MECHANICS OF SLIDE DAMS

by G. A. Young*

INTRODUCTION

Studies which promote the use of nuclear energy for peaceful projects in engineering are sponsored by the Atomic Energy Commission under the Plowshare program. Specific projects being considered include the construction of harbors, canals, and dams. Of these projects, perhaps the most difficult to accomplish will be the latter.

This paper which is in two parts considers the problems which are associated with the construction of slide dams with nuclear explosives. It examines first the characteristics of conventional earth and rock-fill dams which are based upon proven techniques developed after many years of experience. The characteristics of natural landslide dams are also briefly considered to identify potential problems that must be overcome by slide dam construction techniques. Second, the mechanics of slide dams as determined from small-scale laboratory studies are presented. It is concluded that slide dams can be constructed and that small-scale field tests and additional laboratory studies are justified.

CHARACTERISTICS OF CONVENTIONAL EARTH- AND ROCK-FILL DAMS

Earth-Fill Dams

Today most earth-fill dams are constructed by rolled-fill methods. Modern practice[1] requires that fill material of carefully controlled moisture content be placed in layers, usually less than 1 foot thick. After being placed, each layer is thoroughly compacted by an appropriate roller to provide the maximum relative density. A nearly uniform gradation of material may be used throughout the dam or, when materials permit, zones with different gradations of material and different permeabilities may be created (Figures 1 and 2). An arrangement in which the less permeable zone is placed near the upstream face provides good drainage and prevents a build up of seepage pressure near the downstream toe of the embankment. However, the upstream face is susceptible to failure during reservoir drawdown conditions.

An important element in all earth-fill dams is the graded filter drain shown at the downstream toe in Figures 1 and 2. The filter drain improves the stability of the downstream face. Without a filter,[2] fills of nearly uniform permeability become almost entirely saturated, and the seepage line intersects the downstream slope, as illustrated by the broken line in Figure 1. This condition develops regardless of the base width. A properly designed filter drain, however, draws the seepage line well down into the fill and greatly increases stability (see Figure 1). The filter is constructed of layers of pervious material, each layer having a different range of particle sizes,

**Director of Engineering, Agbabian-Jacobsen Associates, Los Angeles*

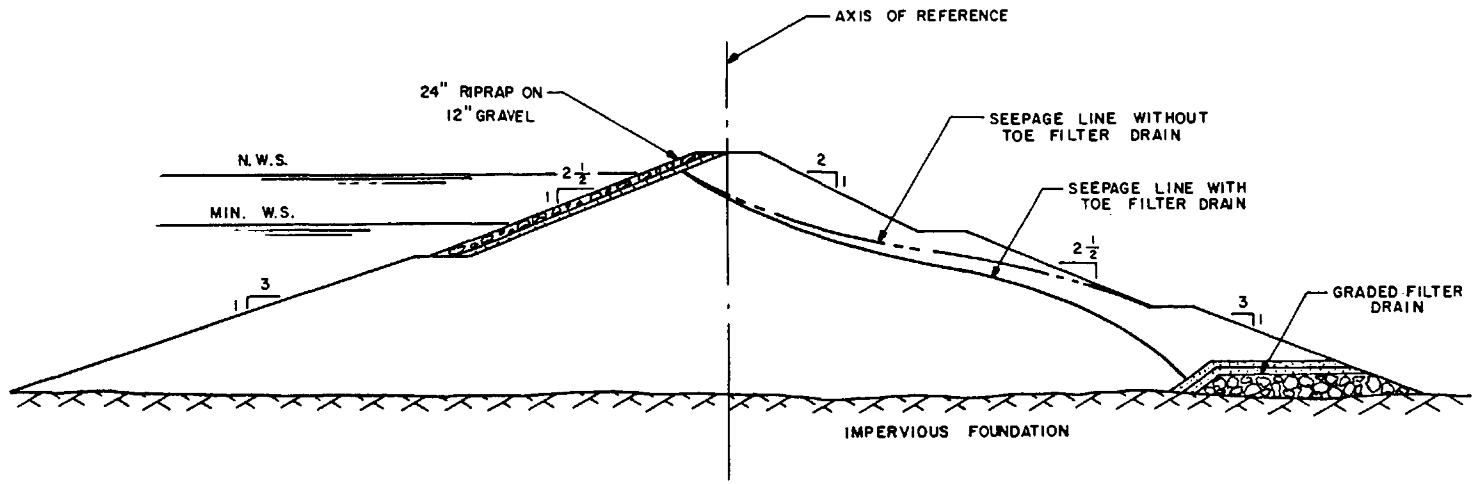


Figure 1. Typical Section of a Rolled Earth-fill Dam Built with Uniform Materials

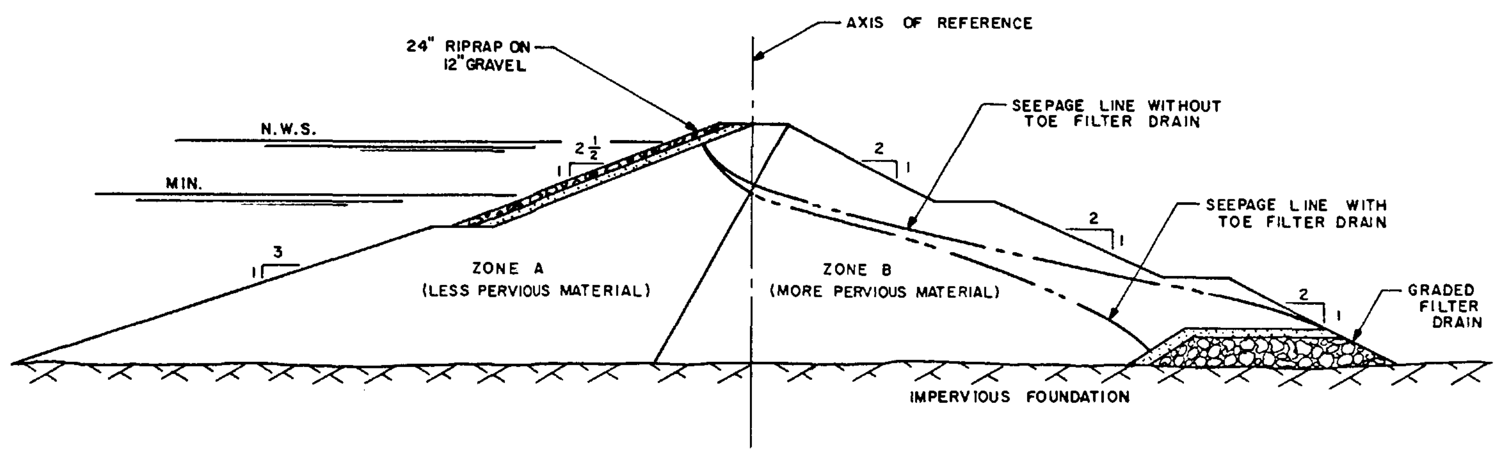


Figure 2. Typical Section of a Rolled Earth-fill Dam Built with Zoned Materials

selected such that the void diameter of each layer is smaller than the diameter of the finer particles of the preceding layer. The layer immediately adjacent to the fill has the smallest particle and void diameters and retains the fine particles in the fill. Without a filter, fine particles are washed out of the fill, thereby increasing the velocity of the seepage flow. Still larger particles are then eroded, until eventually a free-flowing passageway or "pipe" develops through the dam and causes failure. This phenomenon is called "piping." The exact filter drain arrangement required depends greatly upon the relative permeabilities of the fill and the foundation. The arrangements shown in Figures 1 and 2 assume a relatively impervious foundation.

Slopes of earth-fill dams are normally determined by the shearing strength of either the foundation or the fill material. As a result, earth dams usually have a downstream slope of about 2.5 to 1 and an upstream slope of about 2.5-3 to 1. However, if the foundation is poor, the slopes may be flatter. Provisions for slope protection are important for earth-fill dams. The upstream slope must be protected from wave action, (see Reference 3) and the downstream slope must include provisions for the drainage of surface storm water. The former normally requires slope paving of 2 to 3 feet of riprap. Even though riprap is provided, fluctuations of the reservoir water surface and wave action will gradually erode the fine particles from the fill, unless a graded filter is placed between the riprap and the fill. A normal arrangement is shown in Figures 1 and 2.

Rock-Fill Dams

Rock-fill dams are classified according to the type of core or surface used to retain the reservoir water. In general, modern rock-fill dams fall into one of three classes: those with a concrete upstream face, those with a central vertical earth core, and those with an internal earth core which slopes upstream.

Examples of rock-fill dams utilizing a reinforced-concrete upstream face are the Salt Springs Dam[4] (328 feet high) and the Bear River Dam[5] (233 feet high). A section of the latter, completed in 1952, is shown in Figure 3. This type of rock-fill dam is characterized by relatively steep slopes both upstream and downstream and by a thin, reinforced-concrete upstream-face slab supported by a 10- to 20-foot-thick layer of rubble (mortarless) masonry. The remainder of the rock fill is dumped in place from high lifts and sluiced with high-velocity water jets[3]. Because of the upstream location of the water barrier, the entire fill remains unsaturated. As a result, stable slopes as steep as 1.3-1.4 to 1 can be attained and a big saving in material realized over other types of rock-fill dams. Objections sometimes made to this arrangement are that the concrete slab is susceptable to damage during settlement of the fill and that too much labor is required to place the rubble masonry layer.

The central, vertical earth-core arrangements is perhaps the type most frequently used in recent years. The Ambuklao Dam[6] (413 feet high) and the Derbendi-Khan Dam[7] (440 feet high) are two examples. A section of the latter is shown in Figure 4. In this arrangement, the central earth core is placed by conventional rolled-fill procedures, while the rock-fill shell is placed by dumping in the manner described above. The earth core is protected by graded filter drains on both the upstream and downstream surfaces (see Figure 4). Because the shell is pervious, the upstream slope is quite stable, even though reservoir operation may permit a sudden drawdown. As a result of this stability, slopes on both the upstream and downstream surfaces can frequently be 1.75 to 1.

The sloping earth core is an arrangement intermediate between the two previously discussed. The Nantahala Dam[3] (230 feet high) and the Brownlee Dam[8] (407 feet high) are typical of this arrangement (see Figure 5). A comparison of Figures 3 and 5 shows that the downstream slope of the Brownlee Dam is as steep as that of the Bear River Dam. Because the sloping upstream core is susceptible to sliding during sudden reservoir drawdown conditions, a flatter upstream slope was provided for the Brownlee Dam.

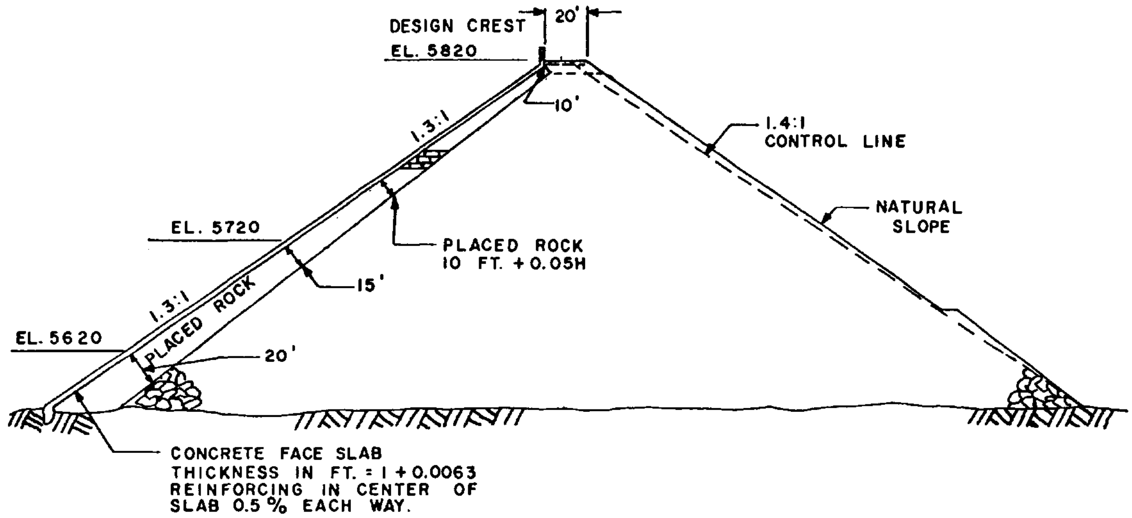


Figure 3. Section of the Bear River Rock-fill Dam

(After I. C. Steele and J. B. Cooke, "Rockfill Dams: Salt Spring and Lower Bear River Concrete Face Dams," *Journal of the Power Division of the ASCE*, August 1958, p. 1737-25.)

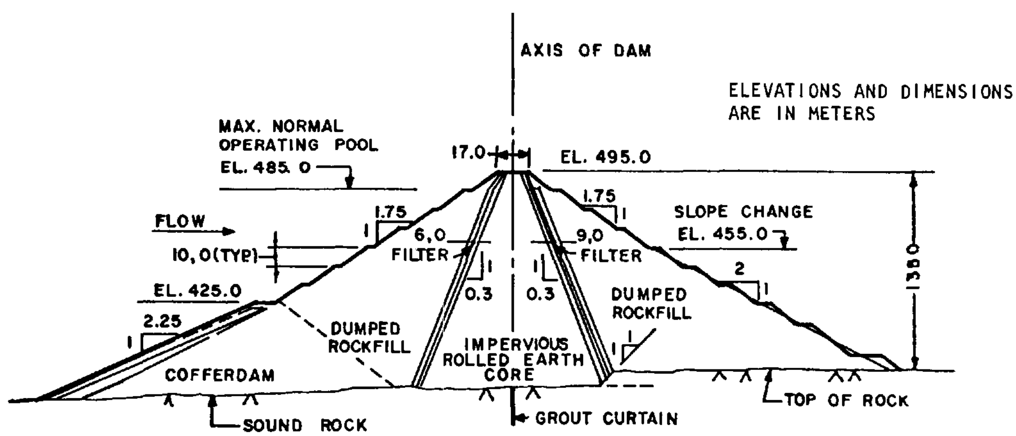


Figure 4. Section of the Derbendi-Kahn Dam

(From C. V. Davis, "Rockfill Dams: the Derbendi Khan Dam," *Journal of the Power Division of the ASCE*, August 1958, p. 1741-16.)

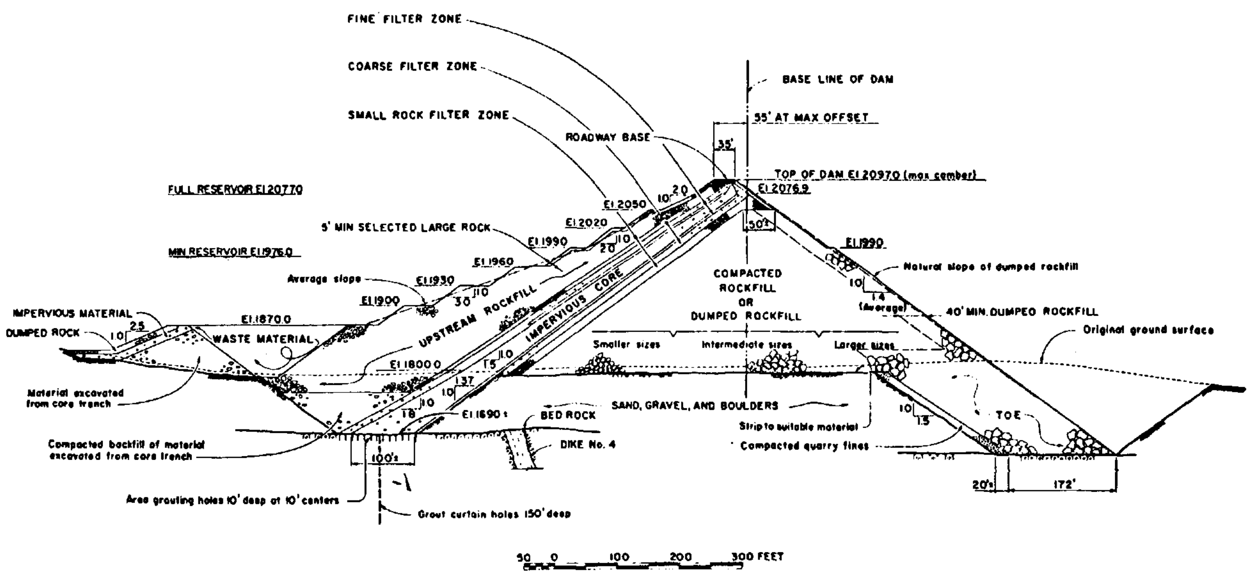


Figure 5. Maximum Section of the Brownlee Project Rock-fill Dam

(From T. Murdal, "Rock-fill Dams: Brownlee Sloping Core Dam," *Journal of the Power Division of the ASCE*, August 1958, p. 1734-7.)

To control seepage, a cutoff which makes a junction with the core in the dam is usually grouted in the rock foundation. On high dams, the cutoff frequently extends more than 100 feet into the foundation. The object of grouting is to seal open joints, cracks, fissures, and seams in the foundation rock. Weathered zones and solution channels often have to be cleaned out and backfilled with concrete before being grouted. Both the importance and the cost of these operations are frequently underestimated. This type of cutoff is shown in Figure 5. When an earth core is placed on a rock foundation, there is always considerable danger that compaction will not be uniform at the rock-earth contact because of the uneven rock surface and that, as a consequence, piping may occur through a loosely compacted zone. Two procedures are followed to prevent this possibility. In the first, one or more shallow, reinforced-concrete cutoff walls are placed nearly parallel to the longitudinal axis of the dam; the walls project 5 to 10 feet up into the earth fill and down into the rock foundation. When a reinforced-concrete upstream face slab is used to retain the water, the slab ties into a shallow cutoff wall which extends 10 to 25 feet into the foundation (Figure 3). In the second approach to this problem, the rock surface is paved with concrete or gunite to provide a smooth surface upon which the earth fill can be uniformly compacted. The foundation is also grouted as noted above.

It will be pointed out in the next section that natural landslide dams have frequently failed because of piping or because of other phenomena resulting from a lack of drainage or a lack of other provisions typical of conventional fill dams. It is well, therefore, to recognize that provisions for settlement and for seepage, slope protection, and slope stability are as essential for slide dams as for conventional fill dams.

CHARACTERISTICS OF NATURAL LANDSLIDE DAMS

Technical and scientific journals record numerous natural landslides in the past few decades, of which some have created huge natural dams. The volumes of some slides have approached the volumes of the largest embankments constructed by man. Landslide events producing these huge embankments have normally been initiated and completed in a matter of minutes in contrast to the years of effort required for equivalent man-made embankments. Unfortunately, in many cases the natural embankments have later failed and released floods which have taken human lives and caused considerable property damage. Because of both the manner in which natural dams have been produced and the manner in which some have failed, a review of the characteristics of natural landslides is essential.

CAUSES OF LANDSLIDES

Terzaghi[9] has indicated that natural landslides result from either internal or external causes. With the former, no change in shearing stress occurs along the plane of ultimate sliding; rather, sliding occurs because some internal action causes a reduction in the shearing resistance of the material. With the latter, some external action causes an increase in shearing stress, but the shearing resistance of the material along the surface of sliding remains unaltered. Terzaghi has further classified the processes which result in landslides as:

1. Construction operations or erosion
2. Tectonic movements
3. Earthquakes or blasting
4. Creep in a weak stratum below the slope
5. Rains or melting snow, causing
    a. Increase in pore water pressure
    b. Loss of apparent cohesion
    c. Swelling
    d. Chemical weathering
6. Frost action
7. Shrinkage in clays, causing cracks and loss of cohesion
8. Rapid changes of water level in lakes or of the water table in the ground
9. Slow rise of the water table or a slow increase in artesian pressure
10. Seepage from lakes, canals, etc.

It is significant that all but the first four processes listed involve the action of water in one manner or another. Also, while slides frequently take place in a very short period of time, most of the processes listed are of a continuing nature and are probably in action long before a slide occurs. Actually, none of these processes are typical of the action most likely to be utilized to initiate a slide or rock fall through the use of nuclear explosions. Indeed, some of the causes indicate conditions unfavorable to dam construction. A study of actual case histories of landslides makes this fact more evident.

CASE HISTORIES

Although several natural landslide events reported in the technical and scientific literature should prove of interest to most readers, not too much of the information is directly transferable to the problem being considered. Consequently, only four representative examples will be presented.

Gohna Slide[10,11]

This slide occurred in northern India in 1893 and is of significance because of its size and because of the manner in which the dam formed by the slide ultimately failed. The slide was initiated at an elevation of 4000 feet above the stream bed in a region where limestone beds dip toward the river at an angle of 45 to 50 degrees. Possible processes contributing to the slide were increased pore water pressure between rock layers, undercutting of the toe of the slope by the river, and a weakening of the limestone beds due to dolomitization. The slide created a dam 900 feet high with a crest length of 3000 feet and a base width of 11,000 feet (Figure 6). The initial slopes were relatively flat, on the average about 5 to 1. The volume of the slide exceeded 50 million cubic yards, and the volume of the lake ultimately impounded was slightly less than 400,000 acrefeet.

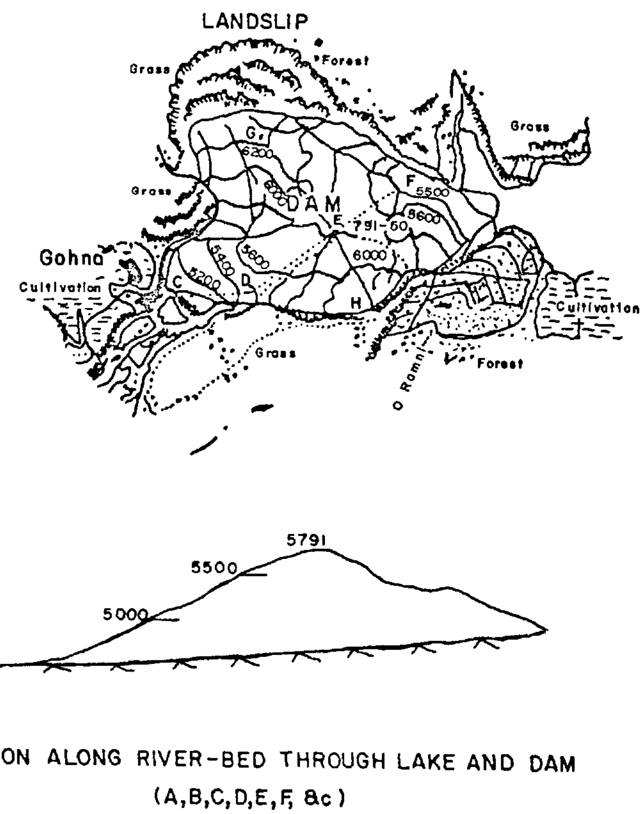


Figure 6. Plan and Profile of the Gohna Slide

(From J. H. Glass, "The Great Landslip of Gohna...," *Journal of the Society of Arts*, London, March 27, 1896, p. 432.)

The slide occurred in September, and the resulting dam was breached in August of the following year. The dam was rather closely observed, for it early was recognized that the dam would be overtopped and would probably fail. In early July, leakage from the dam was estimated as about 50 cubic feet per second. As the reservoir filled, the rate rapidly increased and was estimated by mid-August to be about 350 cubic feet per second. Following a heavy rain which occurred about this time at the dam site, a large portion of the dam slipped downstream, leaving a nearly perpendicular wall about 400 feet high. Although huge stones were also present, the slide revealed very finely ground rock material, approaching rock flour. This slide occurred when the reservoir water surface was within 80 feet of the crest of the dam.

Prior to failure, the seepage rate increased still further, and only a few hours after the crest was lightly overtopped during another rain, approximately 400 feet of the top of the embankment swept away. Failure was apparently due to piping, although failure would no doubt also have resulted from overtopping.

Gros Ventre Dam[12,13]

A slide occurred in Wyoming on the Gros Ventre River in 1925. The slide dam created is also of significance because of both its size and the manner in which it failed. In the region of the slide, beds of limestone, sandstone, and clay shales dip toward the river at an angle of about 20 degrees. The instability of the general region, particularly to the north, had been long recognized.[14]

On the day of the slide, a mass of 50 million cubic yards of material slid down a 2100-foot scarp and created a dam about 250 feet high. As shown in Figure 7, the slide was oriented upstream at an appreciable angle. Because of this orientation and also because of the low shear strength of the material, very flat slopes resulted; the base width of the dam approached 10,000 feet, and the average slopes were about 15 to 1.

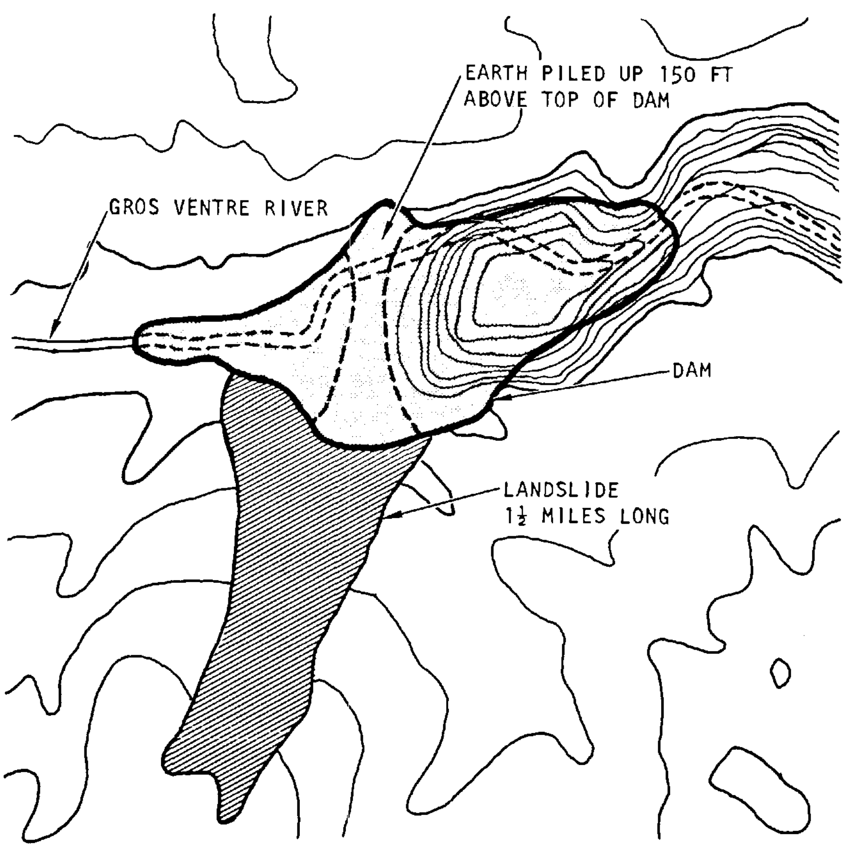


Figure 7. Gros Ventre Landslide Dam

(From R. H. Carlson, *Nuclear Explosives and Landslide Dams*, SC-4403(RR), Sandia Corporation, 1960, p. 58.)

Because the river was in flood at the time of the slide, the lake reached a depth of 200 feet within 3 weeks. No leakage appeared below the dam for about 12 days, but once seepage broke through the downstream face, its rate increased until it equaled the flow rate of the stream. At this point, the lake level stabilized at a depth of about 200 feet and later dropped. Seepage was estimated to have reached a rate of about 500 cubic feet per second.

Considerable confidence developed that the dam would not fail, but during the flood runoff 2 years later, it was overtopped, and a channel 100 feet deep and 300 feet wide was scoured across its crest. The lake had impounded about 165,000 acre-feet of water when the dam was overtopped, approximately half of which was released in only a few hours. The result was heavy damage and the loss of several lives downstream. The failure can be attributed to overtopping, but piping may also have been a factor, since the channel scoured down to the depth where the leakage was greatest.

Frank, Alberta, Slide[15]

This slide, which occurred in April 1903, destroyed a part of the town and killed about 70 people. The river valley in the region of the slide was more than 1/2 mile wide and was filled with boulder clay and gravel, above which projected several sandstone ridges and knolls. The valley was flanked on each side by low terraces, above which (on the side from which the slide was initiated) a talus slope extended at an angle of about 30 degrees. The cliff slope consisted of limestone beds which dipped into the hillside at an angle of 50 degrees. The limestone beds and the talus slope overlay shales and sandstones which also dipped into the hillside, but at a steeper angle of about 82 degrees.

Due to creep in the shale beds, which may have been aggravated by coal-mining operations at the foot of the slope, a huge mass 1/2 mile square and 400 to 500 feet thick sheared across the limestone beds (Figure 8), plowed through the boulder clays and gravel in the valley, and climbed the opposite slope to a height of 400 feet. In some places, sandstone ridges deflected the slide by as much as 90 degrees. In the valley, the slide spread over more than one square mile to a depth which varied from 3 to 150 feet, with an average of about 65 feet. Although small ponds were formed, no effective dam was created.

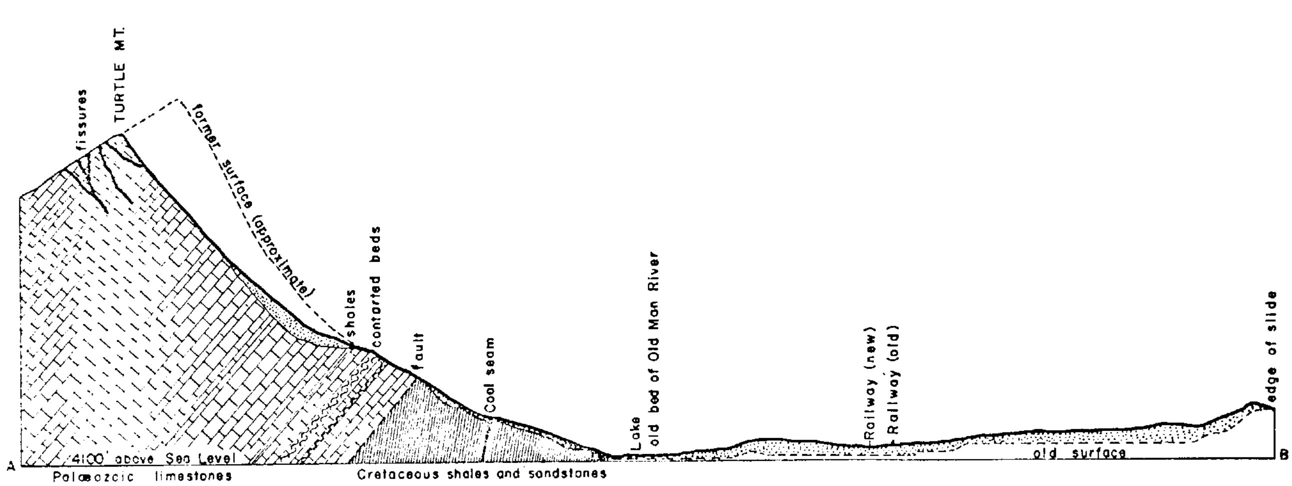


Figure 8. Section Along the Frank, Alberta, Slide

(From R. G. McConnel and R. W. Brock, *Report on the Great Landslide at Frank, Alberta*, Extract from Part VIII, Annual Report, 1903, Department of the Interior, Dominion of Canada

Madison Canyon Slide[16,17,18]

This slide, initiated by a severe earthquake near the Montana-Wyoming border in August 1959, occurred about 7 miles below Hebgen Dam and completely blocked the Madison valley. The main portion of the slide consisted of micaceous schist and gneiss which were weathered to a depth of about 100 feet. This material was held in place by a tapered buttress of dolomite at the base of the slope. The earthquake motion caused the dolomite buttress to shear, releasing the unstable mass of schist and gneiss. The volume of the slide was more than 30 million cubic yards.

The dam formed by the slide had a base width of about 4500 feet (measured along the valley) and a height of about 200 feet above the river bed. The average slopes exceeded 10 to 1 (Figure 9). The volume of the lake which would have formed was estimated at about 80,000 acre-feet. Because it was recognized that there was danger of a flood wave downstream if the dam should be overtopped in its natural condition, steps were taken to reduce the crest height by 50 feet by a process of hydraulic degradation controlled by excavating equipment. This action reduced the lake volume by more than one-half and provided a channel approximately 3500 feet long and with a controlled slope across the crest of the dam. The great length of the channel made a sudden failure no longer possible. Also, the smaller lake volume reduced the magnitude of any potential flood. In all probability, the channel across the dam will continue to degrade slowly until the channel is stabilized and a much smaller permanent lake is created.

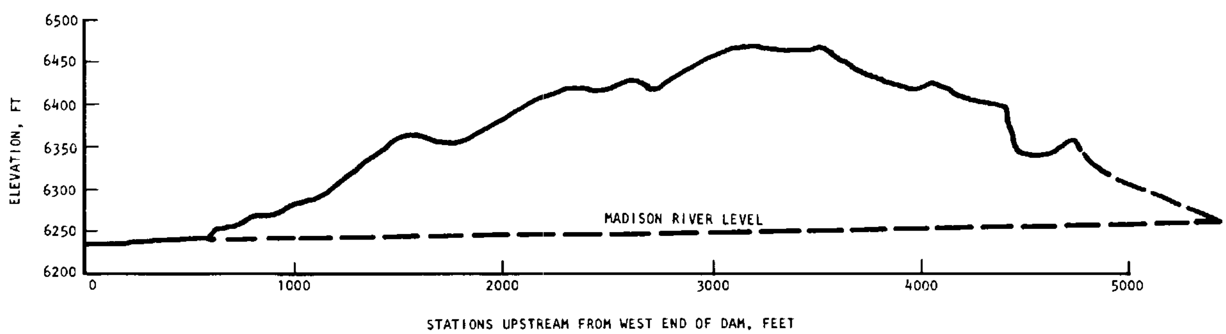


Figure 9. Profile of the Madison Canyon Slide

(From R. H. Carlson, *Nuclear Explosives and Landslide Dams*, SC-4403(RR), Sandia Corporation, 1960, p. 68.)

OBSERVATIONS FROM NATURAL LANDSLIDES

As mentioned previously, many natural landslides can be studied, but not too much can be learned which is directly applicable to the problem being considered. The following represent observations which appear pertinent.

Materials

A great variety of materials have produced notable slides, as indicated by Terzaghi[9] and by the case histories presented. Also, the materials incorporated into the slide are usually not the types of materials which would be used for fill-dam construction. Weathered schist and gneiss (Madison Canyon), unstable clays and weathered shales (Gros Ventre) are examples of such unsuitable materials.

Even when competent materials have been incorporated, the height of fall has in some instances been more than optimum. As a result, the kinetic energy imparted to the slide has caused materials as hard as limestone to be ground into fine powder (Gohna and Frank, Alberta, slides). In conventional rock-fill dam construction, it has been found that less settlement occurs when the rock is dropped from high lifts. Lifts as high as 100 to 200 feet have given good results. The reason is that settlement largely occurs in rock-fill dams when the reservoir is filled, because the increased pressure causes sharp points of contact between rocks to break off, thereby developing larger areas of contact and allowing the rocks to move closer together. When rock is dropped from high lifts, impact and sliding tend to break off sharp points and produce more rounded particles. But slides from heights of several thousand feet appear to impart too much kinetic energy; the result is flat slopes and perhaps an over-grinding of the particles. The latter is not too objectionable, but it does result in a lower angle of internal friction for the rock mass. Also fine particles are more susceptible to piping.

Because of the impact and grinding action which takes place in drops from a favorable height, it cannot be assumed that a slide in hard, sound rock would not have a favorable relative density or that settlement would not be of acceptable proportions. Judgment suggests that favorable conditions could develop for rock. Whether this statement is also true for earth-slide embankments is uncertain. Research is obviously needed to establish the relationship between height of fall and the resulting relative density of the slide embankments in both rock and earth materials.

Natural slides have all been characterized by a wide variation in the particle sizes produced. In rock slides from great heights, the gradation has ranged from rock flour to boulders with a volume of several cubic yards. Even though such embankments may be relatively dense, slides such as Gohna and Gros Ventre show that they have a marked susceptibility to piping. This would be expected; even though a rolled-fill dam is carefully controlled and placed, seepage problems will result unless proper provisions are made for drainage. Thus, experience with both natural slide dams and man-made, rolled-fill dams indicates that slide dams produced by nuclear explosives will have to be provided with an earth core or an impervious face slab to control seepage or provisions will have to be made for adequate drainage to prevent piping.

Sites

The sites of many large landslide dams have been unfavorable. At Frank, Alberta, the valley was extremely broad and was deeply covered with boulder clay and gravel--certainly an unfavorable site for either a conventional or a slide dam. While the Gohna slide occurred in a valley having a reasonable foundation, some treatment would obviously have been provided before the placement of a conventional or a slide dam. Indeed, experience with conventional dams indicates that, for either conventional or slide dams, sites should be given some treatment before placement of fill.

Spillway, diversion, and outlet arrangements for high earth- and rock-fill dams present problems which can be satisfactorily solved only by some initial construction prior to placement of the fill material. Because the foundation must also be prepared and treated before the fill is placed, natural slide dams are almost foredoomed to failure because of these omissions. Experience would indicate that a procedure for constructing slide dams by the use of nuclear explosives must also include provisions for diversion and proper spillway and outlet capacity. Some construction for these items must be provided before initiation of the slide.

Slide Profiles

A study of the cross sections or profiles of natural slide dams reveals the same general characteristics. Slopes are usually very flat, varying from 5 to 1 to as much as 10 or 15 to 1. The crest is usually quite wide and rounded. The net result is a dam with a much broader base than those of conventional dams. This is not a favorable characteristic, as will be shown later.

The flat slopes are apparently caused by more than one factor. In some cases, the slide has not been oriented normal to the valley (for example, Gros Ventre and Madison Canyon). In other cases, such as Frank, Alberta, the valley was quite broad but contained ridges which deflected the slide. Other important factors are the kinetic energy of the material in motion and its internal resistance to flow. These are probably quite variable. For example, the saturated clays at Gros Ventre may have been extremely sensitive and may have experienced spontaneous liquification. However, at the present time, not much is known about the mechanics of the slide motion. Certainly the kinetic energy of the slide particles, which can be variable with depth as well as with the lateral extent of the slide, and the kinematic viscosity of the flowing mass are factors which influence the slope at which the slide ultimately comes to rest. Thus, in contrast to conventional construction, the static shear strength of the material is not a satisfactory parameter for predicting the slope angles of a slide.

In many cases, earth- and rock-fill dams have proved feasible because it was possible to provide spillway and outlet structures at some location other than in the valley containing the fill dam. In such cases, flat earth slopes would not be objectionable. Frequently, however, the spillway, diversion, and outlet works must be in the same valley as the fill dam. This usually means that a shaft, side channel, or chute spillway must be provided and that diversion and outlet releases must be furnished by driving tunnels through the abutments around the base of the fill. In such cases, flat dam slopes are expensive; 5 or 10 to 1 slopes double or quadruple the cost of these items in comparison with 2.5 or 3 to 1 slopes. Thus, techniques that ensure satisfactory slopes for slide dams produced by nuclear explosives must be developed.

Observation Summary

From the preceding considerations, the following observations are important and indicate the requirements of slide dams:

1. The first requirement is an adequate site, one which has a good foundation and good slide materials suitably located. The site must also permit construction of economical spillway, diversion, and outlet structures.

2. It should be anticipated that foundation treatment will be required for all foundations and that this work must normally be done before initiation of the slide.

3. Theory and experience with both conventional dams and natural slide dams show the importance of adequate provisions for drainage and seepage control. Graded filter drains will be required in some cases; an impervious membrane or core adequately protected by filters may be required in others.

4. In many cases, it will be desirable to create slopes steeper than those occurring in natural slides. Techniques for achieving this objective are needed.

5. The settlement and drainage characteristics of slide dams may be influenced by the kinetic energy imparted to the slide. Thus,

relationships should be established between valley and slide geometry; rock properties; and the geometry, relative density, and permeability of the resulting slide dam.

SLIDE MECHANICS

The two previous sections are based upon a literature search and theoretical study by the author which was reported in Reference 19. This study concluded that although it is possible to develop a nuclear-induced slide dam at an ideally chosen site, a better understanding of the slide initiation techniques and of the mechanics of the slide are required to develop efficient designs. Laboratory studies were also recommended to establish the effects of valley and slide geometry, rock properties and foundation conditions upon the geometry, permeability, and settlement characteristics of the resulting slide dams.

Pilot studies consisting of small-scale laboratory experiments have been performed to provide information on the general phenomena of rock slides and to determine whether this type of behavior can be scaled. The results of these studies have been reported in Reference 20. A brief review of some of the results are of interest.

Importance of Valley Geometry

In planning the slide mechanics experiments, consideration was given to the influence of the geometry of the valley cross section. Two limiting valley cross sections are illustrated in Figure 10. In Figure 10a, the valley side slopes are nearly vertical. If, as indicated, the canyon is deep and narrow relative to the dam dimensions, the slide phenomenon becomes almost a rock fall, particularly when slides are initiated high on each side slope. For this geometry, the rock will fall nearly vertically downward until contact is made with the foundation, after which it will move laterally outward normal to the longitudinal axis of the dam. The rock particles, therefore, will have almost plane motion.

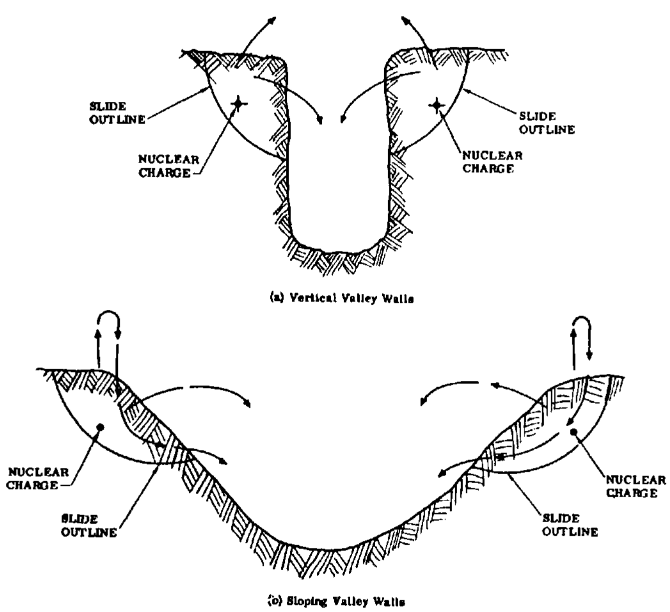


Figure 10. Valley Wall Profiles

Figure 10b represents the other limiting cross-section geometry. In this case, the slope is the flattest upon which a slide can be maintained by gravity. To initiate a slide on such a valley wall, rock would have to be heaved from the crater, after which it would fall back to the crater and slope before sliding to the valley floor. Rock particles, after falling back to the slope, would have three-dimensional motion. That is, most rock particles would have one vertical and two horizontal components as they move down the slope and onto the valley floor.

Experimental Models

Most valleys provide cross-section geometries intermediate between the two extremes described above. However, the plane motion example represents a real condition and is the simpler to consider experimentally. Because of this, it was selected for the initial studies. After some success was achieved with the plane motion studies, general phenomena were observed on a three-dimensional model. In both cases, the models were simplified idealizations of actual field conditions.

The plane motion model is shown in Figure 11a. An ideal plane motion resulted when a granular, cohesionless material, sand, was dropped vertically from a hopper to a flat concrete surface. The controlled variables were the volume of granular material (Q), the width of the hopper (b) and the height of the fall (h). The primary test objective was to observe general phenomena as the model dam was formed. Relationships were established between the controlled variables and the X and Y dimensions of the profile. Attempts were also made to produce geometrically similar profiles to establish the feasibility of scaling.

Figure 11b shows the general features of the three-dimensional model. This model, even when restricted to a consideration of geometric factors, is much more complex than the one used in the plane motion study. Only observation of general phenomena and a limited effort to produce geometrically similar profiles was attempted with this model. Controlled variables were the volume of granular material (Q), the hopper width (b), the shape of hopper cross section (b/b') the slope angle (θ), the length of slope (K), and the width of the valley (W_V).

Dimensional Relationships

A consideration of the dimensional relationships associated with the plane motion model identified eighteen dimensionless products (π terms) of which four terms involved slide and hopper geometry, four introduced properties of the flowing materials, and ten represented ratios of the physical properties of the dam and foundation materials. By using the same slide and foundation material (i.e., Ottawa sand and a concrete floor), similarity was guaranteed for the ten π terms relating the material properties of the dam and foundation. Thus, only the following π terms were significant in the plane motion model experiments:

$$\pi_1 = X/b \text{ or } Y/b \qquad \pi_5 = Vb/\nu$$

$$\pi_2 = h/b \text{ or } V^2/gb \qquad \pi_6 = \gamma V^2/gE$$

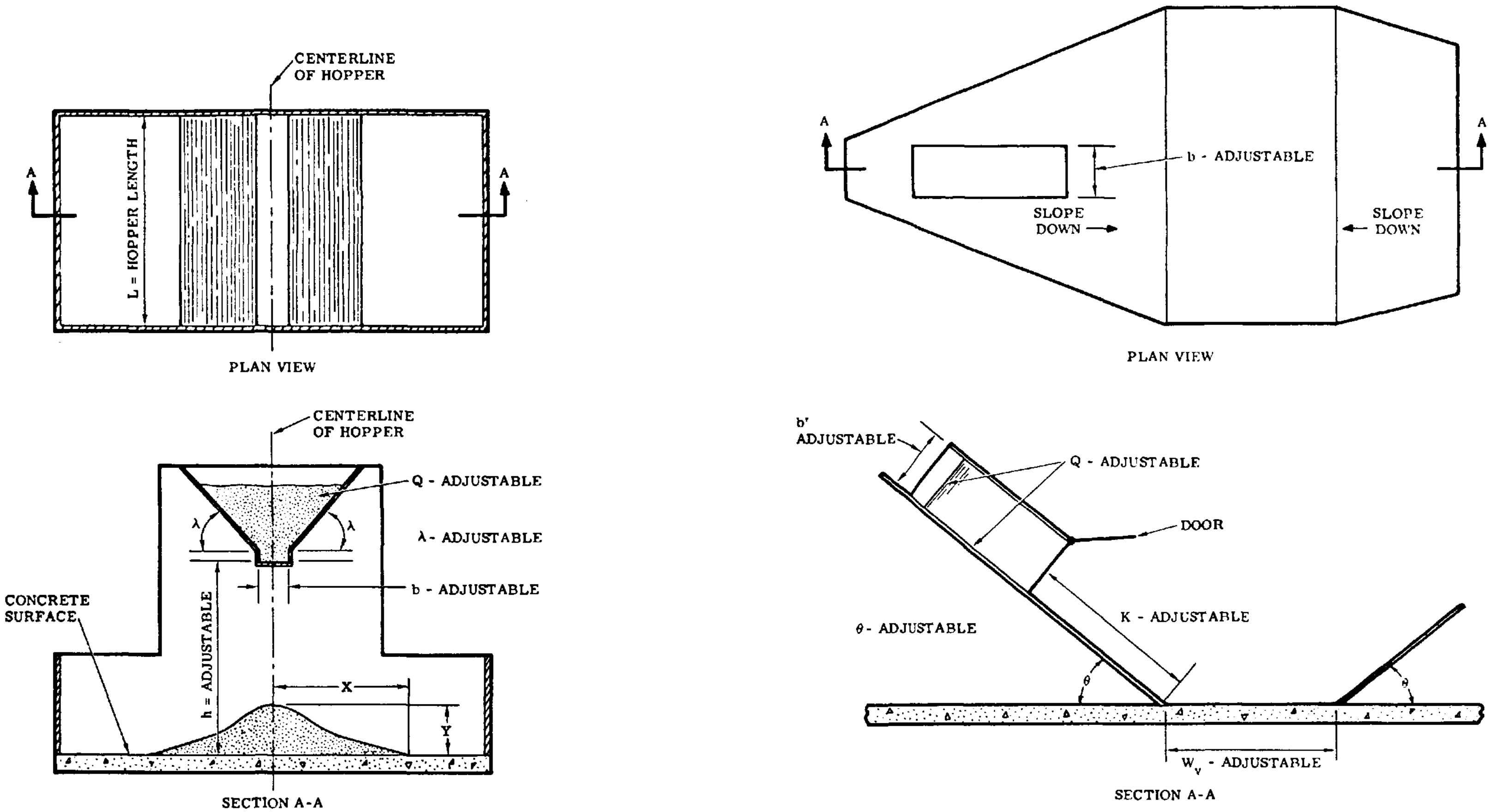
$$\pi_3 = b/d \qquad \pi_7 = \mu V/dE$$

$$\pi_4 = Q/Lb^2 = q/b^2 \qquad \pi_8 = \phi$$

Nomenclature not previously defined in the text or in Figure 11 is as follows:

g = acceleration of gravity

V = average velocity of falling particles, computed from $V = \sqrt{2gh}$



(a) Plane Motion Model

(b) Three-Dimensional Model

Figure 11. Schematic Views of Slide Mechanics Models

d = particle diameter of slide material

q = volume of slide material in hopper per foot of hopper length, $q = Q/L$

ν = kinematic coefficient of viscosity of slide material

γ = unit weight of slide material in hopper

E = elastic modulus of slide material

μ = strain rate modulus of slide material

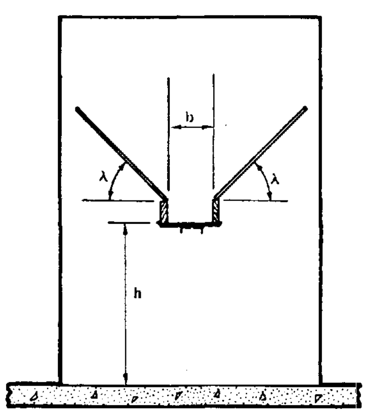
ϕ = angle of internal friction of slide material.

Two of the terms, π_5 and π_7, involve properties of flowing material which to date have not been satisfactorily measured or controlled. Therefore, no attempt was made to control these terms in any of the experiments and some model distortion resulted. It is well to note that it would be extremely difficult, if not impossible, to provide geometric scaling without a distortion in these two terms since the velocity appears to the second power in π_2 and π_6 but is a first power quantity in π_5 and π_7.

Slide Geometry Tests

As stated earlier, the primary objective of the plane motion experiments was to observe the effects of hopper geometry upon the model dam profile. Ottawa sand was selected as the principal test material. A series of tests were conducted which provided variations in the quantity of slide material, Q, the width of hopper opening, b, and the height of drop, h. The hopper configurations and heights of drop considered are indicated in Figure 12.

A limited number of experiments were performed in which the physical properties of the slide materials were varied to obtain some indication of the importance of these properties. Variations in particle density, particle size, and angle of internal friction of the granular slide material were provided. Magnetite and quartz blast sands were used to provide the variations in physical properties. A few experiments were also made to determine whether revetments could be used effectively to improve slide dam profiles. The reader should refer to Reference 20 for discussion of these studies.



CONFIGURATIONS TESTED

HOPPER WIDTH b (ft)	HOPPER HEIGHT h (ft)	HOPPER SLOPE λ (Degrees)
0.25	2.0	90°
0.25	4.0	90°
0.25	6.0	90°
0.50	2.0	90°
0.50	4.0	90°
0.50	6.0	90°
0.75	2.0	90°
0.75	4.0	90°
0.75	6.0	90°
1.0	2.0	90°
1.0	4.0	90°
1.0	6.0	90°
0.25	4.5	45°
0.50	2.5	45°
0.50	4.5	45°
0.50	2.5	60°
0.50	4.5	60°

Figure 12. Hopper Configurations Tested

General Slide Behavior

An explanation of the general slide behavior was not possible until an end wall of the test apparatus was replaced with a glass plate and high-speed photographs were taken of test runs. Visual observation through the glass and study of the pictures indicates that the behavior of the slide material upon impact with the foundation is somewhat similar to that of a water jet impinging upon a flat plate. Sand particles are first deflected outwardly parallel to the foundation surface at a high velocity. The first particles travel a great distance, and a few even travel beyond the width of the base of the final plotted profile. If the quantity of slide material is small, that is, the duration of the flow is short, the resulting profile is very flat as shown in Figure 13a for $q/b^2 = 6$ and 8. The slopes for these particular examples are about 17:1.

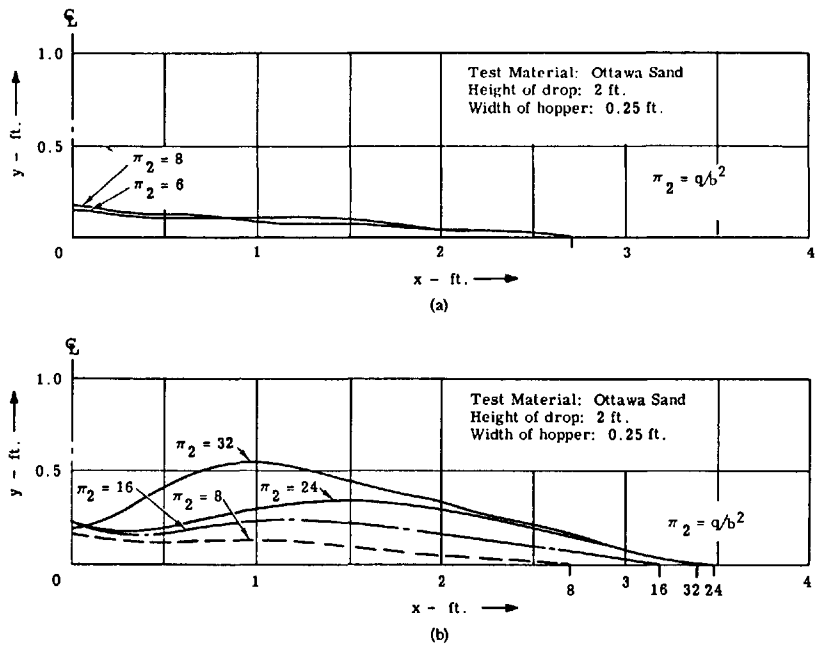


Figure 13. Comparison of Half Profiles; Quantity of Slide Material Variable

If the flow continues for a longer time, as is the case for a larger volume of slide materials (width of hopper (b) remaining the same), the lateral deflection pattern of the particles changes, and a thin stream of particles starts to flow upward and outward over previously deposited particles. A cup-like depression forms on each side of the vertical stream of particles falling from the hopper. If the flow of particles from the hopper ceases at this point, profiles similar to those shown in Figure 13b for $q/b^2 = 24$ and 32 result. The final slopes for the latter are slightly flatter than 4:1. When the flow of particles continues still longer, the pattern is much the same, but steeper final slopes result. While the flow of materials is in progress, the slopes are very steep near the crest of the model profile, but after the flow stops, the stream of material continues to move down the slopes and flatter final slopes result. The flow pattern at two different stages of profile construction is represented in Figure 14. It will be noted that the depth of the flowing material decreases outwardly from a maximum near the crest. Maximum final slopes achieved with a large volume of hopper material were approximately 2:1 with Ottawa sand.

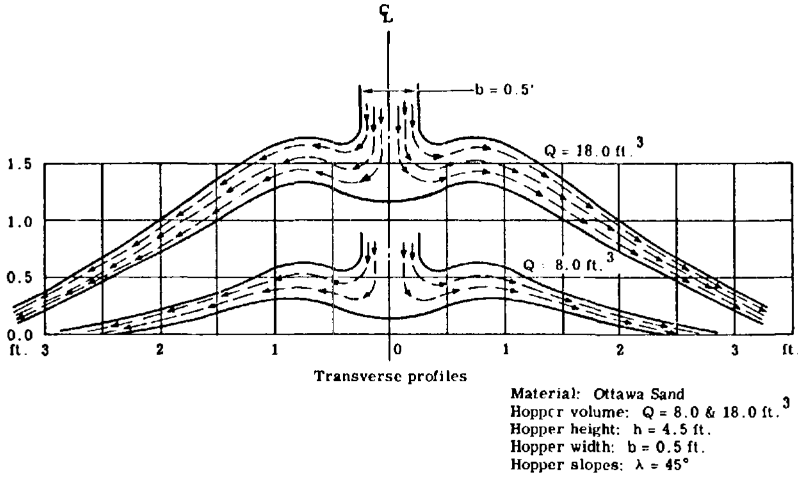


Figure 14. Flow Patterns

Effect of Hopper Width

The above discussion considered only the effect of the volume of slide material. Actually, similar profile variations can be produced by holding the volume of slide material constant and changing the hopper width. Figure 15 indicates the influence of the width of opening (b). Figure 15a suggests that decreasing the width of the hopper decreases the maximum height of dam profile. This is true, however, for only small relative quantities of slide material. When the quantity of material is doubled, as represented in Figure 15b, the maximum profile height for b = 0.25 foot is slightly greater than for b = 0.50 foot. Increasing the quantity of slide material further will cause the smaller hopper opening to have the greater maximum height of profile and the steeper slopes. These effects are more clearly depicted in Figure 16a. It should be noted that for the larger quantities of slide materials, the maximum height of profile does not occur at the centerline.

Effect of Hopper Height

Figure 16b provides two examples of constant volume slide material and width of opening, but variable height of drop. As expected, flatter profiles are experienced with the greater heights of drop.

Geometric Scaling

Two types of geometric scaling were attempted. In the first, the geometry of the hopper and the height of drop were scaled but particle size of the granular material was kept constant. Figure 17, 18, and 19 provide typical results for the normalized profiles. Excellent agreement was demonstrated in Figures 18 and 19, but the very small hopper opening of 0.25 feet appeared to introduce some model distortion in Figure 17. However, a comparison was then made between the normalized profile for the 0.25 feet hopper opening and a normalized profile for a geometrically similar test having a hopper opening of 0.5 feet which utilized a blast sand having a larger particle size. The results are shown in Figure 20. Obviously much better scaling resulted, and particle size is considered to be a factor in scaling.

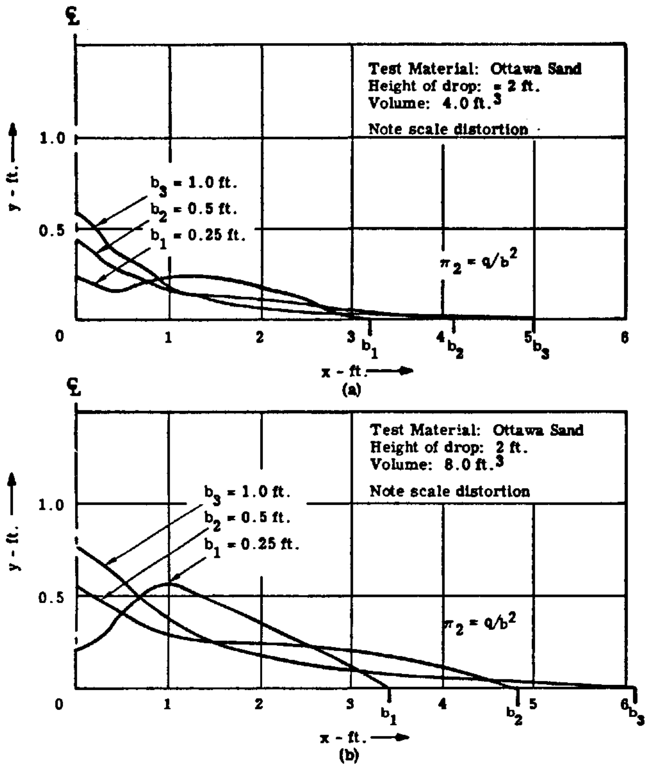
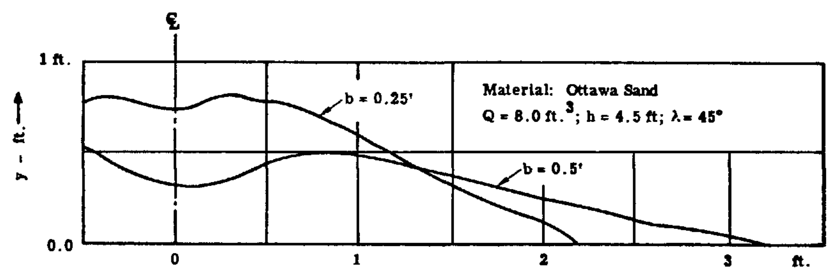
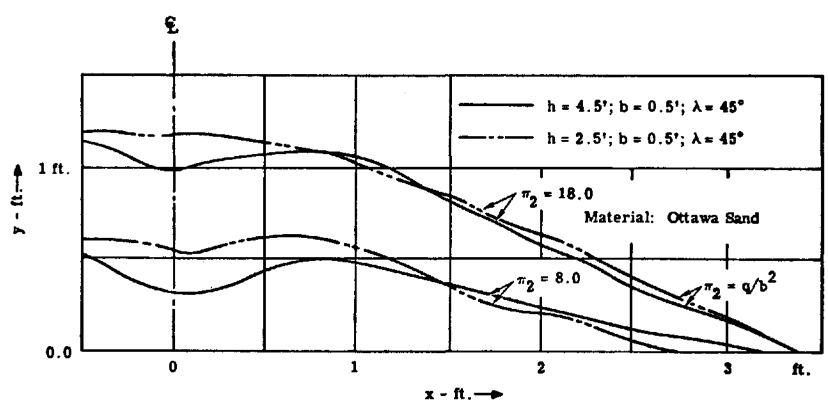


Figure 15.

Comparison of Half Profiles with Vertical Hopper, Width of Hopper Variable



(a) Half Profiles Showing Effect of Width of Hopper Opening



(b) Half Profiles Showing Effect of Hopper Height

Figure 16.

Compariaon of Test Profiles

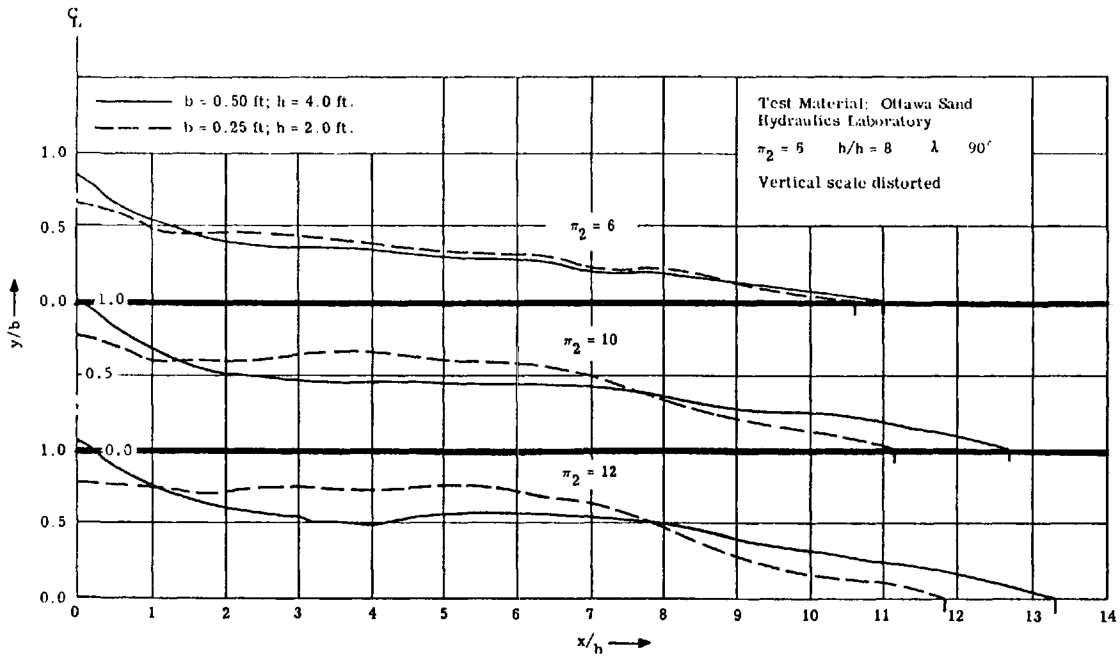


Figure 17. Comparison of Geometrically Scaled Profiles

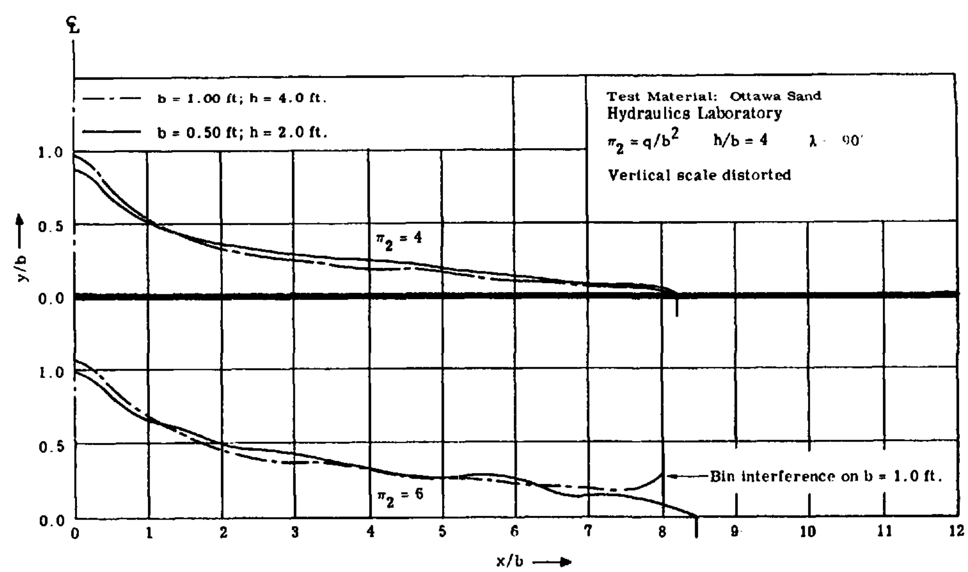


Figure 18. Comparison of Geometrically Scaled Profiles

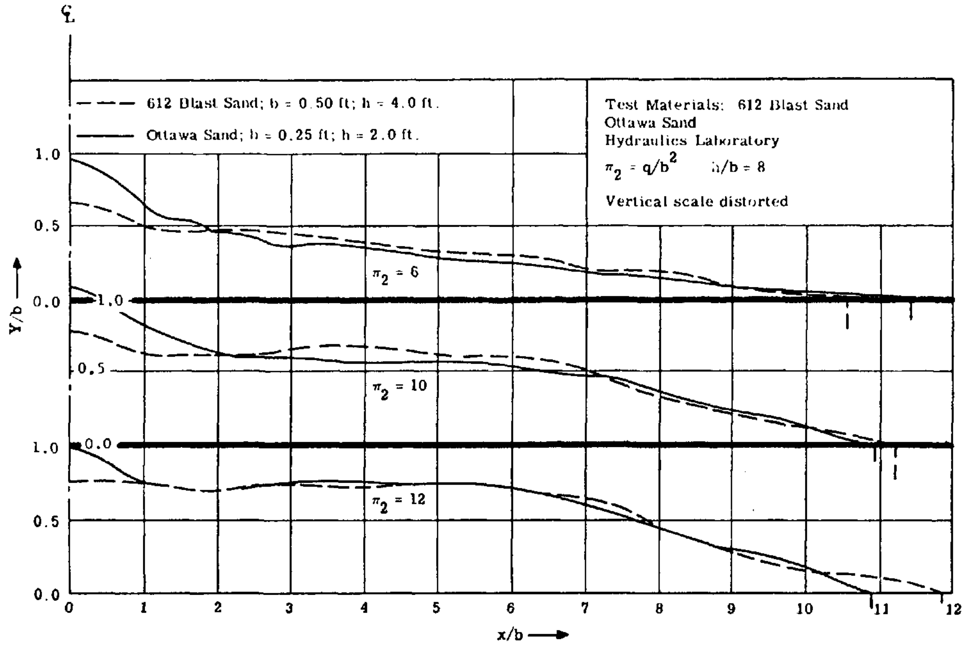


Figure 19. Normalized Half Profiles; Particle Size Scaled Geometrically

Three-Dimensional Motion Model Results

It is not possible to elaborate upon the three-dimensional motion model results, but it is important to indicate that the same general type of phenomena was observed with this model as discussed above for the plane motion studies. Figures 20 and 21 are provided to demonstrate that geometric scaling was also successfully achieved with this model.

SUMMARY AND RECOMMENDATIONS

In this paper it has been demonstrated that there are many important features of conventional dams that must be incorporated into the designs for slide dams. For example, foundation treatment and adequate diversion and spillway capacity must be provided. Also, adequate provisions must be made to control seepage and prevent piping. Important lessons can be learned from observing the characteristics of natural landslides. Slide dams created by man, however, must provide better results than have been observed in nature. Limited laboratory studies have helped define the general phenomena of the mechanics of slides and gives indications that the phenomena can be scaled. The information obtained is useful in planning more extensive laboratory experiments and small-scale field tests. In the field tests, it is assumed that the slides will be initiated with high explosives. Such experiments are now justified and would help develop a more complete understanding of slide dam mechanisms.

ACKNOWLEDGEMENT

This paper is based upon studies conducted by the author which were sponsored by the Sandia Corporation, Albuquerque, New Mexico. Mr. Luke Vortman served as technical adviser for the Sandia Corporation.

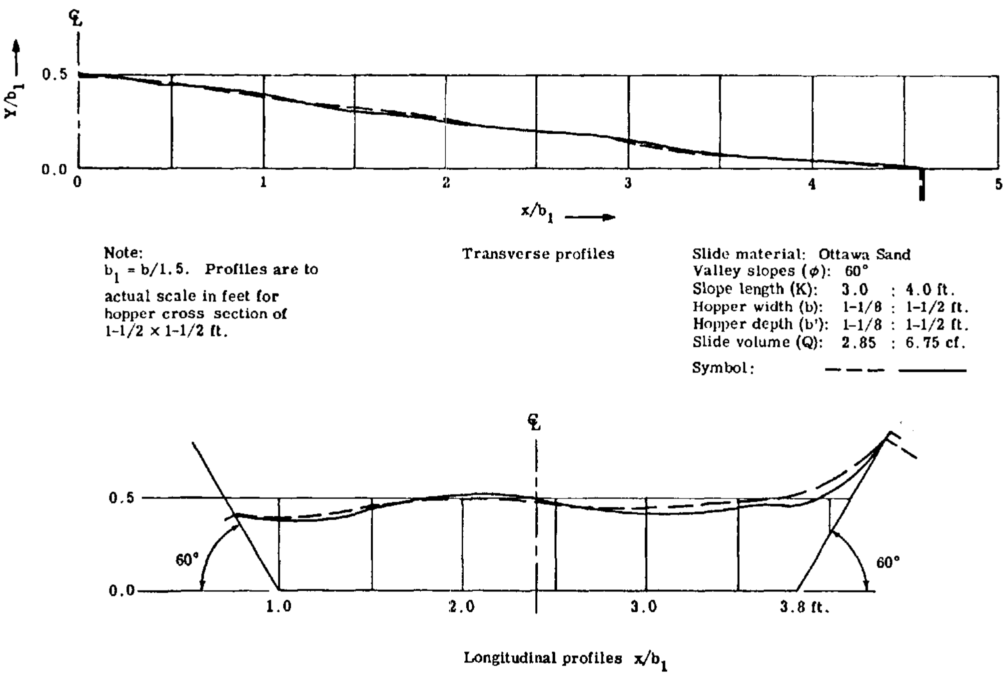


Figure 20. Comparison of Geometrically Scaled Profiles

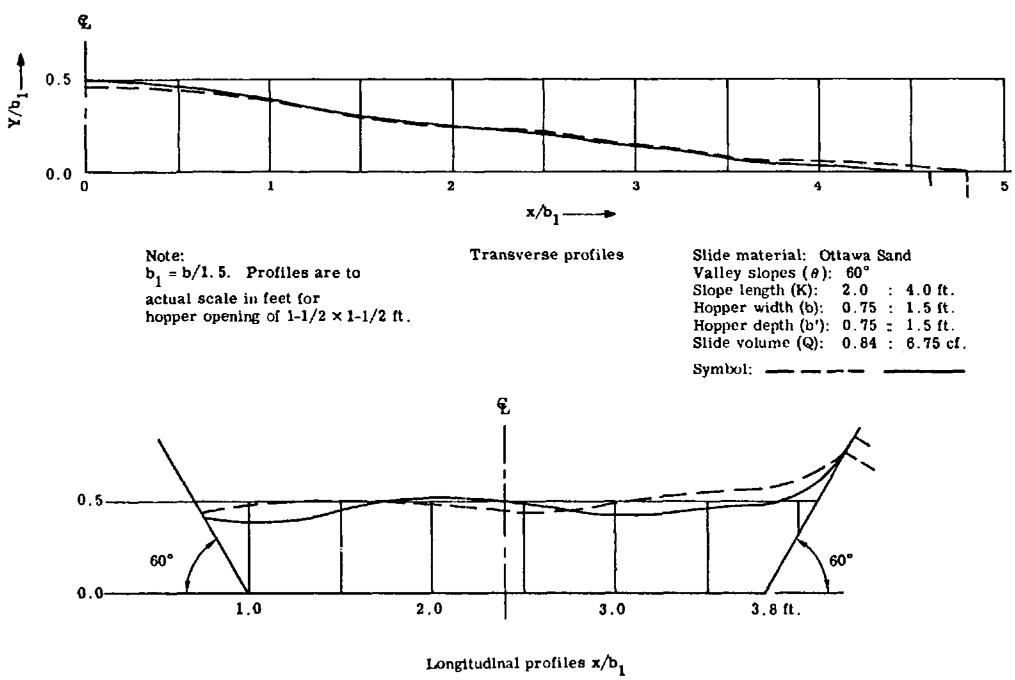


Figure 21. Comparison of Geometrically Scaled Profiles

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