil and Mineral Engineering of the University of Minnesota. He is active in hydropower analysis, design, and research, including the analysis of water quality impacts and conveyance facility design, and is the technical editor of "Hydro Review."

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