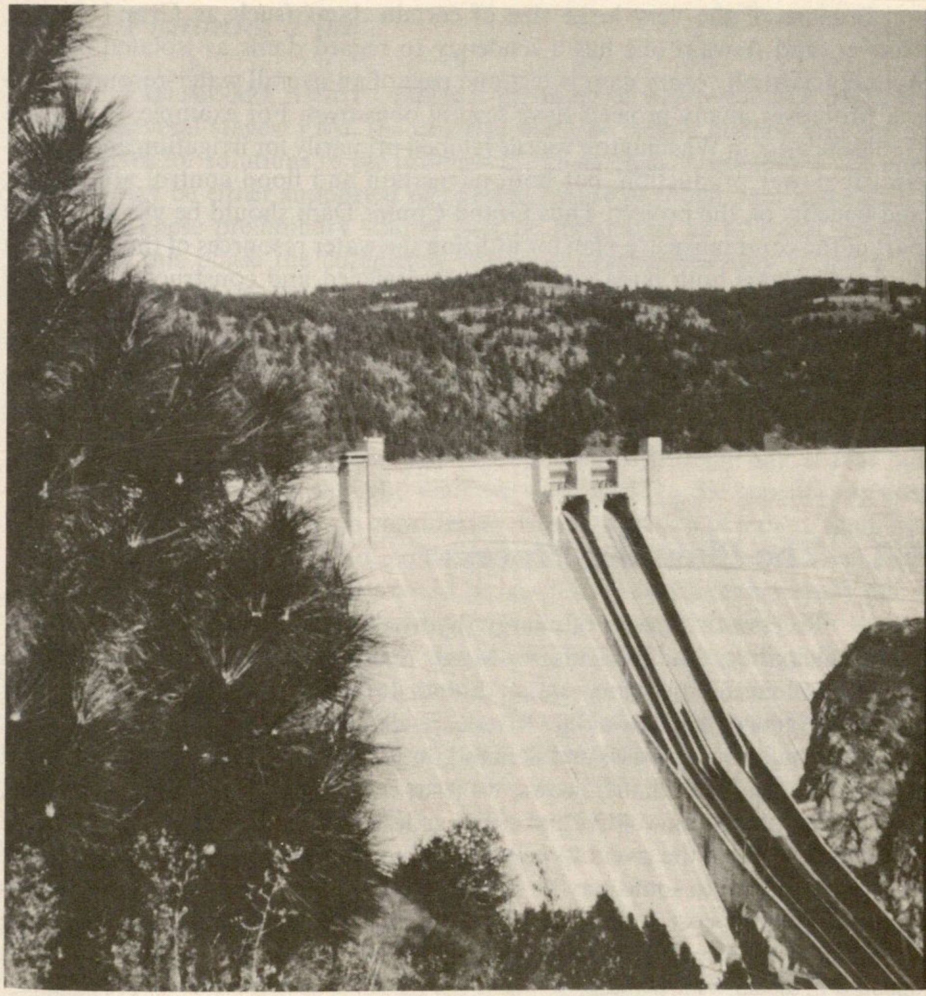


6

Dworshak Dam on the North Fork of the Clearwater River near Orofino, Idaho is a concrete gravity dam with a height of 717 feet above bedrock. The gross storage in the reservoir developed by the dam is 3,500,000 acre feet and the usable storage for power and flood control is 2,000,000 acre feet. (Photo by Kimberle A. Fennema)

Dams and Reservoirs

Because of the very large size of certain dams (such as Grand Coulee, Hoover, and Aswan) one has a tendency to regard dams as isolated entities. Actually, virtually every dam is just one part of an overall water-resource project. Moreover, many projects have several objectives. For example, the Grand Coulee project in Washington was developed primarily for irrigation and hydroelectric power production, but both navigation and flood control are part of, and benefits of, the project. Thus Grand Coulee Dam should be viewed as one part of the comprehensive plan for utilizing the water resources of the Columbia River. All dams both large and small are designed and constructed to satisfy one or more water-resource objectives. The next section is a brief summary of the type of planning process that eventually leads to the constructed dam. Subsequent sections deal with the actual design and construction of dams. The last section in this chapter focuses on the role of the reservoir in the water resource scheme.

6-1 The Planning Process

Rivers are sources of energy (hydroelectric power) and water supply for municipalities and agriculture. Many rivers also serve as transportation arteries and are sources of recreation. Flooded rivers often wreak havoc, causing property damage and loss of life. Rivers are also often used for sewage disposal. As our population increases and demand for food, water, power, recreation, and land (often on flood plains) grows, we (that is, local and federal governments, private companies, and individual citizens) design and construct water-resource projects to control the rivers to our advantage. However, a use of the river that may be beneficial to some may be detrimental to others. Moreover, most water-resource projects cost a good deal of money. Careful planning should therefore be done to achieve optimum utilization of river basins as a whole as well as specific projects within them.

Definition of Planning

Planning means determining, in an orderly manner, the best way to accomplish a particular objective by evaluating various alternatives. For example, in the context of water resources, a problem may exist of not having enough water for the demands of a large city during drought periods. Careful planning should be done to bring about a solution to the problem. Planning involves evaluating several possible solutions. Some solutions entail building structures such as dams and supply pipes. Therefore, planning also involves designing these features and estimating their cost; cost comparisons often determine the "best" alternative.

Planning Phase

Preliminary Study Efficient planning of water-resource projects is done in several stages. First, the problem must be clearly defined and various alternatives for solutions to the problem are envisioned. A number of the alternatives may be either supported or rejected by use of rough data and shortcut analyses. These preliminary studies may be thought of as a coarse screening phase. In this phase the completely infeasible alternatives may be ruled out.

For example, let us return to the problem of the major city that does not have enough water to satisfy the demand during drought periods. Alternatives to this problem may include (1) water conservation programs, (2) constructing a dam on a river to provide storage from which water may be drawn during a drought, (3) a combination of water conservation and dam construction. Perhaps the water conservation can be implemented initially and a dam built eventually. In any event, thought will have to be given to the benefits and costs of each alternative project. In considering the construction of a dam, there may be several streams on which it could be built, and on any given stream, there may be several sites at which it could be located. The preliminary study allows all these different possibilities to be narrowed down to perhaps two or three of the best choices. By using rough data and simplified analyses in the reconnaissance study much time and money is saved over that which would be involved if a more thorough study were done on all the alternatives. The preliminary study will either result in a recommendation to cease further study (all alternatives are unsatisfactory for economic, environmental, or social reasons) or a recommendation will be made to proceed to the next phase, the feasibility study.

Feasibility Study During the feasibility study, a project or plan is developed to determine whether or not it should be implemented. The project or plan must be technically and economically feasible and environmentally acceptable. The preliminary study narrows the possible solutions down to a few reasonable alternatives, and the feasibility study establishes the scope, magnitude, and all essential features of these alternatives. Cost estimates are also refined so that a good assessment of cost versus benefits may be made for each alternative. Moreover, environmental issues are addressed and environmental impact statements (EIS) are prepared in this phase. Comparisons of the alternatives are then made, and the best project or plan is determined. Then a decision is made either to cease the study (too costly, or insurmountable environmental or social problems exist) or to proceed to the final phase.

Final Study In the final phase of planning, detailed designs are made and plans and specifications are developed. The cost and benefit estimates may not change greatly from the feasibility phase to the final phase, though they may; for example, the cost of money (interest rates) may change significantly during that period. The possibility of major changes in the project should therefore be considered even after the final study has been done.

Planning Considerations of Projects Involving a Dam and Reservoir

Whether it be for municipal water supply, irrigation, or hydropower, several items must be considered in the planning and design of a dam and reservoir:

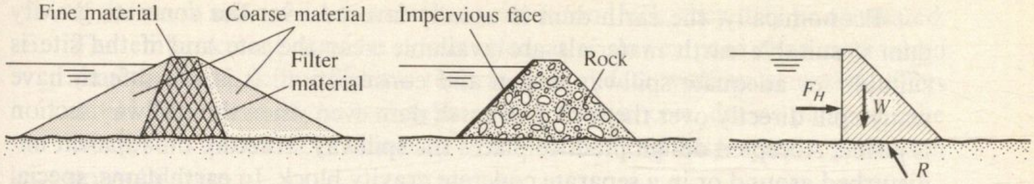
1. Hydrological data. Data of the stream that the dam is to be built on are analyzed to determine flood and drought flows and to determine the required capacity and operating procedure for the reservoir. Also, the required spillway capacity can be determined from the hydrologic data.
2. Geologic data. Some data of the geology of the area that the dam is to be built on will no doubt exist; however, on-site inspection, geologic mapping the drilling of exploratory holes, and collection of core-sample data by geologists are usually required. These data reveal the structural ability of the foundation material to withstand the loads that may act on it and indicate the leakage and erosion problems that may be encountered.
3. Reservoir data. A complete assessment of the area to be inundated by the reservoir must be made. This includes topographic maps, land ownership, land classification, and location of roads and public utilities. These data are used to estimate the cost of land acquisition and relocation of roads and utilities.

Many other factors, such as availability of materials and manpower, environmental aspects, sedimentation and dam safety must also be considered in the planning and design of a dam and reservoir.*

6-2 Types of Dams

Dams are classified according to the material (earth, rock, concrete) from which they are built (see Fig. 6-1) and according to their configuration and the way in which they resist the forces imposed on them. Thus a gravity dam is one in which the gravitational forces (such as the weight of the dam itself) are great enough to resist the overturning moment and sliding force of the hydrostatic forces imposed on it (Fig. 6-1*c*). Another type of gravity dam is the

* For more details on planning considerations, see Peterson (12) and the U.S. Bureau of Reclamation publication on the design of dams (20).



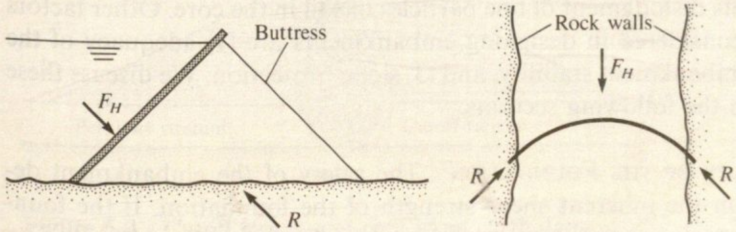
(a) Earthfill dam (b) Rockfill dam (c) Concrete gravity dam

Figure 6-1 Earth dam, rockfill dam, and concrete gravity dam

buttress dam (Fig. 6-2*a*), in which reinforced concrete slabs constitute the face of the dam and are supported by vertical buttresses at intervals of 50 to 100 ft. In contrast, an arch dam is designed to transfer the imposed loads to adjacent rock walls on either side of the canyon it is located in (Fig. 6-2*b*). Both earthfill and rockfill dams are special types of gravity dams. We consider these types of dams in more detail in the following sections.

Earth Dams

INTRODUCTION The first dams ever built by humans were made of earth. Biswas (1) notes that along the Nile River at about 2000 B.C. during the flood seasons some of the flood water was diverted through a canal to a natural storage depression. The flow in this canal was reportedly controlled by two earthen dams. On the Arabian peninsula in what is now Yemen, an earthfill dam was constructed sometime between 1000 B.C. and 700 B.C. Called Marib dam, it was 33 ft high and 1900 ft long (1). In Sri Lanka, an earth dam 70 ft high and 11 mi long was completed about 500 B.C. and it is still in use (14). Today, the most common type of dam is still the earth dam, which can be designed and built on almost any given site and foundation condition by using a wide range of earth materials available at the site. As for all dams, thorough field investigations are required for the design and construction of sound earth dams.



(a) Section of a buttress dam (b) Plan view of an arch dam

Figure 6-2 Buttress and arch dams

Economically, the earth dam is usually favored over the concrete gravity dam if suitable earth materials are available near the site and if the site is suitable for adequate spillway design and construction. It is not safe to have water spill directly over the top of an earth dam even when the spillway section is paved. Accepted design practice places the spillway structure on adjacent undisturbed ground or in a separate concrete gravity block. In earth dams, special attention must be given to the design, construction and maintenance of the dam to resist internal erosion. If this is properly done, earth dams are as safe as any other type.

Some earthfill dams have been constructed by placing the earth in the dam by pumping the fill material to the site in a slurry and letting the water drain off. A dam constructed in this manner is called a *hydraulic fill dam.* The largest hydraulic fill dam in the United States is the Fort Peck Dam in Montana (126 million cu yd of fill material). However, stability problems can develop when constructing this type of dam. For example, a huge slide (5×10^6 cu yd) occurred during the construction of Fort Peck Dam (11). With the development of large earthmoving equipment, the cost of placing earth with earthmovers is competitive with the hydraulic process. Therefore, almost all earth dams are now constructed by using earthmoving equipment to place and compact the materials in the dam in layers so that a dense stable fill is produced. Such a dam is called a *rolled fill earth dam.*

The built-up section of an earthfill or rockfill dam is the *embankment*. The embankment must resist the hydrostatic forces of the water in the reservoir and must contain a section or zone impervious enough to prevent excessive seepage. If the soil material is relatively fine and abundant, a *homogeneous embankment* may be constructed. However, it is more common to find a variety of materials at the site of the dam. In this case, the usual practice is to design a *zoned embankment* (Fig. 6-1*a*). The finer material is compacted to produce a relatively impervious zone, and this is usually placed near the central part of the dam. The coarser material is placed upstream and downstream of the impervious core. This coarse material primarily provides stability to the dam. The interface zone between the fine material of the *core* and coarse material on the downstream side must be carefully designed and placed to eliminate the possibility of erosion of the fine material as seepage occurs after the reservoir is filled. Thus the interface zone should be a graded filter material that allows passage of seepage water but prevents dislodgment of fine particles of soil in the core. Other factors that should be considered in designing embankments are (1) adequacy of the foundation, (2) embankment stability, and (3) slope protection. We discuss these in more detail in the following sections.

Adequacy of the Foundation The safety of the embankment depends, in part, on the inherent shear strength of the foundation. If the foundation soils are weak, special steps must be taken. For example,

1. The slopes of the embankment may be flattened to distribute the load over a greater area.

2. If the weak soils of the foundation are not too thick, they may be excavated.
3. The embankment may be constructed at a slower pace than normal so that the weak soils will have time to consolidate without excessive differential settlement of the embankment. In this process, drains placed in the weak soils provide for removal of water and can help speed up the soil consolidation.

Whether the foundation is composed of weak or strong soils, the surface of the foundation material must be carefully prepared before the embankment material is laid down. This preparation includes removing all vegetation; digging out stumps and large roots; and stripping off sod, top soil, and any other organic material. Pockets of soft compressible soils should also be removed. When all the organic and soft material has been removed, stump holes and other pockets should be filled with soil and compacted by power tampers. The foundation is then plowed and rolled to compact the upper surface of the foundation. Sometimes a cutoff trench is excavated in the foundation material so that a contact is made between the core material and bedrock (see Fig. 6-3). Constructing a cutoff ensures that any old drains, pervious zones, or abandoned pipes are found and removed. However, if a cutoff trench is not used, then an inspection trench having a minimum width of 6 ft should be excavated to check for and remove undesirable features. The trench should then be backfilled and properly compacted.

If the foundation material is rock, it should be cleaned to remove all loose rock. Open joints in the rock should also be cleaned and filled with concrete. To prevent excessive seepage through fissures and crevices, grouting the foundation rock may be necessary.*

EMBANKMENT STABILITY The design of the embankment should be based on the available soils, their water content, and the need for drying or wetting of the soils to achieve optimum conditions for compaction. Any large boulders in the soil should be removed because they will make rolling of the

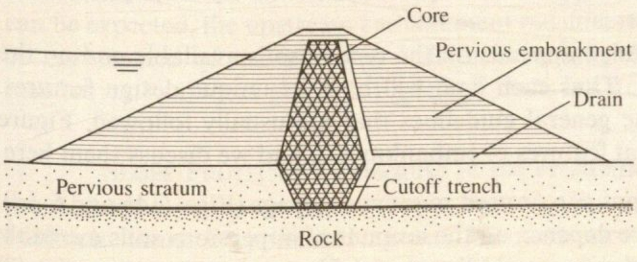


Figure 6-3 Cutoff trench and core of an earth dam

* For more details on foundation and abutment preparation, see Golzé (8) and USBR (20).

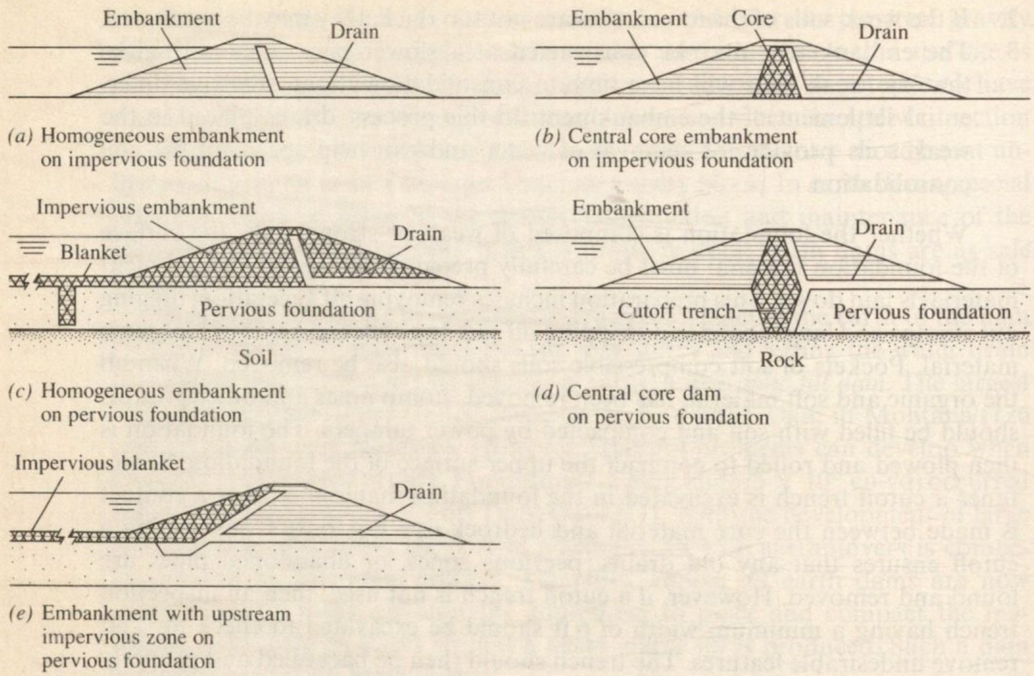


Figure 6-4 Different types of earth dam embankment (8)

soil difficult or impossible. It is impossible to prevent all seepage through any dam. A portion of an embankment dam will always be saturated and the permeability of the material will determine the rate at which water seeps through the embankment. At the downstream side of the core the fine materials of the core could be carried away from the core and into the coarser embankment material if measures are not taken to prevent this erosion. Thus filter zones should be provided on the downstream side of the core and a safe path (often in the form of special drains) should be provided to convey seepage water to the river channel downstream.

The embankment design depends on the type of soils available and on the objectives of the project. Thus each dam will have its unique design features. However, there are some general guidelines that are usually followed. Figure 6-4 illustrates the different features of embankments, and we discuss them here.

1. A relatively impervious core is used to reduce seepage (Figs. 6-4*b* and 6-4*d*). The width of the core depends on the amount of impervious soils available and the method of placement of fill material. However, current practice (8) calls for a bottom width no less than one quarter of the net head between maximum pool level and minimum tailwater level. The top width of an impervious core should not be less than 10 ft to allow movement of equipment for placing and compacting the fill material (8).

2. If the embankment rests on a pervious foundation, a cutoff trench may be used to reduce seepage (Figs. 6-4*c* and 6-4*d*). Upstream blankets may also be used to reduce seepage for embankments on pervious foundations where cutoff trenches would be very deep (Fig. 6-4*e*).
3. Downstream control of seepage water is handled by means of pervious drainage blankets or by constructing most of the embankment of pervious material (Figs. 6-4*a* through 6-4*e*). The transition zone between the zone of fine soil and the coarser drainage blanket must be carefully designed and constructed to prevent the finer soils from being carried into the coarser drain material. Specifications for such a transition zone are given in reference (20).

The strength of the relatively impervious soils depends on their compacted density, and this in turn, depends on the water content and the kind and use of compaction equipment. Thus improving the strength of the soil by increasing or decreasing the water content is possible. Drying can be done by harrowing the fill in dry weather or by heating it in a specially designed mechanical dryer. Sprinkling of the borrow area is a common means of moistening a soil that is too dry.

In designing the slopes of the embankment, the usual practice is to choose slopes that have been found from past experience to be stable. These slopes are then checked for stability using the Swedish-slip-circle method. The factors involved in choosing the embankment slopes are:

1. Character of soil materials available
2. Foundation conditions
3. Height of structure
4. Possibility of rapid drawdown of the upstream pool

Speed of drawdown is important in embankment design because if the pool is drawn down rapidly, a relatively impervious embankment will not drain freely, and the pore-water pressure within the embankment will reduce the shearing resistance and increase the weight of the embankment material over what it would have been had it drained. The reduced shearing resistance may cause the embankment to fail by sliding. Therefore, if rapid drawdown of the reservoir can be expected, the upstream embankment must have a flatter slope, or it must be constructed of material that will drain rapidly. Table 6-1 shows recommended embankment slopes for selected materials and drawdown conditions.

Slope Protection Because of the erodibility of earth dams, special precautions must be taken to prevent erosion. The upstream slope of the embankment must be protected against rain and snowmelt runoff and against wave erosion. The most commonly used type of protection on the upstream face is dumped riprap, which consists of stones or rock fragments placed on a properly graded filter. The filter may be a specially placed blanket of sand and gravel, or it may be the upstream zone of a zoned embankment. The rock for the riprap should be hard, dense, and durable to resist wave action and normal

Table 6-1 Recommended Embankment Slopes for Small Earthfill Dams on Stable Foundations (20)*

Type of Dam	Drawdown Condition	Soil† Type	Upstream Slope	Downstream Slope
Homogeneous	Gradual‡	Silty gravel	$2\frac{1}{2}$:1	2 :1
		Sandy clays	3 :1	$2\frac{1}{2}$:1
		Inorganic clays	$3\frac{1}{4}$:1	$2\frac{1}{2}$:1
Modified homogeneous§	Rapid	Silty gravel	3 :1	2 :1
		Sandy clays	$3\frac{1}{2}$:1	$2\frac{1}{2}$:1
		Inorganic clays	4 :1	$2\frac{1}{2}$:1
Zoned with large core shell material of pervious sand, gravel, or rock	Gradual	Silty gravel	2 :1	2 :1
		Sandy clays	$2\frac{1}{2}$:1	$2\frac{1}{2}$:1
		Inorganic clays	3 :1	3 :1
Zoned with large core shell material of pervious sand, gravel, or rock	Rapid	Silty gravel	$2\frac{1}{2}$:1	2 :1
		Sandy clays	3 :1	$2\frac{1}{2}$:1
		Inorganic clays	$3\frac{1}{2}$:1	3 :1

* These recommendations were abstracted from the USBR for the design of dams less than 50 ft in height.
† Soil types are for embankments of homogeneous dams and cores of zoned dams.
‡ Rapid drawdown is defined as a drawdown rate of more than 6 in. per day.
§ A modified homogeneous dam is one that is entirely homogeneous except for a filter drain constructed at the base of the downstream part of the embankment.

Table 6-2 Thickness and Gradation Limits of Riprap on 3:1 Slopes (21)

		Gradation, Percentage of Stones of Various Weights (lb)*			
Reservoir Fetch (mi)	Nominal Thickness (in.)	Maximum Size	40 to 50 Percent Greater Than	50 to 60 Percent From — To —	0 to 10 Percent Less Than†
2.5 and less	30	2500	1250	75–1250	75
More than 2.5	36	4500	2250	100–2250	100

* Sand and rock dust shall be less than 5 percent, by weight, of the total riprap material.
† The percentage of this size material shall not exceed an amount that will fill the voids in larger rock.

weathering over a long period. The maximum size of rock and thickness of the riprap layer depends primarily on the size of waves that might be expected, and this in turn, depends on the reservoir *fetch.** Table 6-2 gives recommended riprap thickness and rock sizes for small dams. The upstream slope protection should extend from the crest of the dam to several feet below the minimum water level.

* Fetch is the unobstructed overwater distance from the dam to the nearest land mass upwind from the dam. We discuss this topic in more detail in Sec. 6-5.

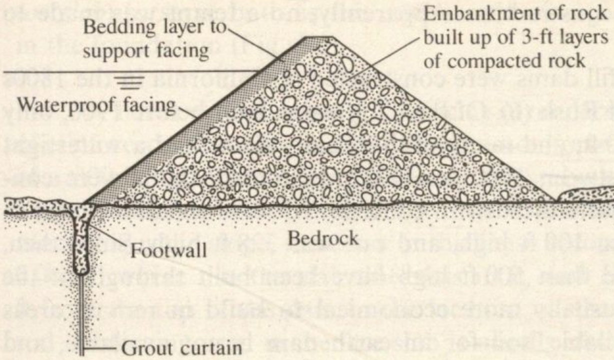


Figure 6-5 Rockfill dam

The downstream slope of the embankment requires less protection than the upstream slope. If the downstream zone of the embankment consists of rock or cobble fill, no special treatment of the slope is necessary. If the downstream embankment consists of fine materials (sand, gravel, or finer), common practice is to protect this surface with a 1- or 2-ft thick layer of rock or cobble. Grass turf also is often used for downstream slope protection where rainfall is adequate to maintain growth.*

Rockfill Dams

Introduction A rockfill dam is an embankment that uses variable sizes of rock to provide stability and a thin membrane on its upstream face or a compacted earth core in the embankment for water tightness. The most commonly used membrane is a reinforced concrete slab (see Fig. 6-5). Other membranes are asphaltic concrete, steel plate, and wood timbers.

Rockfill dams with earth cores have essentially the same features as a central core earth dam except that the main embankment consists of rock instead of coarse soil. As in the central core earth dam, the filter zones on either side of the core must be carefully designed and placed to prevent erosion of the core.

The oldest rockfill-dam remains are found in Egypt at the Wadi el-Garawi about 30 km south of Cairo. An interdisciplinary group of engineers and scientists established the date of construction at approximately 2600 B.C., and they have documented it as a true rockfill dam with a central core of earth and rubble (7). The dam was planned to be 110 m long, and much of it was built to its design height of 14 m above its base. Unfortunately, however, the dam was never completed, probably because of a flood that washed out most of it

* For more details on earth dam design, see Golzé (8) and USBR (20).

during the final stages of construction. Apparently, no attempt was made to continue work thereafter.

The first modern rockfill dams were constructed in California in the 1800s during the California Gold Rush (6). Of the early dams built before 1900, only three were higher than 100 ft, and many of these early dams had a watertight facing made of timber. Between 1900 and 1932, 18 rockfill dams were constructed, and of these, 12 were faced with concrete. Most of the dams built in this period were more than 100 ft high, and one was 328 ft high. Since then, several rockfill dams more than 500 ft high have been built throughout the world. A rockfill dam is usually more economical to build in remote areas where rock is readily available, soil for an earth dam is not available, and bringing in materials for a concrete dam is too costly.

The early dams were constructed by dumping loose rock in lifts (layers) up to 25 ft thick. However, dams built in this way had considerable settlement with attendant damage to the upstream face. In recent years, most rockfill dams have been built by placing the rock in thin (no more than 3 ft thick) layers and compacting them with several passes of heavy-smooth drum, vibratory rollers.

THE FOUNDATION Almost all rockfill dams are built on fairly solid rock foundations. As with earthfill dams, the foundation should be cleared of silt, clay, and organic material before construction of the rock embankment.

EMBANKMENT DESIGN FOR CONCRETE FACE DAMS The upstream and downstream slopes of the embankment are constructed to have the angle of repose of the rock, that is, from about 1.3 horizontal to 1 vertical to 1.4 horizontal to 1 vertical. The rock is laid down in layers and each layer is compacted. The upstream face of the embankment requires special treatment to provide a suitable surface on which the impervious concrete face will rest. In current practice, the face supporting zone of the embankment consists of well-graded material with a maximum rock size of about 3 in. and a minimum of sand size (3). This zone is usually about 12 ft wide (horizontal measure) at the top of the dam and remains that width to the base for dams less than 300 ft high (3). For higher dams the width is made greater toward the base. For the 525-ft high Areia Dam in Brazil, the face supporting zone was increased to 30 ft at its base. The material in the face supporting zone is placed in layers about a foot thick and compacted with several passes of a vibratory roller. Then, after the entire embankment is in place, the upstream face is trimmed and compacted further with several passes of the roller, which is guided up and down the slope by means of a cable. In this manner, a sound foundation is prepared for support of the concrete slab face.

UPSTREAM BASE At the upstream base of the dam, an anchor is required for the impervious face. Common practice is to excavate a portion of the foundation rock and pour a concrete wall (sometimes called a *footwall* or *toe slab*) to serve both as this anchor and as a grout cap for a grout curtain in the foundation (Fig. 6-5).

IMPERVIOUS FACING The facing of modern concrete face dams consists of reinforced slabs with vertical joints but not horizontal joints (3). Thus the face of a 300-ft high dam might consist of slabs about 50 ft wide by about 500 ft long. The slab is usually about 1 ft thick at the top of the dam and, with distance down the slope, increases in thickness according to the formula $t = 1 + 0.003H$, where t is the slab thickness in ft, and H is the vertical distance in ft from the top of the dam to the location on the slope. The amount of steel rod reinforcing used in the face slab is usually about 0.4% of the concrete volume. The reinforcing runs both horizontally and vertically (up and down slope) through the slab. Because some settlement will occur in the embankment, the impervious face must be able to conform to the changes in the upstream surface of the dam. This is accomplished by the vertical contraction joints with water stops of rubber, plastic, or expandable metal between the slabs. The bottom slab of the face bears against the footwall. An expansion joint with a waterproof seal is also used here.*

Concrete Gravity Dams

FORCES ON THE DAM Concrete gravity dams are designed so that the weight of the dam itself (the gravity force) is sufficient to resist overturning by the applied forces. The forces that must be considered in the design of the dam are the hydrostatic forces both upstream and downstream, hydrostatic uplift, the weight of the dam, earthquake forces, and ice forces.† Figure 6-6 shows how these forces are applied. We discuss each of these forces separately.

HYDROSTATIC FORCES Because of the pressure of the water in the reservoir and in the downstream channel, hydrostatic forces will be exerted on the dam. In Fig. 6-6, the horizontal force per unit width is $F_{U,H}$. Here $F_{U,H} = \gamma h_U^2/2$, where γ is the specific weight of the water (62.4 lb/ft^3 for fresh water), and h_U is the vertical distance from the water surface to the base of the section of dam. The location of the line of action of this force is at 2/3 of the depth below the water surface. If the dam has a sloping face, there will be a vertical component of hydrostatic force; this is identified as $F_{U,V}$ (see Fig. 6-6). The magnitude of $F_{U,V}$ equals the weight of the water vertically above the sloping face of the dam, and its line of action is through the centroid of this

* For more details on the design and construction of rockfill dams, see Cooke (3), Creager (5), and Golzé (8).

† Forces due to temperature rise in a dam may also be significant if it is a large dam. For details, see Golzé (8).

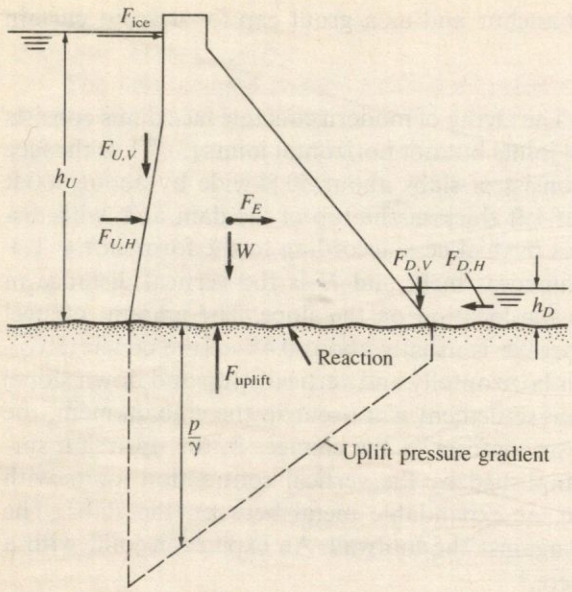


Figure 6-6 Forces acting on a section of a concrete gravity dam

volume of water. Similar hydrostatic forces act on the downstream face of the dam, as shown in Fig. 6-6.

HYDROSTATIC UPLIFT After the reservoir is filled, water (under pressure) will seep into the pores of the concrete of the dam and through the pores and fissures of the foundation rock. Once conditions of equilibrium have been established (that is, once the seepage rate is constant), a pressure head gradient, as is shown in Fig. 6-6, will be established in the pores of the concrete along the base of the dam. The maximum head is at the heel (upstream limit) of the dam, where $p/\gamma = h_U$, and the minimum head is at the toe of the dam and is equal to h_D. Thus the magnitude of the hydrostatic uplift will equal the product of average uplift pressure and the area of the base section. The line of action of the uplift force will act through the centroid of the pressure prism at the base of the dam. Customary practice is to reduce the uplift force by creating a more impervious zone in the rock foundation by boring holes into the foundation rock and pumping cement grout into the holes. The grout then is forced into the fissures and pores of the foundation. These grout holes usually are spaced about every 10 ft along the length of the dam and are intended to create a relatively impervious *grout curtain.* Therefore, when water seeps through this curtain, the head loss is much greater across the curtain than in the rest of the foundation material, thereby reducing the uplift pressure downstream of the grout curtain. To relieve the uplift pressure even more, drains are usually installed between the zone just downstream of the grout curtain and the toe of

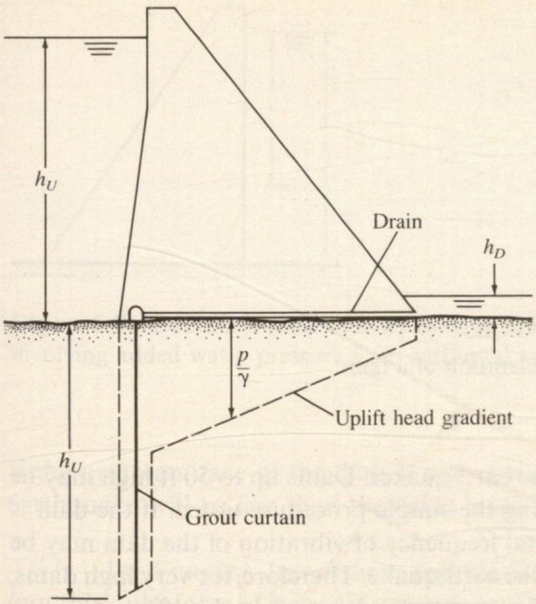


Figure 6-7 Uplift pressure on a dam with grout curtain and drains

the dam. Thus the uplift pressure distribution for a dam with a grout curtain and drains might appear as shown in Fig. 6-7. Even though the grout curtain and drains will greatly reduce the uplift pressure, engineers, assuming a more pessimistic scenario, often design dams to withstand greater pressure than this reduced value. Just how much hydrostatic uplift force is used depends on a given agency's or engineering firm's assumptions and practices.

Weight of Dam In computing the gravity forces, one must include the weight of the concrete (usually assumed to equal 150 lb/ft^3) plus the weight of appurtenances such as gates and bridges. The resultant weight will act through the center of gravity of the entire mass.

Earthquake Forces When an earthquake occurs, the earth shakes (vibrates), as does the dam resting on the earth. From the standpoint of forces resulting from an earthquake, it is convenient to think of an inertial force due to the shaking of the dam. That is, the dam will be accelerated when the quake occurs so that an inertial force will act through the center of gravity of the dam and in a direction opposite to the direction of acceleration. Figure 6-8 shows a dam being accelerated to the left. The inertial force will act in the opposite direction (to the right in this case), and it will be equal to Ma, where a is the acceleration due to the earthquake, and M is mass. In the United States, the design value for the acceleration usually varies from 0.05g to 0.10g depending

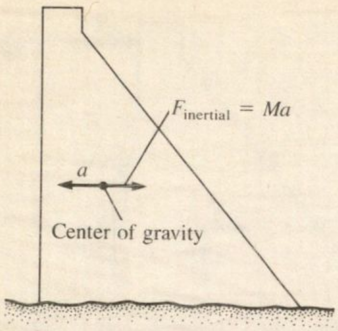


Figure 6-8 Inertial force due to acceleration of a dam dam during an earthquake

on the area's susceptibility to severe earthquakes. Dams up to 50 ft high may be designed for earthquake forces using the simple procedure noted. If the dam is very high, however, the fundamental frequency of vibration of the dam may be in resonance with the vibration of the earthquake. Therefore, for very high dams, more sophisticated dynamic analyses are made for earthquake effects.*

Besides the inertial effects from an earthquake, the water pressure itself will be increased when the dam is accelerated in a direction toward the reservoir. A formula for this added pressure was developed by Zanger (23) and, for dams with a vertical upstream face, is given as

$$P_e = C\lambda\gamma h \tag{6-1}$$

where $$C = 0.365\left[\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right] \tag{6-2}$$

and where λ = the earthquake intensity (the earthquake acceleration divided by the acceleration of gravity)

γ = specific weight of water

h = total depth of reservoir at section being studied

y = the vertical distance from the reservoir surface to the elevation in question (see Fig. 6-9)

According to the USBR (20), it may be shown analytically that the total horizontal force, on the face of the dam, V_e, above the elevation in question, and the total overturning moment, M_e, above that elevation are, respectively,

$$V_e = 0.726P_e y \tag{6-3}$$

$$M_e = 0.299P_e y^2 \tag{6-4}$$

* For a discussion of these analyses, see Golzé (8).

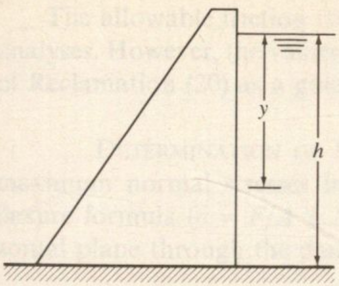


Figure 6-9 Definition sketch for terms in equations involving added water pressure from earthquakes

If the upstream face of the dam is not vertical, the added pressure due to the earthquake will be less than the value given by Eq. (6-1).*

ICE FORCES In northern regions, where ice may become more than a foot thick, the force of ice against a dam may be large. The most severe situation occurs when the temperature of the ice is increasing and expanding. Golzé (8) indicates that the ice force may be taken as 10,000 lb/lineal ft of contact with the dam for ice thicknesses of 2 ft or more.

STABILITY ANALYSIS OF THE DAM Several sophisticated methods are available for analyzing a dam for stability; however, we will describe only the simple gravity method. In the gravity method, a vertical slice of the dam is analyzed for stability, and it is assumed no forces are transmitted to or from this slice by adjacent elements.† The stability analysis checks

1. For resistance to overturning
2. For resistance to sliding
3. To make sure that allowable normal stresses in the concrete are not exceeded

We discuss the procedures and assumptions of each of these determinations.

RESISTANCE TO OVERTURNING If the dam is too thin, it may not have enough weight to resist the action of the water pressure and may fail by tipping in the downstream direction about its toe. If this were to happen, the line of action of the resultant of the applied forces would lie outside the pivot point, as shown in Fig. 6-10. We might then conclude that a dam would be safe from overturning if a rule were adopted stating that the line of action of the resultant

* For the case of a sloping upstream face, see USBR (20).

† The more sophisticated methods of analyses take these forces into account. These forces are generally considered negligible for low dams (less than 50 ft high) but may be significant for high dams.

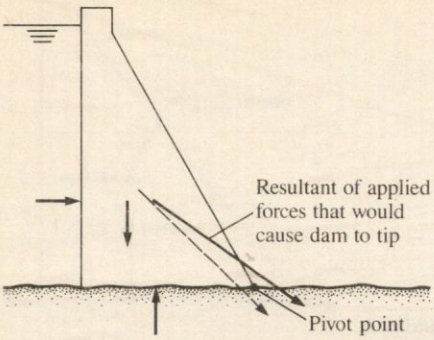


Figure 6-10 Consideration of forces causing overturning

should lie inside the toe of the dam (the broken line in Fig. 6-10). However, this may not be sufficient to guard against overturning. It can be shown that if the resultant acted just a short distance to the left of the pivot point, there would be tension in the concrete at the upstream face of the dam. But to assume unreinforced concrete can take a tensile stress is not prudent; therefore, good engineering practice requires that tensile stress not be allowed at the upstream face of the dam. By applying the flexure formula ($\sigma = F/A \pm Mc/I$) to a horizontal section at the base of the dam or on any other horizontal section, it can be easily shown that the resultant applied forces must lie within the middle third of the section being considered.

Resistance to Sliding The forces that tend to cause sliding are the pressure of the water on the face of the dam, horizontal earthquake forces, ice forces, and wave forces. If these forces are less than the resistance to shear at the base of the dam, or any other horizontal section through the dam, then the dam will not slide. The applied horizontal forces are determined by the methods we discussed in the previous section. The resistance to sliding is equal to the product of the normal force (vertical force for a horizontal shear plane) acting on the shear plane and the coefficient of friction f acting between the two surfaces at the shear plane.

Table 6-3 Representative Friction Factors for Foundation Material (20)

Material*	f
Sound rock, clean and irregular surface	0.8
Rock, some jointing and laminations	0.7
Gravel and coarse sand	0.4
Sand	0.3
Shale	0.3

* For silt and clay, testing is required.

The allowable friction factor, f, for rock is best determined by laboratory analyses. However, the values shown in Table 6-3 were given by the U.S Bureau of Reclamation (20) as a guide for preliminary analysis.

Determination of Maximum Normal Stresses To check whether maximum normal stresses in the concrete will be exceeded, one applies the flexure formula ($\sigma = F/A \pm Mc/I$) to determine the stresses acting on a horizontal plane through the dam at the upstream and downstream faces.

Load Combinations Used in Stability Analyses In the preceding sections, we discussed the kinds of loads that can be exerted on a dam, and we presented methods to determine the stresses that can result from these loads. The question now is whether all the loads should be applied to create the worst possible loading situation, or whether only some of them should be applied to represent a more probable situation. The worst possible situation might be one in which the reservoir is completely full, the maximum ice force is acting, and the maximum earthquake occurs. The worst possible situation is highly improbable; therefore, current design practice makes allowance for these situations by applying a lower factor of safety to them. The procedure is to categorize the possible loading situations according to *usual* load combination, *unusual* load combination, and *extreme* combination. Then, for example, factors of safety of 3.0, 2.0, and 1.0, respectively, are used to determine the concrete's maximum allowable compressive stress (19).

A typical usual load combination for a gravity dam is based on normal design reservoir surface elevation, representative ice load, and normal tailwater elevation. An unusual load combination might include maximum reservoir elevation, maximum ice load, and minimum tailwater elevation. The extreme load combination would be the usual load combination plus the loads resulting from the maximum credible earthquake for the region. The "maximum credible earthquake is one having a magnitude usually larger than any historically recorded event" (8).

To determine the allowable stresses, the factors of safety are applied as follows: The maximum allowable compressive stress for the concrete should be the specified compressive strength of the concrete divided by the safety factor (here the safety factor values are 3, 2, or 1 depending on the type of loading combination). According to Golzé (8), in no case should the compressive stress exceed 1500 psi for the usual load combination or 2250 psi for the unusual load combination. As noted earlier, usual practice is to design the dam so that tensile stresses do not occur in the concrete. More specifically, for the case of usual loading, tensile stresses are not allowed; however, for the cases of unusual or extreme loading, some tensile stresses are allowed (8).

The Design Process In designing a gravity dam, the usual procedure is to first choose a shape of the section of the dam based on previous experience with similar dams. Then, a stress and stability analysis of the structure is made

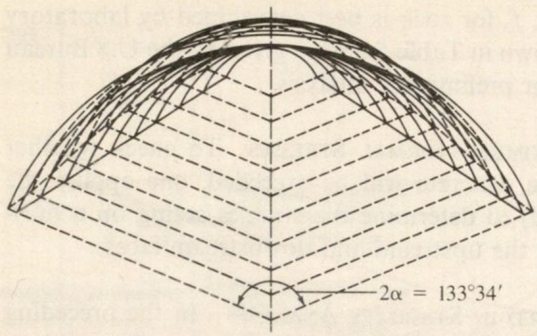


Figure 6-11 Arch dam geometry — plan view

to see if the structure passes the stability tests. If the dam is not stable or if stresses are too extreme, the section is reshaped to improve the design. Likewise, if the initial shape produces stresses much below the allowable limits or much safer than need be for stability, reshaping is also done to produce a more economical design. That process is continued until a satisfactory design is achieved.

Some guidelines for initial shaping of the section are (1) the upstream face is usually made vertical or near vertical, and (2) the downstream face usually has a slope of about 0.75 horizontal to 1.0 vertical.

Dams Categorized According to Special Features

Some types of dams have gotten their names from the distinguishing features that set them apart from the dams we have already discussed. These are arch dams and buttress dams.

ARCH DAM The arch dam is usually built in narrow canyons where the abutments are of massive sound rock so that the horizontal load acting on the dam (water pressure force) may be safely transferred to the canyon walls by arch action. A rule of thumb is that if the ratio of the width at the top of the dam to the height of the dam is less than 5, then an arch dam should be considered for the site (8). Figure 6-11 is a plan view showing the basic geometry of an arch dam. A dam like this is called a constant angle arch dam because the 2α is constant (approximately 133.6°).* Because the width of the canyon increases from the bottom of the dam to the top, the arch radius will also have

* It can be shown that for $2\alpha = 133.6°$, the volume of concrete needed for the arch is minimum. This assumes all the force is transferred to the canyon walls.

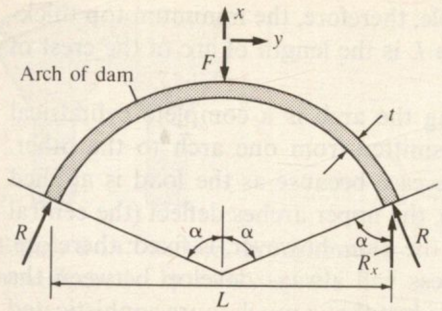


Figure 6-12 Forces on a single arch

to increase with height, as Fig. 6-11 shows. A simple approach to designing an arch dam is to divide it into several horizontal slices (separate arches) and to analyze each of these from the standpoint of hoop stress produced by the water pressure. For example, consider the arch shown in Fig. 6-12, where the width between the canyon walls is L. The horizontal component of water force acting downstream on one half of the arch will be resisted by one of the reaction components R_x, or

$$-\gamma h\,\Delta h\,\frac{1}{2}L + R_x = 0$$

where h is the depth of water above the arch, and Δh is the thickness of the horizontal slice of arch being analyzed.

But $\quad R_x = R\cos(90° - \alpha)$

therefore,

$$R = \frac{\gamma h\,\Delta h(L/2)}{\cos(90° - \alpha)} \tag{6-5}$$

Also $\quad R = \sigma t\,\Delta h \qquad (6\text{-}6)$

where σ is the normal stress in the concrete, and t is the thickness of the arch. Then, eliminating R from Eqs. (6-5) and (6-6) yields

$$t = \frac{\gamma h(L/2)}{\cos(90° - \alpha)\sigma} \tag{6-7}$$

By applying Eq. (6-7) from the crest of the dam to the bottom, we can determine how the thickness of the dam should vary from top to bottom. Of course, an extremely thin dam at the top is undesirable; therefore, the minimum top thickness is usually taken as about $L/60$, where L is the length of arc of the crest of the dam (4).

Equation (6-7) is derived by assuming the arch is a complete cylindrical shell; that is, no transverse stress is transmitted from one arch to the other. However, in the arch dam, this is not the case because as the load is applied (as the water surface in the reservoir rises), the upper arches deflect (the central portion moves downstream). The base of the dam, however, is fixed; therefore, because of this differential deflection, stress will always develop between the arches under full load. This all points to the fact that a much more sophisticated method of stress analysis is needed to design the most economical arch dam.

To actually design an arch dam, the engineer makes an approximate configuration of the dam based on the topography of the site and approximate formulas. Then, a stress analysis of the entire dam is made to reveal where unusually high or low stresses occur in the structure. This stress analysis is usually done using the finite element method. Once the stress analysis is made, changes in the original configuration are made to improve the stress distribution. Another stress analysis is done and any necessary further modifications are made. This process is repeated until a design is achieved that has (1) a reasonably uniform distribution of stress, (2) a compressive stress level throughout as near to the allowable limits as practicable, and (3) a minimum volume of concrete.

BUTTRESS DAM A buttress dam is essentially a hollow gravity dam. Buttresses of reinforced concrete rest on the rock foundation and support a watertight sloping face of the dam. Figure 6-13 shows the general configuration of the nonoverflow section of a buttress dam. The facing of this dam is a flat slab. Other types of facings include the *round head deck* (Fig. 6-14) and the *multiple arch deck* (Fig. 6-15). The gravity part of the stability of this dam comes

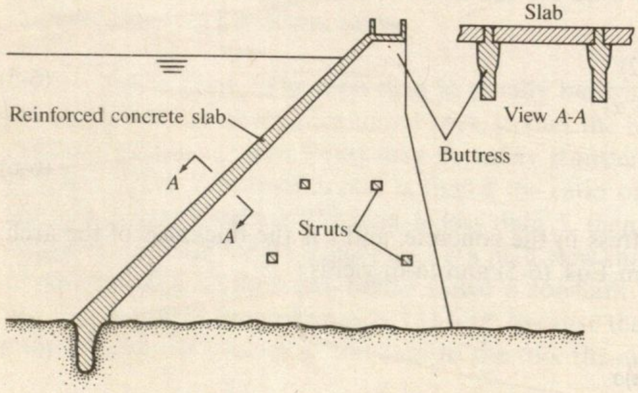


Figure 6-13 Nonoverflow section of a buttress dam

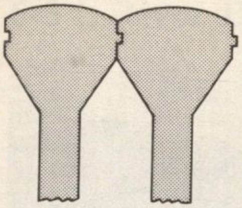


Figure 6-14 Section through round head deck of a buttress dam

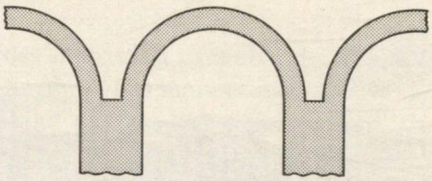


Figure 6-15 Section through multiple arch deck of a buttress dam

from the dead load of the concrete in the buttresses and face and, more important, from the weight of the water above the sloping face. The spillway part of the dam consists of a reinforced downstream face that is also supported by the buttress (see Fig. 6-16).

The main advantage of the buttress dam is that it needs as little as 30 or 40% of the concrete needed for a solid gravity dam (4). However, this advantage is usually offset by the added labor costs of building forms and placing reinforcing steel. Moreover, in areas subject to freezing temperatures, the face slabs of buttress dams have been known to deteriorate from this type of weathering.

The flat-slab face is well suited for low buttress dams. For high buttress dams, however, it is difficult to satisfactorily transmit the large slab load to the buttress without creating serious stress concentrations. Therefore, the multiple arch or round head types are most often used on high head dams.

Figures 6-17 and 6-18 are examples of buttress dams built by the U.S. Bureau of Reclamation.

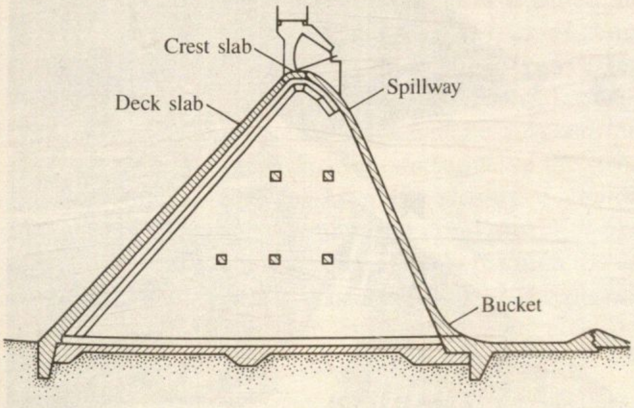


Figure 6-16 Spillway section of a buttress dam — elevation view

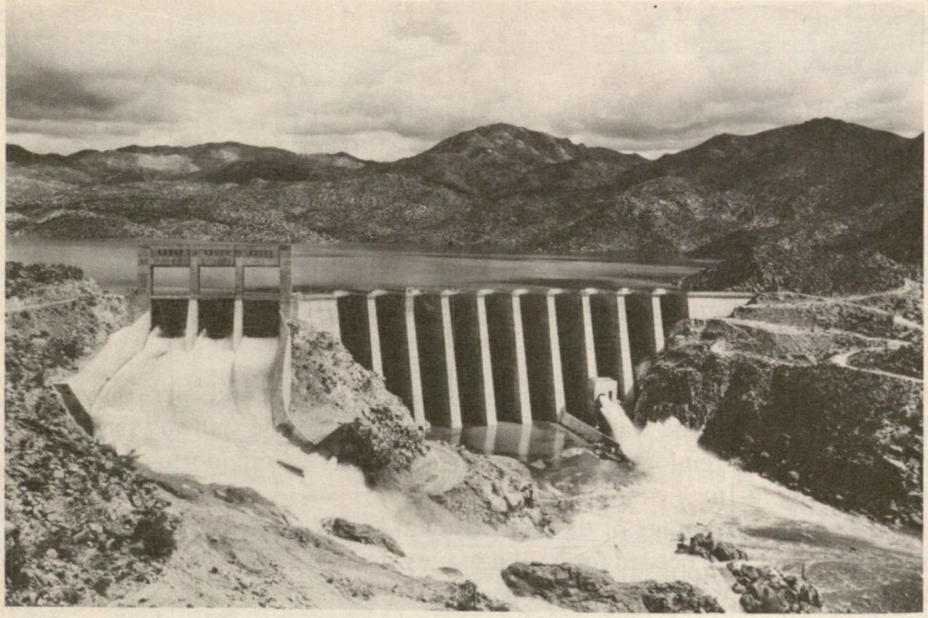
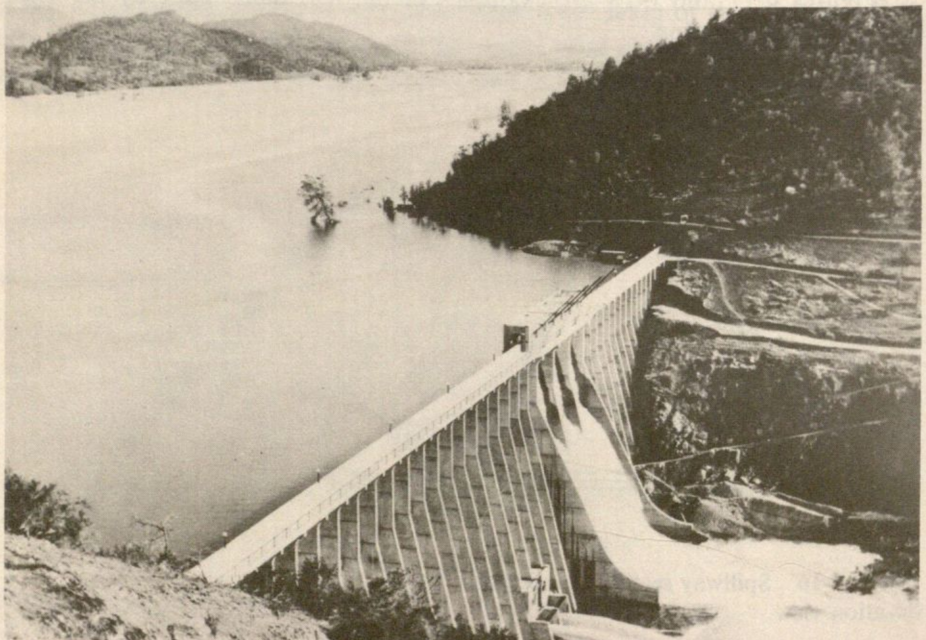


Figure 6-17 Bartlett Dam in Arizona — multiple arch buttress (Courtesy of U.S. Bureau of Reclamation)

Figure 6-18 Stony Gorge Dam in California — slab buttress (Courtesy of U.S. Bureau of Reclamation)



6-3 River Diversion

Whenever a dam is to be built across an existing river channel, the river must be diverted so that construction can be done. The manner in which the diversion is accomplished depends on the kind of dam being constructed, the character of the site, and the characteristics of the streamflow. We discuss several common methods of diversion and present the factors that favor one type of diversion over other types.

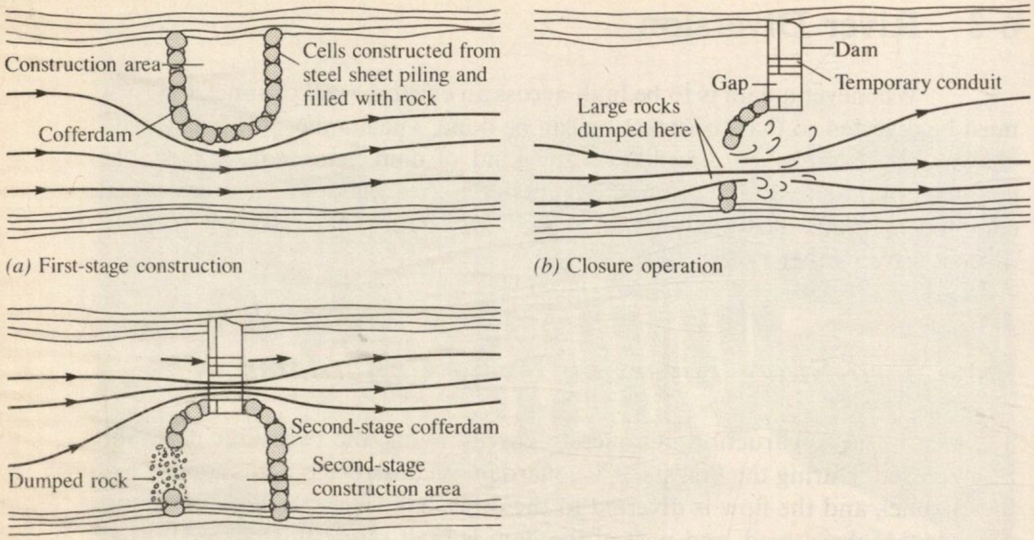
Two-Stage Diversion Using Cofferdams

In the construction of concrete gravity dams, the two-stage diversion is often used. During the first stage, a cofferdam is constructed across about half the channel, and the flow is diverted to the other half. Then the area inside the cofferdam is dewatered, and part of the dam is built inside this cofferdam (see Fig. 6-19*a*).

Part of the first-stage dam construction is often completed only up to a limited height, thus leaving a gap (Fig. 6-19*b*) in the dam that the river can flow through in the second stage of construction. A temporary diversion tunnel also may be included in the first-stage construction through which flow can be diverted in the second stage.

In the second stage, the first-stage cofferdam is removed and construction of part of the second-stage cofferdam is started. At this time, the flow is restricted to a small opening in the second-stage cofferdam (Fig. 6-19*b*). In a large river, cutting off the flow through the second-stage cofferdam is a critical operation because the flow velocity through the opening in the already constructed cofferdam frequently will be large and will increase as the opening becomes smaller. The process of stopping this flow is called the *closure* operation. Often this closure is effected by dumping large rock or concrete tetrahedrons into the restricted channel.* As the rocks or tetrahedrons are dumped, the bottom of the channel becomes progressively higher in elevation and the upstream water level rises. Finally an upstream level will be reached such that the flow will be diverted through a gap or diversion tunnel of the structure that was built inside the first-stage cofferdam. After the closure is completed, a more impervious blanket of fine material (small rocks and gravel) is placed on the upstream part of the closure section to reduce seepage through the large rocks or tetrahedrons of the closed-off section. When the seepage is reduced to a low level, the rest of the second-stage cofferdam can be completed and construction within the area of the second-stage cofferdam can be started (Fig. 6-19*c*).

* For example, 12-ton tetrahedrons were used in the closure operation of McNary Dam on the Columbia River.



(a) First-stage construction (b) Closure operation (c) Second-stage construction

Figure 6-19 Two-stage diversion scheme

After construction is completed inside the second-stage cofferdam, the cofferdam is removed and flow is diverted through that part of the dam (spillway or powerhouse section). Then construction is completed on the part of the structure only partially completed during the first stage of construction. That is, the gap in the first-stage construction is filled, or the temporary diversion tunnel is plugged.

Cofferdams are often of the cellular type, in which linked steel piles are driven in a cellular pattern, as shown in Fig. 6-20, and each of the cells is filled with rock to provide stability against overturning. These cofferdams are also very durable in case they are overtopped during flood season. Figure 6-20 shows how the steel piles are linked.

Scheduling the different phases of construction of the cofferdams must be synchronized with the normally expected variation in streamflow. For example, the first cofferdam will normally be built during a low-flow period, and then the stream will be diverted to the completed structure during another low-flow season. Thus depending on the stream characteristics, the designer may have to think in terms 1-yr time increments between low-flow seasons.

The designer also will have to decide how high to design the cofferdams. Normally, they are not made so high that they would never be overtopped by the design flood for the dam. The designer must balance the added cost of a very high cofferdam against the damage that would result from its overtopping. Damages might include the costs due to work stoppage and cleanup. For major dams, designing the cofferdam so that it will withstand a 20-yr flood is common.

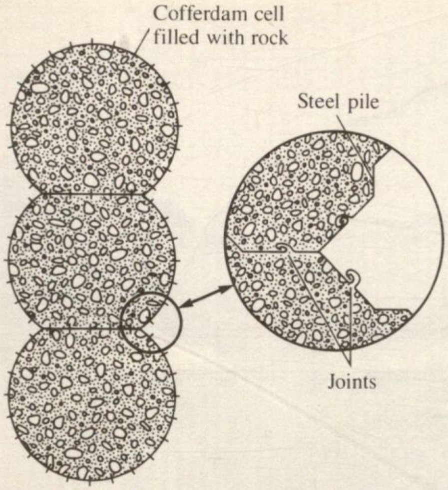


Figure 6-20 Details of cellular cofferdam — plan view

Tunnel Diversion Using Cofferdams

If the dam is to be built in a very narrow canyon, the usual case for arch dams, then the two-stage diversion described above may not be suitable because of the limited working space in the canyon. A common procedure is to excavate a tunnel through one abutment and then build cofferdams upstream and downstream of the damsite. When the cofferdams are closed, the water is diverted through the tunnel, and then construction of the dam can take place inside the cofferdams (see Fig. 6-21).

In designing arch dams, it is fairly common to include a tunnel as part of the spillway. In these cases, part of the spillway tunnel can usually be used as part of the diversion tunnel. The spillway tunnel often has an intake at an elevation near the crest of the dam, which would be too high for diverting flow during construction. To use the spillway tunnel, the typical procedure is to construct a

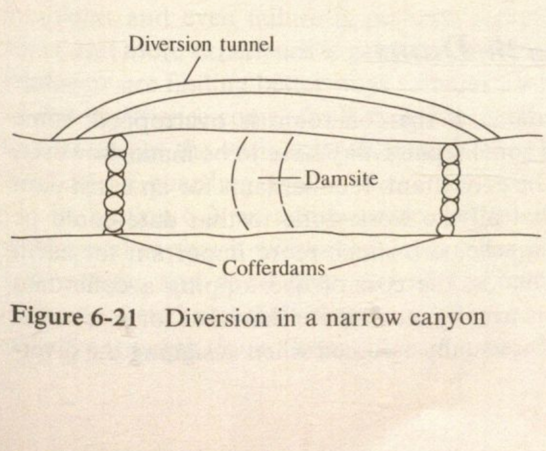


Figure 6-21 Diversion in a narrow canyon

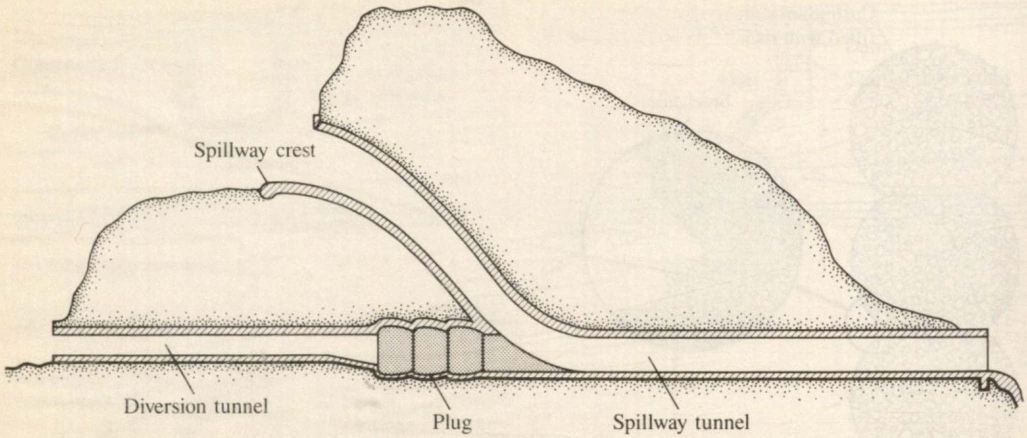


Figure 6-22 Typical arrangement for using part of a spillway tunnel in a diversion scheme

separate diversion tunnel, which is ultimately connected to the spillway tunnel at the lower level. When the dam is completed, the diversion tunnel is plugged, as shown in Fig. 6-22.

Flumes

If a dam is to be built on a fairly small river in a wide canyon, it may be possible to divert the flow away from the construction area by means of a separate channel, or *flume.* A flume, which is often made of a steel or timber frame with a timber lining, usually diverts the flow around the damsite or over a low block in the dam. As construction proceeds, the flume can be moved to another area. Figure 6-23 shows a diversion flume used on the construction of the Canyon Ferry damsite in Montana. In Fig. 6-23, also note the steel sheet piling used to guide the flow into the flume. This is the same type of piling used to construct the cellular cofferdams.

Diversions for Earth Dams

In the case of concrete dams, if the cofferdam is overtopped, some cleanup work will be required and some repairs may have to be made; however, the cost for this work should not be exorbitant. If cofferdams for an earth dam were overtopped, it is possible that all the work done to that date could be wiped out; therefore, the diversion scheme is much more important for earth dams than for concrete dams. That is, the cost of overtopping a cofferdam protecting an earth embankment is usually much more than that for a concrete dam. Therefore, a much lower risk is usually assumed when designing the diver-

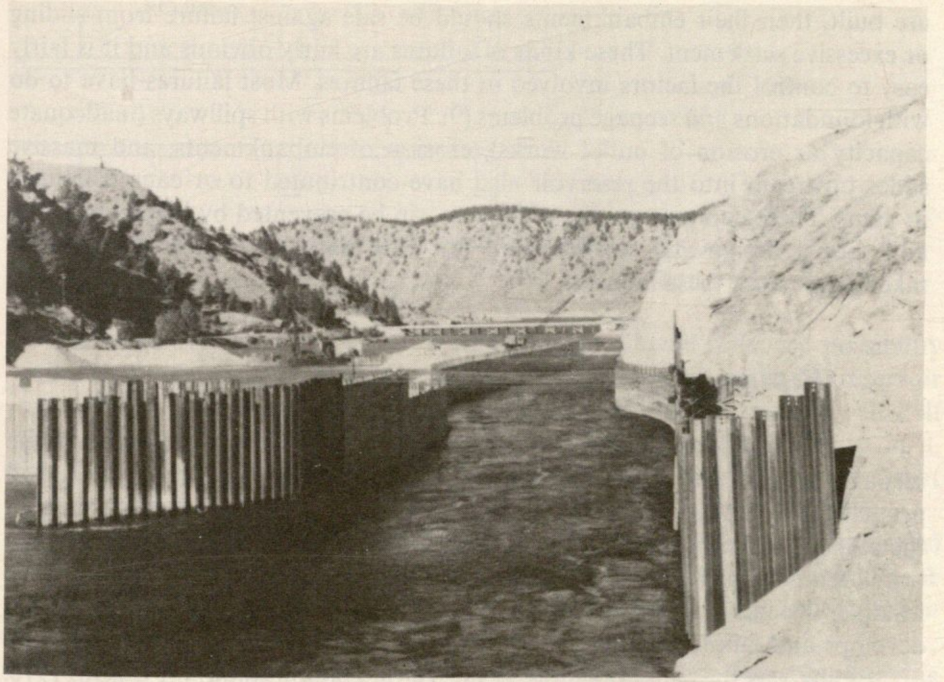


Figure 6-23 Flume diversion

sion scheme of an earth dam; it must be designed to accommodate very large floods. A thorough study of the meteorology of the region and the hydrology of the stream will help schedule critical operations such as closure to reduce the risk of failure.

6-4 Dam Safety

Because the sites for future dams are usually less suitable (canyons not as narrow, weaker foundations) than the sites already used, the potential for problems and even failure is perhaps greater now than before. However, as more and more experience is gained in the construction and operation of dams, engineers are finding better ways to handle potentially serious problems. Some of this experience comes from studying dams that have failed. This part of the text examines the various problems that can lead to the failure of a dam and methods of guarding against the occurrence of such problems are discussed.

In Section 6-2, we addressed the safety of the concrete gravity dam, which is designed so that it will have a certain factor of safety with respect to sliding, overturning, and development of maximum stress. Also, if earthfill dams are well compacted and their slopes are designed and constructed to be consistent with the strength (found from laboratory tests) of the materials from which they are built, then their embankments should be safe against failure from sliding or excessive settlement. These kinds of failures are fairly obvious and it is fairly easy to control the factors involved in these failures. Most failures have to do with foundations and seepage problems (9). Problems with spillways (inadequate capacity or erosion of outlet works), erosion of embankments, and massive slides upstream into the reservoir also have contributed to or caused failures in dams. These common causes of failures can be prevented by careful analyses and proper design and construction. We now discuss measures that can be taken to prevent these failures.

Piping

All earth dams with a central core of compacted earth have some seepage through them. If seepage occurs without dislodging and removing soil particles, no damage will result. However, if soil particles are washed away in the seepage, severe problems may develop. *Piping* is the term given to concentrated leaks that erode surrounding material (soil particles) along the path of leakage. The flow passage enlarges and leakage increases until a serious problem develops and failure possibly occurs.

Piping was the cause of failure of the Teton Dam in Idaho on June 5, 1976. This was a central-core zoned earthfill dam having a height of 305 ft, and the dam failed during the initial filling of the reservoir. More than a score of people died in the ensuing flood, and property damage was estimated at $400 million. Piping was first observed during the morning, and by noon, internal erosion had progressed so far that the top of the dam collapsed.*

Piping can develop in earth dams where the filter between fine soil and coarse material in the embankment is not properly designed to prevent the movement of the fine particles. Thus careful design *and* careful construction of the filter zones are required for safety against piping. All filters should be constructed from screened sands and gravels (9).

Piping can also occur where the fill material joins the abutment material or where it joins a solid structure such as an outlet conduit. If the fill material is not carefully placed and hand tamped next to surfaces of discontinuity, the less consolidated material may be the starting point for piping. One way of reducing the possibility of piping around buried conduits is to construct collars around the conduit, as Fig. 6-24 shows.

In theory, the collars tend to block any flow passages along the exterior of the conduit, and the distance the seepage water must travel is increased by the collars. However, proper compaction is difficult to achieve around these collars, and some authorities discount their effectiveness.

* For more details on the cause of the dam's failure, see Jansen (9).

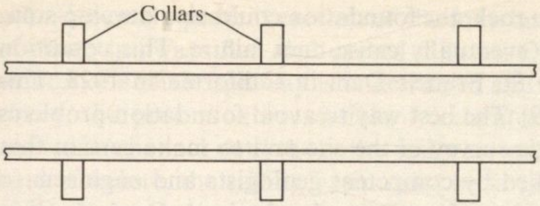


Figure 6-24 Collars around conduit to eliminate piping

When earth dams are built on compressible alluvial soils, the possibility exists that differential settlement of the embankment will occur as the reservoir is filled and the embankment and foundation material become saturated. If differential settlement of the embankment occurs, cracks may develop in it, which may lead to excessive leakage and piping. Sherard (15) has indicated that certain gradations of soil materials are especially susceptible to cracking.

In small earth dams, burrowing animals (muskrats and ground squirrels) can be a souce of piping problems. Therefore, regular inspections should be made of the embankment at low pool levels and if burrows are found, action must be taken to fill them and to prevent new ones from forming.

Piping can be detected by observing the leakage that may be occurring from the downstream side of the dam. If the leakage is in the form of clear water, piping is not occurring. If, however, the leakage is muddy, piping is to be suspected, and one must assume the dam is in danger of imminent failure. Piping that develops along cracks that extend to the exterior of the dam can sometimes be stopped by trenching along the crack and backfilling with compacted impervious soil (15).

Once piping occurs, rapid remedial action is usually called for. First, the reservoir pool should be drawn down as fast as possible by discharging water through outlet conduits. After the pool level has been drawn down, repairs can be made to the defective parts of the embankment or foundation. In the design of the dam, the outlet works should be large enough so that the reservoir can be drawn down quickly.

Foundation Problems

The weight of the dam itself must be carried by the foundation; therefore, the designer must be aware of the character of foundation material, including how impervious it is to seepage. Information about the foundation is obtained by geologic investigations that may include core drilling, geophysical tests, and geologic mapping of the area. If foundation exploration is deficient, the designer cannot anticipate possible difficulties, and the dam may develop problems relating to differential settlement, sliding, and leakage. If the foundation is composed of water soluble rock, the foundation could also develop solution cavities, become weak, and eventually cause dam failure. This condition contributed to the failure of the St. Francis Dam in California in 1928. This was a 205-ft high concrete dam (9). The best way to avoid foundation problems is to conduct a thorough geologic survey of the site and to make certain that the resulting information is studied by competent geologists and engineers so that a complete understanding of the foundation is obtained. Once that has been achieved, a safe design can be executed, or the site may be found unsuitable for economical development.

Slides on Reservoir Slopes

When a reservoir is filled or drawn down after filling, physical changes (saturation of base material, development of excessive pore pressure, added weight to reservoir foundation) of the earth in and around the reservoir can lead to slides when unstable materials exist. Slides can produce waves that completely overtop the dam, as occurred at the Vaiont Dam in Italy in 1963. In this particular disaster, about 315 million cu yd of earth slid into the reservoir and caused a 300-ft wave of water to wash over the crest of the 869-ft high dam. About 2600 people died from the flood wave that traveled down the canyon and devastated everything in its path (9). The basic concrete structure of the dam itself was not damaged!

Geologic investigations should be made of the reservoir area to locate potential slide hazards. According to Jansen (9), "the potential for landslides may exist in nearly any kind of rock, some slates and schists are notoriously susceptible to movement." If a slide hazard exists, special precautions may have to be taken to alleviate or design against the event. An alternative may include changing the site of the dam.

Spillway Problems

Dams may be overtopped if the spillway capacity is not great enough to carry the flood flow. This is especially serious for earthfill dams because of their susceptibility to erosion during overtopping. The most common cause for inadequate spillway capacity is underestimating the peak flow rate or volume of the design flood. Only competent, experienced hydrologists should be given the responsibility of determining the design flood. The design flood should be carefully chosen in the light of potential hazards resulting from dam failures. In Chapter 2, (Sec. 2-7, page 78) we provide more details on design floods.

Failure of the spillway itself has occurred in several dams. One common cause of spillway failure has been inadequate design of the stilling basin. In the early years of dam construction in the Midwest, this type of failure occurred on several concrete gravity dams where the spillway aprons and cut-off walls did not adequately prevent erosion of the foundation material. Erosion at the end of chute spillways has also caused failures. Once a structure such as the end section of a chute spillway is undermined and it starts to drop into the eroded cavity, the process is irreversible. Accelerated erosion proceeds up the slope under the channel until the entire spillway section is destroyed. To prevent this kind of failure, the stilling basin must be large enough to dissipate the energy of the high-velocity water from the chute, adequate wing walls must be built at the end of the chute to eliminate possible undermining, and riprap of adequate size and gradation must be used in the stilling basin itself to prevent erosion of the base material.

Chute spillways have also failed from other causes such as earth slides obstructing the channel and misalignment of joints of the spillway floor sections, which can lead to cavitation and eventual destruction of the spillway.

Design and Inspection

In the preceding sections, we have briefly referred to the necessity for thorough foundation exploration by engineering geologists, design by competent professional engineers, and careful construction practices. The philosophy, organization, and action required for *safe* design and construction of a dam has been most effectively summarized by Jansen (9):

> Safety of dams requires consideration of more than the technical factors. Looking at the organization, for example, one thing which must be assured is that all voices are heard. Ideas may come from within the organization — from nearly any level — or from outside. The latter includes, of course and particularly, consultants. One of the greatest hazards in the engineering of major structures is the exclusion of the ideas of those who may have valuable contributions to make. The management of any organization must exert special effort to assure that this does not happen.
>
> In case histories of projects gone wrong, the dominance of single decisionmakers — sometimes authorities whose reputations for expertise were well earned — is not uncommon. Even experts can make mistakes, and probably the worst is to assume that an expert's judgment need not be questioned by those qualified to question.
>
> Another consideration, especially in large organizations composed of many compartments, is to assure that information flows among the units. The many ideas essential to good engineering must be shared freely across the internal boundaries. The integration of separate efforts should be continuous throughout the evolution of designs, rather than simply gluing together individual final products. This means that designing must start with a general perspective and then focus on the individual parts — not vice versa.
>
> There must be recognition of the inseparable relationship of design and construction. These functions are best considered as a single

> process. Design is not completed until construction is accomplished. Designers and construction engineers have to work in concert during design and while the dam is being built so that the site conditions disclosed can be weighed against design objectives. Any necessary modifications in design during this period should be a collaborative effort of designers, geologists, and construction engineers.
>
> The vital relationship between the engineer and the geologist needs continuing emphasis. They must work as closely together as must the dam and its foundation. . . .
>
> The tendency to think of averages in the engineering of dams must be resisted. Failures occur where the dam or its foundation is weakest, not where it is in average condition. The design must focus on the potential weaknesses. Exploration and testing will necessarily depend on sampling techniques, with results varying sometimes over a wide range. Natural materials available for construction may exhibit average characteristics that meet requirements; yet, they may be judged totally unacceptable when their variations are considered.
>
> The variability of natural conditions does not encourage unreserved faith in standard guidelines or "cookbook" approaches to dam engineering. No matter how many exploratory holes are drilled or how many samples are tested, the reservoir site may still contain surprises — and these may appear at any time during the life of the structure.
>
> Flexibility is the key. Rigid criteria are useful only as long as conditions match the underlying assumptions. They fail when deviations are not perceived or when the latitude and the judgment are not available to make the necessary adjustments. The trouble with a "cookbook" is that some of its users may come to think that it contains all the recipes. Design by the book is especially hazardous in an organization insulated from professional interchange.

Once the dam is completed, surveillance of the dam and reservoir is required to guard against potential hazards over the years of operation. Surveillance includes

1. *Periodic visual inspection by experienced engineers.* These inspections will reveal uneven settlement, cracks, discoloration, or increase of seepage and embankment sloughing.
2. *Monitoring of instruments.* The design of all major dams includes installing instruments that will reveal changes within the dam. For example, sensors should be included in gravity dams to allow detection of structural and foundation movement as the reservoir is filled, and piezometers should be included in earth dams to reveal any changes in the pressure field within the dam and its foundation. Any anomaly that shows up in the pressure field should be a warning of possible abnormal leakage problems.
3. *Interpretation of information.* Information obtained from field inspection and measurements from instruments will be virtually worthless unless intelligently analyzed by experienced people. Observed data must be analyzed for deviations from reading to reading and for slow trends that may have

subtle meanings. The implications of long-term changes are sometimes overlooked, for example, serious piping may take years to develop. Thus a change in pressure gradient (by observing (piezometers) may reveal the progressive development of seepage channels that have less resistance to flow than a well-compacted embankment.

Employment of Qualified Personnel

This aspect of dam safety is succinctly presented by Golzé (8):

> The use of qualified personnel is another basic element. The design must not only be by competent engineers but should be supervised by registered or licensed engineers for the State in which the structure is to be located. In like manner, construction is a professional undertaking which, again, for maximum performance and safety, should be supervised and directed by graduate registered or licensed engineers. The operation of completed facilities should be done by men skilled by years of experience working under the supervision of one or more licensed engineers qualified in this particular field.
>
> In connection with the use of qualified personnel which may either be persons in the employ of the owner or persons employed by firms of private engineers, independent consultants should be available at all stages of design and construction of the facility to advise and counsel the engineers in charge. In connection with the periodic and special inspections of the structure, independent consultants should likewise be employed. Their advice to the dam owners concerning the physical condition and the efficiency of his facility brings to bear a professional judgement supported by years of experience.

Governmental Controls

Besides the safeguards we have already presented, which are generally under the control of the owners and designers of the project, state and federal statutes dictate certain actions that must be taken to ensure a safe project. All major hydropower projects must be licensed by the federal government, and all these (currently more than 400 projects) must be inspected periodically by the Federal Energy Regulating Commission (FERC). These inspections are made at individual dams at least every five years, and if deficiencies are found, the owners of the project must advise FERC within 30 days of the corrective measures they plan to take.

Many states also have enacted laws that exercise control over the design, construction, maintenance, and operation of dams and reservoirs.*

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* For more information on statutes governing the safety of dams, see Golzé (8). For more details on all aspects of dam safety, see Jansen (9).

6-5 Reservoirs

A reservoir is a manmade lake or structure used to store water. Inherent in the definition of a reservoir is that people have the major control over the use of water in it.

For one type of reservoir, for example, elevated tanks used in municipal water supply systems, the inflow to the reservoir is completely controlled, but the outflow is primarily dictated by consumers' needs and desires. For a system like this, the source of supply to the reservoir may be a river from which water may be withdrawn as needed. In this kind of system, the reservoir makes it possible to use pumps of moderate size pumping into the reservoir at a fairly constant rate. During peak demand periods, the reservoir is drawn down, for during these periods, the outflow rate is much more than the inflow provided by the pumps. If reservoirs were not included in this system, many more pumps or pumps having a much greater range of discharge would be needed. The reservoir also supplies water needs during emergencies such as power failures, fires, or pipe ruptures.

Another type of reservoir, for example, one created by damming a stream, has an uncontrolled inflow* but a largely controlled outflow. Natural occurrences and environmental factors are important in the design and operation of this type of reservoir. For example, the water available for storage is totally a function of the natural streamflow that empties into it. Moreover, because the streamflow downstream of the reservoir will be altered by operation of the reservoir, changes in the stream environment (often detrimental) will usually occur.

Other questions that have to be answered in the design and construction of a reservoir on a stream are

1. What height of dam will be needed to yield the desired objectives of the reservoir?
2. Will leakage from the reservoir be a problem?
3. Will evaporation significantly affect the yield from the reservoir?
4. Will there be any problems regarding stability of the earth around the reservoir (slides) when the reservoir is filled and operated?
5. Will incoming sediment be a problem?
6. What is involved in preparing the area of the reservoir (tree clearing, soil removal)?

Other concerns that relate more to social problems are

1. Relocation of utilities and transportation facilities such as roads and railroads.

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* This assumes the reservoir is on a stream that has no other reservoirs or other kind of control upstream.

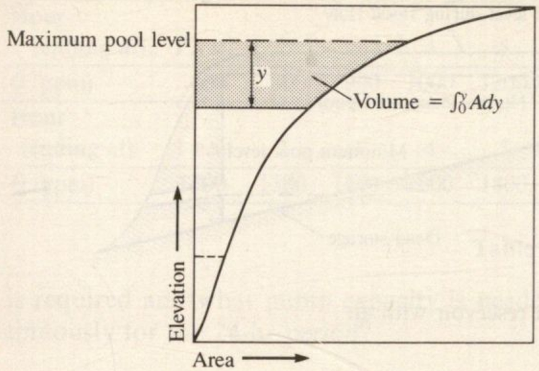


Figure 6-25 Area versus elevation for a reservoir

2. Purchase of land for the reservoir and condemnation of property owned by people who resist selling.*
3. Downstream pollution problems.
4. Development of recreation sites around the reservoir.
5. Impact on natural resources and mitigation measures required.

Some of these topics will be discussed in more detail in the following sections.

Reservoir Capacity

Reservoir capacity is the volume of water that can be stored in the particular reservoir. In the case of manmade tanks, it is simply the inside volume below the maximum water surface level in the tank. Likewise, in a reservoir behind a dam, it is the volume in the reservoir below the *normal maximum* pool level. This can be calculated by using a topographic map of the region. First, the area inside different elevation contours within the reservoir is measured, then a curve of area versus elevation can be constructed as shown in Fig. 6-25. At any given elevation, the increment of storage in the reservoir at that elevation will be $A\,dy$, where dy is a differential depth. Then the total storage below maximum pool level to any depth will be given by $\int_0^y A\,dy$. In other words, the shaded area in Fig. 6-25 will be the total storage between the maximum pool level and depth y.

The *normal maximum pool* level is the maximum possible level in the reservoir when the spillway gates are closed (level at the top of the gates). However, if there are no gates on the spillway (uncontrolled spillway), the normal maximum pool level is assumed to be at the crest of the spillway. The minimum

* Condemnation proceedings apply only to government projects and to certain private power projects.

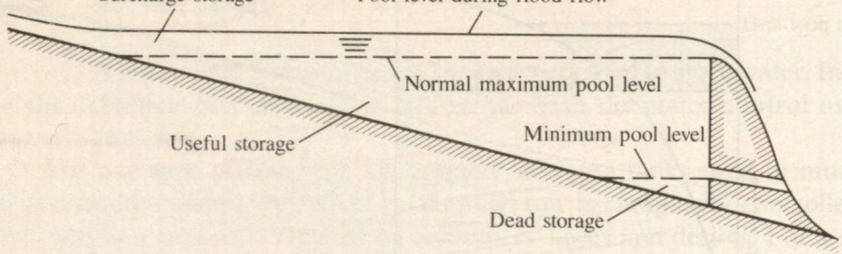


Figure 6-26 Storage relations for a reservoir with an uncontrolled spillway

pool level is usually taken as the lowest level at which flow can be released from the reservoir. This is the invert elevation of the lowest outlet pipe, as shown in Fig. 6-26. The *useful storage* in a reservoir is the storage between normal maximum pool level and minimum pool level. The storage below minimum pool level is called *dead* storage. When water is being discharged over the spillway, the reservoir pool may actually rise above the level of normal *maximum* pool level. This storage above normal maximum pool level, called surcharge storage or flood storage, is important in routing a flood through the reservoir (see Sec. 4-5, page 217).

The foregoing discussion pertains to the storage available because of the volume above ground surface in the reservoir. Besides this obvious storage, some water (as much as 2 or 3% of useful storage in some cases) will be stored in the soil and rocks of the banks of the reservoir. This stored water is called *bank storage*. If the bank material is quite porous, bank storage may be useful; it will be released when the pool is drawn down. However, if the bank material is relatively impervious and if the reservoir is normally drawn down rapidly, the water in bank storage may not drain into the reservoir fast enough to be useful.

How to Determine What Storage Capacity Is Needed

When an elevated tank, such as in a city water system, is used to augment peak flow demands, one can determine the needed pumping capacity and storage volume for a given demand by a simple numerical calculation. Example 6-1 illustrates the procedure.

EXAMPLE 6-1 Table A shows the average water demand for each hour of a common 24-hr working day of a small city.

A pump takes water from a well and delivers it to a reservoir from which the water system is supplied. Based on the demand data, what reservoir volume

Hour (ending at)	1 A.M.	2	3	4	5	6	7	8	9	10	11	12N
Q (gpm)	800	800	900	1000	1200	1425	1900	2200	2000	1575	1600	1700
Hour (ending at)	1 P.M.	2	3	4	5	6	7	8	9	10	11	12M
Q (gpm)	1500	1300	1400	1600	1800	2300	1800	1500	1200	1000	900	800

Table A

is required and what pump capacity is needed if the pump is to operate continuously for the 24-hr period?

SOLUTION

(1) Hour (ending at)	(2) Q_{system}	(3) Q_{pump}	(4) Q_{reserv}	(5) $Q_{reserv}\,\Delta t$ (gals)
1 A.M.	800	1,425	−625	
2	800	1,425	−625	
3	900	1,425	−525	
4	1,000	1,425	−425	
5	1,200	1,425	−225	
6	1,425	1,425	0	0 (reserv full)
7	1,900	1,425	475	28,500
8	2,200	1,425	775	75,000
9	2,000	1,425	575	109,500
10	1,575	1,425	150	118,500
11	1,600	1,425	175	129,000
12N	1,700	1,425	275	145,500
1 P.M.	1,500	1,425	75	150,000
2	1,300	1,425	−125	142,500
3	1,400	1,425	−25	141,000
4	1,600	1,425	175	151,500
5	1,800	1,425	375	174,000
6	2,300	1,425	875	226,500
7	1,800	1,425	375	249,000
8	1,500	1,425	75	253,500 (peak withdrawal)
9	1,200	1,425	−225	240,000
10	1,000	1,425	−425	214,500
11	900	1,425	−525	183,000
12M	800	1,425	−625	145,500

Table B

First, determine the average pumping rate. It will be equal to the average of the demand rate:

$$Q_{\text{pump}} = \frac{\sum_{i=1}^{24} Q_i}{24}$$

where Q_i is the demand discharge as given in Table A.

$$Q_{\text{pump}} = \frac{34{,}200 \text{ gpm-hr}}{24} = 1425 \text{ gpm}$$

The required storage is obtained by setting up Table B. Column 1 is the end time of the period in question, column 2 is the system demand discharge, column 3 is the pumping rate, and column 4 is the difference between the system demand rate and the pumping rate ($Q_{\text{system}} - Q_{\text{pump}}$). Therefore, column 4 gives the rate of reservoir filling if the values in this column are negative, or the rate of reservoir emptying if the values are positive. Assume the reservoir is full at 6 A.M. after the night of low demand when the reservoir is being filled. Then, by summing the $Q_{\text{reserv}}\,\Delta t$ from that time throughout the day (column 5), it is found that the greatest value of $\Sigma Q_{\text{reserv}}\Delta t$ occurs at 8 P.M., and the value of $\Sigma Q_{\text{reserv}}\Delta t$ at that time is the maximum volume of water that had to be drawn from the tank to satisfy the demand. Thus the storage required is 253,500 gallons. ■

One common method of storage requirement analysis for a reservoir on a natural stream uses the mass diagram of flow in the stream entering the reservoir. That is, by constructing a curve of accumulated volume of flow versus time and analyzing it, one can determine the necessary storage to yield a desired dry-period flow, or given a certain volume of storage, one can determine what dry-period flow can be achieved. Consider the mass diagram of a stream as shown in Fig. 6-27. The ordinate is the total volume of flow passing the given station starting from an initial time (in this case, October 1, 1974, the beginning of the 1975 water year).* By definition, the discharge Q is volume rate of flow, or $d\forall/dt$; therefore, the slope of the mass curve at any point on the curve is the river discharge at that time. Thus the steepest parts of the curve indicate flood periods, and the flattest parts indicate low-flow or dry periods. To determine storage needs, one can scan the streamflow records for all years of record and pick out the severest period so far as drought conditions are concerned. That period is then plotted as a mass curve and analyzed. In the next paragraph, we give a detailed description of how such an analysis is made.

* The volume unit used for the ordinate of Fig. 6-27 is sec-ft-mo or sfm which is the abbreviation for a flow of 1 cfs for a period of 1 month. Thus for a 30 day month 1 sfm = 1 ft^3/s × 30 da/mo × 24 hr/da × 3600 sec/h = 2.592 × 10^6 ft^3.

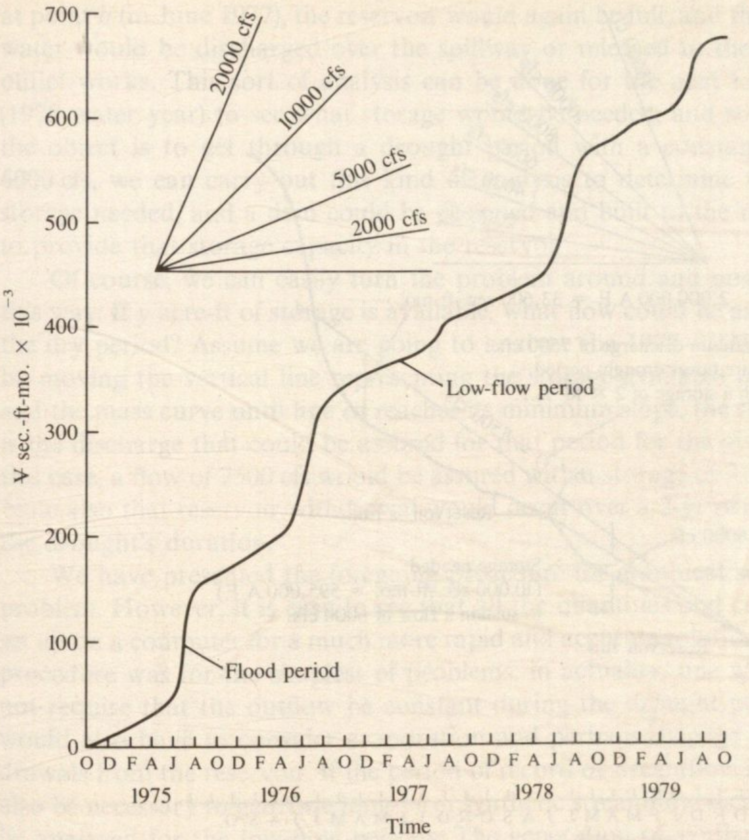


Figure 6-27 Mass diagram for the Salmon River at White Bird, Idaho, from October 1974 to October 1979

Figure 6-28 is the mass curve for the Salmon River for one of the driest periods of record, and it is to be analyzed to determine what reservoir storage would be needed to produce a minimum rate of flow of 6000 cfs downstream of the reservoir for that period. In the upper-left corner of Fig. 6-28, a chart shows the slopes that represent discharges from 2000 to 15,000 cfs. Now, focusing on the mass-flow chart of Fig. 6-28 in the June–July period of 1976, we can see that in June the curve was very steep ($Q = 45{,}000$ cfs); then, from August 1976 to April 1977, it flattens out (in February 1977, the discharge was only about 4000 cfs). During this period, it is obvious that water from storage in the reservoir is needed to augment the flow. To determine the amount of storage needed to maintain the flow at 6000 cfs, we draw a line with a slope representing 6000 cfs tangent to the mass curve of the 1976 summer season, as shown in Fig. 6-28. The point of tangency (point *a*) of that line is the time that the natural

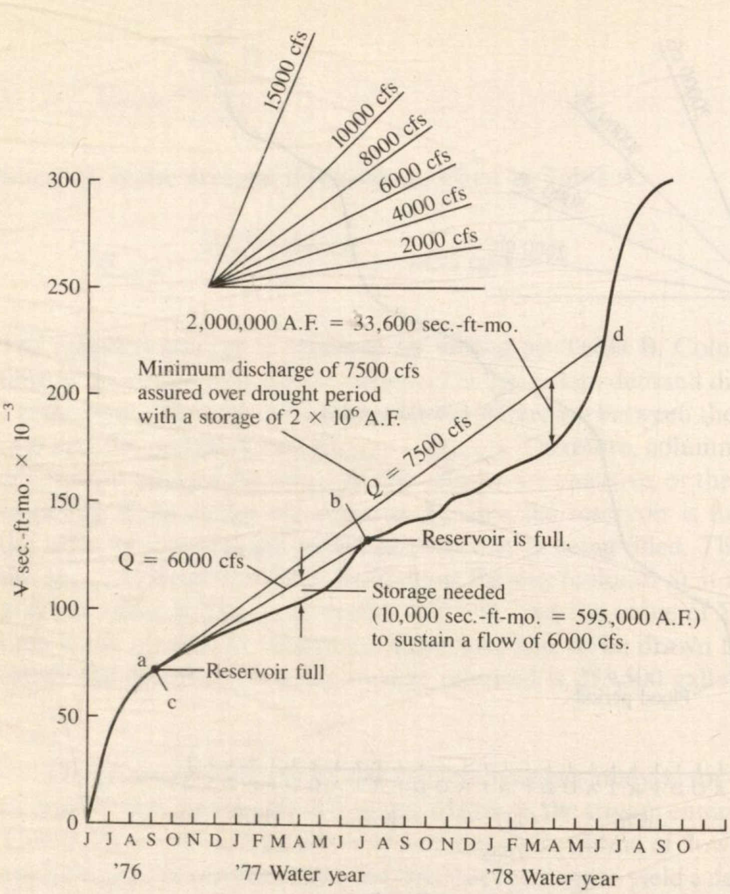


Figure 6-28 Storage-discharge relations for Salmon River at White Bird, Idaho, for 1977 and 1978 water years

streamflow went from a discharge greater than 6000 cfs to a discharge less than 6000 cfs (the slope of the mass curve drops below 6000 cfs). Then, as long as the natural streamflow slope is less than the slope of the 6000 cfs line (the 6000 cfs is the discharge being released from the reservoir), it is obvious that the discharge leaving the reservoir is greater than that coming in so that stored water is being used up. The maximum amount of stored water needed is given by the maximum vertical spread between the 6000 cfs line and the mass curve, as Fig. 6-28 shows. In this case, the maximum spread between two curves is 10,000 s-ft-mo. If the reservoir were full when releases from the reservoir were started (point of tangency), the lowest pool level would occur where the greatest spread between the two lines (6000 cfs line and mass curve) occurs, and after that instant, the reservoir would begin to fill again. When the two curves cross at point *b* (in June 1977), the reservoir would again be full, and thereafter excess water would be discharged over the spillway or released to the river through outlet works. This sort of analysis can be done for the next low-flow period (1978 water year) to see what storage would be needed, and so forth. Thus if the object is to get through a drought period with a constant flow rate of 6000 cfs, we can carry out this kind of analysis to determine the amount of storage needed, and a dam could be designed and built to the required height to provide that storage capacity in the reservoir.

Of course, we can easily turn the problem around and pose the problem this way: If *y* acre-ft of storage is available, what flow could be assured through the dry period? Assume we are going to analyze the 1977–1978 period. Then, by moving the vertical line representing the storage available between line *cd* and the mass curve until line *cd* reaches its minimum slope, the slope of *cd* then is the discharge that could be assured for that period for the given storage. In this case, a flow of 7500 cfs would be assured with a storage of 2,000,000 acre-ft. Note also that reservoir withdrawal would occur over a 2-yr period because of the drought's duration.

We have presented the foregoing procedure for graphical solutions of the problem. However, it is easy to see that all the quantities and concepts can be set up on a computer for a much more rapid and accurate solution. The solution procedure was for the simplest of problems; in actuality, one would probably not require that the outflow be constant during the drought period, and one would also have to consider evaporation and perhaps seepage as other withdrawals from the reservoir. If the period of record of streamflow is short, it may also be necessary to generate long-term synthetic streamflow records that could be analyzed for the low-flow periods. The generation of synthetic records involves applying statistical methods and is part of the science of *stochastic hydrology*. One of the weakest features of the yield analysis using a mass curve of historical records is that the sequence of inflows can never be expected to recur. As a result, it is impossible to assign any probability to the expected yield. In Chapter 2, beginning on page 49, we provide some further considerations on the determination of yield based on return period.

EXAMPLE 6-2 For the conditions shown in Fig. 6-28, what flow could be assured for this period if a storage of 1 million acre-ft were available to draw on?

SOLUTION First, convert 1 million acre ft (A.F.) to s-ft-mo. (sfm) to have storage units consistent with Fig. 6-28.

$$\begin{aligned} 1 \text{ sfm} &= 1 \text{ ft}^3/\text{s} \times 30 \text{ da/mo} \times 24 \text{ hr/da} \times 3600 \text{ s/hr} \\ &= 2.592 \times 10^6 \text{ ft}^3 \\ &= 59.504 \text{ A.F.} \end{aligned}$$

Thus 1 sfm for a 30-day month is equivalent to 59.504 A.F. Then, 1×10^6 A.F. = $(1 \times 10^6$ A.F.)/(59.504 A.F./sfm) = 16,806 sfm. With a storage of 16,806 sfm, it is found by trial and error that the greatest drawdown in the reservoir occurs around April 1978, and that water during the drought period is first drawn from the reservoir around September 1976. The sustained flow for this period is found to be about 6600 cfs. ■

Storage Allocations

INTRODUCTION The storage in a reservoir is rarely allocated for just one use. Even though a dam may be built primarily for power production, consideration will almost always have to be given to operation of the reservoir to mitigate floods, to mitigate damage to fish and wildlife, and to be compatible to a degree to recreational activities on the stream. To satisfy these various demands for the reservoir storage, special terms have been adopted for each of these storage requirements. They are storage for *instream flow requirements* (fish and other aquatic life, recreation, wildlife, and navigation), *flood control, irrigation,* and *power production.*

CONFLICTING STORAGE DEMANDS Storage requirements often conflict with one another. For example, for maximum power production, keeping the reservoir at full pool level is necessary to maintain the highest head possible. However, if the reservoir is to mitigate flood damage, it would have to be drawn down considerably over time before floods are expected so that a significant portion of the reservoir is available to store the flood waters and reduce the flood peaks. Another conflict can arise between power interests and recreational activity on the river below the reservoir. In many power systems, hydropower is used to supply peak load demand. Thus on many streams, the flow varies greatly throughout the day. Highly varying flows from such an operation can be a severe annoyance to fishermen and boaters on the river, and in several cases, lives have been lost because people have been stranded in places (such as on islands) where they could not escape the rising water.

The most pronounced conflict often develops between instream flow needs and irrigation requirements. On many watersheds, most or all of the water may be allocated for irrigation. In that case, the stream below the irrigation diversions will be depleted, and much or all of the fish and other aquatic life will be eliminated, navigation and recreational boating will be made impractical, and other forms of wildlife usually will be adversely affected. A severe situation like this may develop on streams with little or no storage initially developed on it. Thus dam and reservoir projects are often developed for these streams to help restore some of the benefits that existed before excessive withdrawals for irrigation. The design of such a project should include storage allocations for irrigation, instream requirements, and perhaps flood control.

When a project is being planned, decisions must be made about the allocation of water for the various demands. If this cannot be worked out amicably between the parties that have the various interests, legal action is often taken to resolve the matter. In any case, by the time the final design of the dam and reservoir are being carried out, there should be storage allocations for various needs. That is, specific volumes within the reservoir should be reserved for specific purposes. For example, the U.S. Bureau of Reclamation will assign specific volumes in the reservoir for *flood control*, *joint use*, and *conservation* (19). The top part of the reservoir is reserved for flood control, and the next level is reserved for joint use. The joint use part is assigned for flood control during a period of the year and for conservation during a different period of the year. The part of the reservoir below the joint use part is reserved for conservation, by which is meant all water used for irrigation, power production, municipal and industrial water supply, fish and wildlife, recreation, navigation, and for water quality enhancement.

Once the different storage capacities have been allocated, reservoir operating procedures must be developed. These procedures include the way and time of year the reservoir is to be drawn down to provide maximum flood control, the way the water is to be released when flood flows arrive, the way the water is to be released for instream uses, the power and irrigation demands, and so on. Obviously, much care and analysis is needed to develop operating plans for a multiple-use project. Often, linear program formulations systems analysis is used to develop the operating procedure that will optimize the system.* We present additional information on storage allocation for flood control in Chapter 9.

Wind-Generated Waves, Setup, and Freeboard

INTRODUCTION Whenever wind blows over an open stretch of water, waves develop, and the mean level of the water surface may change. The latter phenomenon, called *setup* or *wind tide*, is significant only in relatively shallow reservoirs. When a dam is being designed, the crest of the dam must be made higher than the maximum pool level in the reservoir to prevent overtopping of the dam as the wind-generated waves strike the face of it. The additional height given to the crest of the dam to take care of wave action, setup, and possibly settlement of the dam (if it is earthfill) is called *freeboard*. In the next two sections, we discuss the factors that control setup and wave height.

SETUP Consider the basin of water shown in Fig. 6-29. The solid line depicting the water surface is the case when no wind is blowing; the water surface

* For a more thorough discussion of reservoir system operation, see Louchs (10) and Toebes (16).

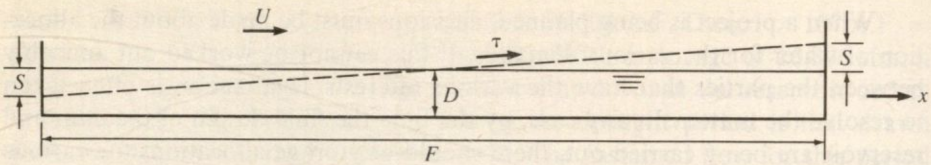


Figure 6-29 Definition sketch for setup

is horizontal. When the wind is blowing, a shear stress acts on the water surface, and because of this, the surface will tilt, as shown by the broken line in the figure. If one considers just the static forces relating to this problem, determining the amount of setup S is easy.* Consider the forces acting on the water in the basin in the x direction. Assume the basin has a length F in the direction of the wind velocity U, and assume the dimension normal to the page is ℓ. The forces in the x direction will be the shearing force produced by the wind and the hydrostatic forces on the ends of the basin. The force balance equation is

$$\sum F_x = 0$$

$$\frac{\gamma_w(D-S)^2\ell}{2} - \frac{\gamma_w(D+S)^2\ell}{2} + \tau_0 F\ell = 0 \tag{6-8}$$

but τ_0 can be given as

$$\tau_0 = \frac{C_f \rho_{\text{air}} U^2}{2} \tag{6-9}$$

where C_f is the average shear stress coefficient. Solving Eq. (6-8) for S yields

$$S = \frac{\gamma_a}{\gamma_w}\frac{C_f}{8}\frac{V^2F}{gD} \tag{6-10}$$

where γ_a = specific weight of air
γ_w = specific weight of water
g = acceleration due to gravity

If one assumes γ_a/γ_w and C_f are essentially constant, Eq. (6-10) can be expressed as

$$S = K\frac{V^2F}{gD} \tag{6-11}$$

* Actually the problem is more complicated than this because once the shear stress acts, circulation will start; however, this simple derivation illustrates the role of the primary variables.

Analysis of work by Saville (13) shows that a reasonable K value is 2.025×10^{-6} for reservoirs; therefore, Eq. (6-11) can be written as

$$S = 2.025 \times 10^{-6} \frac{V^2 F}{gD} \tag{6-12}$$

where V = velocity in ft/s (or m/s)
D = average depth of the reservoir in ft (m)
F = fetch in ft (m)

EXAMPLE 6-3 A reservoir is oval shaped with a length of 10 mi and a width of 5 mi. If the wind blows in a direction lengthwise to the reservoir with a velocity of 80 mph, what will be the setup if the average depth is 20 ft?

SOLUTION The fetch will be 10 mi or 52,800 ft, and the wind velocity is 117.3 ft/s; therefore, the setup will be

$$S = 2.025 \times 10^{-6} \times \frac{V^2 F}{gD}$$

$$= 2.025 \times 10^{-6} \times \frac{(117.3 \text{ ft/s})^2 \times 52{,}800 \text{ ft}}{32.2 \text{ ft/s}^2 \times 20 \text{ ft}}$$

$$= 2.28 \text{ ft}$$ ■

HEIGHT OF WIND WAVES Starting in 1950, the U.S. Army conducted tests to evaluate the wave heights that might be expected in a reservoir with a given fetch and wind speed. *Fetch* is the open water distance (in the direction of the wind velocity) upwind of the point in question. Thus if one were to determine the fetch for the dam on the reservoir shown in Fig. 6-30, and with the given wind direction, the fetch, F, would be as shown.* The results of the U.S.

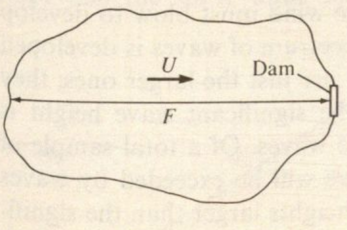


Figure 6-30 Definition sketch for fetch

* Saville showed that if the reservoir is fairly narrow in a direction normal to the wind direction, then the effective fetch would be less than the simple definition given here. For the method of computing effective fetch for long narrow reservoirs see Saville (13).

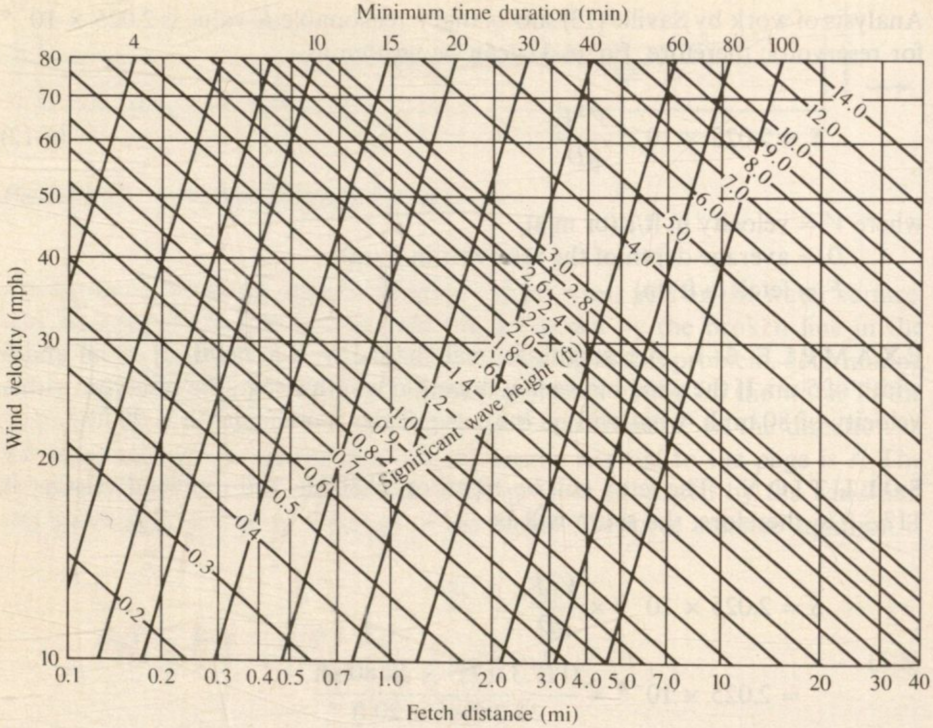


Figure 6-31 Significant wave height, H_s, as a function of wind speed and fetch (13)

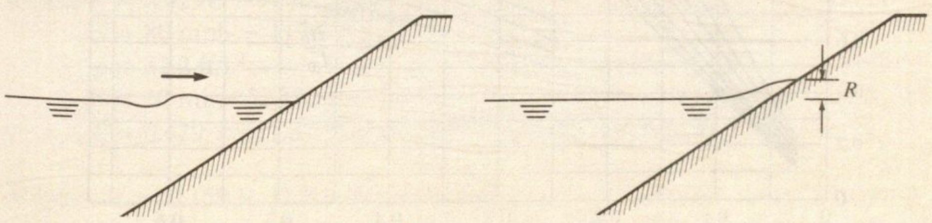
Army studies were published by Saville in 1963 (13). A particularly useful chart developed from that study allows one to determine wave height as a function of wind velocity and fetch. Figure 6-31 is an adaptation of the chart developed by Saville. By entering the chart with wind velocity (given on the ordinate) and fetch in miles (abscissa), one can read off significant wave height (lines that slope downward to the right). Focusing on the lines that slope upward to the right, one can find the minimum time duration that the wind must blow to develop that particular wave height. Actually, an entire spectrum of waves is developed in a storm, and the heights indicated in Fig. 6-31 are just the larger ones; they are called *significant waves.* In Saville's study, the significant wave height is the average height of the highest one third of the waves. Of a total sample of waves, it was also found that the significant wave will be exceeded by waves of greater height 13% of the time. Waves having heights larger than the significant wave height will be exceeded by fewer than 13% of the waves depending on the given height. The relative wave height (the given wave height to the significant wave height) as a function of distribution is given in Table 6-4. Thus a wave height 1.67 times the significant wave height would have only 0.4% of

Table 6-4 Wave Heights Distributions (13)

Total Number of Waves in Series Averaged to Compute Specific Wave Height, H, (%)	Ratio of Specific Wave Height, H, to Significant Wave Height, H_s (H/H_s)	Waves Exceeding Specific Wave Height, H (%)
1	1.67	0.4
5	1.40	2
10	1.27	4
20	1.12	8
25	1.07	10
30	1.02	12
$33\frac{1}{3}$	1.00	13
40	0.95	16
50	0.89	20
75	0.75	32
100	0.62	46

waves exceeding its height. Table 6-4 can be used with Fig. 6-31 to choose the freeboard needed for a given dam. The designer must choose what wave height to design for. One may wish to be conservative and choose the $H/H_s = 1.67$. If the freeboard were designed for this, only about 0.4% of the waves would splash to the top of the dam. A design based on a lesser wave height would allow more waves to splash to the top of the dam, which might be acceptable if a drainage system were designed to carry the excess water away without erosion. At first glance, one might be tempted to choose a freeboard based on the very largest wave that might be expected to reach the top of the dam, but proper drainage design might allow some splashing of the larger waves over the top of the dam.

Once the wave height has been determined, the amount of freeboard chosen depends on the amount of wave *runup* on the face of the dam. Wave *runup*, R, is the difference between maximum elevation attained by wave runup on a slope and the static water elevation at the toe of the slope. The concept of



(*a*) Solitary wave approaching an embankment (*b*) Maximum runup of wave

Figure 6-32 Wave runup

runup is best perceived by visualizing a solitary wave approaching an embankment, as shown in Fig. 6-32*a*. When the wave reaches the embankment, it will break and run up the slope, as shown in Fig. 6-32*b*.

The amount of runup for a wave of given size has been shown by Saville (13) to be a function of the slope of the embankment, the degree of roughness of the embankment, and the relative steepness of the wave, H_0/L_0 (see Fig. 6-33), where H_0 is the wave height in ft or m, and L_0 is the wave length. Here L_0 is given as

$$L_0 = 0.159gT^2 \tag{6-13}$$

where T is the wave period in seconds, and g is the acceleration due to gravity in ft/s^2 or m/s^2. T is a function of fetch and wind speed:

$$T = \frac{0.429U^{0.44}F^{0.28}}{g^{0.72}} \tag{6-14}$$

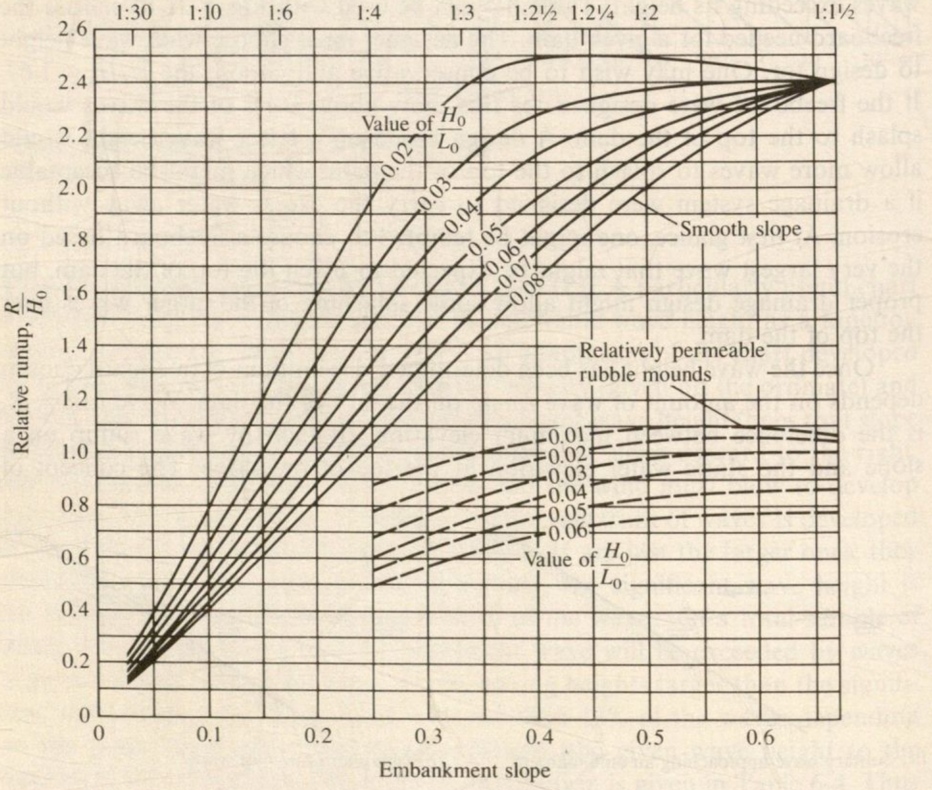


Figure 6-33 Wave runup ratios versus wave steepness and embankment slopes (13)

In Eq. (6-14), U is in ft/s or m/s, and F is in ft or m. The preceding formulas and Figs. 6-1, 6-33, and Table 6-4 are for deep water waves; that is, for depths greater than about one half the wave length (13). Therefore, if the reservoir depth is greater than 0.50 $(0.159gT^2)$ (from Eq. 6-13), one can reliably use these charts and formulas. However, if shoal water occurs before the embankment or dam face, the shallow wave case exists, and one must use different relations.*

FREEBOARD DETERMINATION Freeboard is calculated by summing the amount of setup and the runup that might be expected and adding another increment for intangible effects. These intangibles could include settlement of the embankment. Example 6-4 gives the process of calculating freeboard.

EXAMPLE 6-4 Considering the wind conditions and reservoir size of Example 6-3, what freeboard should be used for an earth dam having an upstream slope of 1 V to 3.0 H? Assume the upstream embankment is faced with well-designed riprap. Further assume the freeboard is to be based on the wave height that will be exceeded in height by only 2% of the waves. The average reservoir depth is 100 ft, and the dam is 200 ft high.

SOLUTION The freeboard will be equal to setup plus runup plus allowance for settlement of the embankment plus an amount for contingencies.

First determine runup: Runup is a function of the wave height and slope and roughness of embankment. The significant wave height H_s is obtained from Fig. 6-31 and for the conditions of this example ($F = 10$ mi and $V = 80$ mph), is found to be 10.6 ft. It is also found from Fig. 6-31 that the wind must blow for about 75 min for this wave height to develop. From Table 6-4, page 359, the wave height that will be exceeded by only 2% of the waves is equal to $1.40H_s$ or $H = 1.40 \times 10.6 = 14.8$ ft.

Next, we must determine the wave length so that we may determine runup with the use of Fig. 6-33. From Eq. (6-13),

$$L = 0.159gT^2$$

where $T = 0.429U^{0.44}F^{0.28}/g^{0.72}$

$U = 80 \text{ mph} = 117.3 \text{ ft/s}$

$g = 32.2 \text{ ft/s}^2$

$F = 10 \text{ mi} = 52{,}800 \text{ ft}$

$T = 0.429 \times 117.3^{0.44} \times 52{,}800^{0.280}/32.2^{0.72} = 6.02 \text{ s}$

Then $L = 0.159 \times 32.2 \times 6.02^2 = 186 \text{ ft}$

* For information on the shallow wave relation, see Saville (13). Other design data on waves and runup may be found in the U.S. Army Corps of Engineers manual on shore protection (17).

$$\frac{H}{L} = \frac{14.8\text{ ft}}{186\text{ ft}} = 0.080$$

From Fig. 6-33, for a value of $H/L = 0.080$ and a slope of 0.33, it is estimated that $R/H \approx 0.50$, (extrapolation of Fig. 6-33) or

$$R = 0.50 \times 14.8 = 7.40\text{ ft}$$

Setup: From Eq. (6-12), we have

$$S = \frac{2.025 \times 10^{-6} V^2 F}{gD}$$

or, for this example,

$$S = \frac{2.025 \times 10^{-6} \times 117.3^2 \times 52{,}800}{32.2 \times 100}$$

$$= 0.46\text{ ft}$$

Settlement: Assume a 1% settlement in the embankment, so settlement freeboard would be 2.0 ft.
Contingencies: For other uncertainties, allow 1.5 ft.

$$\text{Total freeboard} = 7.40 + 0.46 + 2.0 + 1.5 = 11.36\text{ ft} \quad \blacksquare$$

Sedimentation in Reservoirs

INTRODUCTION All streams carry sediments that originate from erosion processes in the basins that feed the streams. In some streams, the average rate of sediment inflow to the reach will equal the rate of outflow; the stream will be in equilibrium. However, if a dam is constructed across the stream and a reservoir is produced, the velocity in the reservoir will be negligible so that virtually all the sediment coming into the reservoir will settle out and be trapped. Therefore, the reservoir should be designed with enough volume to hold the sediment and still operate as a water storage reservoir over the project's design life. For large projects, the design life is often considered 100 years.

Sediment carried in a stream is classified as either *bed load* or *suspended load.* The bed load consists of the coarsest fractions of the sediment (sands and gravels), and it rolls, slides, and bounces along the bottom of the stream. The finer sediments are suspended by the turbulence of the stream. When the sediment enters the lower velocity zone of the reservoir, the coarser sediments will be deposited first, and it is in this region that a delta will be formed (see Fig. 6-34). The finer sediments will be deposited beyond the delta at the bottom of the reservoir.

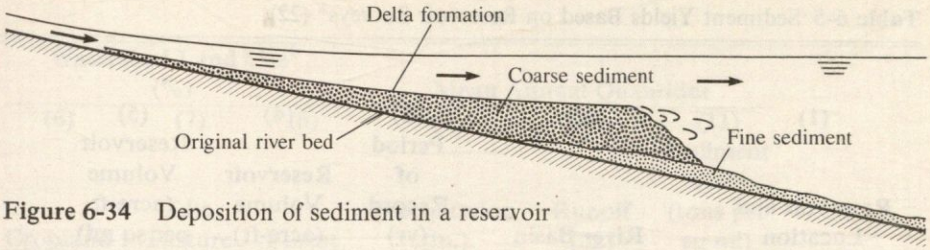


Figure 6-34 Deposition of sediment in a reservoir

Problems relating to sedimentation so far as design and operation of the reservoir are concerned are

1. Estimating the rate of accumulation of sediment in the reservoir so that enough volume can be reserved for the sediment accumulation.
2. Including the anticipated sediment accumulation in the operating plan of the reservoir. For example, because the initial phases of delta formation will be in the higher elevation zone of the reservoir, more reservoir storage will be lost in that zone than in zones at lower elevations. Thus flood-control storage may be reduced at a much faster rate than instream flow storage if the storage for flood control were initially allocated in the upper part of the reservoir. As sediment accumulates, it may be desirable to adjust the storage allocation for the different classes of storage.
3. Planning for location of recreational sites. For example, a boat dock would not be included in an area where a delta would likely form in a few years.

In the next two sections, we address the process of estimating the rate of flow of sediment into the reservoir and the volume it may be expected to occupy.

Sediment Yield The total sediment outflow from a watershed or drainage basin measured in a specified period is the sediment yield. Often, the yield is expressed in terms of tons per acre per year. The engineer designing a reservoir must estimate the average sediment yield for the basin supplying the reservoir to determine at what rate the reservoir will fill with sediment. The estimate of sediment yield is usually based on one or more of the following procedures, which we list from highest to lowest expected accuracy:

1. Obtain records of measured sediment discharge at the reservoir site or near the site. These records should cover ten or more years to establish a reasonable dependable average yield. The methods of measuring sediment discharge are given by Vanoni (22).
2. Look for measured sediment discharge records on basins that have characteristics similar to the basin under consideration. Start a sediment discharge measuring program at the reservoir site as soon as possible, and compare short-term records for the site with similar records from basin(s) having similar characteristics. If a good correlation between the records for the two basins exists, the long-term averages from the other basin can possibly be used for preliminary estimates to design the reservoir.

Table 6-5 Sediment Yields Based on Reservoir Surveys* (22)

(1) Reservoir and Location	(2) River Basin	(3) Period of Record (yr)	(4) Reservoir Volume (acre-ft)	(5) Reservoir Volume (acre-ft per sq mi)
Sardis near Sardis, Miss.	Little Tallahatchie River	20.7	91,900	59.0
Bodeman near Brunswick, Ind.	Tributary of West Creek	13.0	52	19.0
Lake Tandy near Hopkinsville, Ky.	Little River	52.5	770	130.0
Kiser Lake near St. Paris, Ohio	Mosquito Creek	14.5	3,330	380.0
Upper Hocking #2 near Lancaster, Ohio	Hunter's Run	5.0	57	30.0
Lake Dante near Wagner, S. Dak.	Tributary to Choteau Creek	17.9	261	90.0
John Martin near Caddoa, Colo.	Arkansas River	15.2	423,000	22.0
Altus near Altus, Okla.	North Fork Red River	12.6	157,000	62.0
Bellerud Pond near Adams, N. Dak.	Dark River	13.0	17	15.0
Walnut Cove near Walnut Cove, N.C.	Dan River	9.0	970	2.5
Mission Lake near Horton, Kans.	Mission Creek	30.2	1,900	229.0
Guernsey near Guernsey, Wyo.	Platte River	30.3	74,000	14.0§

* Randomly selected from the Summary of Reservoir Sediment Deposition Surveys made in the United States through 1960, United States Department of Agriculture, Miscellaneous Publication No. 964, Appendix A, Feb., 1965.

† Computed from Conservation Needs Inventories of Individual States—usually available by inquiry to state offices of Soil Conservation Service, United States Department of Agriculture.

‡ Estimated.

§ Based on sediment-contributing area only.

Watershed Land Use† (%)			Mean Annual Quantities		
(6)	(7)	(8)	(9)	(10)	(11)
Cropland	Idle Pasture	Forest	Precipitation (in.)	Runoff (in.)	Sediment Yield (tons per sq mi)
30	35	30	52	20	1,100
55	30	5	36	10	280
55	10	30	47	18	1,700
65	20	10	38	12	4,900
65	20	10	36	13	950
60	35	—	22	0.5	260
15	80	5	15	0.3	850
60	30	5	24	0.9	1,300
75	15	5	18	0.7	110
25	10	65	50	19	240
70	20	5	32	4.4‡	740
15	80	—	8–24	1.3	210

3. Study the records of sedimentation in existing reservoirs throughout the region. Base the estimate of sediment yield on basins having similar characteristics. Since about 1940, the U.S. Army Corps of Engineers, the U.S. Soil Conservation Service, and the U.S. Bureau of Reclamation have been measuring the amount of sedimentation in reservoirs throughout the U.S. These data are forwarded to the Federal Interagency Committee on Sedimentation, which compiles the data and distributes it to all offices of these three agencies. The designer may find information by contacting any of these agencies. Table 6-5 shows a summary of sediment yields for several basins based on reservoir surveys.

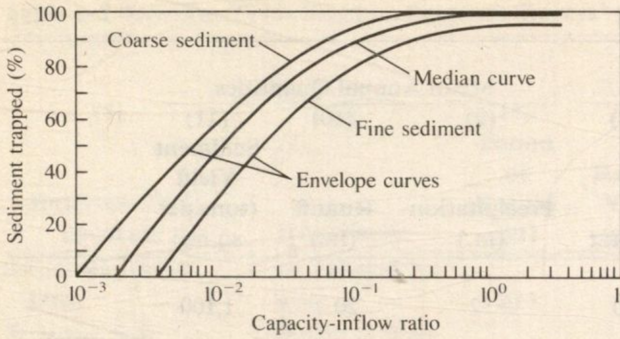


Figure 6-35 Trap efficiency curve (2)

TRAP EFFICIENCY Once the sediment reaches the reservoir, most or all of it will be "trapped" in the reservoir if it has a long enough period to settle out. Thus most sediment of sand size and larger will be trapped in a new reservoir. However, some of the fine suspended sediment may pass through the reservoir if the reservoir is relatively small in comparison to the average discharge of the stream that feeds it. Brune (2) studied 44 reservoirs and developed a relationship between trap efficiency and the capacity-inflow ratio, as shown in Fig. 6-35. Here, the capacity-inflow ratio is storage volume in the reservoir divided by the mean annual volume of water flow into the reservoir. Obviously, as the reservoir fills up with sediment (capacity-inflow ratio decreases), the velocity of water flow through the reservoir increases because the cross-sectional flow area decreases; therefore, a smaller percentage of sediment will be trapped, as Fig. 6-35 shows. Thus one may use trap efficiency curves to estimate the useful life of a reservoir.

SEDIMENT WEIGHT Because sediment yield is given in weight (tons/year), it is necessary to know the unit weight of the sediment deposit to estimate the actual volume it will occupy when it gets into the reservoir. Coarse sediments usually do not change volume much after they are deposited in a reservoir; however, fine sediments may compress considerably with time. Moreover, if the reservoir is drawn down to a level where the fine sediments are exposed and they dry out, they will consolidate much more than if never dried. Even when covered with water after drying, they will not expand appreciably. An empirical formula developed by Lane and Koelzer and presented in Vanoni (22) can be used to approximate the specific weight of sediment deposits in terms of age of deposit T in years and its initial specific weight, γ_1, taken after a year of consolidation:

$$\gamma = \gamma_1 + B \log_{10} T \tag{6-15}$$

Table 6-6 Constants in Eq. 6-15 for Estimating the Specific Weight of Reservoir Sediments* (22)

Reservoir Operation	Sand γ_1	Sand B	Silt γ_1	Silt B	Clay γ_1	Clay B
Sediment always submerged or nearly submerged	93	0	65	5.7	30	16.0
Normally a moderate reservoir drawdown	93	0	74	2.7	46	10.7
Normally considerable reservoir drawdown	93	0	79	1.0	60	6.0
Reservoir normally empty	93	0	82	0.0	78	0.0

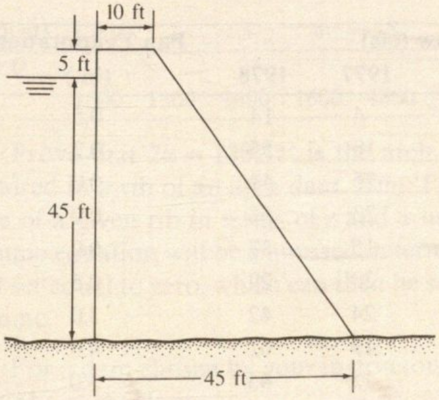
* All values given in lb/ft^3.

In Eq. (6-15), γ_1 and the coefficient B (both given in weight per unit volume) are functions of the type of sediment (sand, silt, or clay) and the manner in which the reservoir is operated (this reflects the effects of drying of the sediment deposit). Table 6-6 gives values of γ_1 and B in lb/ft^3 for various sediments and conditions.

PROBLEMS

6-1 Using the flexure formula, prove that the resultant force acting on a gravity dam must pass within the middle one third of the base of the dam to ensure that none of the concrete is in tension along the base.

6-2 Determine the normal stresses at the heel and toe of this concrete dam using the simple flexure formula for solving for the stress. Consider weight of dam, 2/3 hydrostatic uplift, and hydrostatic force on the face of the dam in your analysis.



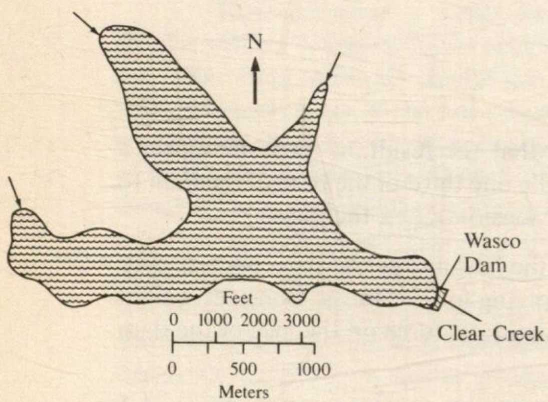
PROBLEM 6-2

6-3 For the dam of Prob. 6-2, what would be the stresses if earthquake forces were also included? Consider only horizontal acceleration toward reservoir.

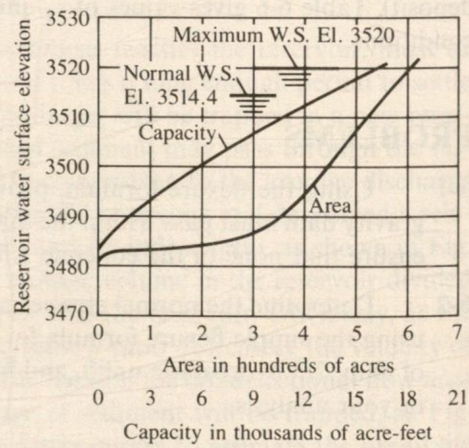
6-4 Consider the dam and conditions of Prob. 6-3. Is the dam stable against sliding? Assume the dam rests on sound rock.

6-5 Using simple arch analysis, design an arch for an arch dam. This particular arch is to be at elevation 1000 ft, and the width of the canyon wall at this elevation is 1970 ft. The maximum pool level for the reservoir is to be 1270 ft. Assume the allowable compressive stress in the concrete is 2000 psi.

6-6 Figure A shows the plan view of Wasco reservoir east of Portland, Oregon. Figure B shows the area-capacity curves for the reservoir. The



(*a*) Wasco Reservoir



(*b*) Area-Capacity curves for Wasco Reservoir

	Flow (cfs)				Pan Evaporation
Month	**1975**	**1976**	**1977**	**1978**	**(in.)**
O	13	6	4	14	0.5
N	67	20	18	55	0.2
D	30	15	25	41	0.0
J	62	30	23	56	0.0
F	23	10	15	37	0.0
M	26	12	20	29	0.5
A	39	14	24	42	3.0
M	56	20	19	61	4.0
J	57	10	15	45	5.0
J	9	5	8	12	7.0
A	5	3	6	7	6.0
S	7	2	8	6	4.0

storage reserved for conservation (irrigation) is between elevation 3488 ft and 3514 ft. The data in the table are the mean monthly flows for the creeks discharging into the Wasco reservoir and pan evaporation estimates during a drought period. For this reservoir, what outflow could be assured during the drought period?

6-7 Determine the wave height and runup for Wasco Dam (refer to Prob. 6-6 for information about the reservoir). Wasco Dam is a 50-ft high earth dam with an upstream slope of 1 V to $2\frac{1}{2}$ H. The reservoir side of the embankment is surfaced with 24-in. riprap.

6-8 Design a nonoverflow section of a gravity dam at a site where the design earthquake intensity is assumed to be 0.1 g. Assume the normal maximum pool elevation in the reservoir is 400 ft, the surface of the rock foundation is at elevation 100 ft, and the normal downstream water surface elevation is 150 ft. Make your own assumptions for other data needed in your design.

6-9 For a site chosen by your instructor, select the type of dam most appropriate for the site and give reasons for your choice.

6-10 The following table gives the anticipated average hourly water demand for a small city for a particular time of year. A reservoir is to be designed to even out the pumping rate from the well source to the reservoir. If the pumping rate is to be constant throughout the day and if it is assumed the reservoir is full when the demand is greater than Q_{avg} in early morning, what reservoir volume is required and what will be the pumping rate?

Hour (ending at)	1 A.M.	2	3	4	5	6	7	8	9	10	11	12N
Demand Q (gpm)	800	800	900	1000	1200	1425	1900	2200	2000	1575	1600	1700
Hour (ending at)	1 P.M.	2	3	4	5	6	7	8	9	10	11	12M
Demand Q (gpm)	1500	1300	1400	1600	1800	2300	1800	1500	1200	1000	900	800

6-11 Prove that $2\alpha = 133.57°$ is the arch angle that minimizes the volume required in a rib of an arch dam. *Hint*: First write an equation for the volume of a given rib in terms of r and α using Eq. (6-7), page 331. Then the volume equation will be expressed in terms of α, which can be differentiated and set equal to zero, which can then be solved to obtain the α for minimum volume.

6-12 For a dam chosen by your instructor, determine the following information about the dam:

a. What type of dam is it?
b. What earthquake intensity was used in design?

c. Is there a grout curtain in the foundation?
d. Are there drains at the base of the dam?
e. Are there any other drains?
f. Was uplift considered in the design? To what extent?
g. Were ice forces considered in the design?
h. What is the foundation rock?
i. Was any other type of dam considered for the site?
j. Is a powerhouse a part of the project? Is so, what type, number, and size (power generating capacity) of turbines are installed in the powerhouse?
k. What are the basic dimensions of the dam?
l. Have there been any leakage problems? If so, what are the details about it? How were they remedied?
m. How was the river diverted during construction?
n. Have there been any problems relating to the reservoir, such as slides or abnormal sedimentation?
o. Are any monitoring instruments installed in the dam to detect dam or foundation movement?
p. What is the magnitude of the design flood? Was it based on the probable maximum storm?

6-13 Lake Okechobee in Florida has an average depth of 7 ft and is about 37 mi long and 30 mi wide. If winds of 40 mph were to blow over the lake, what would be the setup?

6-14 For a 1000-acre reservoir site in your region of the country (specific location to be chosen by you or your instructor), estimate the sediment yield (tons/year) for the site. What volume in acre-feet is this equivalent to?

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