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April 22, 1997

Ministry of Energy, Mines & Petroleum Resources,
Mineral Resources Division,
1810 Blanshard Street,
Victoria, British Columbia
V8A 1X4

Attention: Mr. George Headley, P.Eng.

Re: **Geotechnical Review, Drainage Aspects,**
Main Embankment Dam
Tailings Storage Facility,
Mt. Polley Project, B.C.

MP00018

Dear Sirs;

By way of follow-up to telephone discussions regarding the above last week with your Mr. George Headley, the writer is pleased to forward herewith copies of excerpts from four references as follows:

1. H.R., Cedergren. "Seepage, Drainage and Flow Nets". 1947

2. J.L. Sherard; R.J.Woodward; S.F. Gizienski; W.A. Clevenger. "Earth and Earth-Rock Dams". 1963

3. Robert B. Jansen (Editor). "Advanced Dam Engineering for Design, Construction and Rehabilitation". 1988

4. K.Terzaghi and R.B. Peck. "Soil Mechanics in Engineering Practice". 1948

5. H.D. Plewes and T.McDonald. "Investigation of Chemical Clogging of Drains at Inco's Central Area Tailings Dams".

The first four excerpts are from this writer's working file on the subject review and, as you know, were copied and provided to Mr. Ken Brouwer, P.Eng. of Knight Piésold Ltd. at his request. The fifth paper deals with some experiences at Inco's Tailings Area in Sudbury, Ontario with which this writer has been associated for many years in a consulting capacity and currently as Member of a Peer Review Board. This paper may be of interest for your technical data files.

GR 1A 2692

The writer trusts that the enclosed is in sufficient detail for your purposes and would ask that you call in the event that you wish to discuss any aspect.

As indicated below, a copy is also being forwarded to Mr. Brian Kynoch, P.Eng., by way of keeping him informed.

Yours very truly,

MAJM Corporation Ltd.

M.A.J. Matich

M.A.J. (Fred) Matich, FEIC, FCAE, P.Eng.(Ont.)

Copy with attachments to:

- Mr. Brian Kynoch, P.Eng.,
  Imperial Metals Corporation

Copy to:

- Mr. Ken Brouwer, P.Eng.
  Knight Piésold Ltd.

From

SEEPAGE, DRAINAGE AND FLOW NETS

by

Harry R. Cedergren

(John Wiley & Sons, Inc. 1947)

MAJM

b[illegible]andled and placed with care to avoid contamination and segrega-ti[illegible] (Sec. 5.8). Also, they must be well compacted to reduce the possibilities of localized changes in grading taking place by the dropping of fines through void spaces. Close control is required in the production, handling, and placement of the materials, since even a single improperly constructed portion of a filter can lead to failure.

Many of the problems associated with the design of adequate filters and drains stems from the needs for satisfying two conflicting requirements.

1. Piping requirement. The pore spaces in drains and filters that are in contact with erodible soils and rocks must be *small* enough to prevent particles from being washed in or through them.

2. Permeability requirement. The pore spaces in drains and filters must be *large* enough to impart sufficient permeability to permit seepage to escape freely and thus provide a high degree of control over seepage forces and hydrostatic pressures.

When small quantities of seepage are to be removed, a single layer of well-graded, moderately permeable material meeting both of these requirements may serve the dual roles of filter and drain. But when large quantities of seepage are to be removed, a fine filter layer usually is needed for the prevention of piping, and a coarse layer is needed for the removal of water. Such systems are called *graded filters*. They may contain *more* than two layers. Filters that are covered with surcharges to prevent uplifting by seepage forces are called *loaded* filters or *weighted* filters.

5.2 PREVENTION OF PIPING

General

To prevent piping, water-bearing erodible soils and rocks must never be in direct contact with passageways larger than some of the coarsest soil or rock particles. In nature piping failures often are exhibited by sink holes that form in arid and semiarid lands when fine sand, silt, loess, clay, etc., wash into subterranean tubes or cracks. Parker (1963) points out that piping is an important geomorphic agent in the development of landforms in the drylands.

Many engineering works produce large hydraulic gradients that are conducive to piping. When sewers are constructed below the water table in erodible sand or silt, joints must be meticulously sealed, otherwise serious infiltration is likely to occur. When cracks developed in a deep trunk sewer in Seattle, Washington, a tremend[illegible]s cavity formed in the soil above it, causing a major repair prob[illegible]. Piping is a common cause of failure in overflow weirs, earth dams, reservoirs, and other hydraulic structures (Chap. 1). Whenever filters and drains are required for the contrtol of seepage and groundwater in relation to structures, they should have a high degree of resistance to piping.

Grading of Drainage Aggregates to Control Piping

To prevent the movement of erodible soils and rocks into or through filters, the pore spaces between the filter particles should be small enough to hold some of the larger particles of the protected materials in place. Taylor (1948) shows that if three perfect spheres have diameters greater than six and one-half times the diameter of a smaller sphere (Fig. 5.1*a*), the smaller spheres can move through the larger. Soils and aggregates are always composed of *ranges* of particle sizes, and if the pore spaces in filters are small enough to hold the 85% size (D_{85}) of adjacent soils in place, the finer soil particles will also be held in place (Fig. 5.1*b*). Exceptions are gap-graded soil and soil-rock mixtures (Sec. 5.3).

Bertram (1940), with the advice of Terzaghi and Casagrande, made laboratory investigations at the Graduate School of Engineering, Harvard University, to test filter criteria that had been suggested by Terzaghi; he established the validity of the following criteria for filter design:

$$\frac{D_{15}(\text{of filter})}{D_{85}(\text{of soil})} < 4 \text{ to } 5 < \frac{D_{15}(\text{of filter})}{D_{15}(\text{of soil})} \tag{5.1}$$

The left half of Eq. 5.1 may be stated as follows.

Criterion 1. The 15% size (D_{15}) of a filter material must be not more than four or five times the 85% size (D_{85}) of a protected soil. The ratio of D_{15} of a filter to D_{85} of a soil is called the *piping ratio*.

The right half of Eq. 5.1 may be stated as follows.

Criterion 2. The 15% size (D_{15}) of a filter material should be at least four or five times the 15% size (D_{15}) of a protected soil.

The intent of criterion 2 is to guarantee sufficient permeability to prevent the buildup of large seepage forces and hydrostatic pressures in filters and drains. This criterion is discussed in detail in Sec. 5.4.

The work of Bertram was expanded by further experiments by the U. S. Army Corps of Engineers (1941) and the U. S. Bureau of Reclamation (Karpoff, 1955) and others. Frequently some requirements in addition to criteria 1 and 2 are placed on the grading of muddy water entered the drain pipes, topsoil penetrated the filter layer only a small fraction of an inch.

In 1963 the author supervised experiments in which a two-inch layer of screenings was placed over a layer of silt. With the screenings filled with water the surface was compressed many times with a kneading compactor. These tests, which were intended to simulate the action of concentrated highway traffic on saturated subgrades, indicated that when criterion 1 was satisfied, negligible intrusion occurred at the boundary between the soil and the screenings, but when the piping ratio was much above 4 or 5, substantial intrusion took place under the kneading action.

Experience indicates that if the basic filter criteria described in preceding paragraphs are satisfied in every part of a filter, piping cannot occur under even extremely severe conditions. Bertram's original investigations indicated that the grain sizes of uniform filter materials may be up to ten times those of uniform soils before appreciable amounts of soil will move through filters and that Eq. 5.1 usually is conservative. If a protected soil is a plastic clay, the *piping ratio* often can be much higher than 5 or 10, as indicated by U. S. Army Corps of Engineers practice previously noted. But if cohesionless silts, fine sands, or similar soils are in direct contact with filter materials which have piping ratios much above 5 or 10, erosion is very likely to occur. In 1940 the author witnessed earth dam construction with loess soil being compacted adjacent to a drain composed of 6-inch diameter boulders having a piping ratio relative to the loess of around 2000! During the first filling of the reservoir the drain caused serious internal erosion. Eventually the drain was pumped full of cement grout to save the dam.

When coarse rock, coarse gravel, or other coarse materials are used in drains, erodible soils and rocks should be separated from these materials by two or more intervening filters as required, with each adjacent pair designed to prevent piping. Mechanical analysis plots such as are shown in Figs. 5.2 to 5.6 offer a good visual picture of the grain size distributions of individual soils and filter materials, and are useful in developing filter designs.

Although filter criteria are almost foolproof, experience and judgment will reduce the danger of mistakes being made in their application. Several examples of "normal" designs of filters to prevent piping are given in Section 5.3. Precautions that must be taken in designing filters to protect gap-graded soil-rock mixtures and the dangers of severe segregation in filters are described in the last part of Section 5.3. In these examples, the primary control is assumed to be criterion 1, the ratio of the D_{15} size of the filter to the D_{85} size of the protected soils.

Pipe Joints, Holes and Slots

When pipes are embedded in filters and drains, no unplugged ends should be allowed, and the filter materials in contact with pipes must be coarse enough not to enter joints, holes, or slots. The U. S. Army Corps of Engineers (1955a) uses the following criteria for gradation of filter materials in relation to slots and holes:

For slots

$$\frac{85\% \text{ size of filter material}}{\text{slot width}} > 1.2 \tag{5.4}$$

For circular holes

$$\frac{85\% \text{ size of filter material}}{\text{hole diameter}} > 1.0 \tag{5.5}$$

The U. S. Bureau of Reclamation (1965) uses the following criterion for grain size of filter materials in relation to openings in pipes:

$$\frac{D_{85} \text{ of the filter nearest the pipe}}{\text{maximum opening of pipe drain}} = 2 \text{ or more} \tag{5.6}$$

Equations 5.4, 5.5, and 5.6 represent a reasonable range over which satisfactory performance can be expected. The design of a filter containing a drain pipe is illustrated by an example in Sec. 5.3.

5.3 EXAMPLES OF FILTER DESIGNS TO PREVENT PIPING

Historical

Before the development of rational and experimental filter design criteria, drain design was considered more of an art than a science. Designers depended on judgment, instinct, or precedent. In many instances coarse stone or gravel was placed in direct contact with fine-grained soils, with the result that drains often became clogged or soil piped through causing structural failures. Such was the case with *French drains* and *macadam* rock bases used in highway construction after about 1800 A.D. But some of the early road builders wisely placed fine gravel or screenings between fine soils and coarse stone bases and drains; and some of the early dam builders used several layers of stone grading from finer material in contact with the soil to coarser rock or gravel at the centers of drains. Creager, et al. (1950) describe

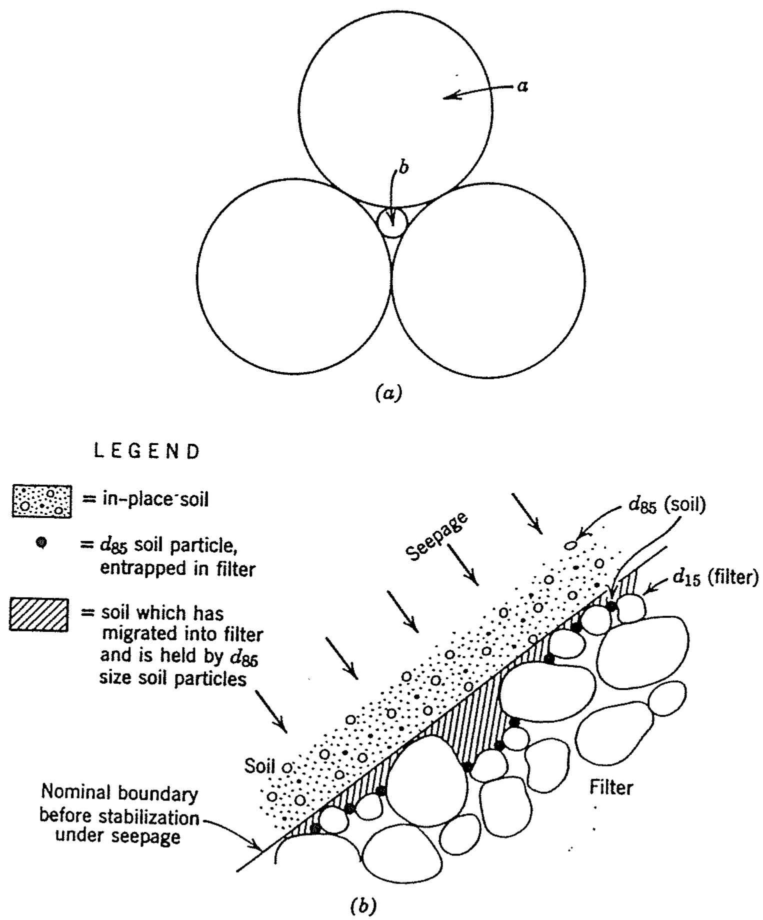


FIG. 5.1 Illustration of prevention of piping by filters. (*a*) Spherical particle *b* will just pass through pore space between three spheres six and one-half times the diameter of *b* (Taylor, 1948). (*b*) Conditions at a boundary between a soil and a protective filter.

filter aggregates. For example, the U. S. Bureau of Reclamation limits the maximum size of filter aggregates to 3 in. to minimize segregation and bridging of large particles during placement. To prevent the movement of soil particles into or through filters, the U. S. Army Corps of Engineers (1955) requires that the following conditions be satisfied:

$$\frac{15\%\text{ size of filter material}}{85\%\text{ size of protected soil}} \leqq 5 \qquad (5.2)$$

and

$$\frac{50\%\text{ size of filter material}}{50\%\text{ size of protected soil}} \leqq 25 \qquad (5.3)$$

It is seen that Eq. 5.2 is another expression for the relationship given by the left half of Eq. 5.1.

The U. S. Army Corps of Engineers (1955) also states.

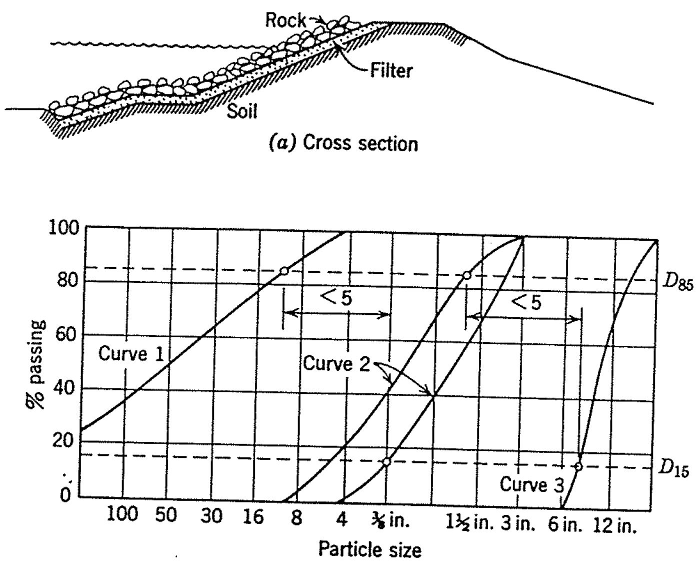
> The above criteria will be used when protecting all soils except for medium to highly plastic clays without sand or silt partings, which by the above criteria may require multiple-stage filters. For these clay soils, the D_{15} size of the filter may be as great as 0.4 mm. and the above D_{50} criteria will be disregarded. This relaxation in criteria for protecting medium to highly plastic clays will allow the use of a one-stage filter material; however, the filter must be well graded, and to insure nonsegregation of the filter material, a coefficient of uniformity (ratio of D_{60} to D_{10}) of not greater than 20 will be required.

The U. S. Army Corps of Engineers recommends limiting the piping ratio (D_{15} of filter to D_{85} of soil) to something less than 5 if crushed stone is used for the filter material. The safe ratio is usually checked on important projects by performing laboratory tests with materials to be used in the work. The Corps and the U. S. Bureau of Reclamation also recommend that the grain-size curves of filters and protected layers be somewhat parallel to each other. This is the objective of the relationship expressed by Eq. 5.3.

Sherard, et al. (1963) make the following additional rule for the design of filters. "When the protected soil contains a large percentage of gravels, the filter should be designed on the basis of the gradation curve of the portion of the material which is finer than the 1-inch sieve."

Many other experimenters in addition to Bertram, the U. S. Army Corps of Engineers, and the U. S. Bureau of Reclamation have satisfied themselves that criterion 1 will prevent piping. In 1940 the author conducted a series of experiments in which soils were mixed with water, and slurries were poured over filter materials meeting criterion 1. Under these extremely severe conditions a small amount of clay and colloids washed through but nearly all of the material stayed on top of the filters.

In the construction of a military air base in the Pacific Northwest in 1942, an unexpected storm washed topsoil into partially completed trench drains along the edges of the runways. Fortunately a filter layer meeting criterion 1 had been placed in the trenches. Although



(a) Cross section

(b) Grading curves

FIG. 5.2 Rock slope protection design to prevent undermining. Intermediate filter or cushion course (curve 2) prevents soil (curve 1) from washing through coarse rock (curve 3). Care must be taken to prevent segregation of intermediate course.

the Tabeaud Dam in California which was constructed in 1902 with a rock drain having two progressively coarser filter zones between the soil and the drain rock.

With the development of rational and experimental filter criteria, the design of filters and drains has become more of a science than an art. Several examples of the application of filter criteria to the design of filters to prevent piping are given in the following paragraphs.

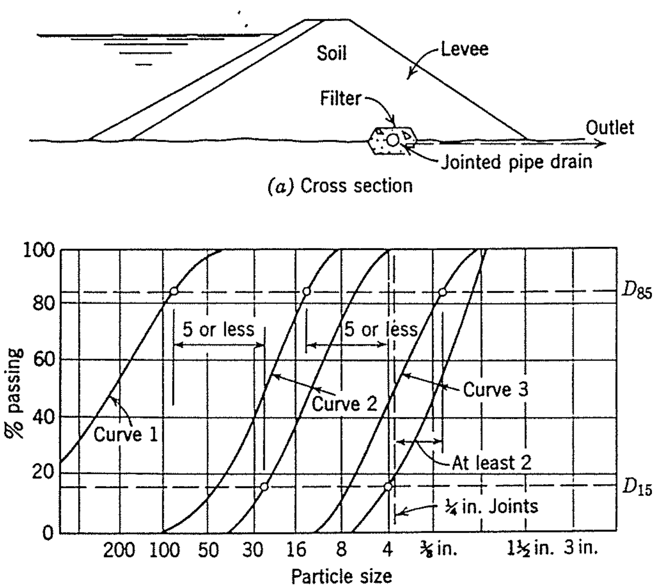
Rock Slope Protection

Frequently coarse rock is placed on the banks of levees, on the upstream faces of earth dams, and in other situations where erodible soils must be protected from fast currents and wave action. If coarse rock is placed directly on fine soil, currents and waves may wash the soil out from under the rock and lead to undermining and failure of expensive protective works, even to failure of the works being protected.

Soil erosion under rock slope protection usually can be prevented by the placement of a filter layer of intermediate-sized material between the soil and the rock. Sometimes erosion can be prevented by the use of well-graded rock containing suitable fines which work to the bottom during placement. A typical rock slope protection with an underlying filter is shown in Fig. 5.2. The slope protection rock (curve no. 3) has a particle size range from 6 in. to 24 in., and a 15% size of about 7 in. The filter layer (curve 2) has a minimum 85% size of about 1.4 in. The piping ratio (criterion 1) = 7 in./1.4 in. = 5. In turn, the soil has an 85% size between sieves 8 and 16. The fine filter has a maximum 15% size of $\frac{3}{8}$ in., which is less than five times the 85% size of the soil; hence, according to criterion 1 the soil will not erode through the filter, and the filter is safe from washing out through the rock.

Levee Drain with Perforated or Jointed Pipe

Figure 5.3 shows a typical longitudinal levee drain composed of two grades of filter aggregate surrounding a perforated or jointed



(a) Cross section

(b) Grading curves

FIG. 5.3 Design of graded filter to protect soil (curve 1) from washing into $\frac{1}{4}$ in. joints in drain pipe [fine material (curve 2) adjacent to soil; coarse material (curve 3) surrounds pipe].

are dictated by seepage considerations. The more common methods for controlling seepage in earth dams and levees and in their foundations are described in subsequent sections, which give examples that not only point up the advantages and weaknesses of various methods but also illustrate the use of analytical methods in the design of safe earth dams and levees.

Not only must dams and levees be *designed* for safety, they must be *constructed* with safety in mind since unsafe construction practices can lead to failures. Adequate stripping of unsatisfactory materials from foundations and abutments is essential to the security of earth dams and levees. Impervious cores should be in intimate contact with nonerodible, relatively watertight soil or rock formations. Loose materials should be removed from all exposed fissures or joints and such areas backfilled with "dental" concrete or slush grout that is protected from damage and sealed with an impervious membrane curing compound. Pneumatically applied mortar (gunnite) often is used for the protection of impervious cores against piping along contacts with jointed, fissured, sheared, and erodible rocks.

Impervious fill that is in direct contact with rough or uneven surfaces should be placed slightly wet of optimum to assure its molding to the shape of the surface. Care must be taken to avoid the use of excessively wet impervious fill in the bottoms of cutoff trenches or other locations where seepage or groundwater may be encountered. Compacting impervious fill very densely at the *optimum* moisture content produces material with greatest resistance to piping. Conversely, the placement of impervious fill at high or low water contents or with very little compactive effort produces material having little resistance to piping. In the construction of impervious cutoffs into pervious water-bearing soils, excavations often become badly flooded unless adequate dewatering measures are employed to remove the incoming seepage (Sec. 7.1). Sometimes in an effort to reduce foundation dewatering costs impervious backfill for cutoffs is bulldozed into the water until sufficient thickness has been placed to support construction equipment. Then the surface is rolled lightly and additional fill placed in layers and *thoroughly compacted.* This practice is an invitation to trouble because well-compacted fill tends to arch over cavities that form in loose underlying material because of piping or reduction in volume produced by saturation. When cavities or pipes erode to the reservoir side, a sudden rush of water may cause rapid failure of the structure. The author examined one earth dam which washed out in 1965 because of this condition and another which leaked so badly that it could not serve its intended purpose of storing water.

If it occurs at one point only, the efficiency is considerably greater than if it is distributed among several openings. Thus a cutoff with 5% of open area at one point reduces the seepage by 60%, whereas a cutoff with the same amount of open space equally divided among 8 openings is less than 20% efficient.

Impermeable Upstream Blankets

Levee systems often extend many miles along one or both sides of rivers, frequently on foundations having natural impermeable covers over pervious sands and gravels. Often the impermeable cover is thin or substantially missing on the river side as shown in Fig. 6.5*a*. Under these conditions levees may successfully withstand a number of low flood stages, only to fail suddenly when a large flood comes along. Since extensive blankets or other effective seepage control measures can be very costly, designers often prefer to construct levee systems with minimum seepage control initially and determine the need for additional measures by observation of the behavior at moderate river stages. When this can be done safely, considerable savings in cost may be possible; however, such levees may be potentially unsafe and may fail if subjected to a high river stage before the necessary seepage control measures are constructed.

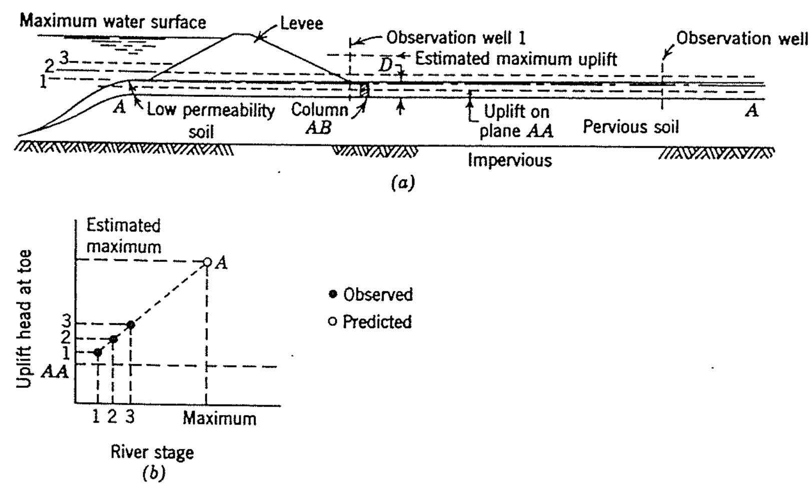


FIG. 6.5 Levee with dangerous uplift condition at landward toe. (*a*) Cross section. (1), (2), (3) are low and intermediate levels at which readings have been taken in observation wells. (*b*) River stage versus head at toe.

Observation wells or piezometers installed at the landward toe and at one or more additional points, as shown in Fig. 6.5*a*, permit uplift pressures to be measured and the degree of security evaluated. If readings can be obtained at several low to moderate river stages, such as 1, 2, and 3 in Fig. 6.5*a*, a plot of "river stage" versus "uplift head" at the landward toe can be plotted as shown in Fig. 6.5*b*, and the uplift at maximum river stage (point A) can be estimated by extrapolation. The degree of security against blowouts and sand boils depends on the magnitude of the maximum *uplift gradient* (Sec. 3.4) at the landward side of the levee. The factor of safety G_s against uplift failures can be calculated as the ratio of the downward forces acting on column of soil AB to the upward forces. Assuming that column AB has an area of 1 sq ft and a height D, the downward force due to its submerged weight is $(D)(62.5 \text{ lb/cu ft})(G-1)/(1+e)$, and the upward seepage force is (D) (62.5 lb/cu ft) (i). The factor of safety is therefore

$$G_s = \frac{(D)(62.5)(G-1)/(1+e)}{(D)(62.5)i} = \frac{(G-1)/(1+e)}{i}$$

Since the hydraulic gradient i is equal to the uplift head h_t on plane AA divided by the height D of column AB,

$$G_s = \frac{(D)(G-1)/(1+e)}{h_t}$$

And the maximum safe uplift head at the toe is

$$h_t = \frac{(D)(G-1)/(1+e)}{G_s}$$

To be reasonably secure against blowouts and boils, the factor of safety in the above expression should be at least 2 to 2.5. If the predicted uplift h_t is greater than a safe value, control measures should be provided. Turnbull and Mansur (1961) in describing underseepage under the Mississippi River levees conclude that ". . . it appears that heavy seepage and sand boils should be anticipated whenever estimated upward gradients exceed 0.5 to 0.8, depending on site conditions." Seepage conditions as related to upward gradients through the top stratum as measured by piezometers during the 1950 high water are summarized in Table 6.4.

For soil conditions similar to those shown in Fig. 6.5 impermeable upstream blankets often are used for controlling seepage. Their effectiveness depends on (1) securing blanket materials that are relatively low in permeability, and (2) completely covering the permeable strata exposed to water pressure. If both of these requirements cannot be met, control may be very ineffective, and it may be necessary to resort to other methods, such as cutoffs, relief wells, landside berms, or sublevees.

Figure 6.6 illustrates the need for completeness of blankets. The cross section in Fig. 6.6*a* shows a levee on an impermeable soil layer underlaid by considerably more permeable formations which are exposed to the hydrostatic head of a river. The exposed width L is

TABLE 6.4 Seepage Conditions and Measured upward Gradients, Mississippi River Levees.

Seepage conditions	i
Light to none	0 to 0.5
Medium seepage	0.2 to 0.6
Heavy seepage	0.4 to 0.7
Sand boils	0.5 to 0.8

(After Turnbull and Mansur, 1961)

partially covered with an impermeable blanket, as in Fig. 6.6*b*, except for a width x which is not covered. A typical flow net is given in Fig. 6.6*b* with 90% of the distance L blanketed ($x/L = 0.10$). The hydrostatic uplift at the bottom of the impermeable layer as determined from this flow net is shown as a dashed line, and the uplift head h_t at the toe is 0.54 h. The curves in Figs. 6.6*c* and 6.6*d* give the relationships between the ratio x/L and the relative seepage quantity and h_t/h. These curves show that to be highly effective the blanket must be substantially 100% complete.

Thin Sloping Membranes

A fundamental principle in the design of any dam or levee is that the energy of the water pressing against the structure must be safely consumed. A theoretically ideal design is furnished by a thin, highly impervious sloping membrane, such as steel or reinforced concrete, on the upstream face of a rock or gravel embankment (Fig. 6.7). The resultant hydrostatic pressure P (Fig. 6.7*a*) presses downward into the foundation, increasing frictional resistance to sliding on the base.

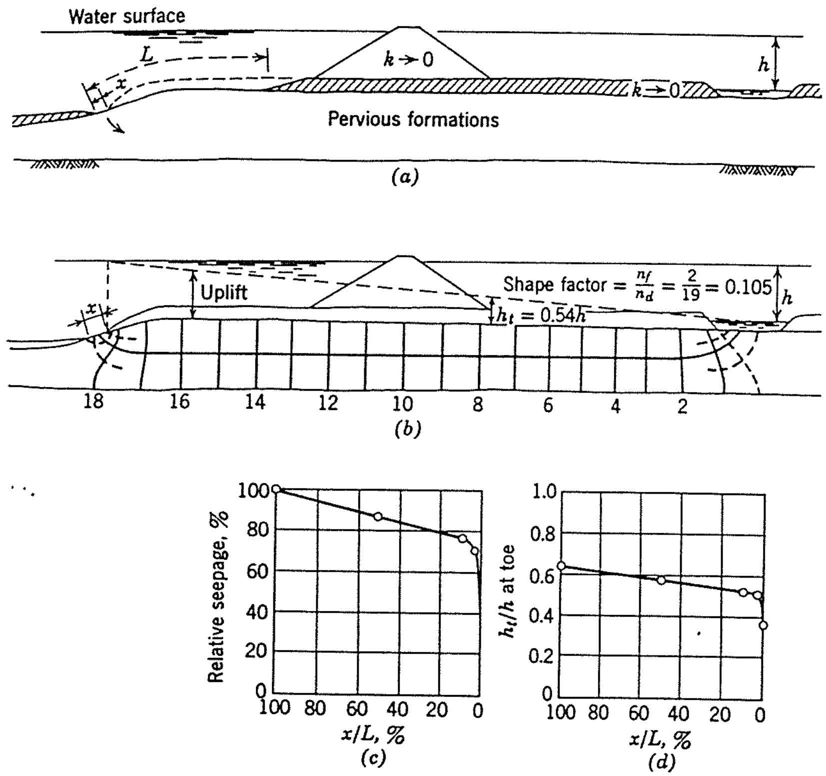


FIG. 6.6 Study of blankets for levees that shows the need for thorough blanketing. (*a*) Cross section of levee and foundation. (*b*) Typical flow net with incomplete blanket ($x/L = 0.10$). (*c*) Relative seepage quantity. (*d*) Uplift at toe (for simplification horizontal and vertical permeabilities are assumed equal).

Thin, impervious membranes are susceptible to cracking due to differential settlements caused by high pressures (Fig. 6.7*b*) and are relatively costly. Tight cutoffs and well-anchored surface slabs are essential to the safety of membrane dams (Fig. 6.7*c*). At high altitudes where impermeable soils usually are scarce, concrete-faced rockfill dams sometimes are built; but when ample supplies of impermeable earth are available near damsites, *zoned* earth dams almost always are more economical than any other kind.

The cases described represent seepage control by methods that depend on the principle of keeping the water out or reducing seepage quantities. Flow net studies have shown that these methods must have a high degree of perfection; otherwise they are not very effective.

Almost always such methods must be used in combination with drainage facilities. In the next paragraphs methods for draining dams and levees and their foundations are illustrated. The first to be described is the zoning of embankments to force seepage into safe patterns.

6.3 SEEPAGE CONTROL BY DRAINAGE METHODS

Embankment Zoning

Zoned earth dams have an internal impervious section called a *core*, which furnishes watertightness, and outer sections on both sides of the core called *shells*, which furnish strength. Depending on the availability of materials and personal preference, dam designers vary the location and thickness of impervious cores in zoned dams (Fig. 6.8). Some designers prefer extremely thin, sloping cores (Fig. 6.8*a*), sometimes called "Growdon" dams after J. P. Growdon, the originator of this type of dam. Others prefer a somewhat thicker, moderately sloping core, as shown in Fig. 6.8*b*; still others a thick, centrally located core, as illustrated in Fig. 6.8*c*.

Filters must always be used when required for the protection of cores. Thin, inclined cores dissipate hydrostatic energy quickly but are subjected to large hydraulic gradients which increase the dangers of piping if filter protection is not extremely good. Thin cores offer less protection than thick cores against offset displacements due to

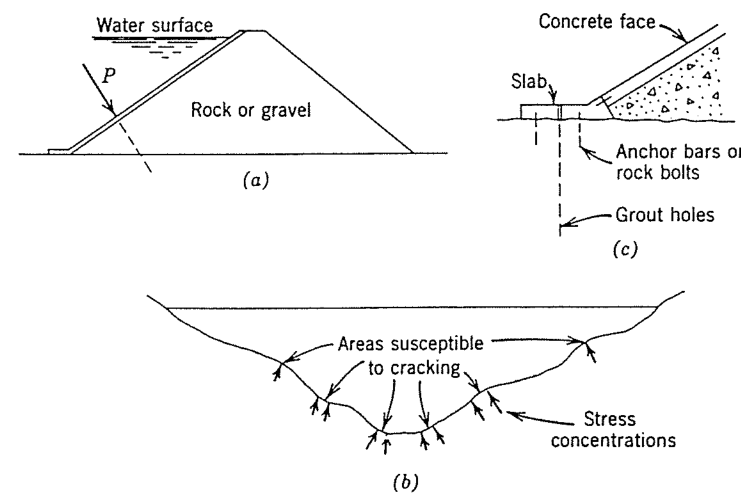


FIG. 6.7 Dam with thin impervious membrane. (*a*) Cross section. (*b*) Profile. (*c*) Toe detail.

tion. It can be installed by excavating into the landward toe of an existing levee or dam when the water level is low and the ground is well drained. These drains should be provided with perforated or jointed pipes connected to gravity outlets. They are best suited to levees or small dams on relatively homogeneous foundations or on pervious foundations covered by shallow topsoil layers which can be penetrated by the drainage trench. If this type of drain is to improve uplift conditions at the landward side of a levee or small dam, it must be in contact with water-bearing strata. The flow net in Fig. 6.18*b* shows potentially unstable conditions that existed before the installation of a toe drain. The flow net in Fig. 6.18*c* shows the seepage pattern after the installation of such a drain.

When foundation conditions are such that a toe drain would be separated from underlying pervious strata by impervious layers, only a portion of the seepage could be removed by the drain and only limited control obtained. In such cases, a toe drain can be combined with relief wells or replaced by a system of relief wells.

Relief Wells

When pervious strata beneath dams or levees are too deep to be penetrated by cutoffs or shallow drains, relief wells offer a means of relieving uplift pressures because they can penetrate the most pervious waterbearing strata in a foundation. Their discharge resistance must be small, and they should be spaced sufficiently close together to lower the adjacent water pressures to a safe level. Relief wells must be designed with screens or filters that prevent the loss of soil out through the wells, and they must be resistant to corrosion and to deterioration caused by bacteria. To assure permanent performance without movement of soil, filter criterion 1 (Sec. 5.2) must be satisfied.

An advantage of relief well systems is the ease with which they can be expanded if an initial installation should not furnish the needed control. An initial system can be installed, based on the best available knowledge of soil conditions, and if this system does not furnish the degree of control desired, additional wells can be added until an adequate system is obtained. This procedure is most suitable in situations where the level of the water surface behind a structure can be controlled. If there can be no control over the water level, an initial system should be the minimum judged adequate to prevent failures.

Middlebrooks and Jervis (1947) developed formulas for the design of fully penetrating relief well systems based on seepage theory and model studies. Turnbull and Mansur (1961) describe methods developed by the U. S. Army Corps of Engineers for designing partially penetrating wells.

Designing a relief well system requires determination of the most economical spacing and penetration of wells that will lower the uplift pressures to a safe level. The U. S. Army Corps of Engineers' method starts with an infinite line of wells, and the spacing is reduced where necessary to allow for the lower efficiency of a finite line of wells as compared with an infinite line.

An approximate method for estimating relief well spacings can be developed as follows (Fig. 6.19). An impervious embankment rests on a thin impervious stratum which lies over a pervious layer connected with a reservoir or river at the left. As shown in Fig. 6.19*a*, a row of relief wells at the toe discharges the steady seepage through outlets located a moderate distance below the ground surface.

The assumed seepage condition under the dam or levee is shown in Fig. 6.19*b*. In this analysis the seepage quantity Q_1 is the total underseepage in a strip having a length equal to the well spacing $2r$. The quantity Q_1 equals $2rkH(n_f/n_d)$, in which k is the coefficient of permeability of the soil, H is the net head on the structure, and the ratio (n_f/n_d) is the shape factor for the foundation. The quantity Q_2 (Fig. 6.19*c*) is the discharge that bypasses the drains. In the analysis that follows, Q_2 is assumed to be zero, since this assumption places the maximum load on the relief wells. Section *A-A* (Fig. 6.19*d*) shows the hydrostatic conditions assumed in the plane of the wells. A head difference h_m is assumed to exist between the edge of each well and the midpoints between wells.

Reasonable allowances should be made for head losses at the wells h_w; however they cannot be calculated, as they depend on factors such as unknown screen resistances and smear effects caused by the well drilling processes.

Well formulas may be used in developing solutions to this problem, assuming that flow to the wells occurs only in the half-circle upstream from the wells. The simple well formula is

$$Q = \frac{\pi k(h_2^2 - h_1^2)}{2.3 \log_{10} (r_2/r_1)} \tag{6.1}$$

This equation is for flow into a simple, fully penetrating well for a complete circle of 360°. Modified for 180°, Eq. 6.1 becomes

$$Q_3 = 0.685k \frac{(h_2^2 - h_1^2)}{\log_{10} (r_2/r_1)}$$

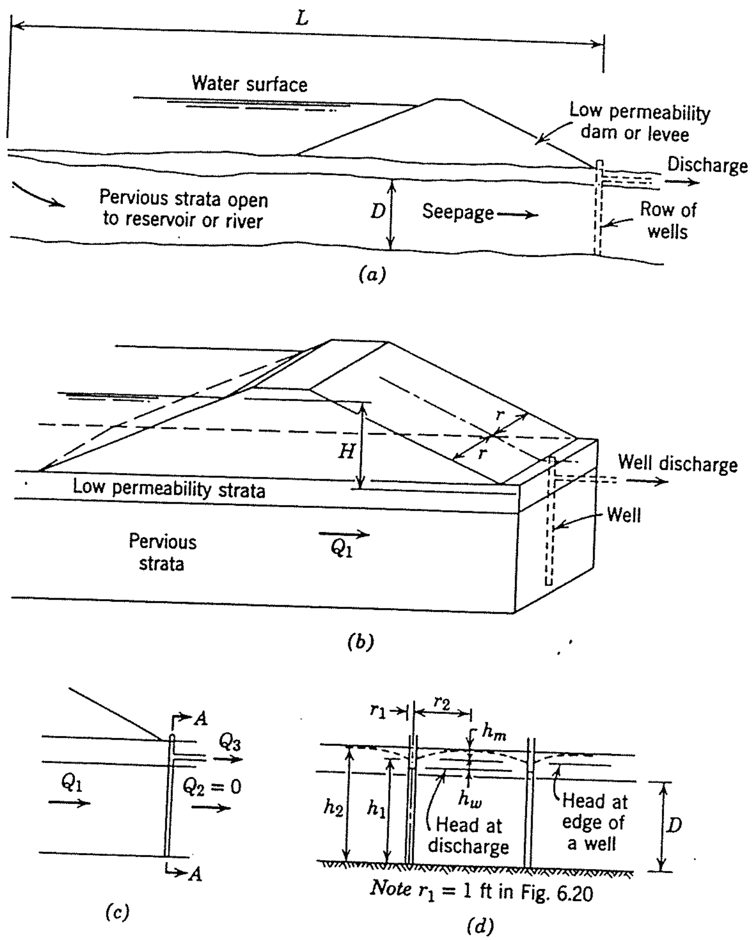


FIG. 6.19 Conditions of simplified relief well theory. (*a*) Cross section. (*b*) Seepage conditions. (*c*) Discharge assumption. (*d*) Section *A-A*.

If it is assumed that the drains intercept all of the seepage ($Q_3 = Q_1$),

$$0.685k \frac{(h_2{}^2 - h_1{}^2)}{\log_{10}(r_2/r_1)} = 2rHk\left(\frac{n_f}{n_d}\right)$$

Simplifying and making $r = r_2$,

$$h_1{}^2 = h_2{}^2 - 2.93H\left(\frac{n_f}{n_d}\right) r \log_{10}\left(\frac{r}{r_1}\right) \qquad (6.2)$$

The well discharge conditions shown in Fig. 6.19 may be more nearly approximated by artesian conditions than by flow to simple

wells. A formula for flow to artesian wells is

$$Q = \frac{2\pi kD(h_2 - h_1)}{2.3 \log_{10}(r_2/r_1)} \qquad (6.3)$$

For a half-circle Eq. 6.3 becomes

$$Q_3 = \frac{1.37kD(h_2 - h_1)}{\log_{10}(r_2/r_1)}$$

If $Q_3 = Q_1$, then

$$\frac{1.37kD(h_2 - h_1)}{\log_{10}(r_2/r_1)} = 2rkH\left(\frac{n_f}{n_d}\right)$$

Simplifying and making $r = r_2$, we have

$$h_2 - h_1 = 1.46\frac{H}{D}(n_f/n_d)r \log_{10}(r/r_1) \qquad (6.4)$$

But $h_2 - h_1 = h_m$; hence this equation gives values for h_m.

Typical solutions for Eqs. 6.2 and 6.4 are given in the charts of Fig. 6.20, which relate half the well spacing r to shape factors for underseepage and h_m.

In developing Fig. 6.20, twenty points on each of the three charts were calculated using the simple well formula and twenty with the artesian formula. Interestingly, no significant differences could be noted. It thus appears that the results are not sensitive to moderate differences in assumptions.

To reduce the number of variables in this sample solution, three charts are given for $H = 100$ ft. Figure 6.20*a* is for sections in which the thickness of the foundation D is equal to H; Fig. 6.20*b*, for $D = 0.5\ H$; and Fig. 6.20*c*, for $D = 0.2\ H$. These charts can be applied to dams and levees of other sizes by multiplying the dimensions in the charts by the ratio the *real H* bears to 100 ft. Thus if $H = 50$ ft, the dimensions in the charts should be multiplied by $\frac{50}{100}$ or 0.5.

The head producing flow under a dam or levee should be measured from the elevation of the water surface against the structure to the height of the median pressure head along the row of wells. If h_m is relatively small, the head can be measured to the elevation of the drain discharge or simply to the elevation of the ground at the downstream toe. Refinements in the measurement of the head can be made as desired.

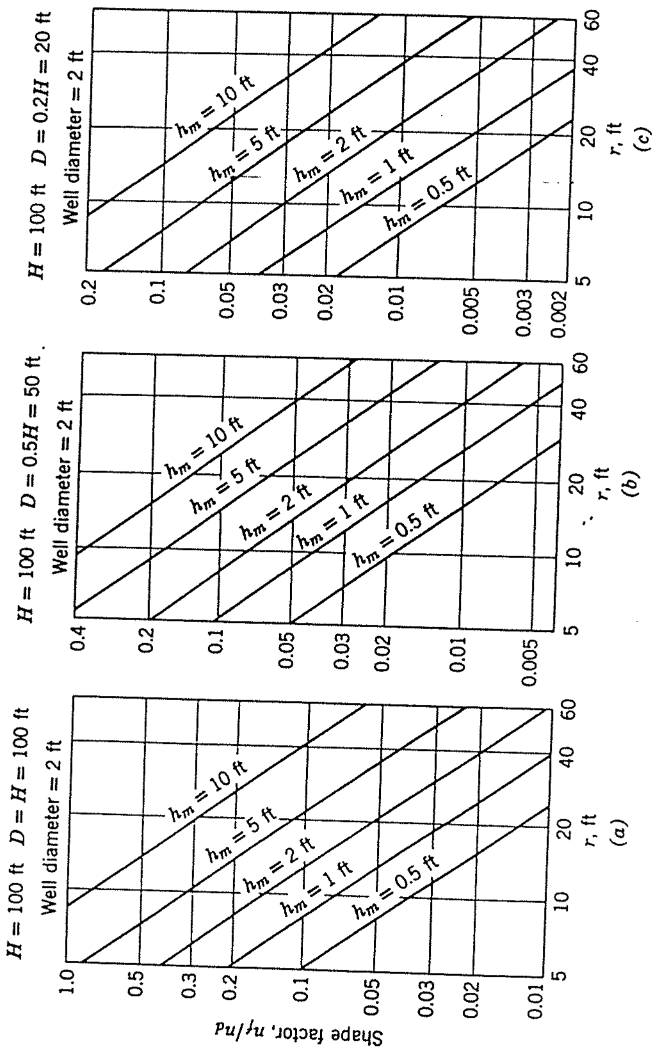


FIG. 6.20 Relief well design charts (approximate solutions). (Refer to Fig. 6.19.) (*a*) Relief well chart for $D = H$. (*b*) Relief well chart for $D = 0.5H$. (*c*) Relief well chart for $D = 0.2H$.

Foundation seepage shape factors n_f/n_d used in Fig. 6.20 should be determined from flow nets, or they can be roughly approximated by substituting the ratio $D/(L + 0.4D)$. (See Fig. 6.19*a*.)

This solution is somewhat approximate, since it does not represent a rigorous analysis. The charts in Fig. 6.20 are for wells having a diameter of 2 ft. Charts for other conditions can be readily developed using the methods described here. Other more accurate solutions have previously been noted.

6.4 EFFECTIVE DRAINAGE AND EARTHQUAKE RESISTANCE

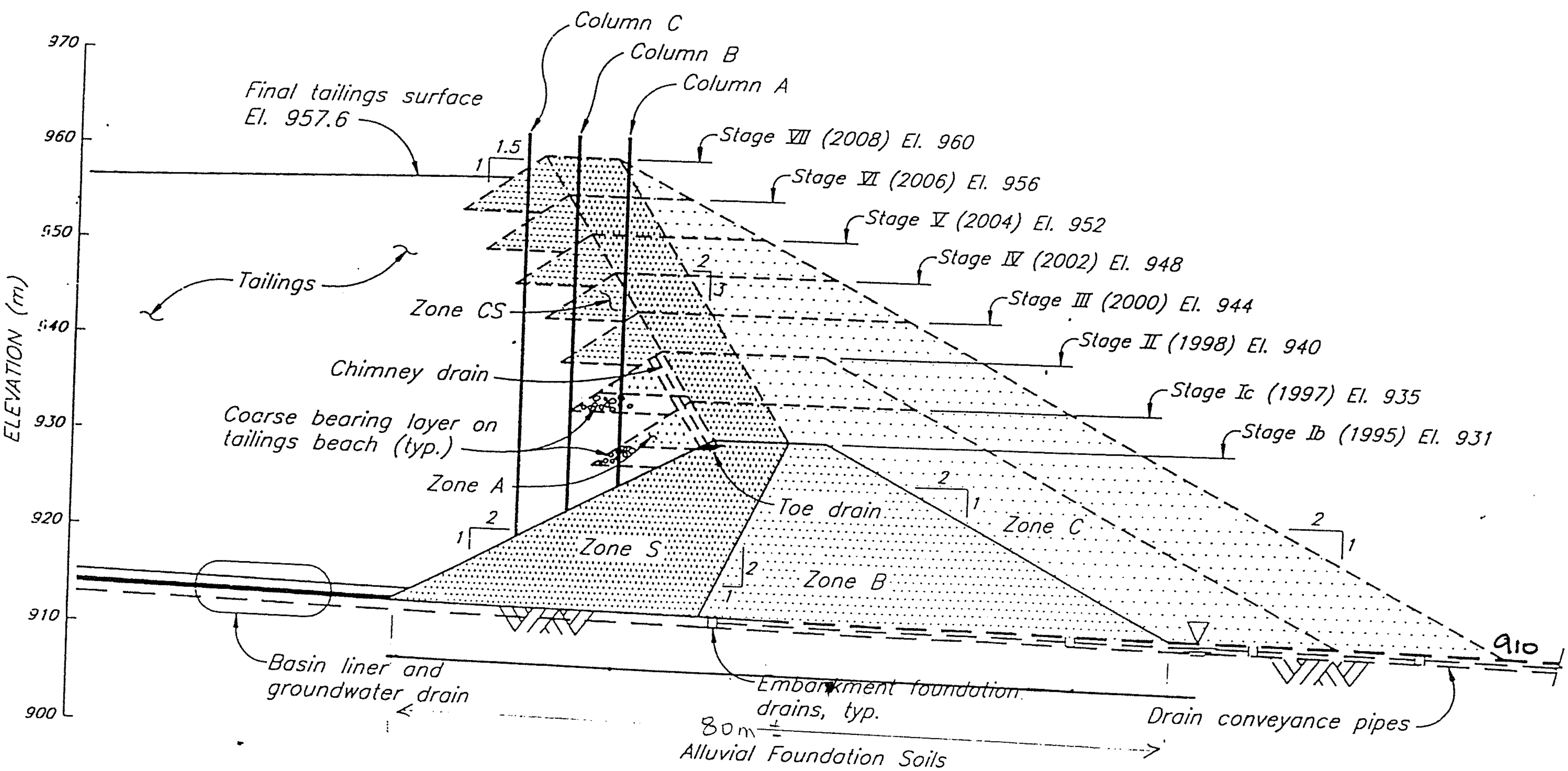
Basic Considerations

The massive landslides of recorded history nearly always occurred in saturated earth. Many landslides take place during or immediately following earthquakes (Sec. 8.1). Sherard et al. (1963b) report that the only dam known to have failed completely during an earthquake (Sheffield Dam, California) was an unzoned dam, loosely compacted, and with the foundation and lower part of the embankment saturated at the time the earthquake occurred.

The mechanics of failure of earth masses under earthquake shocks are discussed in Chapter 8. The conclusion is reached (Sec. 8.2) that good drainage is one of the most effective means of improving slope stability during earthquakes. Also strong, dense materials are considerably more resistant to damage by earthquakes than loose, weak materials.

At one time it was thought that if sands and gravels were well compacted they could not *liquefy* or otherwise be substantially weakened by earthquake shocks. Evidence now points to the probability that even rather well compacted gravels *may* liquefy under severe shocks if they are under the great pressures that sometimes may develop deep within such massive earth dams as are now being constructed and planned. The surest way to prevent liquefaction failures in dams of major proportions is to require thorough compaction and the best possible drainage.

If the saturation level is permitted to rise substantially in the downstream shells of major dams, all such saturated zones must be considered potentially susceptible to liquefaction under severe earthquakes. Unless watertight upstream membranes are used, it is impossible to keep water out of the upstream portions of zoned dams; however, the pressure of the water helps to counteract outward earthquake forces. High compaction, the use of strong permeable rock or gravel



1cm = 7m.

$$\frac{N_f}{N_d} = \frac{1}{18} = 0.05 \text{ say.}$$

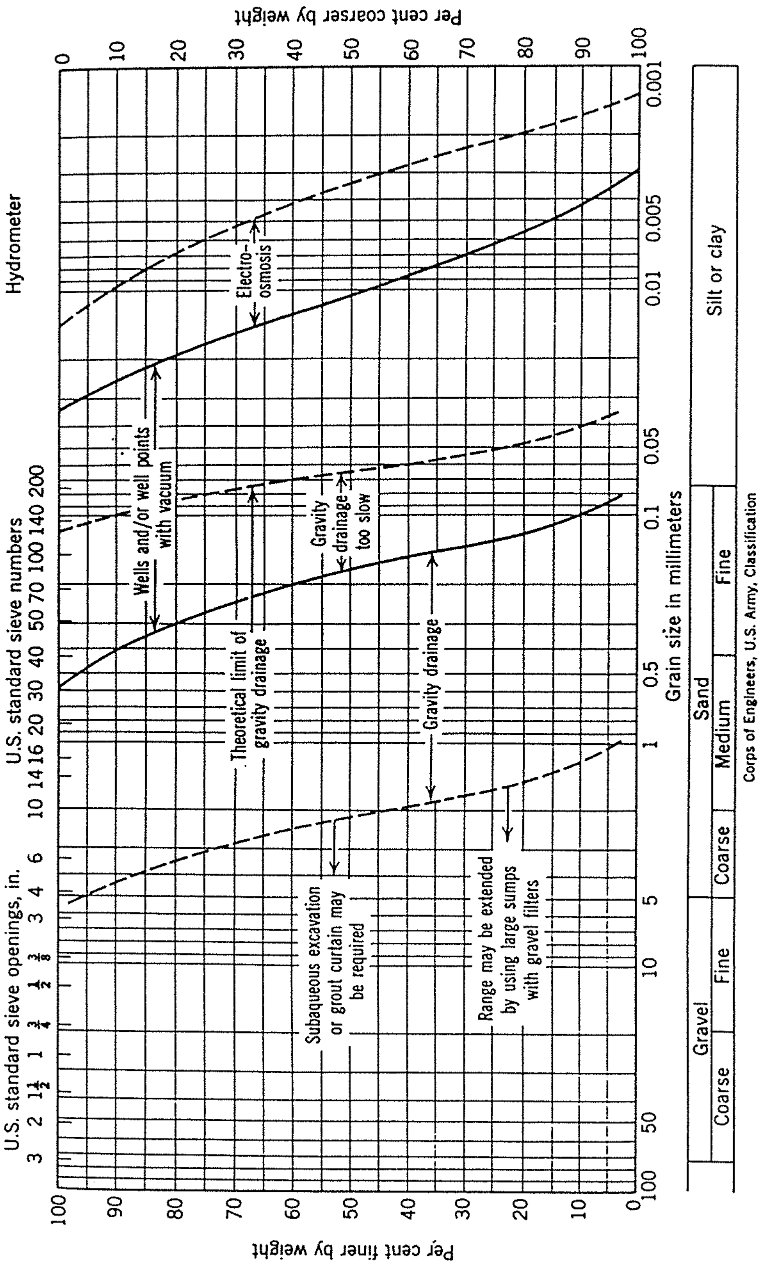


FIG. 7.5 Dewatering systems most suitable for various soil types (Moretrench Corp.): (Copied from *Foundation Engineering*, Leonards, McGraw-Hill Book Co., New York, 1962.)

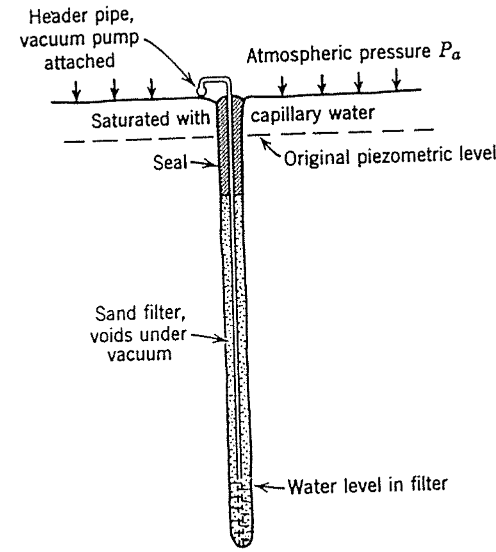


FIG. 7.6 Diagram illustrating the vacuum method of drainage. (Terzaghi and Peck, John Wiley and Sons, 1948.)

by the vacuum method remain completely filled with water; hence, reasonable precautions should be taken to avoid severe jarring of the soil structure. If the soil has a loose structure, it is possible that severe shocks could produce a collapse of the soil structure leading to liquefaction.

Electro-osmosis

Freundlich (1926) developed the double layer theory of soil water, which offers an explanation for the phenomena of electro-osmosis. According to this theory, the water near the soil particles is made up of two layers. One layer is assumed to be bonded to the particles; the other is free moisture. The layer adjacent to the particles has an excess of anions, the outer layer an excess of cations. As a result, when an external voltage is applied, the unattached cations are free to migrate toward a negative cathode. Esrig and Majtenyi (1965) reviewed theories of osmotic flow and developed an equation which suggests that the electro-osmotic velocity of water flow in porous media such as soils is related to pore ion conductivity, permeability, porosity, and soil plasticity.

Dr. Leo Casagrande (1947) made use of the principle of electro-

This correlation between foundation soil type and piping potential has some value as a guide to judgment . . . but it should not be used as a design criterion. Regardless of the average hydraulic gradient (or creep ratio), any earth dam foundation can be made safe against piping by the installation of properly proportioned graded filters.

Terzaghi (1922) presented an explanation of the mechanics of piping due to seepage pressure, clearly showing the advantages of theoretical and experimental methods over empirical methods. His theoretical explanation was supported by tests with models, which verified the conclusion that piping at the toe of a sheet pile wall occurs when the uplift forces exceed the available downward forces. He also explained his idea of the inverted filter which has since become a standard feature of design for many kinds of hydraulic structures. In a later publication Terzaghi (1929) demonstrated by means of flow nets that minor geological details can have a major influence on actual seepage patterns in nonuniform foundations, and he restated the physical causes of piping as follows:

> The fundamental requirement is that the upward pull exerted by the flowing water overcomes at some point on the bottom of the tailrace the downward pull exerted by the force of gravity. As soon as this event occurs the dam is lost, no matter where it takes place.

Casagrande (1935, 1937, 1961) has long been a prominent advocate of the use of rational methods for the design of structures with seepage and for the control over seepage forces and pressures with drainage. (See also Sec. 10.4.) In modern design, dams and overflow weirs on earth foundations should always be developed with the aid of rational methods and planned seepage control measures.

Mechanics of Piping Due to Heave. Nonuniformities in the deposition of soils and vertical holes made by burrowing animals, rotted roots, *unfilled drill holes,* abandoned water wells, and the like often permit seepage to concentrate and emerge in the form of *boils* at the landward toe of dams and other hydraulic structures. Sherard et al. (1963a) describe numerous causes of localized piping failures in dams. Piping failures caused by heave can be expected to occur at the downstream side of a hydraulic structure when the uplift forces of seepage exceed the downward forces due to the submerged weight of the soil (Sec. 3.4).

The method described by Terzaghi and Peck (1948b) for determining the factor of safety against piping is illustrated with reference to a row of sheet piles in sand (Fig. 10.29). The principles developed

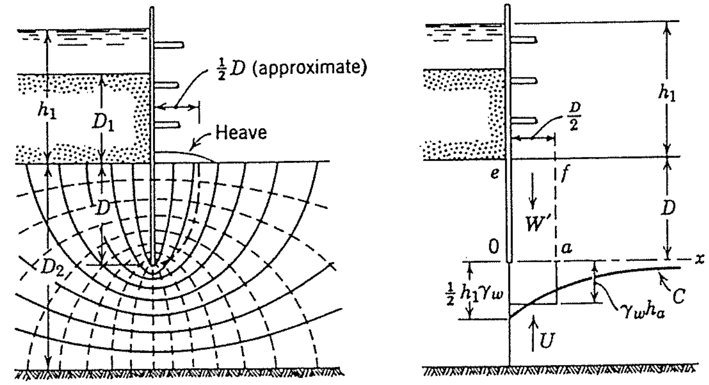


FIG. 10.29 Use of flow net to determine factor of safety of row of sheet piles in sand with respect to piping. (After Terzaghi and Peck 1948.) (Fig. 104, p. 231, *Soil Mechanics in Engineering Practice,* K. Terzaghi, and R. B. Peck, John Wiley and Sons, New York, 1948.)

here apply equally to the soil at the landward sides of weirs or dams on permeable earth foundations. First, a flow net is drawn as shown in Fig. 10.29*a*, from which the excess hydrostatic pressure on a horizontal plane such as Ox at a depth D can be determined as explained in Sec. 3.4. For a head h_1 on the left side of the sheet pile wall, the uplift pressure (excess hydrostatic pressure) can be represented by the ordinates of curve C using line Ox as a reference line. It is seen that the uplift pressure is greatest just to the right of the wall; hence the greatest danger of uplift exists near the wall. By tests with models, Terzaghi found that when the upward forces of seepage on a portion of Ox near the wall become equal to the downward forces exerted by the submerged soil, the surface of the sand rises as shown in Fig. 10.29*a*. This heave occurs simultaneously with an expansion of the volume of the sand, which causes its permeability to increase. Additional seepage causes the sand to boil, which accelerates the flow of water and leads to complete failure. Terzaghi's model tests demonstrated that heave occurs within a distance of about $D/2$ from the sheet piles. To calculate a factor of safety against failure, forces are determined on the prism $efaO$ (Fig. 10.29*b*) which has a depth D and a width $D/2$.

The average excess hydrostatic pressure on the base of prism $efaO$ is equal to $\gamma_w h_a$, and the uplift force U is equal to $\gamma_w h_a D/2$. Piping failure occurs when U becomes equal to the submerged weight of the sand which is its volume $D^2/2$ times its unit submerged weight γ' or $W' = 1/2D^2\gamma'$.

The factor of safety with respect to piping can therefore be expressed as

$$G_s = \frac{W'}{U} = \frac{D\gamma'}{h_a\gamma_w} \tag{10.4}$$

If it is not economical to drive the sheet piles deeply enough to prevent heave, the factor of safety can be increased by placing a weighted filter over prism *efaO*. If the weight of such a filter is W, the total downward force within a distance $D/2$ of the pile wall is $W + W'$, and the factor of safety is increased to

$$G_s' = \frac{W + W'}{U} \tag{10.5}$$

The upward force U exerted by the seeping water can also be determined with hydraulic gradients obtained from the flow net, since the seepage force is equal to the average hydraulic gradient in prism *efaO* multiplied by its volume and the unit weight of water (Sec. 3.4). Thus

$$F = \gamma_w i(\text{vol}) = 62.5i(\text{V}) \tag{3.19}$$

When the hydraulic gradient becomes equal to 1.0, the uplift exerted on a cubic foot of submerged soil is 62.5 lb/cu ft. Under this gradient, a soil with a unit weight double that of water (125 lb/cu ft) has a unit submerged weight of 62.5 lb/cu ft; hence with an uplift gradient of 1.0, the effective downward force is zero. This must be, since the *body force* (Sec. 3.4) must be zero. For this state of stress the frictional resistance at the base of prism *efaO* is zero. When these conditions exist, a state of *flotation* exists in the soil that leads to heave and to boiling and piping, as described previously.

Hydraulic gradients that produce flotation and heave vary with the unit weight of the soil, as shown by Fig. 10.30, which relates the upward hydraulic gradient that will just lift a soil with the unit submerged weight of the soil. It is seen that extremely lightweight soils are lifted by very small gradients, whereas heavy soils such as most sands and gravels require an uplift gradient of about 1.0 or somewhat greater.

In designing masonry dams and weirs on earth foundations, flow nets should be used in studying seepage patterns, and Eqs. 10.4 or 10.5 used for calculating factors of safety with respect to heave. Equation 3.19 may be used for determining the uplift gradients at discharge exits.

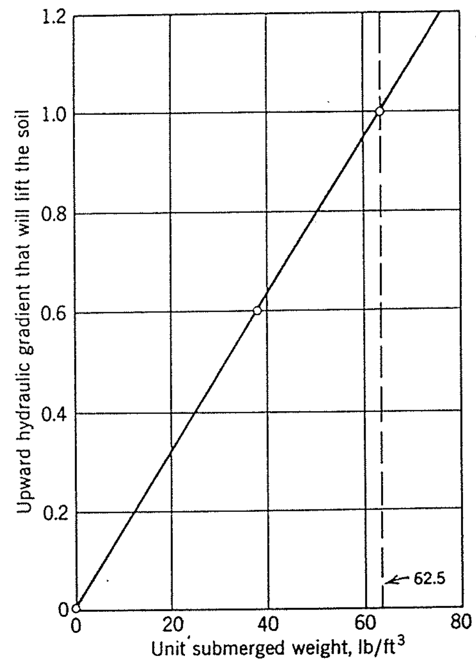


FIG. 10.30 Relationship between submerged soil weight and uplift gradient that will cause flotation.

Designing Weirs on Earth Foundations. To design and build safe, yet economical weirs on alluvial foundations, the following general procedures should be followed:

1. Investigate the foundation with adequate soils and geologic surveys and subsurface explorations.
2. Perform such field and laboratory tests as are required to obtain the important soil constants for representative soil specimens.
3. Evaluate soils and geological conditions.
4. Develop a design that will provide a safe structure for the most unfavorable conditions that can be expected.
5. Carry out the work according to properly prepared plans and specifications.

6. Follow up with piezometric measurements and such other field observations as are required to verify the performance of the completed work.

As with other structures, the design of overflow weirs is largely based on experience; however, analytical seepage methods can be of great value in pointing up the good and bad features of alternate seepage control measures and in establishing sound design principles.

To illustrate the possibilities of the flow net for studying this type of problem, consider the overflow weir in Fig. 10.31, which is assumed to be constructed without any seepage control measures on a homogeneous and nonstratified foundation. *If soils are stratified, this should always be considered as outlined in Sec. 3.2.* This weir is assumed to have a width B, and its downstream edge lies at a small depth d below the ground surface downstream from the structure. For these conditions the escape gradient is smaller than would exist if the weir rested entirely above the foundation, since for $d = 0$ the escape gradient at point x approaches infinity.

For the weir shown in Fig. 10.31, the escape gradient should be determined at x by measuring a distance Δl, between two equipotential lines at x, and determining the corresponding value of Δh, since $i_x = \Delta h/\Delta l$. Full squares should be subdivided when required to obtain a reasonably accurate value for i at the exit point x. For this example, Δl is about 2.5 ft, and the corresponding $\Delta h = h/n_d = 24$ ft$/9 = 2.7$ ft; hence $i_x = \Delta h/\Delta l = 2.7/2.5 =$ approximately 1.0, which is unsafe. It is evident that the design in Fig. 10.31 does not provide security against piping failures, since escape gradients should not exceed 0.30 to 0.40.

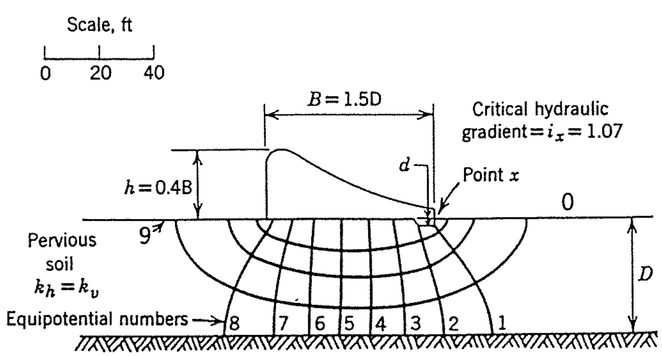


FIG. 10.31 Flow net for seepage under an overflow weir having no seepage control measures. ($k_h = k_v$).

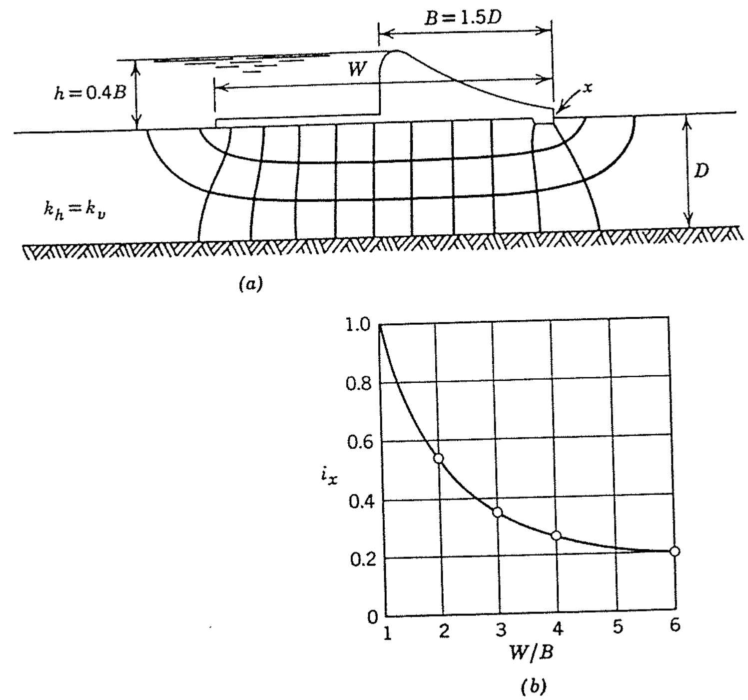


FIG. 10.32 Flow net study of weirs with upstream aprons. (*a*) Typical flow net. ($k_h = k_v$). (*b*) Curve relating total width of weir and apron to critical hydraulic gradient (hydraulic gradient at point X).

To investigate possible improvements in gradients that can be obtained by lengthening the contact between this structure and its foundation, a number of flow nets such as the one in Fig. 10.32*a* were drawn. If the head acting on the structure is h, the width of the weir is B, and the total length of the contact width W, the escape gradient i_x, varies as shown in Fig. 10.32*b*. It is seen that the total horizontal width of weir and apron W must be four times B if the escape gradient is to be reduced to a reasonably safe level of about 0.3. Depending on upstream blankets alone for seepage control is not efficient, and it provides negligible protection against piping due to "roofing."

Although the modified empirical line of creep methods recognize that inclined or vertical contacts between structures and foundations offer greater resistance to seepage than horizontal contacts, these methods generally fail to evaluate details that can have a major influence on the security of hydraulic structures. For example, the

Many of India's major rivers have alluvial beds of great widths and pass large floods. Their overflow dams, which are of a type that has been developed by the engineers of India, have heights in the range of 6 to 15 ft and lengths varying from a few hundred feet to thousands of feet. An example of an Indian dam is the Dauleshwiram Dam, built in 1840, which has a head of 17 ft on its crest and a length of 12,000 ft. The crest, 11 ft above normal river bed, has a cover of ashlar masonry. Its section (Fig. 10.26) is 232 ft wide upstream and downstream. Structures of this type must be able to pass large floods without being washed out and be safe against uplift, underseepage, and piping. Large downstream aprons have an important part in dissipating the river's energy. Cutoffs and drains are necessary to prevent piping and uplift failures.

Many of the early weirs and dams built in India and Egypt failed due to piping; however, the true cause remained obscure until the twentieth century. A serious failure in 1898 of the Narora Dam on the Ganges River in India focused attention to the problem of building safe weirs and dams on earth foundations. Prior to this time, engineers considered that the design of earth dams and other types of dams on earth foundations was more of an art than a science. It was widely believed that mathematical and analytical methods, although very appropriate for the design of masonry dams, were out of place in the design of earthen dams or masonry dams and weirs on earth foundations.

Terzaghi and Peck (1948a) point out that after the failure of the Narora Dam in 1898, a serious effort was made to analyze experience with dams on earth foundations and establish rules for their design. The methods that evolved started with the concept that piping tends to begin along the contacts between rigid structures and their foundations (Sec. 10.1). The shortest path that a particle of water could take in flowing under a structure was called the *line of creep*. According to line of creep methods, it is only necessary to make

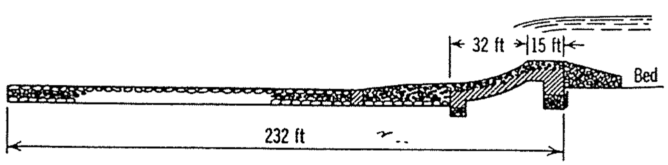


FIG. 10.26 Dauleshwiram Dam, India. (Item 2, Fig. 17, p. 1551, *American Civil Engineer's Handbook*, Merriman and Wiggin, John Wiley and Sons, New York, 1946.)

the length of the creep line sufficient for the type of soil in the foundation, and the structure will be safe against piping failures.

The original "line of creep" theory for dams on permeable earth foundations was advocated by W. G. Bligh, based on experience with weirs on alluvial foundations in Egypt and India. He treated this subject in the second edition of his work, *The Practical Design of Irrigation Works*. The general type of structure contemplated has a long impervious blanket upstream, a long apron downstream, and rock riprap or rubble downstream for the prevention of scour. Partial cutoffs may be provided to reduce the danger of underseepage failures, but drainage is not contemplated.

To be safe from undermining, according to the line of creep concept, the length of creep, L must be at least

$$L = cH \tag{10.3}$$

in which H is the greatest difference between the water level in the reservior and below the weir or dam, and c is an imperical constant called the *creep ratio*, which depends on the type of soil in the foundation and varies from 5 to 18. A few typical values for c are given in Table 10.2.

Designers who used the line of creep theory generally recognized that vertical contact surfaces offer greater resistance to piping than horizontal surfaces.

Lane (1935) revised the line of creep theory to allow for reduced resistance to piping along horizontal contacts as compared with vertical or inclined contacts. On this basis he developed his weighted creep method, in which horizontal contacts and slopes flatter than 45°, being less likely to have intimate contact, are weighted at one-third the value of steeper contacts.

Lane's *weighted creep ratios* varied from 2.5 for boulders with some cobbles and gravel to 8.5 for very fine sand or silt.

TABLE 10.2 Typical Bligh Creep Ratios.

Type of soil	c	Average Hydraulic Gradient
Light silt and sand	18	$\frac{1}{18}$
Fine micaceous sand	15	$\frac{1}{15}$
Coarse-grained sand	12	$\frac{1}{12}$
Boulders or shingle and gravel and sand mixtures	5–9	$\frac{1}{5}$–$\frac{1}{9}$

Creager, Justin, and Hinds (1950) developed further modifications of the *weighted creep* method, allowing for two important considerations: (1) whether or not drainage is provided, and (2) whether or not a flow net analysis has been made. Their modified weighted creep ratios vary from 1.5 for medium to hard clay foundations *with drains* and *with seepage analysis* to 8.5 for very fine sand and silt foundations, *without drains* and *without a seepage analysis*. The type of design contemplated by Creager et al., as shown in Fig. 10.27, has an upstream blanket and cutoff and a filter drain with weeps through a downstream concrete apron, to furnish control over underseepage.

Many overflow weirs and dams have been constructed in the United States and in other countries with combinations of methods for the control of underseepage. Borovoi, Razin, and Eristov (1963) describe the spillway dam for the Volga River Hydroelectric Station (Fig. 10.28), which has a head of 27 m and a length of 725 m. This spillway dam, which is expected to pass a flow of 37,600 cu m/sec, was constructed on 6 to 10 m of alluvium. Seepage control is obtained by two rows of steel sheeting driven through the alluvium into the sand-aleurite rock, which has a permeability one-seventieth that of the sand. Two rows of relief wells relieve uplift head under the dam and stilling basin.

The use of empirical methods often leads to designs having unknown factors of safety; for this reason, most leading dam designers prefer the use of analytical methods that evaluate the physical forces of soil and water. Sherard et al. (1963) state with reference to Lane's work that,

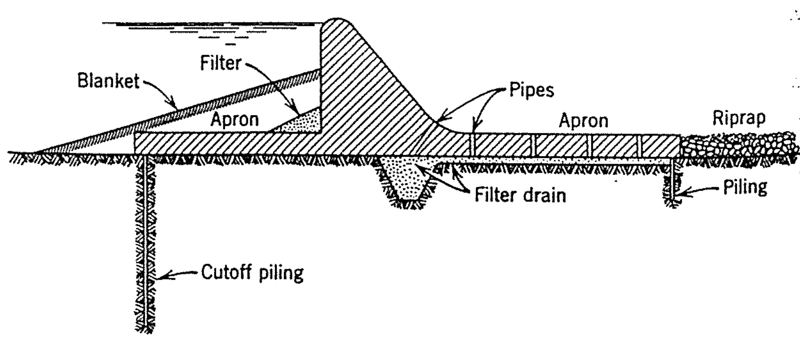


FIG. 10.27 Diagrammatic example of overflow dam on earth foundation. (Fig. 13, p. 68, *Engineering for Dams,* Creager, Justin, and Hinds, John Wiley and Sons, New York, 1950.)

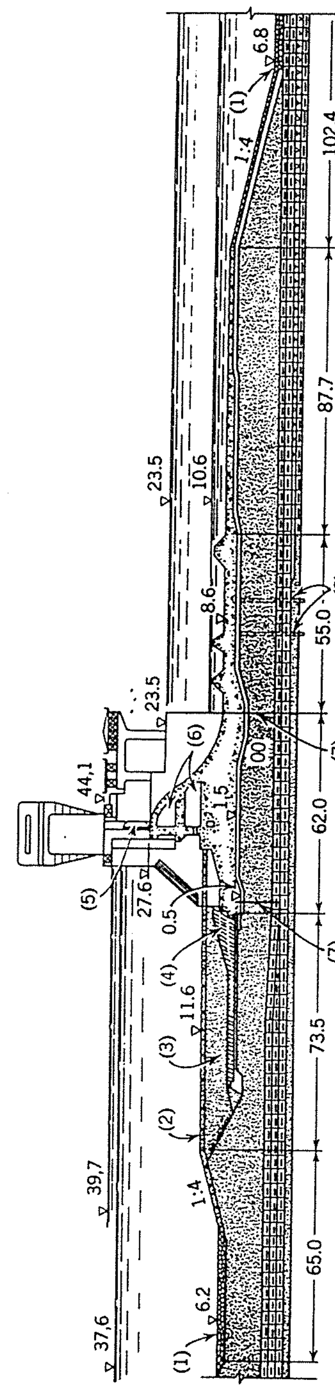


FIG. 10.28 Spillway dam for Volga Hydroelectric Station, USSR. (After Borovoi, Razin, and Eristov.) (1) rockfill, (2) concrete plates, (3) sand, (4) compacted clay, (5) main gate, (6) hollows, (7) steel sheeting, (8) relief wells. Dimensions are given in meters. (See "Some Large Dams of Hydro-projects in the U.S.S.R.," contribution to "Topmost Dams of the World," The Japan Dam Assn., Tokyo, Japan, Oct. 1963, pp. 224-5.)

position of a cutoff greatly influences the magnitude of escape gradients and the degree of safety against piping. Thus Fig. 10.33*a* shows a flow net for an overflow weir with a sheet pile cutoff under the *upstream edge,* and Fig. 10.33*b* shows a flow net with a cutoff of the same dimensions under the *downstream edge*. Except for the direction of flow, these two flow systems are identical, and the line of creep, or weighted line of creep, for these two designs is identical. Nevertheless the factor of safety against piping is vastly different for these two cases. The escape gradient for the section in Fig. 10.33*a* approaches infinity at point x, and the factor of safety against heaving

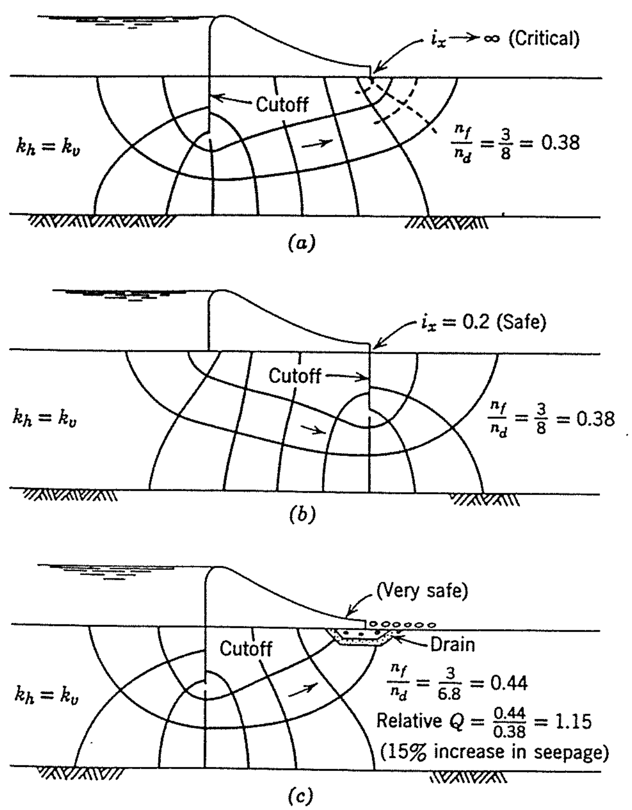


FIG. 10.33 Flow net study of seepage control measures versus safety against piping ($k_h = k_v$). (*a*) Cutoff wall under upstream toe, no drain. (*b*) Cutoff wall under downstream toe, no drain. (*c*) Cutoff wall under upstream toe and drain at downstream toe.

at this point approaches zero; whereas the maximum escape gradient in Fig. 10.33*b* with the sheet pile wall at the downstream edge is about 0.2, which is a reasonably safe level. A still more satisfactory design (Fig. 10.33*c*) utilizes a cutoff at the upstream toe in combination with a drain under the downstream toe of the weir.

In designing overflow weirs and small masonry dams on alluvial foundations, *uplift pressures* must also be kept below safe levels or aprons can be lifted and damaged. The influence of design details on uplift pressures under overflow weirs can readily be studied with flow nets as illustrated by Fig. 10.34. The design in Fig. 10.34*a*, which provides a downstream apron and cutoff, reduces the escape gradient to a moderate level but creates excessive uplift pressures beneath the downstream apron. In Fig. 10.34*b* the design is modified by placing a deep cutoff at the upstream toe and a shallow cutoff below the edge of the apron. This modification reduces the uplift pressures somewhat but not sufficiently to prevent uplifting of the apron. The design as modified further in Fig. 10.34*c* provides an upstream apron, a shallow upstream cutoff, and a downstream drain. This design has a large factor of safety with respect to uplift pressures and piping. If foundations are highly stratified, the cutoff should be deepened or relief wells installed below the drain or both.

Spillway Chutes

Overflow weirs on pervious foundations are designed to pass the surplus flows of rivers; hence they are *spillways.* Masonry dams on rock foundations frequently are designed with spillway sections that pass the surplus flows of rivers over these structures. These dams (Sec. 10.4) are made safe against underseepage by grouting and drainage.

Almost all earth dams and some masonry dams are designed with spillways that are cut into rock formations that are weathered to various degrees. Unless the rock is fresh and nonerodible, at least the upper portion of a spillway must be lined with concrete. If seepage and uplift pressures under the lining are not controlled, spillways may be severely damaged during critical river stages. In severe cases, failures of dams have been initiated by spillway failures; consequently the control of seepage and uplift pressures under spillway linings is of extreme importance.

Whenever a rigid structure is built on erodible or compressible weathered foundations, the slightest erosion or settlement of the foundation opens channels along which seepage concentrates and causes

From

EARTH AND EARTH-ROCK DAMS

by

J.L. Sherard; R.J. Woodward;
S.F. Gizienski; W.A. Clevenger

(John Wiley & Sons, Inc. 1963)

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Fig. 6.2:15 Views of the appearance of slurry trench cutoff material as exposed in an excavation three months after completion—Wanapum Dam.

great, the excavation can be made with trenching machines or backhoes, and trench widths of 3 to 4 ft. (or even less) can be employed and unit costs of \$2.00/ft.^{2} or less, can be obtained.

6.3 REDUCING UNDERSEEPAGE

If the construction of a complete seepage barrier for a dam founded on pervious soil is not practicable or economical, the designer has the choice either of making a partial cutoff or no cutoff, and then providing for the control of the seepage which develops. Underseepage can be controlled by methods discussed in Sec. 6.4. It can be reduced in quantity by one or both of the following principal measures:

1. Constructing a partial vertical cutoff usually extending down to an intermediate soil stratum of lower permeability, Fig. 6.3:1*a*.
2. Increasing the width of the base of the impervious section of the dam by constructing a horizontal blanket of impervious soil connected to the dam core and extending upstream, Fig. 6.3:1*b*.

Each of these methods reduces the quantity of underseepage by lengthening the seepage path. In doing so, each also reduces the downstream pore water pressures, and thus increases the stability. Another method sometimes used to increase the stability is to provide a downstream berm with or without a horizontal drainage blanket, Fig. 6.3:1*c*.

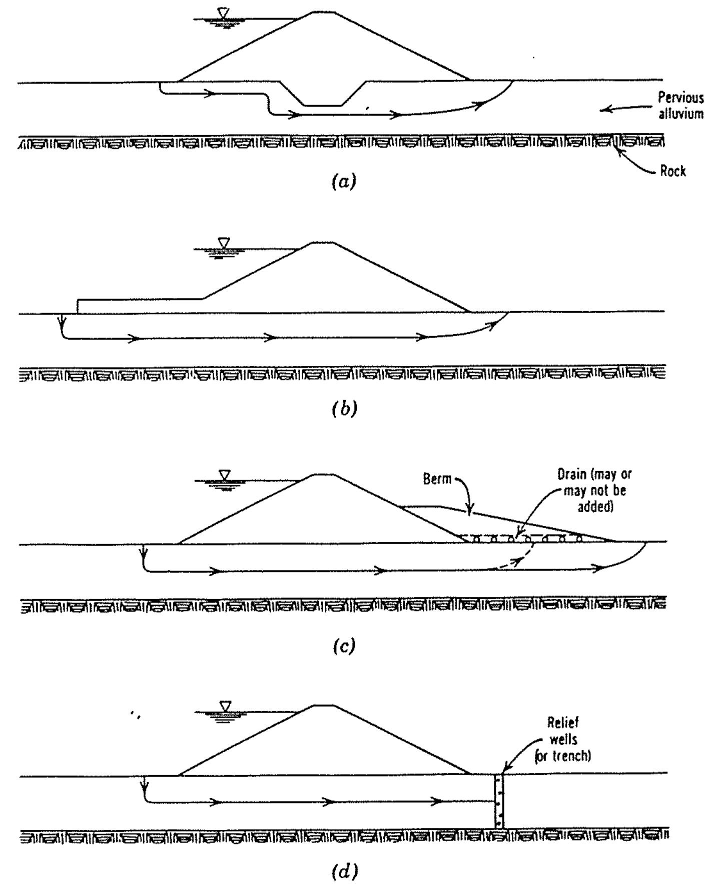

Fig. 6.3:1 Methods of underseepage control for dams on pervious foundations without complete seepage cutoffs. (a) Partial vertical cutoff. (b) Upstream impervious blanket. (c) Downstream berm. (d) Relief wells.

If no drain is provided, the berm will act to increase the seepage path in the same way as the upstream impervious blanket.

6.3a Partial Vertical Cutoffs

At sites where the average over-all coefficient of permeability of the foundation soil is practically the same in both directions and does not decrease with depth, a partial seepage cutoff has little influence on the underseepage quantities or pressures. Theory and model tests (Ref.

120) indicate that it is necessary for a cutoff to penetrate a homogeneous pervious soil foundation at least 95% of the full depth before there is any appreciable underseepage reduction. For this reason, only a complete cutoff should be considered at a site with a homogeneous foundation.

On the other hand, a vertical cutoff extending partially through the pervious soil may be of considerable value in reducing the underseepage in certain circumstances. Partial seepage barriers are valuable at sites where the average permeability of the foundation soil decreases with depth below the surface or where there is a single continuous impervious layer into which the cutoff can be connected.

In the circumstance where a complete or even a partial vertical cutoff is not warranted, the designer should always specify a nominal minimum cutoff trench with a depth of 6 to 10 ft. The purpose is to provide a continuous excavation through the upper few feet of the soil for inspection. Frequently a much better understanding of the subsoil profile may be obtained from examining the walls of this trench, and sometimes conditions are exposed which indicate that deeper exploration should be carried out. The trench is subsequently backfilled with rolled impervious core materials. This gives a secondary advantage by cutting off localized leakage which may tend to develop through the surface soil layer in drying cracks, animal burrows, or root holes. The cost of such a trench is relatively very small and is always well justified even for low dams.

6.3b Horizontal Upstream Impervious Blankets

The horizontal upstream impervious blanket, which increases the horizontal length of the average path of underseepage, is more effective in controlling seepage through a homogeneous soil foundation than the partial vertical cutoff.[1] If the blanket is very impervious compared to the natural foundation, so that relatively little seepage through the blanket occurs, the reduction in the seepage quantities and pressures at the downstream toe is directly related to the length of the blanket. If the blanket is only slightly less pervious than the foundation material, there is a maximum length of blanket beyond which no appreciable additional value is obtained by increasing the length.

Good practice for a blanket which is relied upon to control the underseepage requires that it be constructed of impervious soil in the

[1] See Ref. 271 for a review of the performance of upstream blankets installed at three major dams of the U.S. Corps of Engineers.

same manner and with the same care as the impervious core of the dam. At some dams nominal upstream blankets have been constructed by dumping impervious soil (sometimes waste stripping or other material) in a random manner and compacting only by the travel of the hauling equipment. At many sites a natural surface blanket of impervious material already exists, and it is necessary only to fill the holes or gaps to make a continuous seal. In such cases at least the upper surface of the natural blanket should be scarified, brought to a good water content, and compacted.

The necessary thickness and length of a blanket depend on the permeability of the blanketing material, the stratification and thickness of the pervious foundation, and the reservoir depth. Thicknesses varying from 2 to 10 ft. are most frequently used. If the blanket is not very tight with respect to the natural underlying soil, its effectiveness can be increased by making it thicker in the portion of its length which is directly upstream from the core of the dam.

Although the effectiveness of the blanket can be analyzed by using flow nets obtained from graphical solutions or electric models, these methods are tedious and cumbersome when many solutions are desired for varying sizes of blankets and estimates of permeability. A convenient mathematical solution has been developed by P. T. Bennett for the condition where the foundation consists of a single horizontal pervious layer with a more impervious surface blanket (Ref. 90). The errors resulting from the simplifying assumptions made in the development of Bennett's theory are small compared with the accuracy with which the coefficients of permeability of the various layers can be determined. The equations for underseepage quantities and pressures using Bennett's theory are given in Fig. 6.3:2.

6.4 CONTROLLING UNDERSEEPAGE[1]

Underseepage threatens the safety of a dam in two ways:

1. The pressure in the water through the foundation at some point below the downstream toe may approach or equal the weight of the overlying soil. Such high water pressures reduce the stability of the slope against sliding and in the extreme situation could theoretically heave the upper layers of the foundation.
2. The discharging seepage water may lead to piping failure.

[1] Suggested general reading on underseepage control and analysis—Refs. 98, 101, 314, 424, 464–469.

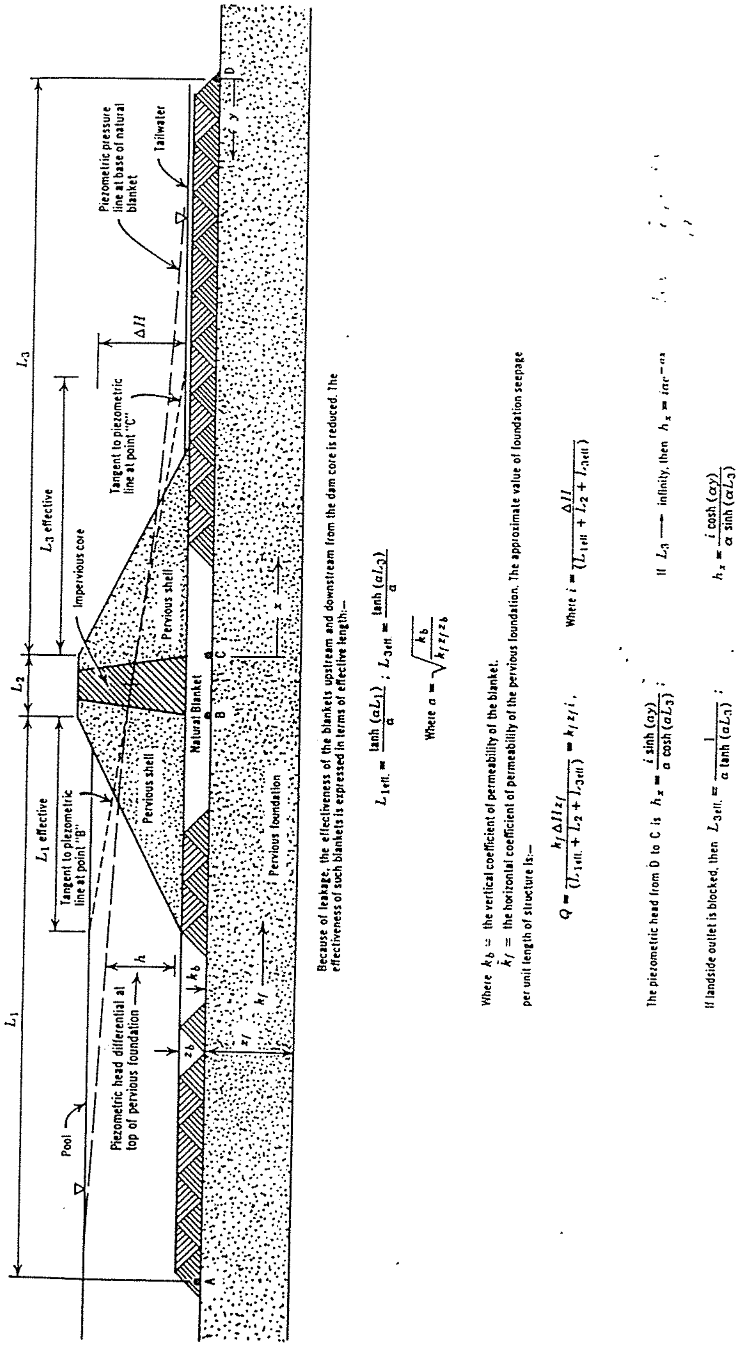

Fig. 6.3.2 *Effectiveness of finite upstream and downstream impervious blankets (after U.S. Army Engineers, Ref. 388).*

6.4a Underseepage Analysis

HIGH PORE PRESSURES

The uplift pressure in the foundation near the downstream toe of the dam depends primarily on the configuration and permeability of the various subsoil layers, and it may be high even if the quantity of seepage is not great. The highest pore pressures develop in the upper elevations of the foundation in the circumstance where there is a relatively impervious horizontal surface layer which prevents the free discharge of the seepage water. If this impervious surface layer is not sufficiently thick, the seeping water usually breaks through a crack or other hole, and may form a sand boil (Sec. 2.2*b*). On the other hand, if it is thick, the underseepage may never be seen at the surface even though dangerously high pressures can be measured with piezometers.

There are no reported failures of earth dams in which the foundation soil below the dam heaved, although there has been much concern at many dams when very high pressures were measured.[1] Undoubtedly, high downstream seepage pressures have contributed to the instability of most of the dams which have had downstream slope slides (Sec. 2.4). The influence of the pore water pressures on the stability of the downstream portion of the dam is subject to analysis (Sec. 7.6). Where high pore pressures are anticipated, the maximum values which may develop can be estimated in advance and the dam section proportioned to have an adequate factor of safety against sliding under the worst conceivable conditions. The actual pressures which develop after the dam is built and the reservoir fills can then be measured to assure that the stability condition is not more severe than anticipated.

PIPING

The piping potential, or factor of safety against internal erosion, in a given dam cannot be analyzed, but piping can be completely and reliably prevented by controlling the underseepage in such a way that (1) the exit velocities are not high, and (2) the water discharges

[1] Since the seeping water is flowing under a gradient toward the downstream toe, the pressure head in the foundation below the dam is always above the tailwater elevation. Therefore, if the surface of the ground downstream from the dam is relatively flat, the pressure head measured in a piezometer will nearly always be above the elevation of the ground surface.

through adequate thicknesses of progressively coarser soils which meet the gradation requirements for filters (Sec. 1.3*b*).

Early efforts to analyze the factor of safety against piping under dams on pervious foundations were made through the use of a concept called the "line of creep," which was defined as the shortest path that a particle of water would have to travel in seeping under the dam. The ratio between the line of creep and the pressure head loss was termed the "creep ratio," which is the inverse of the average hydraulic gradient. Later, when it was realized that the foundation stratification had a major influence on the piping potential, a "weighted creep ratio" was defined for which the length of the line of creep was computed as the sum of the vertical components of the shortest seepage path plus one-third of the length of the horizontal seepage path.

Using this weighted creep ratio as a criterion of piping potential, E. W. Lane made a study of 280 dam foundations, which included 150 failures (Ref. 480). As a result of this study, safe values of weighted creep ratio were established as a function of the foundation soil type. They ranged approximately from a minimum of 3 for foundations of gravels and boulders to a maximum of 8 for very fine sands. This correlation between foundation soil type and piping potential has some value as a guide to judgment for the engineer designing an earth dam on a pervious soil foundation, but it should not be used as a design criterion. Regardless of the average hydraulic gradient (or creep ratio), any earth dam foundation can be made safe against piping by the installation of properly proportioned graded filters.

SEEPAGE FLOW PATTERN

If the subsoil strata comprising the foundation are relatively uniform, the seepage flow pattern under the dam can be estimated readily with graphical flow nets (Chap. 5). This procedure rapidly becomes difficult and tedious for non-uniform subsoil conditions; however, by making simplifying assumptions, the designer can arrive at a fair estimate of the seepage pattern. The three-dimensional seepage which occurs through the pervious abutments has also been studied with reasonable success by means of flow nets. In this case electric models have been very helpful because of their rapid construction of the complex flow pattern.

At sites where the subsoil profile is very erratic, experience indicates that a simplified seepage analysis, based on a roughly estimated average permeability of the foundation soil, is likely to be as reliable

as the most detailed study which attempts to evaluate the seepage which may develop through the individual lenses or strata. In extremely erratic soils, which frequently occur in glaciated regions, the most thorough possible subsurface investigation, with a great number of exploratory borings, does not give sufficient information to allow a reliable estimate of the seepage pressure distribution, the quantity of flow, or even of the locations, if any, where leakage is most likely to emerge downstream from the dam. Under such circumstances the value of a competent geological study of the subsoil conditions cannot be overemphasized. After accumulating all the information possible, the designer must anticipate the most unfavorable conditions which might develop. Piezometers must be installed to check the actual pressures which develop after construction, and provisions must be made to control whatever leakage occurs.[1]

6.4b Regulation of Leaks

Any uncontrolled seepage erupting in the form of springs in the natural ground downstream from the core of the dam is potentially dangerous from the standpoint of piping. The greater the quantity and velocity of the discharging underseepage, the greater the piping potential. The danger is worse if the natural surface soils are fine, cohesionless sands or silts. It decreases as the soil becomes coarser, because well-graded coarse soils tend to form natural graded filters at the discharge points. Therefore leaks discharging through coarse soils may or may not need to be protected by specially constructed filters. On the other hand, leaks through fine-grained cohesionless soils always need filters to prevent progressive subsurface erosion.

With a dam which has a considerable quantity of underflow, it is necessary to locate and observe the point of seepage discharge, if any, on the ground surface, and to install filters if they are needed to prevent erosion. Underseepage which discharges uniformly over a large, more or less flat area downstream from the dam can be covered with thin layers of progressively coarser filter material. Where the seepage occurs in the form of a few concentrated leaks in one area, it can be controlled by channeling all the leaks into an excavated drainage

[1] Excellent descriptions of the step-by-step procedures used for the design of two dams on very erratic, pervious foundations are given in Refs. 93 and 456. Also see Ref. 463 for a description of the design measures to control seepage through the very pervious abutments at the Chief Joseph and McNary Dams on the Columbia River.

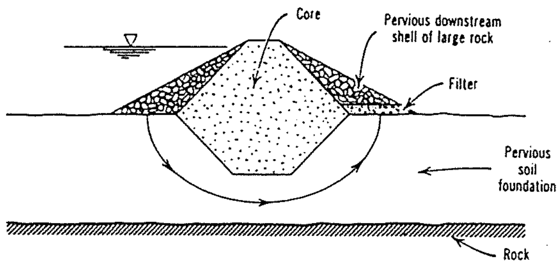

Fig. 6.4:1 Need for filter under downstream rockfill shells on soil foundation.

trench which is backfilled with graded filter material. Drainage tunnels or relief wells (Sec. 6.4*c*) can be installed to control the discharge of the underseepage before it reaches the ground surface. This action decreases the pore pressures deep in the abutments or foundations and thus increases the stability of the downstream portion of the dam.

A dam cannot be considered safe unless all downstream leaks are discovered. For this reason, it is dangerous to allow large ponds of water to stand in the vicinity of the downstream toe of a dam which has appreciable underseepage. Such ponds may form in the borrow pits or as the backwater from the outlet or spillway discharge. If large areas downstream from the dam are covered with water, piping of concentrated leaks may progress unobserved and lead to failure.

Unobserved piping can also occur under downstream embankment sections composed of large rock, as shown in Fig. 6.4:1. For this reason, a horizontal filter is needed between the foundation soil and the downstream rockfill section.

6.4c Relief Wells

Relief wells to control the pressure of seeping water under the foundation of an earth dam (Fig. 6.3:1*d*) were first used by the U.S. Army Corps of Engineers. They were installed below several of the major Army dams in the late 1930's as an emergency measure when unexpectedly high pressures developed during reservoir operation. One of the first times they were included in the design stage was in 1940 at the Arkabutla Dam in Mississippi. Because they are relatively inexpensive and enormously effective in controlling seepage pressures, they have been used with increasing frequency since then. At present,

with few exceptions, most engineers consider relief wells at any site where there is a possibility that high pressures could develop.

The primary disadvantages of relief wells are:

1. They require inspection and maintenance and may have to be replaced during the lifetime of the dam.
2. They decrease the average seepage path and increase the quantity of underseepage.

In the immediate vicinity of each well, the pressure head in the seepage water is reduced to a value nearly equal to the elevation of the top of the well (hydrostatic pressure). The wells should be spaced close enough so the water pressure between them is not excessive. In a comprehensive paper on this subject, Middlebrooks and Jervis (Ref. 462) have developed theoretical methods for determining well spacings. The theory, however, is approximate and the results of such computations can only be considered a rough guide.[1] The most practical approach is to install the wells at reasonable spacings and to measure the pore water pressures which develop between them. Wells are commonly spaced between 50 and 100 ft. on centers. Intermediate wells can be added later if necessary to keep the pressure below any desired value. One of the main advantages of a relief well system is this flexibility which allows it to be expanded to meet the need at nominal extra cost.

Relief wells should extend through the full depth of the pervious foundation if possible. This is especially important for erratically bedded soil formations, where the seepage flow pattern cannot be reliably estimated in advance. Extensive laboratory tests on models, performed in the U.S. Army Engineers' laboratory at Vicksburg, Mississippi (Ref. 464), indicate that even in a very homogeneous pervious foundation, the effectiveness of wells which penetrate less than 50% of the thickness of the layer is greatly reduced. Wells which penetrate less than 25% have far less efficiency than fully penetrating wells and if the layer is more pervious in the lower elevations, a partially penetrating well may have almost no effect on the underseepage (Refs. 56, 114).

A relief well should have an interior perforated pipe (well screen) with a minimum inside diameter of 6 in., or larger if heavy flows are

[1] See Ref. 562 for an excellent description of the relief well installation at the U.S. Corps of Engineers' Fort Randall Dam on the Missouri River, including an example of the application of the analytical theory for computing well spacings and discharges.

anticipated. Gravel-filled holes are much less effective than open pipes (Ref. 462). Many kinds of pipe have been used successfully, although wooden pipes have probably been most widely installed because they last longer than other types when completely saturated, Fig. 6.4:2.[1] When it is anticipated that the reservoir may become completely empty and the wells may not remain always full of water, the wooden well screen can be backfilled with concrete in the upper elevations so that the well will remain serviceable even if the wood deteriorates. Asphalt dipped, galvanized metal well screens have been used at a number of large dams in recent years, Fig. 6.4:3. At the USBR's Enders Dam in Nebraska, a 24-in. diameter metal well screen was protected cathodically with buried magnesium anodes (Ref. 42). Perforated well screens of plastic pipe and concrete pipe have also been used on a few major dams, Fig. 6.4:4.

The annular space surrounding the well screen is backfilled with gravel which is graded to meet the filter requirements of the natural foundation soil. At the surface, the annular space is backfilled with impervious soil or concrete in order to prevent an upward flow of water outside the pipe.

Seepage water from the wells is usually discharged on the ground surface at the toe of the dam through a horizontal overflow pipe, Fig. 6.4:5. It should then be collected in a lined drainage ditch. At some dams, the seepage water has been discharged into a horizontal header pipe which is buried below the ground surface, Fig. 6.4:4. This system has the psychological advantage of keeping the leakage hidden in addition to reducing the discharge pressure by a few feet. Whenever the header pipe is buried (at the toe of the dam or inside the dam), the relief well casing should be extended to the surface for inspection and maintenance.

The holes for relief wells should be drilled by a method which will not seal the pervious soil strata exposed in the wall of the hole with fines. Relief wells should be developed in the same way as wells drilled for water supply, and after the hole is completed and cased, it should be pumped out and "surged" to remove all muddy water and "to develop the filter." The surging can be carried out with a heavy rubber piston a little smaller than the inside diameter of the pipe. After cleaning and surging, a pumping test should be made to determine the rate of inflow for a given drawdown and the rate of sand

[1] Corrosion of the initially installed steel relief well pipes at Fort Peck Dam on the Missouri River made it necessary to replace the system with wood pipe after about six years of operation (Ref. 569).

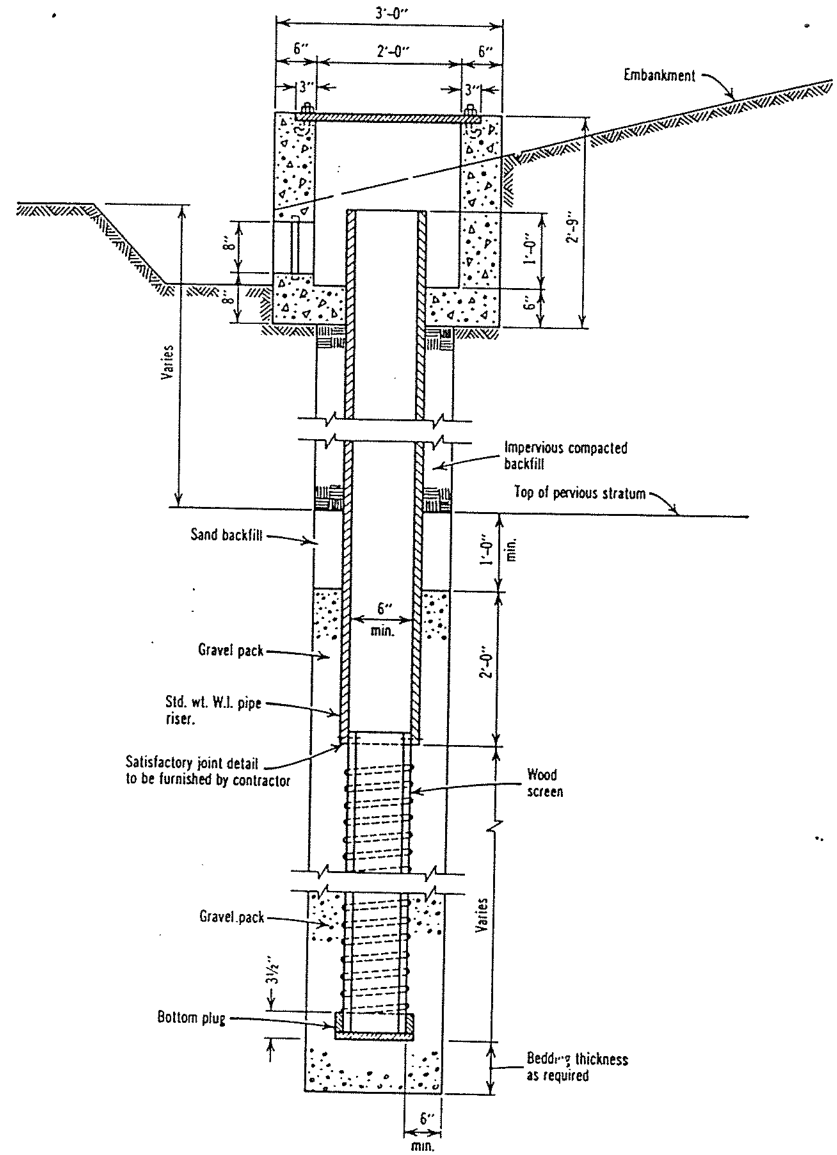

Fig. 6.4:2 Gravel-packed relief well with screen (after U.S. Army Engineers, Ref. 388).

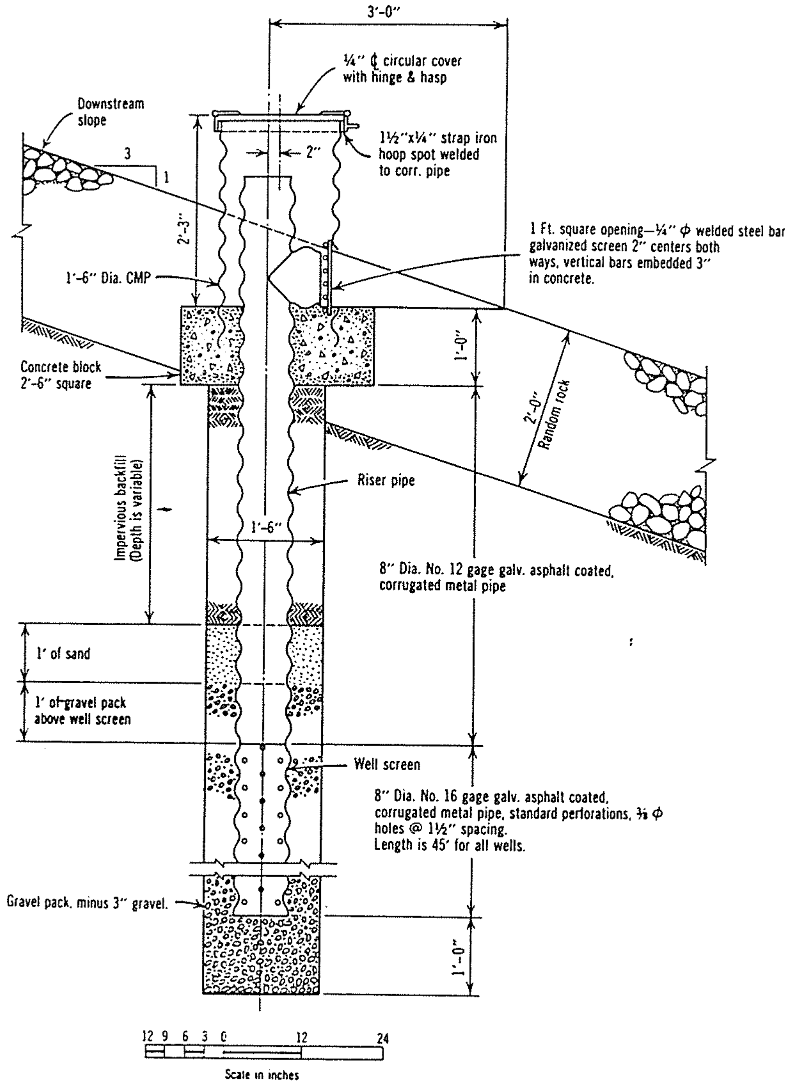

Fig. 6.4:3 Relief well detail—Success Dam. (Courtesy Sacramento District, U.S. Army Corps of Engineers)

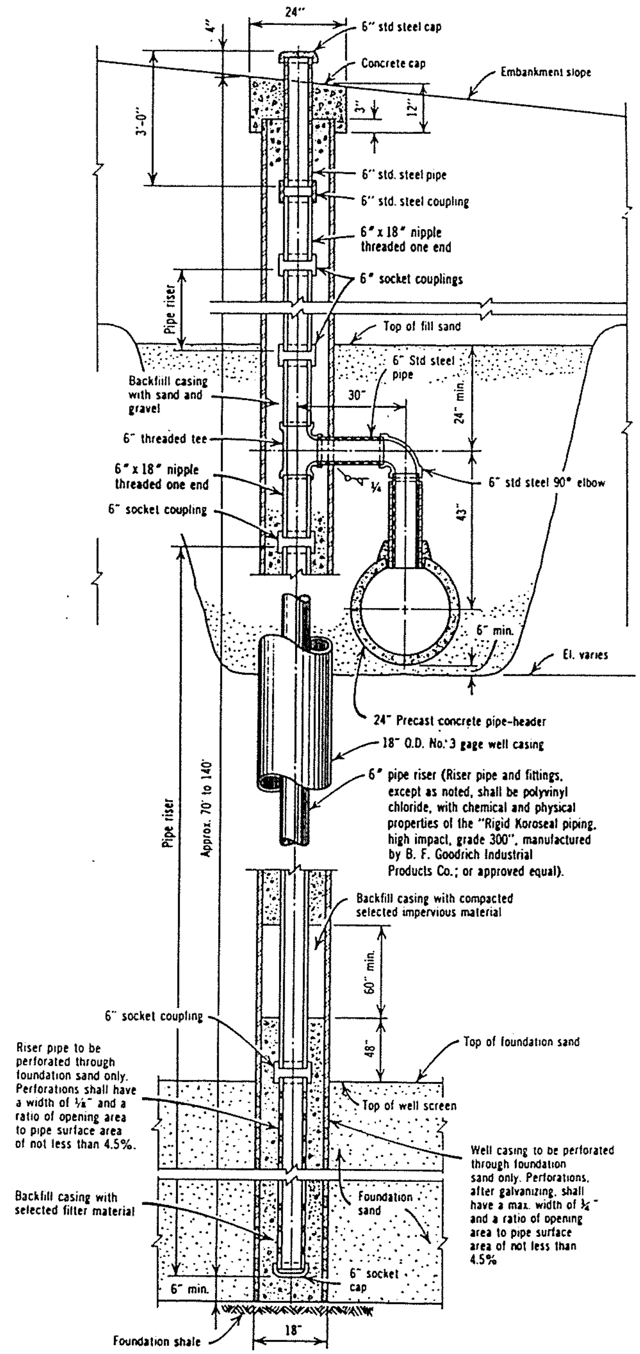

Fig. 6.4:4 Relief well using plastic pipe—Karnafuli Dam, East Pakistan, 1959. (Courtesy International Engineering Co.)

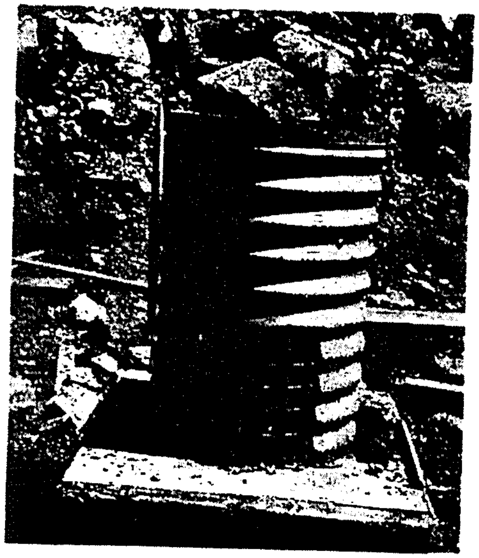

Fig. 6.4:5 Typical overflow discharge pipe at top of relief well.

infiltration. The inflow measurement can be used to compare with subsequent tests on the well to determine if it is becoming less effective with time.

Relief well systems may be supplemented at the abutments with horizontal drainage tunnels. Sheeted and braced tunnels have been provided at several major dams in recent years to tap seepage in earth and rock abutments; such tunnels may have lengths of many hundred feet. In rock abutments the holes are usually left open, and they may or may not be provided with a concrete lining which has weep holes in it. Tunnels in earth abutments may be concrete lined or may be backfilled with pervious filter material.

seven

Stability Analyses

7.1 HISTORICAL DEVELOPMENT

Prior to 1935 few experienced engineers placed much reliance on theoretical embankment stability analyses. Before this time, earth dam side slopes were selected wholly on the basis of past experience, and local rules evolved reflecting local experience, or the opinions of the principal designers in the area. In the period between 1905 and 1915, for example, during a boom in the construction of irrigation works in Colorado local engineers built a number of major earth dams which had steep upstream slopes (1.5:1) and flatter downstream slopes (2:1 to 3:1). But during the same years in California many large reservoirs were constructed with the use of earth dams in which the upstream slope was usually made flatter than the downstream slope.

In France in the late nineteenth and early twentieth centuries, the upstream slope was generally made 1.5:1 and the downstream slope flatter (Ref. 277):

> This belief in the 1.5:1 dogma was so firmly enrooted in the minds of certain French engineers that, having been called into consultation during 1934, on an earth dam 16 meters high, having all the characteristics of a slide, the contractor said in answer to our observations on the stability of the dam . . . . "A 1.5:1 dam always holds" . . . .

In his distinguished book *Earth Dam Projects* Joel D. Justin suggested that slopes be selected on the basis of past experience and judgment, and he summarized his remarks on the subject with the following recommendation (1932):

> Some engineers contend that the upstream slope should be flatter; others have claimed the opposite. A thorough study leads to the conclusion that neither is correct, as sometimes the upstream slope should be flatter and vice versa.

From

ADVANCED DAM ENGINEERING

FOR DESIGN,

CONSTRUCTION AND REHABILITATION

Edited by

Robert B. Jansen

(Van Nostrand Reinhold, 1988)

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they may have limitations that should be recognized. Even with extensive blanketing, a potential for piping may still exist due to high internal water pressures at downstream exits of foundation strata. Hydraulic gradients through blankets may be very high. A typical upstream blanket may be placed upon a nonuniform foundation without a filter layer. The thin earth element is subjected directly to heavy reservoir loading. Under such conditions, a blanket could be susceptible to cracking and/or piping, as was observed at Tarbela Dam in Pakistan. These possible disadvantages must be weighed carefully in analyzing seepage control options. Often a partial cutoff under the embankment will be safer and more effective than a blanket, because of horizontal stratification of foundation materials.

At some sites, the natural soil on the reservoir basin may have superior qualities as a seepage barrier, including filter capability. In such cases, particularly where a positive cutoff would be difficult to secure, care should be taken to preserve the natural blanket and to incorporate it into the embankment design.

Cutoffs. Seepage under an earthfill dam is controlled most effectively by a cutoff into an impervious foundation. At many sites this can be accomplished by excavating a trench and backfilling with compacted earth, which is in effect part of the embankment core. Such a cutoff should be sufficiently wide to ensure an acceptably low seepage gradient, and its excavated slopes should be flat enough to preclude excessive stress concentrations. The base width of the cutoff trench is commonly specified to be at least one-fourth, and preferably should be at least one-half, the maximum net head on the dam. If there is any possibility of piping of the backfilled material into the pervious foundation, a filter layer should be placed on the downstream face of the trench to separate the incompatible materials.

If the cutoff trench would have to be extended to an uneconomical depth, a slurry trench might be a feasible alternative. Trenches of this type can be placed practically to maximum depths of at least 180 ft (55 meters), usually by means of dragline, clamshell, backhoe, or other trenching equipment. The walls of the trench are stabilized by a pool of bentonitic slurry. The backfill that finally displaces the slurry must contain enough fine-grained soils to be effectively impervious and sufficient coarse material to minimize settlement. A generally acceptable range for materials passing a No. 200 sieve is from 10 to 30%. To obtain and retain a uniform slurry mix, backfill components of clay, well-graded sand and gravel, and bentonite preferably should be weight-batched into mixers for blending with a predetermined quantity of water. On some projects, the trench backfill has been mixed by windrowing, dozing, or blading. Although mechanical mixing with aggregate or concrete mixers is generally superior, mixing on the ground has been sufficient in many cases. Properly constructed soil-bentonite slurry trench cutoffs have properties similar to those of stiff clay. Their effectiveness has been demonstrated in many projects where they have adjusted to embankment or foundation deformations without significant cracking. Preferably, they are located just upstream from the dam where their settlement would not affect the embankment. They have been incorporated into original designs of dams, and they can also be effective in remedial work at existing structures.

Another method that offers many advantages involves addition of cement to the bentonite-water slurry just before its introduction into the trench. The cement-bentonite slurry remains to set up and form the permanent cutoff wall.

Techniques have also been developed for the installation of concrete walls or diaphragms through the use of slurry trenches. Tremie concrete has been placed successfully in this way to achieve a positive cutoff. Concrete diaphragms have been built at several major dams, including Manicouagan 3 in Québec. This 350-ft (107-meter)-high earthfill is founded on alluvium over 107 meters deep, consisting of sand, gravel, cobbles, and boulders. The cutoff through this material was established by two parallel concrete walls 2 ft (0.6 meter) thick, spaced with centerlines 10 ft (3 meters) apart, composed of interlocking piles and panels.

When a cutoff is to be constructed at an existing dam, its influence on stability must be carefully analyzed. The preferable location for the cutoff may be at or near the upstream toe.

One of the least effective alternatives for seepage reduction is sheet piling. Although it was installed at many early dams, it has not proved dependable. The use of steel sheet piling cannot be regarded as a positive means of controlling seepage. Vibrating pile hammers and other measures may improve pile alignment, and bentonitic slurry may be helpful in sealing the piling interlocks; but sheet piling in most applications cannot be expected to provide a watertight barrier.

Foundation Drainage. Usually an earthfill can be made impermeable enough that it will not pass much seepage. Reservoir water losses are likely to be greater through the underlying natural formations. Underseepage can be controlled by grouting, cutoffs, and/or earth blanketing, in combination with drain layers, foundation galleries, toe drains, and/or relief wells. Drainpipes have been placed under embankments and under the earth linings of reservoirs. However, such installations have possible disadvantages that should be considered carefully. A conduit laid under an embankment must not be susceptible to rapid deterioration. Any failure may be difficult to detect and to remedy. Access for monitoring and/repair is highly desirable.

In designing a pipe system for drainage of the abutments, two preferred guidelines are: (1) provide two outfalls connected together so that, if one outfall fails because of crushing under construction or movement or plugging during operation, there will still be a reserve discharge line; and (2) extend the upper end of each abutment drain to serve as access for cleaning and testing.

Underdrains in zones subject to movement should be divided into sections with separate outfall systems so that areas of leakage can be identified. Drains on one side of a foundation shear can be isolated from those on the other side to avoid possible fracturing of lines at the shear.

Rigid pipes commonly used for drains, such as clay tile and asbestos cement, require extra care in handling and bedding because they are relatively brittle and easily damaged. Some metal pipes are very susceptible to corrosion, particularly when located in a moist embankment. Although such pipes are found at many older dams, their use in new construction or in rehabilitation would have to be thoroughly justified in comparison with usually preferred alternatives such as sand and gravel drains.

The need for filters must be assessed in designing drains. Also, filter layers may be required at the foundation contact beneath pervious shells if there is a possibility of flushing foundation material into the embankment. This is particularly important on soil foundations. In some cases, the embankment zones may be fully compatible with the underlying material. Thorough investigation is necessary, however, to ascertain that conditions at the contact are sufficiently uniform to justify omission of a filter.

Relief wells are used in combination with various other schemes to control water pressures in the downstream zones of the embankment and in its foundation. Wells drilled in erodible material must be protected from piping. Under usual conditions, the well casing inside diameter should be at least 6 in. (15 cm) so that seepage head loss will be minimized. A filter of not less than 6-in. thickness should be provided around the well screens. The recommended minimum ratio of the 85% size of the filter to the size of the screen opening is 2.0.

Relief wells are especially beneficial where an impervious foundation layer overlies pervious material. The wells should be drilled deep enough into the underlying formation that destabilizing water pressures are reduced to safe limits. Well spacing may vary from about 25 ft (7.6 meters) in very pervious foundations to 100 ft (30.5 meters) in less pervious materials. Spacing of wells generally should not be more than about one-half their depth.

In common practice, a pressure relief system is developed in stages, with a minimum number of wells installed initially and others added later based on monitoring of foundation pressures as the reservoir operating regimen is established. The wells should be designed to be accessible for maintenance. Even with periodic cleaning, their efficiency may gradually decline, so that rehabilitation and/or supplemental wells may be needed eventually.

Where the upper part of the foundation is permeable, relief wells may not be effective seepage interceptors. A toe drain may be beneficial in collecting and conveying flows issuing from the embankment and the foundation. This kind of drain typically consists of a filter-protected pipe in a trench under the toe of the dam. Toe drains often are placed in combination with drain blankets. They are installed also on impervious foundations to lower the groundwater level immediately downstream from the embankment. For any earthfill, they may be useful in detecting and monitoring seepage sources.

Because properly designed toe drains are accessible for maintenance and repair or replacement, various kinds of pipe may be used in their construction, including concrete, clay tile, plastic, or corrugated steel. The minimum trench depth usually specified is about 3 or 4 ft (1.0 or 1.2 meters) (Fig. 9-5). The maximum depth normally is dictated by the need for a continuously positive drain slope on each abutment. The pipe size is determined by drainage requirements and may range from a minimum of 6 in. (15 cm) up to 30 in. (76 cm) or larger. Collector pipes are open-jointed or perforated and are surrounded by filter material. Two-layer filters may be necessary to prevent foundation piping

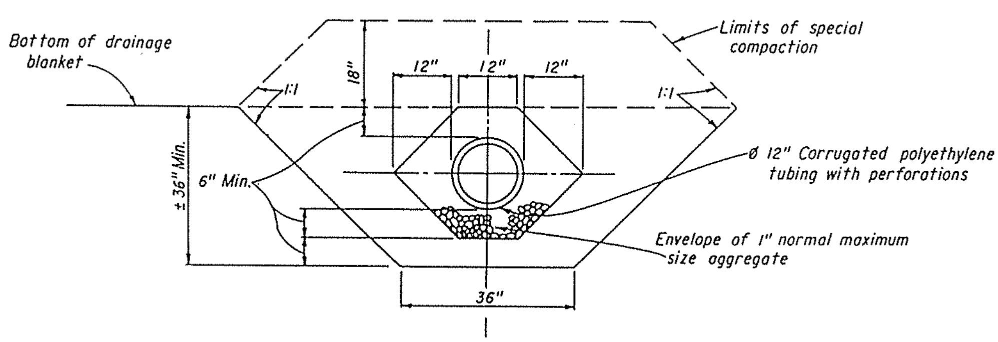
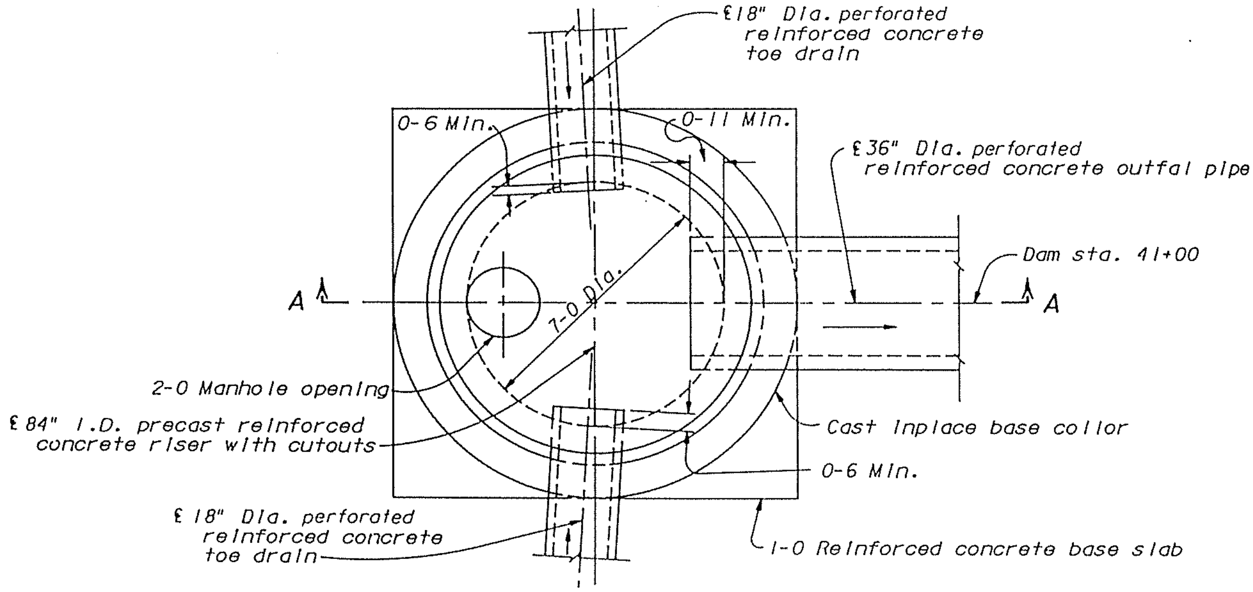
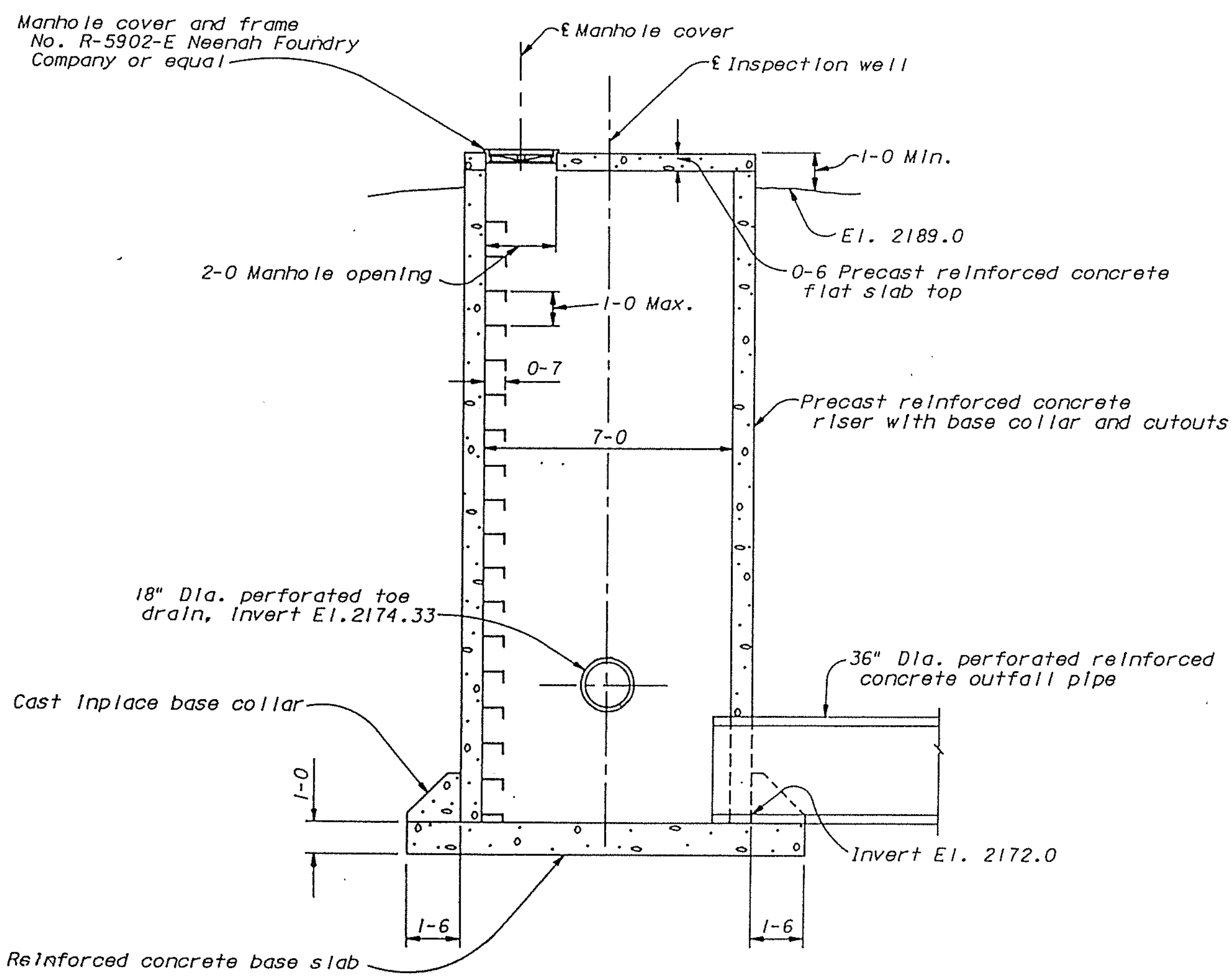


Figure 9-5. Typical toe drain.



OUTFALL INSPECTION WELL NO. 1 - PLAN VIEW



OUTFALL INSPECTION WELL NO.1-SECTION A-A

Figure 9-6. Typical inspection well, Calamus Dam. Courtesy: U.S. Bureau of Reclamation.

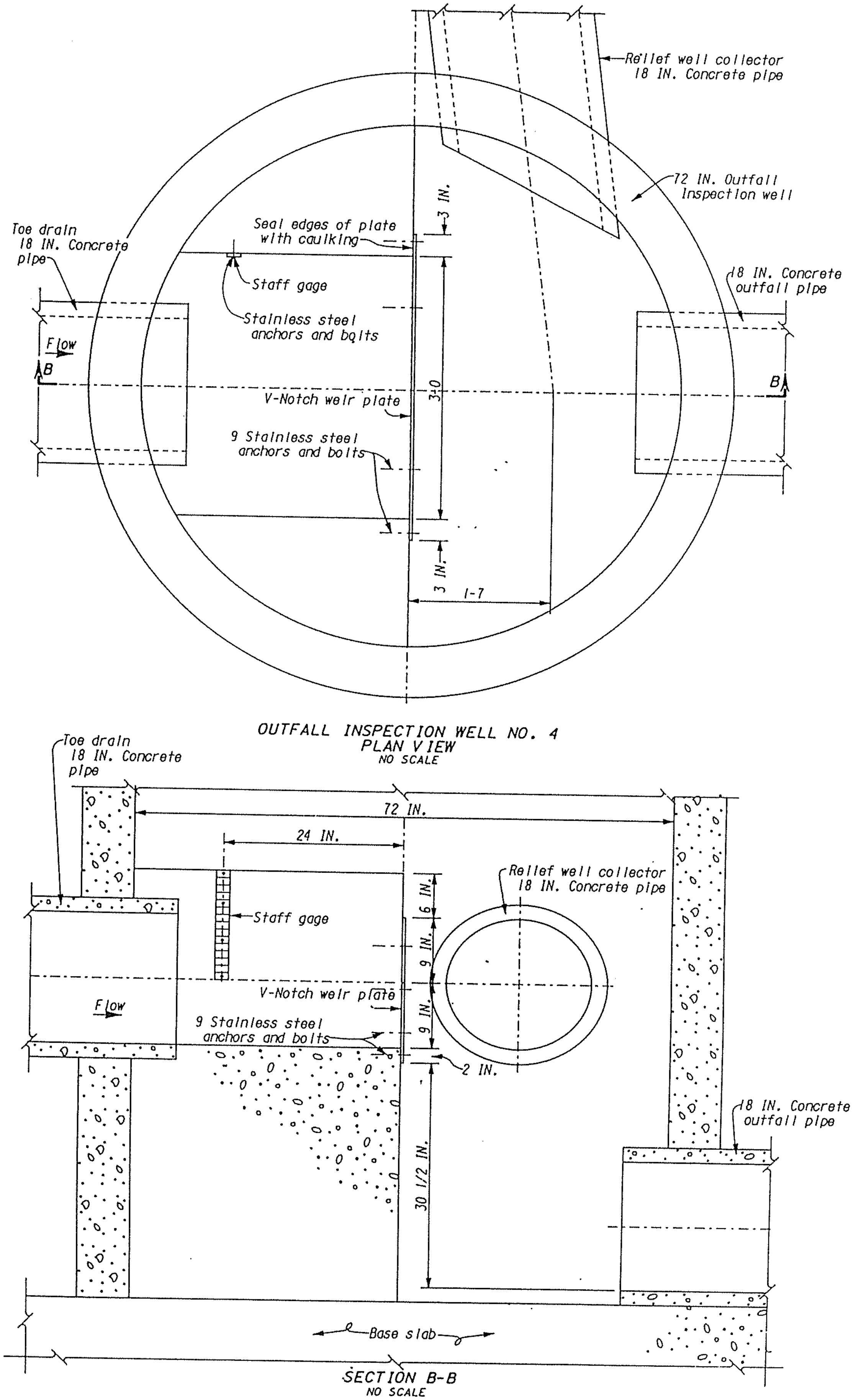
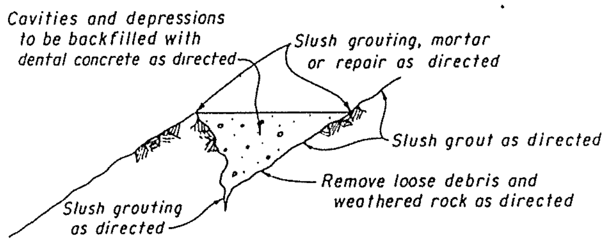
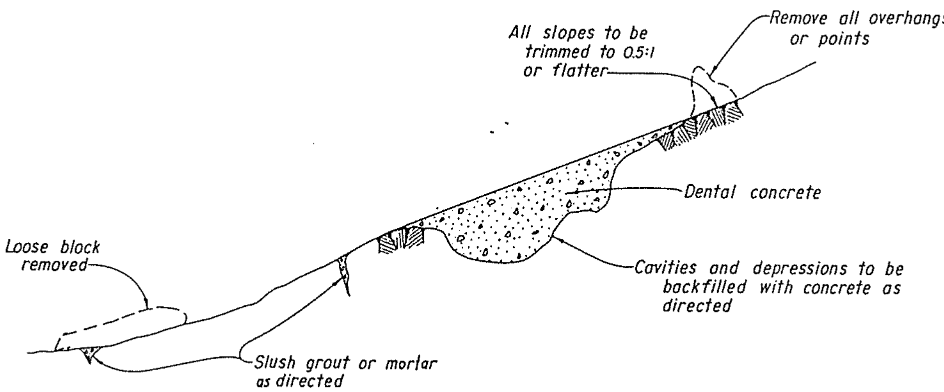


Figure 9-7. Typical inspection well with weir, Calamus Dam. Courtesy: U.S. Bureau of Reclamation.



(d)



(e)

Figure 9-4. (*Continued.*)

deform without excessive cracking. It is also beneficial to defer placement of the upper part of the fill, if possible, to minimize the effects of settlement.

Foundation Seepage Control

Grouting. In designing to control foundation seepage, a reasonable balance must be attained between drainage and provisions for reducing flows (i.e., blankets, cutoffs, and grout curtains). The conditions peculiar to each site will dictate the optimum combination of protective measures. A primary function of a seepage barrier is to prevent excessively concentrated flows through foundation discontinuities. Grouting, for example, tends to close the largest rock openings and thus to distribute flows more evenly so that drainage downstream from the treated zone can be enhanced. Grouting may not be very beneficial by itself, but often it is dependable when combined with adequate drainage systems. The grout must be mixed to proper proportions for the site conditions and has to be injected under controlled pressures to avoid damaging the dam or the foundation. In establishing a grout curtain under an embankment, several rows of grout holes are generally preferable to a single row.

Impervious Blankets. Where an effective foundation cutoff is not feasible, the impervious zone of the embankment can be extended upstream as a blanket. In combination with downstream drain wells, this alternative may provide economical seepage control. A complete blanket would extend into an impervious contact along its full boundary. Partial blanketing is sometimes done to lengthen the path of seepage. The availability of materials and the reservoir topography at the dam will influence the design. To be most effective, an earth blanket normally should have at least a 3-ft (0.9-meter) thickness and should be thoroughly bonded to the adjoining impervious elements. In locations where the blanket might be subjected to erosion, it should be covered by protective material.

The area of the blanket is usually determined by balancing costs with the value of the desired degree of seepage control. The benefits of future sealing by reservoir siltation also should be considered. Although upstream blankets can be useful in decreasing water losses and uplift pressures,

From

SOIL MECHANICS IN ENGINEERING PRACTICE

by

K. Terzaghi and R.B. Peck

(John Wiley and Sons, Inc. 1948)

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cutoff represents a potential source of failure by piping. The mechanics of these failures are discussed in the following article.

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ART. 59. SAFEGUARDS AGAINST PIPING

GENERAL CHARACTERISTICS OF PIPING FAILURES

Unless the foundation of a dam is provided with a perfectly watertight cutoff, water percolates through the subsoil from the reservoir to the downstream side, where it may emerge in the form of springs. Under certain conditions discussed in the following paragraphs, the percolating water may produce one of two phenomena. Either the seepage pressure may lift the entire body of soil located along the downstream toe, or else the water that comes out of the ground at the downstream toe may start a process of erosion that culminates in the formation of a tunnel-shaped passage or *pipe* beneath the structure. A mixture of soil and water then rushes through the passage, undermining the structure and flooding the channel below the dam. Failures of either type are known as *failures due to piping*. The first type has been referred to as failure by *heave*, and the second as failure by *subsurface erosion*.

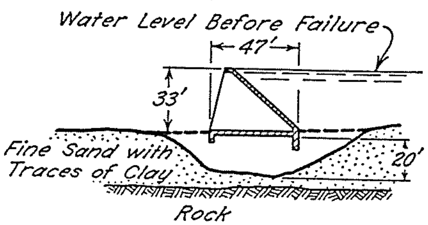

FIG. 202. Diagram illustrating failure of dam foundation due to piping.

The failure of a dam by piping ranks among the most serious accidents in civil engineering. It is likely to include not only failure of the structure

but also extensive damage to the subsoil for a considerable depth. Furthermore, it not infrequently happens without warning and causes loss of life and damage to property located in the lower reaches of the valley. Therefore, the conditions that lead to failure by piping and the means for avoiding the danger deserve special attention.

Figure 202 illustrates a typical failure caused by piping. The dam, of the slab and buttress type, rested on a reinforced-concrete base slab provided with an upstream cutoff wall 9 ft deep and a downstream cutoff 7 ft deep. Failure occurred suddenly by a rush of water beneath the dam. A 52-ft gap was left in the subsoil and was bridged over by the structure.

CAUSES AND MECHANICS OF PIPING

Until the beginning of the 20th century the causes of piping remained unknown, in spite of the fact that piping failures were not uncommon. Designers realized the value of sheet-pile cutoffs but no rules were available for determining the proper depth to which the piles should be driven or for estimating the factor of safety against failure of the completed structure. However, after the catastrophic failure in 1898 of Narora Dam on the Ganges River in India, attention was drawn to the problem, and the first serious effort was made to analyze accumulated experience and to establish a set of rules for the design of dam foundations on permeable strata. These rules were based on the assumption that the sole cause of piping was erosion along the surface of contact between the soil and the base of the dam. The path that a water particle followed along this surface was called the *line of creep*. If the length L of the line of creep was such that the average hydraulic gradient $i = h/L$ was less than a certain critical value for the foundation material, the dam was believed to be safe. The quantity,

$$C_c = \frac{L}{h_{cr}} \tag{59.1}$$

was called the *creep ratio*. The value h_{cr} represented the greatest height to which the water level in the reservoir could rise with reference to tailwater level without producing failure by piping. The available failure records indicated that the ratio C_c increased with increasing fineness of soil from about 4 for gravel to about 18 for fine sand and silt.

The first step in designing a dam on the basis of equation 59.1 was to estimate the creep ratio C_c of the subsoil. This was done by means of a table containing the values of C_c for the principal types of soil. The required length L of the creep line was then obtained by multiplying the creep ratio C_c by the hydraulic head h_{cr} created by the dam. The foundation was laid out in such a manner that the length of the creep line was at least equal to L. For example, the length of the line of creep for the dam shown in Figure 203 is

$$L = t_1 + t_2 + B + t_3 + t_4 = B + \sum t$$

and this distance must be at least as great as $C_c h_{cr}$.

During the next 30 years it was gradually recognized that vertical sections of the line of creep contribute more toward reducing the danger of piping than horizontal sections of equal length. The difference is due to the fact that the subsoil of dams is commonly of sedimentary origin, and

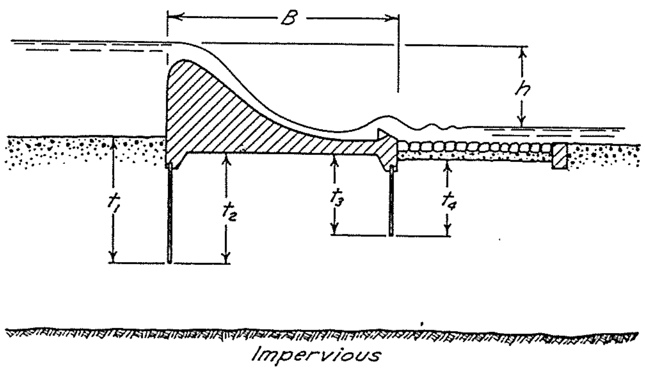

FIG. 203. Diagram indicating dimensions used for computation of length of line of creep.

sedimentary deposits are always much less permeable in the vertical direction than in the horizontal directions (see Article 11). If k_h and k_v are, respectively, the coefficients of permeability in the horizontal and vertical directions, the loss in head per unit of length of vertical sections of the line of creep is roughly equal to the ratio k_h/k_v times that of horizontal sections. The value of the ratio ranges between 2 or 3 and almost infinity, depending on the details of stratification and the importance of the variations of the permeability in the vertical direction.

To take account of the greater efficiency of vertical sections of the line of creep, the original procedure was modified by the assumption that every horizontal section of the line of creep was only one third as effective as a vertical section of the same length. On this assumption, the equation,

$$C_w = \frac{\frac{1}{3}B + \sum t}{h_{cr}} \qquad (59.2)$$

was obtained. The value C_w is known as the *weighted creep ratio*. Since

equation 59.2 corresponds approximately to the ratio $k_h/k_v = 3$, it is obvious that it does not take into account the wide range of values that this ratio can have in the field.

Table 27 is an abstract of a list of safe values for C_w, based on a digest of about 280 dam foundations of which 24 had failed.[59.1]

The line-of-creep approach to the problem is purely empirical. Like every other procedure based solely on statistical data, it leads to design with an unknown factor of safety. Experience and experiments have shown that the values of C_w, equation 59.2, are widely scattered from the statistical average for a given soil. The values of C_w contained in Table 27 represent maximum rather than average values, and the values of h_{cr}

TABLE 27

WEIGHTED CREEP VALUES C_w (EQ. 59.2)

Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5

From E. W. LANE, Security from Underseepage—Masonry Dams on Earth Foundations, *Trans. ASCE*, Vol 100 (1935), p 1257.

obtained by means of equation 59.2 and Table 27 represent the smallest heads at which piping ever occurred. Therefore, the wide scattering of the values of C_w from the statistical average implies that as a rule the factor of safety of dams designed on the basis of equation 59.2 and Table 27 is very high. The factor of safety of some of the dams must be excessive; that of others may be barely tolerable, and an unprecedented coincidence of several unfavorable circumstances may even lead to failure. Similar situations have been noted in the discussions of pile formulas, Articles 30 and 56, and of footing design on the basis of tabulated values for the allowable soil pressure, Article 54. Such situations call for theoretical and experimental investigations to supplement the existing empirical knowledge.

The theoretical evaluation of the factor of safety of dams with respect to piping is based exclusively on the theory of piping by heave, Article 40. To verify this theory the tests illustrated by Figure 204 were performed.[59.2] The weighted creep ratio for the finest sand used in the test was $C_w = 7$. The measured critical heads h_c at which piping occurred, the heads h_c' computed by means of the theory of piping, and the head

h_{cr} computed by means of equation 59.2 are given in Table 28. This table shows that the agreement between the values of h_c observed in the tests and those computed according to the theory of piping, Article 40, is very satisfactory, whereas the values h_{cr} are far too low. If the discharge area is covered with a heavy filter, as in tests *b* and *d*, design on the basis of equation 59.2 would appear to be grossly wasteful. However, it would be dangerous to base the design of a dam foundation on the results of the theory of piping and the laboratory tests without first considering the purely empirical aspects of the process.

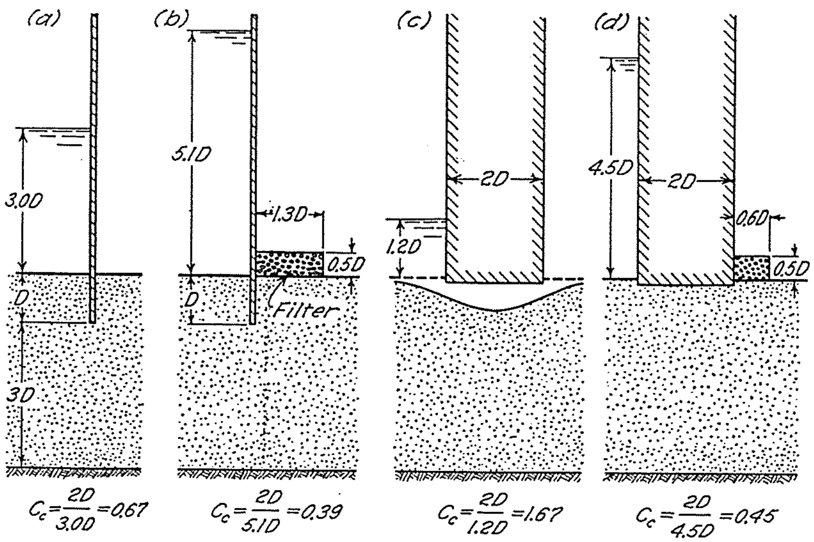

FIG. 204. Diagram showing results of laboratory experiments to determine critical head with respect to piping under different conditions. Corresponding values of creep ratio C_c indicated for each condition.

Both the theory and the tests lead to the conclusion that the factor of safety against piping by heave is practically independent of grain size. Furthermore, the conditions that determine the factor of safety with respect to failure by heave are independent of time. Hence, piping by heave should occur either during the first filling of the reservoir or else not at all. In striking contrast to these characteristics of piping failures by heave, construction experience leaves no doubt that the grain size does have a considerable influence on the critical head. Furthermore, the majority of piping failures have occurred several months or even years after the ill-fated dams were put into operation. Hence, it appears that most if not all piping failures of actual dams were caused by subsurface erosion and not by heave. The frequency of piping failures due to sub-

surface erosion is obviously due to the fact that all natural soil strata are more or less nonhomogeneous. When water percolates through such strata, it follows the most permeable zones, and it leaves the ground in the form of springs. If the discharge of a spring is great enough, and the soil conditions favor underground erosion, the spring may gradually create a tunnel by cutting backward along a line of maximum hydraulic gradient. As soon as the intake end of this natural tunnel arrives near the bottom of the reservoir, the water breaks into the tunnel, and the dam fails by piping.

TABLE 28.

COMPARISON BETWEEN MEASURED AND COMPUTED CRITICAL HEADS
TEST ARRANGEMENT SHOWN IN FIGURE 204

	Sheet Piles		Flat-Bottom Weir	
Test Arrangement	Test *a*, No Filter	Test *b*, with Filter	Test *c*, No Filter	Test *d*, with Filter
h_c observed in test	$3.0D$	$5.1D$	$1.2D$	$4.5D$
h_c' computed by theory, Article 40	$2.9D$	$4.8D$	$1.0D$	$4.6D$
h_{cr} computed by means of equation 59.2	$0.3D$	$0.3D$	$0.14D$	$0.14D$
Value of ratio h_c/h_c'	1.0	1.1	1.2	0.97
Value of ratio h_c/h_{cr}	10	17	8	32

Erosion tunnels with unsupported roofs are conceivable only in soils with at least a trace of cohesion, Article 33. The greater the cohesion, the wider are the spaces that can be bridged by the soil. In a general way, the cohesion of soils increases with decreasing grain size. Therefore, the danger of a piping failure due to subsurface erosion increases with decreasing grain size, and the corresponding values of the creep ratio also increase.

The head required to produce failure of a dam by piping due to subsurface erosion can be very much smaller than the critical head for piping by heave. Therefore, the foundation for a dam cannot safely be designed on the basis of the theory of piping by heave, Article 40, unless the possibility of a failure due to subsurface erosion is eliminated by covering all the areas where springs may develop with inverted filters, Article 11. The design of such filters requires thorough familiarity with all the circumstances attending subsurface erosion in the field.

SUBSURFACE EROSION

The destruction of dams by piping is usually so complete that the sequence of events can seldom be reconstructed. However, subsurface erosion can also be induced by careless pumping from open sumps or by natural events such as the tapping of bodies of ground water by the erosion of river banks. These processes commonly leave evidence that remains open to inspection. Therefore, they constitute the principal sources of our knowledge of the characteristics of subsurface erosion. The following paragraphs contain abstracts of the records of pertinent observations.

Figure 205 represents a cross section through a gently inclined blanket of gravel that rests on a deep bed of very fine uniform loose sand. At A a pit was dug for the foundation of a new machine. Although the pit was surrounded by sheet piles that extended to a considerable depth below

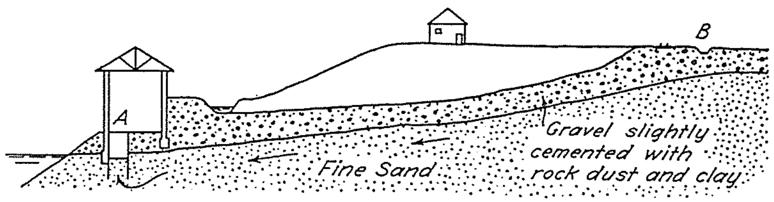

Fig. 205. Diagram illustrating underground erosion produced by pumping mixture of sand and water from sump A. Sinkhole at B 300 ft distant from A.

final grade, the pump discharged a mixture of sand and water. The quantity of sand removed was far in excess of the volume of the pit. Before final grade was reached, the building collapsed. At the same time a sink hole, 3 ft deep and 20 ft in diameter appeared at B, at a distance 300 ft from the pit. Between A and B the ground surface was intact. Hence, the loss of ground can be accounted for only by soil transportation in a relatively narrow subterranean conduit. It is most likely that the conduit was located immediately below the gravel blanket, because the slightly cemented gravel was capable of forming an unsupported roof.

In the Rhineland pumping was kept up for 13 years in a sand pit. The bottom of the pit was located between 16 and 20 ft below the original water table. During this period three of the springs that discharged into the sump cut backward and eroded tunnels in the slightly cohesive sand. Each tunnel terminated in a sink hole on the ground surface. The largest tunnel was 3 to 6 ft wide and in its length of 170 ft had an average grade of only 6 per cent. The sink hole above the end of this tunnel was 8 ft deep and 35 ft in diameter.

In another instance an open cut was excavated for the construction of a sewer. The excavation passed through fairly stiff clay into fine sand that was drained by pumping from an open sump. While pumping proceeded,

a narrow strip of the ground surface subsided about 1 ft. The formation of the trough started at the sump and gradually proceeded to a distance of about 600 ft. The width of the trough increased from a few feet at the sump to more than 10 ft at the farther end.

Examples of underground erosion due to natural causes are also not uncommon. On the east bank of the Mississippi River near Memphis a large-scale subsidence occurred after the high water of 1927. At this location the river bank rises in a bluff about 100 ft high. Without any warning a strip at the top of the bluff about 700 ft long and 100 ft wide started to subside at the rate of 1 ft per hr. The pavement that covered

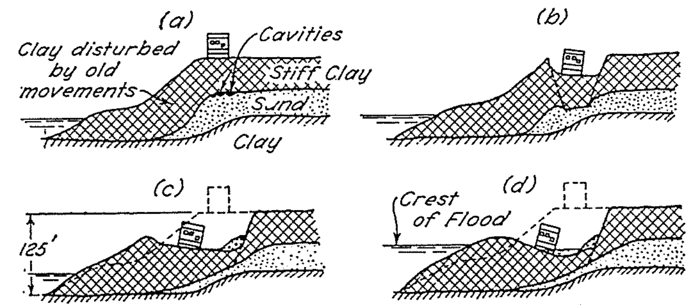

Fig. 206. Diagrams illustrating large-scale subsidence due to underground erosion. (a) Incipient state; (b), (c), and (d) subsidence after 24 hr, two months, and one year, respectively.

the ground surface remained horizontal and fairly intact for a period of about 30 hr. During the following two months the subsidence increased to as much as 60 ft, and the subsided surface broke up, as shown in Figure 206. The trough-like depression was caused by the failure of the roof above the intake section of an underground sand flow.[59.3]

Although the piping phenomena described in the preceding paragraphs took place in very different soil formations, they all had one important feature in common. The subsidence of the roof always occurred at a great distance from the discharge end of the tunnel. This fact indicates that the erosive capacity of a spring increases as the length of the tunnel increases. The reason is illustrated by the flow nets in Figure 207. The thin dash curves indicate equipotential lines, or contour lines of the water table, whereas the solid curves represent the flow lines. The dash-dotted lines indicate the boundary of the intake area. With increasing length of the tunnel, the number of diverted flow lines increases. Thus, the discharge from the spring becomes greater, and the rate of erosion increases.

Progressive subsurface erosion starting at springs near the toe of a dam also proceeds as shown in Figure 207, along lines leading toward the reservoir. The frequent occurrence of springs at the downstream edge is known to everyone who has had experience with dams. If a spring is powerful enough to start erosion in the first place, the erosion will almost certainly become more serious as time goes on, because the flow from a given spring increases with the length of the eroded tunnel, Figure 207. Finally, the dam will fail by piping.

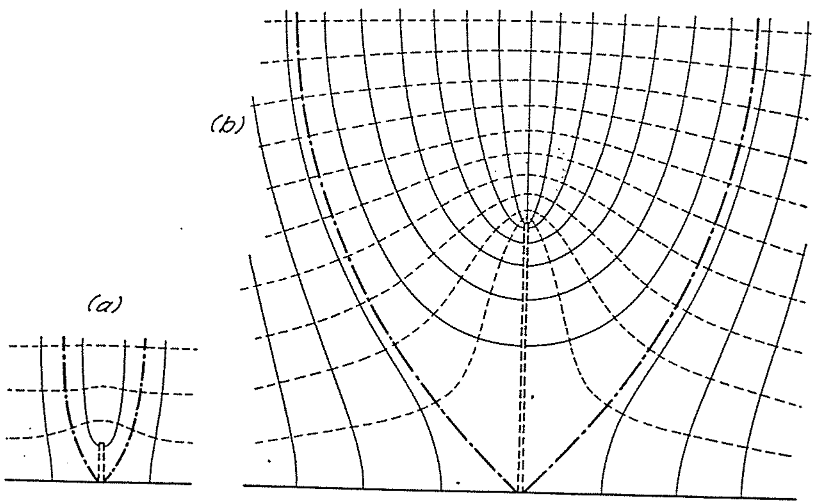

FIG. 207. Flow nets illustrating increase of intake area of spring as length of eroded channel increases. (a) Incipient state; (b) after erosion has proceeded to considerable distance from spring.

MEANS FOR AVOIDING PIPING

In discussing the means for avoiding piping we must make a distinction between small and large jobs. A similar distinction was made between small and large retaining walls and between shallow and deep cuts.

The design of short and low dams is a routine procedure, because the structures are not important enough to justify elaborate preliminary investigations. Dams of this category are protected against piping by making the designs in accordance with the line-of-creep rule expressed by equation 59.2.

A dam designed on the basis of equation 59.2 will be safe, unless poor design or construction combine with exceptionally unfavorable foundation conditions. In addition to compliance with equation 59.2, sound engineering merely requires the avoidance of an unnecessary concentration of flow lines beneath unprotected areas on the downstream side of the dam. The consequences of disregarding this fundamental requirement are illustrated in Figure 208a, which represents a section through Hauser Lake Dam in Montana. The subsoil consisted of 66 ft of gravel. The water was retained by a reinforced-concrete skin supported by a steel framework that rested on large footings. The presence of the footings produced a local concentration of flow lines, as shown in the figure. The dam failed in 1908, one year after the first filling. Since it did not fail immediately, the cause was undoubtedly spring erosion. A second example is shown in Figure 208b, which represents a section through a dam across the Elwha River in Washington. The structure rested on gravel and coarse sand underlain by bedrock. While the reservoir was

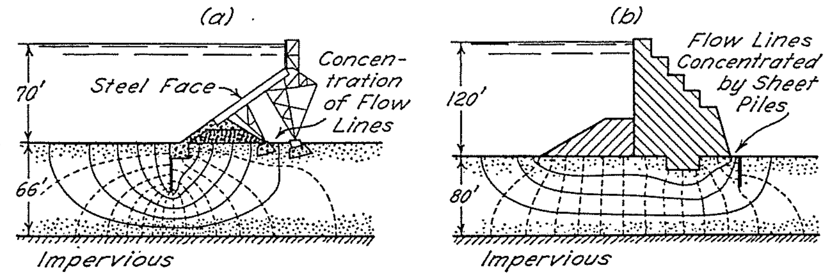

FIG. 208. Flow nets showing concentration of flow lines responsible for failure by piping of two dams; (a) Hauser Lake Dam, Mont. (b) Elwha River Dam, Wash.

being filled, large springs developed at the downstream toe. In order to reduce the flow, a row of sheet piles was driven to a depth between 30 and 40 ft, at a distance of 8 ft from the toe. This obstruction caused a concentration of flow lines, as shown in the figure, and subsurface erosion occurred. The dam failed before the sheet-pile wall was completed.

Routine design on the basis of equation 59.2 is adequately safe provided the most primitive precautions are taken to avoid local concentration of flow lines. However, when practiced in connection with large dams it is certain to be uneconomical. In order to depart without risk from the routine method of design it is necessary, first of all, to make a thorough soil exploration, including the preparation of permeability profiles such as the one shown in Figure 201a. These profiles furnish the data required for establishing a working hypothesis regarding the trend of the flow of seepage out of the reservoir. All those areas where subsurface erosion may conceivably start must be covered by graded inverted filters. The presence of the filters prevents even incipient erosion at all points of the protected area and increases the critical head from the value required to produce erosion to the much larger value required to produce failure by heave. Rules for the design of filters are given in Article 11.

Seepage computations must always be based on simplifying assumptions regarding the permeability pattern of the subsoil, and the difference between the predicted and the real flow of seepage can be very great, irrespective of the thoroughness with which the subsoil has been explored.[59.2] Therefore, it is necessary to find out by means of observation wells whether and to what extent the theoretical and the actual flow of seepage are in agreement. If the observations disclose a strong flow of seepage toward unprotected areas, these areas must also be covered with filters, or else the seepage must be diverted into filter wells or drainage tunnels. Experience has shown that the hydraulic pressure conditions in the subsoil of storage dams may change progressively for many years after construction.[59.4] Hence, supervision of these conditions must be continued until the effects of the fluctuations of the water level in the reservoir become reversible.

Examples of Filter Protection

The rock-fill dam shown in Figure 199*a* rests on a stratum of sand and cemented sand with an erratic permeability profile. Only the middle part of the cutoff wall extends down to the impermeable base, and the impounded water enters the subsoil of the dam by flowing beneath the shallower side portions of the wall. Therefore, springs could emerge at almost any point of the base. After construction the base of the dam would be inaccessible, and subsurface erosion could proceed without being noticed. To eliminate this danger the entire base of the dam except at the two ends was covered with an inverted filter that occupies an area of about 400,000 sq ft. The water that enters the filter is collected in large-diameter open-joint drain pipes that discharge into an open drainage ditch following the toe of the rock fill. The soil conditions are such that clogging of the filter is almost unconceivable. Yet, even if it should occur, it would be without serious practical consequence, because the only function of the filter is to prevent soil particles from being washed into the interstices of the rock fill. Even a completely clogged filter would serve this purpose. Any spring that might develop at a later date beyond the boundaries of the protected area would be located outside the area occupied by the rock fill. It would be plainly visible, and underground erosion by the water vein feeding the spring could easily be stopped by means of a filter plug while erosion was still in an incipient state.

Piping beneath masonry dams is most likely to start just below the downstream toe (see Figure 208*b*). Therefore, this region should be protected by a filter. However, if the dam is of the overflow type, solid matter carried by floods may clog the filter. In such instances it may be

preferable to install the filter beneath the middle of the dam, as shown in Figure 209. This dam, of the bear-trap type, rests on fine sand containing some silt and streaks and layers of gravel. The seepage water drains from the filter into a drain pipe that is embedded in the concrete and discharges into the tail water. According to Table 27, a dam on such soil should have a weighted creep ratio at least equal to 6 or 7. The ratio for the dam as designed and built is only 4.0. Yet, in spite of the low creep

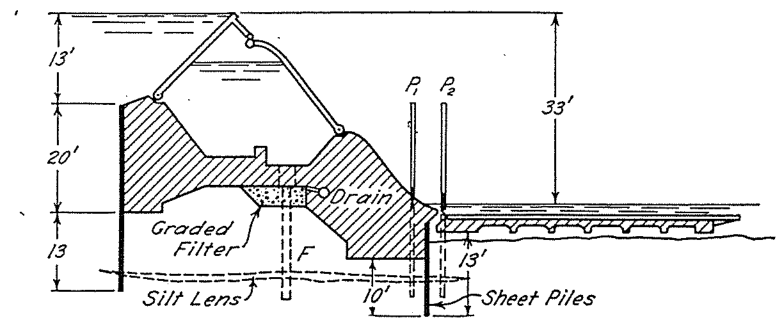

FIG. 209. Overflow dam of bear-trap type with graded filter beneath body of dam. If piezometric observations indicate ineffectiveness of filter due to obstruction of flow by silt or clay seams, bleeder wells *F* are required.

ratio, the dam satisfies all legitimate safety requirements, because the graded filter shown in the figure excludes the possibility of failure due to underground erosion.

Figure 209 also demonstrates the necessity for ascertaining the piezometric levels in the subsoil of the dam at least during the first filling of the reservoir. The design of the filter shown in the figure and the estimate of the factor of safety of the dam with respect to piping were based on the assumption that the subsoil is more or less homogeneous. This assumption seemed justified on the basis of the results of the test borings. However, the sand that constitutes the subsoil might have contained a few undetected thin layers of silt or clay. Discontinuous layers of this kind are harmless, but, if one of them is continuous over the entire area between the upper and lower row of sheet piles, as indicated by the thin dash lines in Figure 209, its presence has two very detrimental consequences. It considerably reduces the effective length of the line of creep, and, in addition, it prevents the flow of seepage toward the filter. Therefore, it is necessary to provide the concrete floor located above the filter area with plugged holes and, during the first filling of the reservoir, to observe the water level in piezometric tubes such as P_1 and P_2. If the water level in these tubes remains close to the tail water, it can be assumed that the filter serves its purpose. On the other hand, if the water level rises perceptibly when the level in the reservoir goes up, the efficacy of the filter is doubtful, and it becomes necessary to tap the permeable soil located below the lower edge of the sheet piles by means of filter wells F. It is very unlikely that such a necessity will arise. However, failures due to piping also occur without being anticipated, and sound engineering requires the elimination of even remote possibilities of failure.

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CHAPTER X

SETTLEMENT DUE TO EXCEPTIONAL CAUSES

ART. 60. SETTLEMENT DUE TO CONSTRUCTION OPERATIONS

EXTRANEOUS CAUSES OF SETTLEMENT

In Chapter IX we have discussed the settlement of buildings and other structures under the influence of their own weight. Although this is the most common type of settlement, other types are important enough to deserve consideration. They include settlement due to increasing the load on the surrounding soil, to excavation in the vicinity, to lowering the ground water table, and to vibrations. In this article we shall consider only the first two categories.

SETTLEMENT DUE TO INCREASING LOAD ON SURROUNDING SOIL

The application of a load to one portion of the ground surface above any type of soil causes the surface of the adjacent soil to tilt (see Figure 210a). The distance within which the tilt is of any practical importance depends, however, on the soil profile as well as the dimensions of the loaded area. If the subsoil contains soft clay, the magnitude and distribution of the settlement can be roughly estimated on the basis of the results of soil tests. If the subsoil is sand, the settlement cannot be computed and estimates can be based only on the records of precedents.

If rafts on sand are designed in accordance with the rules contained in municipal building codes, they are likely to settle as much as 2 in. Exceptionally, they may settle even more (see Article 55). Since the greatest part of this settlement occurs during construction, the structure itself will not be damaged unless it is very sensitive. However, the tilt of the adjoining ground surface toward the loaded area may be great enough to damage neighboring structures. In New York, for example, a 20-story building was constructed on a lot between two 7-story buildings supported by spread footings on a deposit of fine sand. The new building rested on a raft at a depth of 20 ft below the ground surface. The soil pressure was 2 tons per sq ft in excess of the weight of soil removed. Since the building itself settled only 1.8 in., and the settlement was fairly uniform, the building remained intact. Yet, the neighboring buildings

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INVESTIGATION OF CHEMICAL CLOGGING OF DRAINS AT INCO'S CENTRAL AREA TAILINGS DAMS

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INVESTIGATION OF CHEMICAL CLOGGING OF DRAINS AT INCO'S CENTRAL AREA TAILINGS DAMS

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ABSTRACT: The clogging of toe drains with chemical precipitates has been a historical problem at Inco's Central Tailings Area tailings dams. Site investigations were carried out to assess the characteristics of the tailings seepage water and to determine the extent of the drain clogging in various types of drain materials. The results of the work showed the principal factor in the formation of precipitates is the oxidation of the iron-rich seepage water as it emerges from the dam and is exposed to oxygen. Infiltration of oxygen through the porous drain materials can also trigger the formation of precipitates at the contact of the air and the water table inside the dam. This process is self-limiting because the accumulation of chemical precipitates at the air-water interface eventually acts an effective oxygen barrier. The observational evidence gathered at the Inco site also showed that chemical precipitates do not form in saturated soil conditions below the water table. Hence, submergence of drain materials to preclude oxygen is one means to prevent clogging.

1 INTRODUCTION

Inco's Central Tailings Area near Sudbury, Ontario covers an area of about 5,500 acres. Tailings have been continuously deposited in the impoundment since 1936 and future tailings deposition is planned for at least another 30 years. The present Central Tailings Area is comprised of six non-operating impoundments (termed the A, C-D, M, M1, P and Q Areas), and an operating impoundment (R Area).

The tailings deposited within the impoundments contain on average 6% by weight of total sulphides, predominantly as pyrrhotite. Oxidation of the tailings in the vadose zone of the impoundments results in acidic seepage conditions at the toe of the tailings dams. Seepage from the tailings dams is collected and pumped back into the tailings ponds or pumped to a waste water treatment plant. It is predicted that sulphide oxidation reactions may persist for up to 400 years depending upon the thickness of the vadose zone formed in the impoundments and the sulphide content of the tailings (Coggan, 1992). Collection and treatment of seepage water may be required for a similar period under current operating conditions.

A concern for the long-term performance of the Central Tailings Area tailings dams is the extensive formation of ferric hydroxide precipitates which have lead to partial clogging of the toe drains within the dams. It is conceivable that growth of the precipitates over time could completely plug the toe drains and cause the phreatic levels in the dams to rise. This would reduce the overall stability of the dams and may lead to problems of erosion of the dam slopes.

In 1994, field and laboratory studies were carried out to investigate the cause of the chemical precipitates and to determine the extent of the drain clogging in various types of drain materials. The studies included sampling and geochemical testing of the pond and seepage waters to determine their properties and composition; test pits and drill holes to investigate the extent of the precipitate formations; and chemical testing of the precipitates to determine their composition. Based on this work, a model for the tailings seepage water geochemistry and precipitate formations was developed for the Inco site.

This paper presents the results of the 1994 field and laboratory studies and discusses the findings and conclusions derived from the work.

2 DESCRIPTION OF TAILINGS DAMS

The tailings area are impounded by tailings dams constructed using the upstream construction method. In each case, the dams were started by construction of a 30 ft high starter dam. Tailings were then spigotted from the crest of the starter dams and the dams raised by upstream construction methods at 5H:1V slopes to heights of up to 160 ft.

For the older non-operating tailings areas, the starter dams were constructed out of locally borrowed soils which ranged from impervious clay to semi-pervious silty sand and gravel. Problems with erosion of the exterior slopes of the starter dams from surface water runoff and dam seepage were encountered at most of these dams during the early years of operation. The slopes were subsequently dressed with a 10 ft to 20 ft thick drainage blanket of smelter slag in an attempt to prevent further erosion. The starter dams for the more recent R Area were built out of pervious rockfill to overcome these problems.

Figure 1 shows the plan and section of the Rock Dam located in the P Area. This dam is a typical example of a dam in the non-operating tailings areas and was extensively investigated in this study. The dam was started in the early 1960's and was completed in 1984, 4 years before the closure of the P Area in 1988. A drainage blanket of slag was placed on the slope of the starter dam in the late 1960's. After closure, the surface of the tailings slope above the starter dam was capped with 1 ft of compacted clay to minimize infiltration of surface water into the oxidized vadose zone and to stabilize the slope against erosion. The surface of the cap has been reclaimed with vegetation comprised principally of grasses. Seepage from the dam is collected in a seepage collection pond where it is pumped to a waste water treatment plant.

3 FIELD AND LABORATORY INVESTIGATIONS

Field and laboratory investigations were conducted at three selected dams in the non-operating P Area to provide additional information on the tailings geochemistry, and to study the nature and extent of the precipitates formed in the tailings dams. For comparison, six dams in the currently operating R Area were also investigated. The procedures of the investigations are described in the following sections. The results of the investigative work are discussed in Section 4.

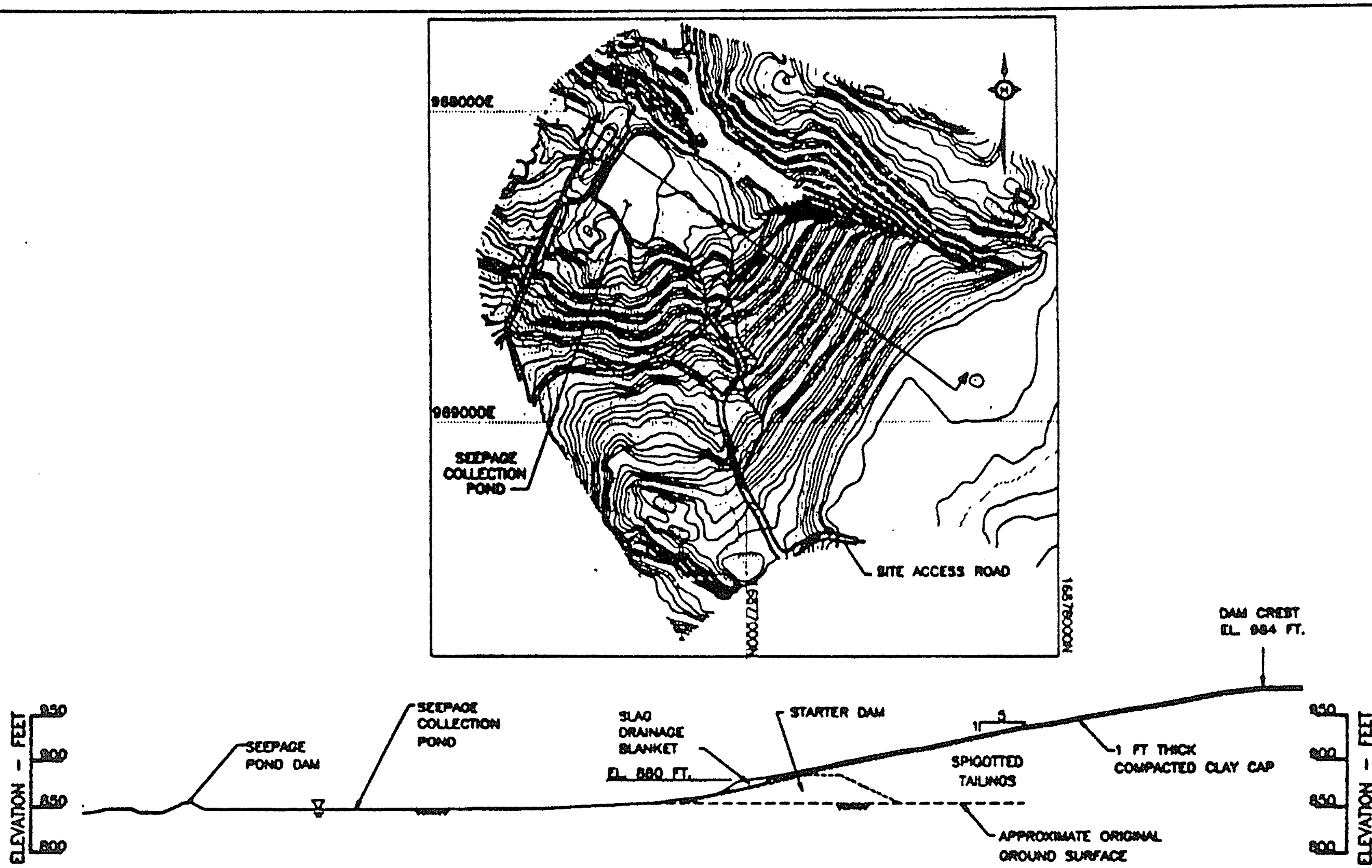


FIGURE 1 PLAN AND TYPICAL SECTION OF ROCK DAM

3.1 *Water Sampling and Testing*

The seepage water emanating from the toe of six tailings dams in the R Area (Dams 1, 3, 6, 11, 12 and 16) and three tailings dams in the P Area (Whissel, Rock and Pistol Dams) were sampled for chemical analysis. Water samples were taken directly at the point of exit from the toe of the dams and, where possible, at several distances (up to 20 m) along the flow path away from the dam. The water in the seepage collections ponds at each dam was also sampled. In addition, the tailings ponds formed in the R and P Areas were sampled. Pond water samples were also taken from the M Area which is another non-operating impoundment.

At each sampling site, field measurements of temperature (°C), conductivity (μS), pH and redox potential (mV) were taken using a portable water tester. At selected sites, 1 litre samples of the water were taken and submitted to Inco's analytical laboratory for testing. The samples were kept chilled in the field in an attempt to preserve the samples. However, some change in water chemistry, principally iron oxidation, occurred prior to testing as evidenced by a drop in pH. The tests conducted by Inco consisted of temperature, conductivity, pH and redox potential; ICP (inductively coupled plasma) scans of the samples to determine metal concentrations; and gravimetric titration to determine concentrations of Fe^{2+} and SO_4^{2-}.

3.2 *Test Pits and Drill Holes*

Test pits and drill holes were carried out at the toe of the Pistol and Rock Dams to investigate the presence of precipitates within the slag toe drain and underlying starter dam fill. In addition, a 90° V-notch weir was installed downstream of the Rock Dam to measure the flow of surface seepage to the seepage collection pond and compare it to historical flow measurements in the 1970's.

The soil stratigraphy and presence of precipitates in the test pits and soil samples from the drillholes were carefully logged and photographed. Samples of the soils were recovered for laboratory water content determination and gradation testing. Where present, samples of the various precipitates were recovered and submitted to Inco for geochemical testing. The tests performed included ICP metal scans for various metal concentrations and titration of Ca and S. Samples of the seepage water from the test pits were also taken for geochemical testing.

4 DISCUSSION OF RESULTS

4.1 *Tailings Water Chemistry*

Figure 2 presents histograms of the pH measurements for the pond water, toe seepages and seepage collection pond water. The data is divided into the R Area dams which have rock fill starter dams and the P Area dams which have starter dams constructed of locally borrowed native soil. Pond water measurements for the M Area are also given. The main observations from these comparisons are as follows:

1. The pH of the non-operating M and P Area ponds were 6.6 and 5.5, respectively. The pH of the R Area pond was between 6.4 and 5.8. There was no clear correlation between pH and the age and activity of the tailings areas.

2. The pH of the toe seepages from the P Area dams were typically between 6 and 7 which is higher than the pH of 5.5 for the pond water. This result indicates that

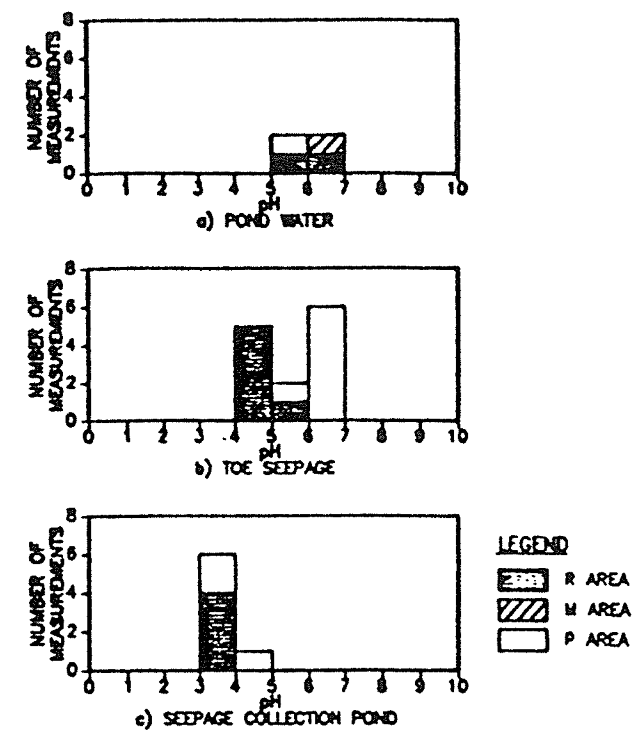


FIGURE 2 HISTOGRAMS OF pH MEASUREMENTS

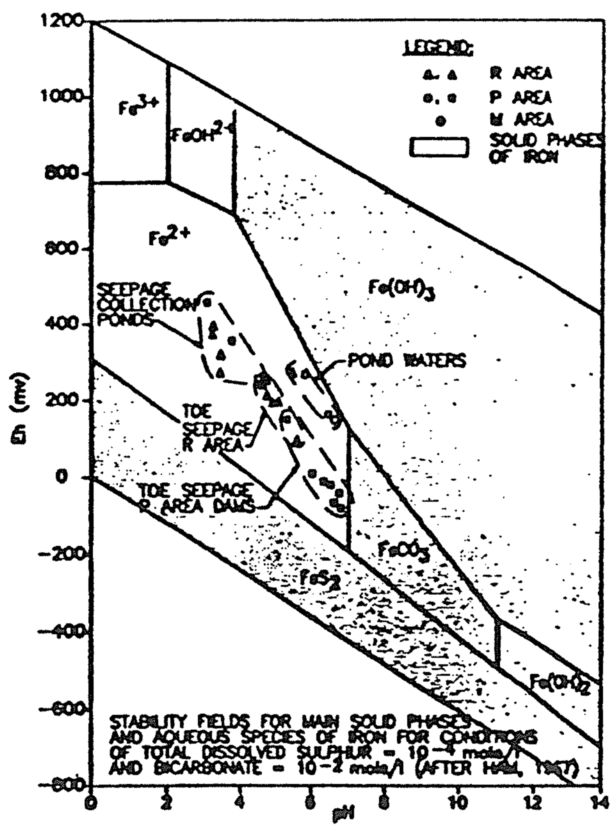


FIGURE 3 Eh–pH DIAGRAM

some pH buffering of the seepage water occurs as it passes through the tailings. The near neutral pH toe seepage indicates that oxidation of iron in the water does not occur to any significant extent prior to its exit as seepage from the dam.

3. The pH of toe seepage from the R Area dams is typically between 4 and 5 which is lower than the pHs of 5.8 and 6.4 for the pond waters. The lower pH of the toe seepage suggests that some oxidation of ferrous iron and precipitation of ferric iron occurs before the seepage exits the dam toe. This oxidation is attributed to the access of oxygen through the coarse porous rock fill of the starter dams. This oxidation of iron has resulted in the formation of precipitate coatings on the surfaces of the rock fill.

4. The pHs of the seepage collection ponds located downstream of the tailings dams are typically between 3 and 4 for both the P and R Areas. The reduced pH conditions are attributed to the oxidation of iron in the seepage water after it exits the dams and is detained in the collection ponds for several weeks.

Figure 3 presents the field chemistry data in terms of an Eh-pH diagram. The approximate stability fields for the main solid phases and aqueous phases of iron in water are also indicated for an iron concentration of 10^{-5} mole/litre. The principal aqueous phase of iron is indicated to be soluble ferrous iron, Fe^{2+}. Theoretically, the data should plot at or just below the boundary between the soluble Fe^{2+} and the insoluble ferric hydroxide $Fe(OH)_3$. The position of the pond water plots higher in the Eh-pH diagram because the concentration of dissolved iron is very low (0.8 to 7 mg/ℓ) as compared to the seepage water (129 to 1522 mg/ℓ). Higher dissolved iron shifts the boundary of the solid $Fe(OH)_3$ phase further to the left. The higher Fe^{2+} concentrations in the tailings seepage are produced by the sulphide-oxidation reactions in the tailings.

The Eh-pH diagram illustrates the progressive oxidation of iron in the tailings seepage water. The presence of oxygen increases the redox potential which moves the state vertically upwards into the solid phase of the diagram. The oxidation of the dissolved Fe^{2+} and precipitation of $Fe(OH)_3$ reduces the pH of the water by removing dissolved hydroxide alkalinity and brings the state horizontally back across to the boundary between the soluble and solid phases. In this manner, the status of the water chemistry incrementally "notches" its way up the boundary between the soluble and solid phases. The formation of $Fe(OH)_3$ produces the abundant precipitates that coat the ground at the toe of the dams.

The tailings water chemistry indicates that oxidation of iron in the seepage water occurs earlier at the R Area dams than in the P Area dams because of access of air into the body of starter dams through the porous rock fill. In comparison, relatively little oxidation occurs within the soil starter dams in the P Area which are finer grained.

4.2 *Precipitate Formations*

An abundance of precipitates, up to 6 inches in thickness, covered the ground at the toe of all the P Area tailings dams. The surface of the precipitates generally consisted of reddish oxide deposits. These formed a soft to hard crust up to about ½ inch in thickness. These precipitates were underlain by soft yellow deposits. Test pits and drill holes were carried out at the Pistol and Rock Dams to investigate the penetration of the precipitates into the body of the dam. Sections showing the soil stratigraphy and zones of observed precipitates are given on Figure 4 and Figure 5. The observations from this work are as follows:

4.2.1 *Pistol Dam*

The soils encountered at the dam toe consist of slag overlying the native silty clay foundation soils. Some intermixing of the slag and silty clay had occurred during placement. Seepage through the slag was encountered in the bottom 2 to 4 inches of the slag, just above the foundation contact.

The upper 1 ft of slag below the ground surface was found to be weathered and intermixed with tailings and clay eroded from the surface cap on the dam slopes. The weathering and interactions between the slag, tailings and clay has resulted in cementation of the slag. This cementation ranged from slight to a hard "concrete" condition. The slag below the surface crust was found to be dry, unweathered and resembled freshly placed slag. This slag was cohesionless and tended to run into the test pits while it was being excavated.

Typically, 1 ft to 2 ft of precipitates were encountered immediately above the zone of seepage at the base of the slag. The precipitates consisted of about 2 to 12 inches of soft to hard red-oxide deposits and coatings on the slag particles, underlain by about 6 to 12 inches of slag plugged with soft, fluid-like, yellow precipitates. The yellow precipitates completely saturated the pore spaces of the slag, thereby reducing the permeability of the slag and acting as a barrier to air diffusion.

Water was used to wash the yellow precipitates off the surface of the slag particles. The surfaces of the slag were hard and fresh, and did not appear to have been chemically altered by contact with the seepage water.

Seepage through the slag exited through the window between the overlying yellow precipitates and the underlying silty foundation clay. Reddish precipitates were encountered within the seepage water and on the slag particles. The precipitates filled about 20% of the pore space and did not appear to impede the flow rate of seepage. It is believed that some of the precipitates present in the seepage water had been dislodged during excavation of the test pits.

4.2.2 *Rock Dam*

The soils encountered at the dam toe consisted of a blanket layer of slag placed over sand, and sand and gravel which was used to construct the starter dam. The upper 1 ft of the slag was found be weathered, but cementation of the slag was not as pronounced as observed at the Pistol Dam.

The water table through the lower slope of the tailings dam, as determined from the test pits, drilling and standpipes, exits the toe of the dam through the slag placed over the dam slope. The water table extends back through the starter dam fill materials and into the tailings. The depth of the water table below the tailings slope is 10 to 14 ft. Evidence of oxidation of the tailings and starter dam fill materials, as noted by the reddish-brown colour, was observed above the water table.

As for the Pistol Dam, 1 ft to 2 ft of precipitates were encountered immediately above the water table through the slag and starter dam materials. Within the slag, the precipitates consisted of about 4 inches of soft to hard red-oxide deposits and coatings on the slag particles, underlain by about 6 to 12 inches of slag plugged with soft, fluid-like, yellow precipitates. Within the starter dam materials, only yellow

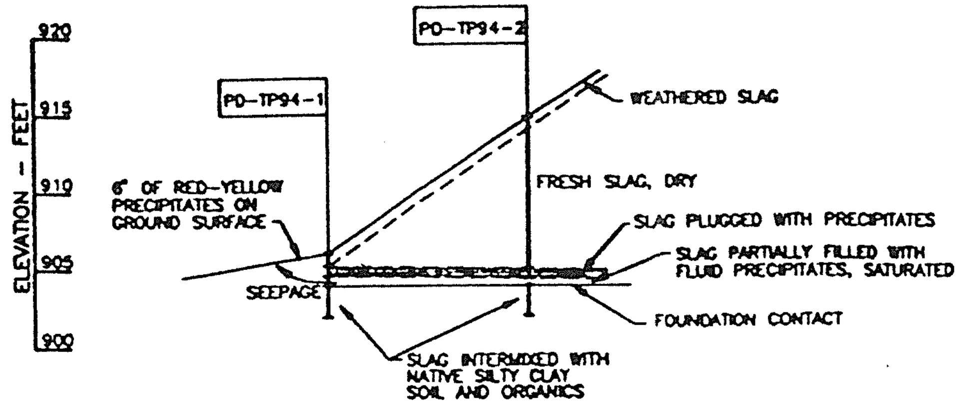


FIGURE 4 SECTION OF PISTOL DAM SHOWING OBSERVED PRECIPITATES

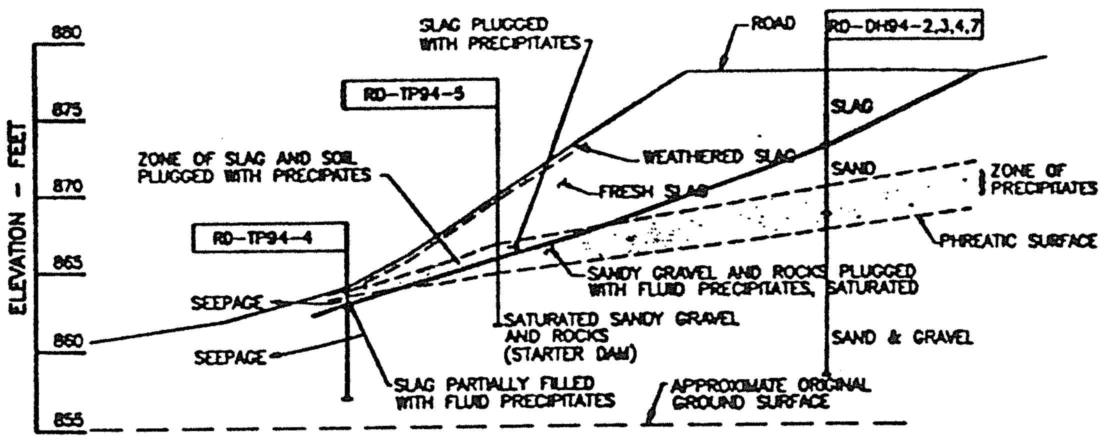


FIGURE 5 SECTION OF ROCK DAM SHOWING OBSERVED PRECIPITATES

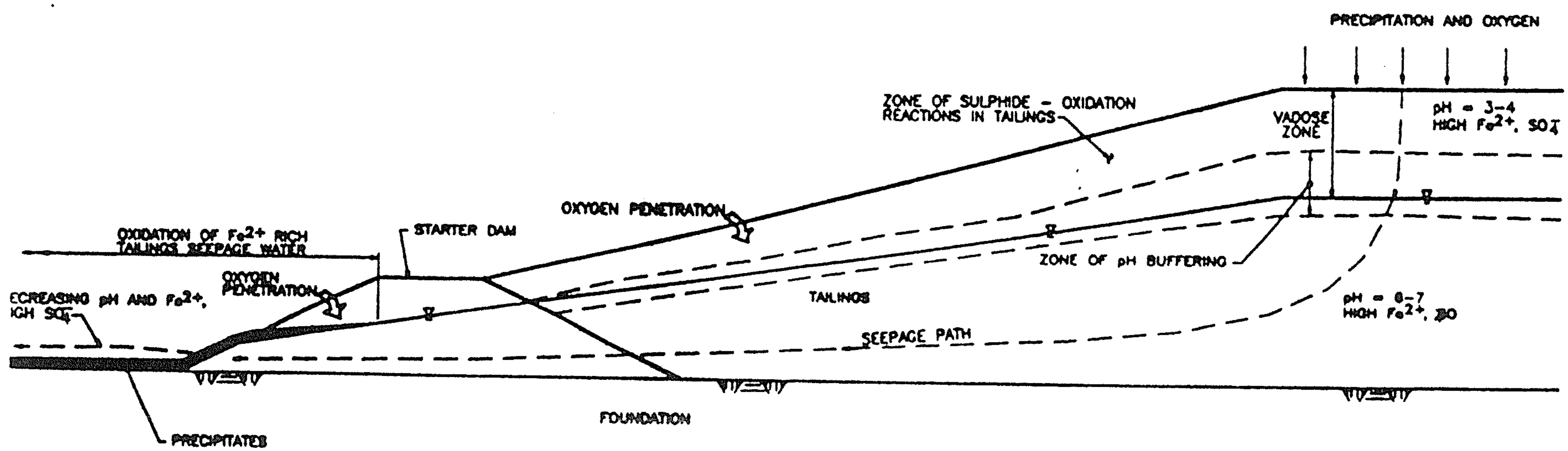


FIGURE 6. SCHEMATIC MODEL OF TAILINGS SEEPAGE WATER CHEMISTRY AND PRECIPITATE FORMATION

precipitates were encountered. The yellow precipitates completely saturated the pore spaces of the slag and starter dam materials, thereby reducing the permeability and acting as a barrier to air diffusion. In contrast, there were no precipitates present and no evidence of oxidation of the starter dam materials below the water table.

A test hole was drilled at the mid-slope of the tailings dam to investigate whether any precipitates were present in the tailings. The drilling indicated the tailings to be highly oxidized to a depth of about 5 ft, with the degree of oxidation decreasing with depth to the water table at 14 ft. No visible evidence of precipitate formation in the tailings materials was observed.

The seepage flow rate measured by the weir installed downstream of the dam is compared in Table 1 with previous seepage measurements recorded in 1970s. The previous measurements were also obtained using weirs. The seepage flow rates are remarkably similar and indicate that the precipitates formed in the body of the starter dam have not blocked the seepage from exiting the toe of the dam.

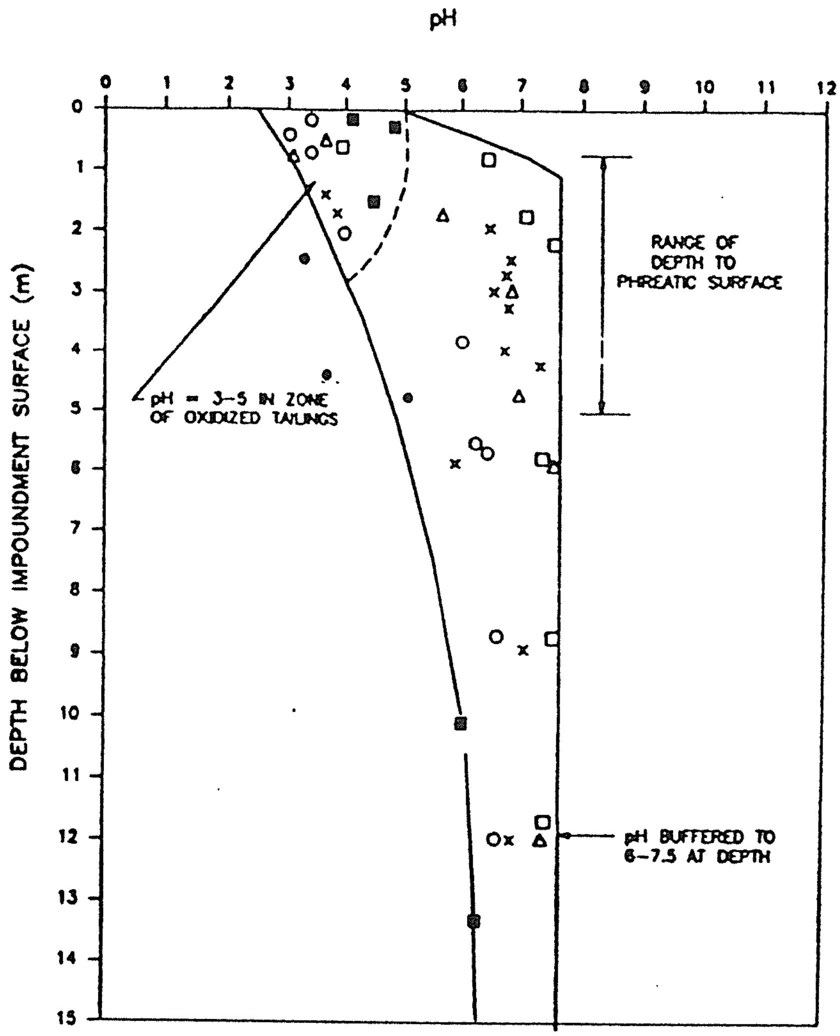
Table 1 Surface Seepage Flow Rates at Rock Dam

DATE	WEIR FLOW MEASUREMENT (Igpm)
September 1970	80
October 1972	55
June 1973	50
March - April 1974	56
December 1994	60

4.3 *Precipitate Chemistry*

Samples of the precipitates recovered from the Pistol and Rock Dams were tested by Inco to determine their elemental composition. Tables 2 and 3 summarize the percentage content of major elements in the hard, red precipitates and the yellow, fluid precipitates observed within the slag at the Pistol and Rock Dams.

The red precipitates are principally iron oxides, as evident by the high Fe content of 40%. Other secondary oxides are present in smaller amounts including CaO, Al_2O_3 and MgO. A significant amount of SiO_2 (16% - 20%) is present in the two samples tested. Generally, SiO_2 content varied widely amongst the samples tested and it is believed that the SiO_2 represents mineral soil that was scraped off the host slag along with the precipitates. An X-ray diffraction test conducted by Inco on one sample of the red precipitates showed that the major constituent is iron oxide hydroxide, FeO (OH).



LEGEND

	SAMPLE LOCATION	TAILINGS AREA
●	IN4	C-D
■	IN1	C-D
O	IN13	M1
□	IN10	M1
Δ	IN11	M1
x	IN12	M

REFERENCE: COGGAN (1992)

FIGURE 7 pH BUFFERING OF TAILINGS WATER

Table 2 Major Elements of Red Precipitates

DAM	TEST PIT NO.	SAMPLE NO.	SAMPLE DESCRIPTION	Fe (%)	S (%)	K (%)	CaO (%)	Al_2O_3 (%)	MgO (%)	SiO_2 (%)
Pistol	PD-TP94-3	Sa-1	Slag with red oxides on surfaces	39.4	0.9	0.4	1.7	4.6	1.3	22.0
Rock	PD-TP94-4	Sa-1	Slag with red oxides on surface	39.6	2.6	0.3	1.0	4.5	0.8	16.6

Table 3 Major Elements of Yellow Precipitates

DAM	TEST PIT NO.	SAMPLE NO.	SAMPLE DESCRIPTION	Fe (%)	S (%)	K (%)	CaO (%)	Al_2O_3 (%)	MgO (%)	SiO_2 (%)
Pistol	Ground Surface	-	Yellow precipitates	35.0	9.4	4.7	0.4	0.8	0.3	3.5
Pistol	RD-TP94-1	Sa-3	Slag with yellow precipitates	34.9	7.3	4.0	0.8	2.0	0.8	9.9
Pistol	RD-TP94-2	Sa-1	Slag with yellow precipitates	35.7	4.3	2.1	0.9	3.5	0.9	20.7
Rock	RD-TP94-4	Sa-2	Slag with yellow precipitates	35.0	7.9	4.0	0.2	1.5	0.5	7.3

The major constituents of the soft yellow precipitates are Fe followed by S (Sulphur) and K (Potassium). In comparison with the red oxide precipitates, there are very little secondary oxides present. The ratios of S and K, and the high iron content indicate that these precipitates are mainly composed of K-Jarosite, $KFe_3(SO_4)_2(OH)_6$. X-ray diffraction testing conducted by Inco on one of the precipitate samples confirmed the identification of the yellow precipitates as K-Jarosite. Evidence of K-Jarosite in the Inco tailings was also found by Coggans (1992) in oxidized tailings and within hardpan layers.

Upon review of the above results, it was concluded that the precipitates observed in the slag above the water table initially formed as K-Jarosites. The red oxides at the top of the precipitate zone subsequently formed by alteration of the K-Jarosite into secondary oxide components. These oxides form a hardpan layer which caps the zone of precipitation. It is considered possible that a portion of the red oxides could have been formed immediately after placement of the slag on the dams, by leaching of the slag in contact with the acidic tailings seepage. The leaching would have ceased once a sufficient layer of K-Jarosite had been deposited to form an oxygen barrier, such that oxygen contact with the seepage water and continued iron precipitation was inhibited.

4.4 *Mechanism of Precipitate Formation*

Figure 6 indicates the mechanism of precipitate formation at the toe of the tailings dam. Sulphide oxidation reactions within the tailings in the vadose zone at the surface of the impoundment releases ferrous iron, Fe^{2+}, and acidity into the tailings water as controlled by the following reaction:

$$FeS_2 + 7/2\ O_2 + H_2O \quad \text{-->} \quad Fe^{2+} + 2\ SO_4^{2-} + 2\ H^+$$

Infiltration of precipitation carries these products downward to the water table and into the regional groundwater flow paths. Acid consuming reactions caused by the dissolution of mixed carbonate minerals along the flow paths buffer the pH back to near neutral levels. Evidence of pH buffering below the zone of oxidation was demonstrated at the M Area by Coggans (1992), as shown on Figure 7. Dissolved Fe^{2+} travels with the seepage water through the tailings dam and is discharged at the toe of the starter dam. As the seepage water comes in contact with air, the dissolved Fe^{2+} is rapidly oxidized by a biologically mediated reaction to ferric iron, Fe^{3+}, which then hydrolyses to form insoluble ferric hydroxide, $Fe(OH)_3$, in the following reactions:

$$Fe^{2+} + 1/4\ O_2 + H^+ \quad \text{-->} \quad Fe^{3+} + 1/2\ H_2O$$

$$Fe^{3+} + 3\ H_2O \quad \text{-->} \quad Fe(OH)_3 + 3\ H^+$$

Secondary phases of $Fe(OH)_3$ can also form minerals such as jarosite, hematite, goethite, lepidocrocite and siderite.

As indicated above, iron precipitation principally takes place at discharge points around the edges of the impoundment where the near-neutral pH tailings waters, rich in Fe^{2+}, emerge from dams to become exposed to oxygen in the air. This results in the abundant precipitates which have accumulated at the toe of all the tailings dams.

Depending on the coarseness and density of the materials at the dam toe and the depth of the water table, oxygen can infiltrate through the air-filled porosity of soils to the water table and initiate iron oxidation and deposition of precipitates within the body of the dams. Such deposits just above the water table were observed at the toe of all dams investigated in this study. However, the process of internal deposition of precipitates is self-limiting because the resultant layer of precipitates acts as an effective barrier to oxygen diffusion, thereby limiting further oxidation. The oxygen barrier formed by the precipitates is responsible for the current near-neutral pH in the toe seepages at the Pistol and Rock Dams.

5 CONCLUSIONS

This paper summarizes the results of the studies which were carried out to investigate the chemical precipitates in the drain at Inco's Central Tailings Area tailings dams. The following summarizes the main findings of the study:

1. The principal factor in the formation of precipitate deposits at the tailings dams is the oxidation of Fe^{2+} in the tailings water as it emerges from the dam to become exposed to oxygen in the air. Infiltration of oxygen through porous dam materials can trigger the formation of precipitates at the contact of the air and the water table within the dam. This process is believed to be self-limiting because the accumulation of chemical precipitates will eventually act as an effective oxygen barrier.
2. The precipitates observed at the Inco tailings dams typically consisted of up to 12 inches of yellow, fluid precipitates overlain by a thinner layer of reddish precipitates that ranged from soft to hard in consistency. Chemical analysis work carried out to date indicates that the yellow precipitates are principally composed of K-Jarosite. The reddish precipitates are interpreted to be oxides formed by alteration of K-Jarosite into secondary oxide components.
3. The observations evidence gathered at the Inco site shows that chemical precipitates do not form in saturated soil conditions below the water table. Hence, submergence of drain materials to preclude oxygen penetration could be an effective means to prevent clogging from chemical precipitates.

REFERENCES

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