L3-37(1)(a)

stability anal

stand

PHILLIPS URANIUM CORPORATION

BOX 26236 4665 INDIAN SCHOOL ROAD N.E.
ALBUQUERQUE, NEW MEXICO 87101

TELEPHONE 505 266-5891

March 19, 1980

RECEIVED
MAR 24 1980
RADIATION PROTECTION SECTION

Mr. J. L. Whiteman, Chief
Design and Construction Section
State of New Mexico Natural Resources Department
Water Resources Division
Bataan Memorial Building
Santa Fe, New Mexico 87503

Dear Mr. Whiteman:

This will confirm receipt of your letter dated March 18, 1980 and our meeting of the same date in our offices.

Pursuant to a telephone request made by Mr. Don Lopez of your staff, a meeting was held to discuss additional needs of the State Engineer's Office in support of Phillips Uranium Corporation's proposed Nose Rock evaporation pond dam design.

As discussed and summarized, the State Engineer's Office is requesting that Phillips provide additional defensive measures to protect against the possibility of embankment cracking, to ensure filter criteria, and to control seepage. In addition, other items discussed were the possibility of placing a protective blanket of material between the dam construction material and the slope protection material riprap, graveling the crest surface of each stage of the dam, providing a map which showed the general borrow areas of the material to be used for construction, and an evaluation of selected shear strength parameters based on any redesign which may be required as a result of this discussion.

More specifically, with regard to the defensive design mechanisms requested, Mr. Lopez indicated that there was a need to incorporate a filter mechanism in the first stage of the dam. With regard to the foundation materials, Mr. Lopez indicated that these materials are sensitive to increases in moisture content, and a defensive mechanism should be incorporated to preclude the possibility of embankment cracking. With regard to the procedure used in compaction, it was requested that we consider the possibility of changing the specification from the use of a modified proctor to the use of standard compaction in order to accommodate a more flexible structure.

There was a lengthy discussion with regard to the filter criteria, and Mr. Lopez requested that we provide him with the gradation curves of the material we propose to use. There appeared to be some confusion as to the excavation depth of the cutoff trench. It should be made clear that, in the first stage, when excavating and constructing the cutoff trench, the trench will be excavated down to the suitable foundation rock.

9805140060 800319
PDR ADOCK 04008731
C PDR

Discussion having been completed, it was agreed that Mr. Jim Tinto and Mr. Bob Booth would evaluate the existing data and determine if any additional field work would be required in preparation of a response to your request. It is anticipated that the additional work which you have requested can be completed prior to July 1, 1980. As I indicated during the meeting, I would prefer to meet with you once again after we have prepared our response to your request and discuss the particulars in order to expedite your review.

Attached herewith, for your information, is a list of the attendees at the meeting. I trust that the contents of this letter accurately reflect your request. If you have any comments or corrections, please advise me at your earliest convenience.

Sincerely yours,

Juan R. Velasquez

Juan R. Velasquez

JRV:imm-(RC)
Attachment
cc: Mr. Don Lopez, State Engineer's Office
Mr. William Fleming, NMEID
Mr. Robert Booth
Mr. James Tinto
Mr. Richard Peacock
Mr. Merle Miller

LIST OF ATTENDEES

March 18, 1980

D. T. Lopez, State Engineer's Office

J. L. Whiteman, State Engineer's Office

Richard Peacock, Phillips Uranium Corporation--Nose Rock

J. H. Tinto, Davy McKee

Merle Miller, Engineering and Services, Phillips Uranium Corporation

R. D. Booth, Sergent, Hauskins & Beckwith

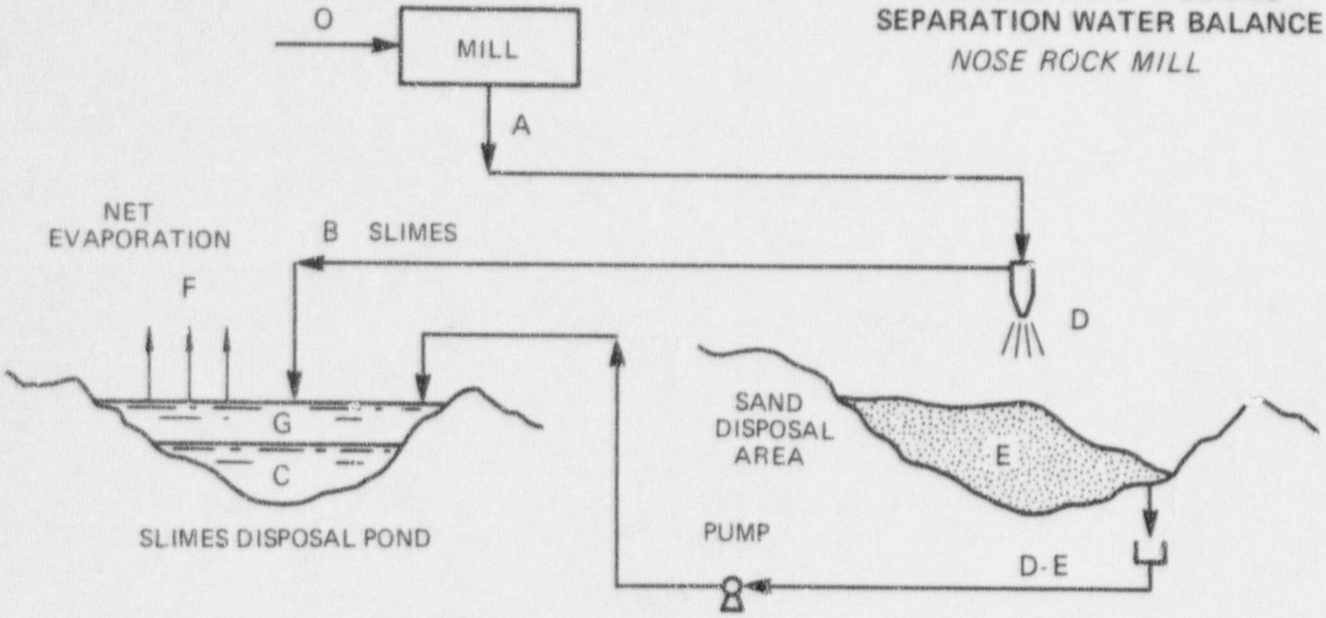
Juan R. Velasquez, Phillips Uranium Corporation

J. L. Whiteman, State Engineer's Office

W. M. Fleming, New Mexico Environmental Improvement Division

3 yrs to build up to 2500 tons/day

TAILINGS SAND – SLIMES SEPARATION WATER BALANCE
NOSE ROCK MILL



NOTE:

1. NET RESERVOIR EVAPORATION INCLUDES 10 INCH AVERAGE ANNUAL PRECIPITATION.
2. EVAPORATION PER YEAR IS BASED ON THE AVERAGE ESTIMATED POND SURFACE AREA DURING THE YEAR.
3. WATER FROM SAND DISPOSAL AREA IS BASED ON SANDS DRAINING FROM 75% SOLIDS TO 85% SOLIDS.
4. ALL VOLUMES ARE SHOWN IN ACRE-FEET.
5. WATER VOLUMES ARE BASED ON PROJECTED MILLING CAPACITIES AND ULTIMATE TAILINGS DISPOSAL CAPACITIES OVER 20 YEARS OF OPERATION.

1. $A = B + D$
2. $G = [(B - C) + (D - E)] - F$

This chart based on 2500 tons/day only:

Year	Dry Tons Solids Per Year	Total Tailings Water Acre-ft	Slime Water Acre-ft	Water Retained in Slimes Acre-ft	Liberated Slime Water	Sand Water Acre-ft	Water Retained in Sand Acre-ft	Water Drained from Sands Acre-ft	Net Evaporation Acre-ft	Year End Water Vol. Acre-ft	Avg. Pond Elev.
	O	A	B	C	B-C	D	E	D-E	F	G	
1	377,500	434.9	365.4	46.3	319.1	69.5	36.8	32.7	200.9	150.9	6402.0
2	627,500	722.9	605.9	76.1	529.8	117.0	61.2	55.8	476.4	260.1	6407.0
3	791,000	923.0	775.5	98.4	677.1	147.5	78.0	69.5	619.8	386.9	6409.0
4	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	700.2	524.2	6411.0
5	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	738.9	622.8	6415.0
6	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	767.6	692.7	6416.0
7	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	792.0	738.2	6417.0
8	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	837.9	737.8	6418.0
9	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	869.5	705.8	6419.0
10	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	881.0	662.3	6420.0
11	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	901.1	598.7	6421.5
12	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	918.3	517.9	6422.5
13	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	921.1	434.3	6422.5
14	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	924.0	347.8	6423.0
15	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	924.0	261.3	6423.0
16	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	924.0	174.8	6424.0
17	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	938.4	73.9	6424.0
18	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	941.2	DRYING	6424.5
19	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	970.6	DRYING	6425.0
20	887,500	1,035.6	870.0	110.4	759.6	165.5	87.6	77.9	970.6	DRYING	6425.0

37 ft hi at this elev

Att meeting Mar 80

BY RFB DATE 4/2/78 SUBJECT Phillips Uranium SHEET NO. 1 OF
CHKD. DATE JOB NO. 2250A

SEEPAGE ANALYSIS

PERMEABILITIES

REF: GEOTECHNICAL INVESTIGATION REPORT TAILINGS DISPOSAL AREA, NOSE ROCK URANIUM MINE & MILLING PROJECT, McKINLEY COUNTY, NEW MEXICO. BY SERGENT, HAUSKINS & BECKWITH. DATED, MARCH 30, 1978.

FIELD PERMEABILITY TESTS WERE PERFORMED BY SERGENT, HAUSKINS & BECKWITH AND REPORTED IN THEIR INVESTIGATION REPORT DATED MARCH 30, 1978, SEE PAGES A-36 AND A-37. THESE RESULTS HAVE BEEN PLOTED ON A CENTER LINE PROFILE OF THE EMBANKMENT (ATTACHED). THE COEFFICIENT OF PERMEABILITY FOR UNDER SEEPAGE IN THE SEEPAGE ANALYSIS WILL BE 0.1 FEET/YEAR FOR FOUNDATION MATERIAL

field K = .1 ft/yr

LABORATORY PERMEABILITY TESTING OF THE CORE MATERIAL WAS PERFORMED BY S,H&B SEE PAGES B-8 & B-16. RESULTS VARIED FROM 0.075 FT./YR. TO 0.197 FT/YR. FOR SEEPAGE ANALYSIS A VALUE OF 0.2 FT/YR WILL BE USED FOR CORE MATERIAL.

lab K of core ≈ .2 ft/yr

LABORATORY PERMEABILITY TESTING OF THE RANDOM FILL MATERIAL WAS PERFORMED BY SH&B, SEE PAGE B-25. THE REPORTED VALUE OF 244 FT/YR. WILL BE USED FOR THE SEEPAGE ANALYSIS

lab K of fill = 244 ft/yr

$$K_1/K_2 = .2/244 = 1/1000$$

ENG-7

SEEPAGE ANALYSIS (CONT.)

ASSUMPTIONS

1.) Since the permeability of the Random material is over 1000 Times that of The Core Material, Head Losses in The Random material are minor and the net will be drawn for the Core only: REF. FUNDAMENTALS OF SOIL MECHANICS, DONALD W. TAYLOR, Fifteenth Printing, 1965, Page 188

2.) THE FOUNDATION ROCK WILL BE CONSIDERED INPERMEABLE. REF. 5, H&B REPORT PAGE ___.

3.) THE ULTIMATE DAM HAS BEEN DIVIDED INTO FIVE SEEPAGE ZONES. SEEPAGE THROUGH EACH ZONE IS ASSUMED TO BE THE AVERAGE SEEPAGE OF THE END SECTIONS OF EACH ZONE. THE INITIAL DAM HAS BEEN DIVIDED INTO THREE SEEPAGE ZONES. (SEE SHT. ___)

SEEPAGE ANALYSIS (CONT)

STARTER EMBANKMENT

where did this figure come from?

ZONE	SEEPAGE END A	SEEPAGE END B	AVERAGE SEEPAGE	ZONE LENGTH	TOTAL SEEPAGE/YR
I	0 FT^3/YR/FT	15.4 FT^3/YR/FT	7.7 FT^3/YR/FT.	2,100 FT	16,170 FT^3
II	15.4	15.4	15.4	2,800 FT.	43,120
III	15.4	0	7.7	1,600 FT.	12,320
TOTAL	—	—	—	6,500 FT	71,610 FT^3

TOTAL SEEPAGE = 1.6 ACRE FT./YR.

$\approx$ 0.002 cfs

$\approx$ 1 gpm

BY RFB DATE 11/2/78 SUBJECT Phillips Uranium SHEET NO. 3 OF

CHKD. DATE JOB NO. 2250A

SEEPAGE ANALYSIS (CONT.)
ULTIMATE EMBANKMENT

where did this come from?

ZONE	LENGTH	SEEPAGE END A	SEEPAGE END B	AVERAGE SEEPAGE	TOTAL SEEPAGE
I	1,900 FT.	0 $FT^3/YR/FT.$	56 $FT^3/YR/FT$	28 $FT^3/YR/FT.$	53,200 FT^3
II	3,500	56	118	87	304,500
III	2,800	118	118	118	330,400
IV	1,600	118	56	87	139,200
V	2,200	56	0	28	61,600
TOTAL	12,000 FT.	—	—	—	888,900 FT^3/YR

TOTAL SEEPAGE = 20 ACRE-FT./YR.

ultimate seepage rate is > 10 X more than starter dam seepage rate.

why?

reasons:

1) total length of ultimate dam is double the starter dam
2) ave seepage rate is 4 X the starter dam

2

why?

ENG-7

$_2 = 1000 K_1$

EMBANKMENT-DAM ENGINEERING

CASAGRANDE VOLUME

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Geotechnical Engineers, Inc.

Steve J. Poulos
Geotechnical Engineers, Inc.

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sand and silt to be in direct contact with the perforated pipe.

Recommendations for Preventing Piping in Embankments and around Drains

Let us summarize and reemphasize the precautions that are needed in the design and construction of filters and drains in earth dams and levees:

1. Do not allow materials larger than 1 to 2 in. in size in the coarser layer of a two-layer drain.
2. Do not permit extremely wide ranges of particle sizes in a filter layer. Broadly graded mixtures with maximum sizes of several inches or more tend to segregate during placement, creating conditions conducive to internal piping.
3. Require filter materials to be placed with spreader boxes or other equipment that does not induce segregation.
4. Require filter materials to be well saturated at the time of placement and compaction. Most dry aggregates tend to segregate badly during handling and placing.
5. Do not permit filters to become contaminated with fines that might be dropped from tires of construction equipment, washed down slopes by rainstorms, or transported by other accidental means.
6. Whenever possible, avoid the use of filter layers that contain only sand sizes, since such materials have little resistance to washing through accidental coarse pockets, holes in pipes, open joints in rock formations, openwork gravel seams, and other large openings.
7. Require careful, thorough inspection of the work.

3. REDUCTION OF SEEPAGE

Basic Considerations

Seepage-reducing methods make use of relatively impermeable cutoffs, grout curtains, and upstream blankets, which consume energy at locations within cross sections where large water pressures and seepage forces can have no detrimental effects. The net result of these methods is that water pressures and seepage forces are reduced in the critical exit regions. These seepage-reducing features are usually used in combination with properly designed filters and drainage features, since seepage reduction alone may be only partially effective, as will be illustrated by several examples. Furthermore, the need for a conservative approach in designing dams makes such a "second line of defense" highly desirable.

Thin Upstream Sloping Cores

Figure 11*a* is a cross section through an earth dam with an upstream sloping core of low permeability. The foundation is assumed to be impervious. Under steady seepage, the small amount of water that seeps through the core flows vertically downward in a partially saturated zone and then more or less horizontally in a thin saturated layer along the impervious foundation. If the permeability k_1 of the core is very low in relation to the permeability k_2 of the downstream zone, as assumed in this example, substantially all of the head loss occurs in the upstream half of the dam, and the downstream half is relatively unaffected by the seepage. The condition shown in Fig. 11*a* is the one that is often assumed to exist in this kind of dam, but if the permeabilities of the core and the downstream zone approach each other, the elevation of the line of seepage in the downstream zone rises. Figure 11*b* shows the flow net for $k_2 = 80k_1$, and Fig. 11*c* shows the lines of seepage in the downstream zone for several ratios of k_2/k_1. It is seen that even for $k_2/k_1 = 200$, the line of seepage is higher than desirable in the downstream zone of the dam. For this type of dam the downstream shell must be several hundred times more permeable than the core.

Partial Cutoffs

If an earth dam were constructed on a pervious or semipervious earth or rock foundation without any cutoff, the line of seepage might rise to a high level in the downstream half of the dam, thus decreasing the stability of the downstream slope. Hence if the dam in Fig. 11 were constructed on a foundation with a coefficient of permeability equal to that of the downstream "pervious" zone (but the core has very low permeability), the line of seepage might rise to a high level, and large exit gradients could exist at the toe (Fig. 12*a*). The height of the line of seepage and the exit gradients can be lowered by a number of methods, such as: (1) making the downstream shell at least 100 times more permeable than the foundation; (2) installing a relatively impervious cutoff into the foundation; (3) grouting the foundation beneath the core; (4) installing an impervious blanket up-

k_1 = permeability of core
k_2 = " of downstream material

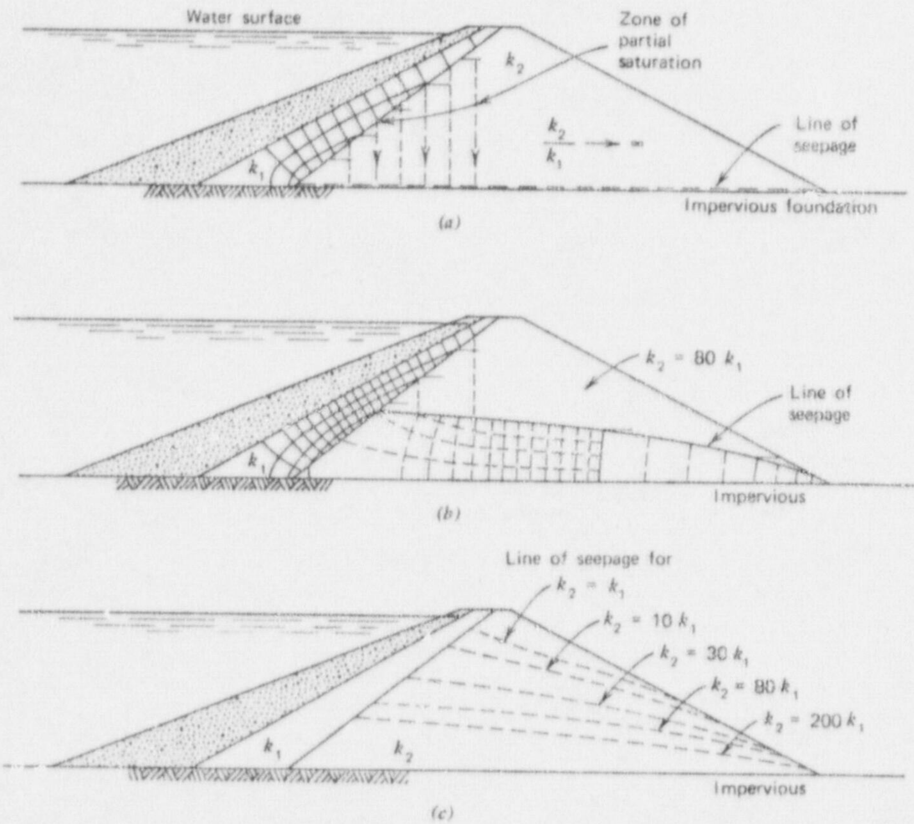


Fig. 11. Effect of relative permeability of upstream sloping core and downstream zone on position of line of seepage in downstream zone. (a) Ideal condition. (b) Typical flow net ($k_2 = 80k_1$). (c) Line of seepage for various ratios of k_2/k_1.

stream from the core; or (5) installing drainage wells or other drainage facilities. Methods (2), (3), and (4) are seepage-reducing methods and will be discussed in the following paragraphs.

It may be seen in Fig. 12*b* that if an impervious cutoff is installed to 60% of the depth of the pervious foundation, the flow net is modified and water rises to a slightly lower level than in Fig. 12*a*, but exit gradients (as reflected by the distance between equipotential lines) are reduced only slightly. Theoretical positions of the line of seepage for several depths of cutoff are given in Fig. 12*c*. It may be seen that the cutoff must be essentially "perfect" if it is to have a major influence on the height of the line of seepage. Since this is virtually impossible to achieve, other seepage control methods generally must be used in conjunction with cutoffs.

Figure 13 gives two flow nets (drawn on transformed sections, for $k_h = 25k_v$) that are part of the study of seepage beneath a dam built in a natural saddle separating a reservoir on the left from a lower valley on the right. Beneath the upper cover of relatively impervious soil is a stratum of highly pervious sand and gravel to a depth of about 50 ft. Under the pervious sand and gravel is a 50- to 60-ft-thick silty formation, which is underlain by gravelly materials.

Preexisting piezometric levels were observed at two or more depths at a number of locations. These observations indicated the existence of downward hydraulic gradients, since smaller head was measured in the bottom gravels than in the upper gravels at a given location. Seepage evidently was escaping relatively freely from the lower gravels on the river bank at the right. Springs that emerged from the silty formation relatively high above the river evidently were being fed by infiltration of rainwater in the general area shown. Without such a supplemental source of water, the line of seepage could not have emerged at such high levels unless the soil profile were substantially different from the cross section shown.

The flow net in Fig. 13*a* was constructed so that the equipotential lines were consistent with the observed piezometer readings. From this flow net it was estimated that the silty formation was about

Davy McKee Corporation

CALCULATION SHEET

BY R.C. Mills DATE 3/14/80 SUBJECT Phillips - Nose Rock SHEET NO. ______ OF ______

CHKD. ______ DATE ______ Filter Capacity JOB NO. ______

Filter Material

Screen Size	% Passing
No. 4	90-100
No. 10	70-90
No. 40	15-30
200	0-10

permeability will be approximately 14 ft/day

seepage of 118 $ft^3/yr/ft$

capacity of filter is 3 ft × 14 ft/day for 1 ft width

$= 42\ ft^3/day/ft = 5110\ ft^3/yr/ft$

ENG-7 REV. 7/79

fine, erodible soil materials into the open-graded layer. If a trench drain is cut into erodible soils, plastic filter cloth can be placed on the sides and bottom of the trench to keep soil out of the backfill material as shown in Fig. 6.8.

Figure 6.9 shows gradations and permeabilities of several open-graded aggregate drainage materials and several blended aggregates in the class of filter materials. They are all high quality, washed, processed aggregates containing moderate percentages of crushed particles. All of the open-graded materials were stabilized by hot-mixing with a small amount of a paving grade asphalt. Three are similar to those for which tests are summarized in Table 6.1. The filter materials have been compacted to only moderate densities.

The permeability values in Fig. 6.9 illustrate general levels of permeability that are possible for the range of material gradations depicted. While open-graded materials of the kind represented are only slightly affected by increased compactive effort, blends of sand and gravel sizes are greatly influenced by compactive effort and water

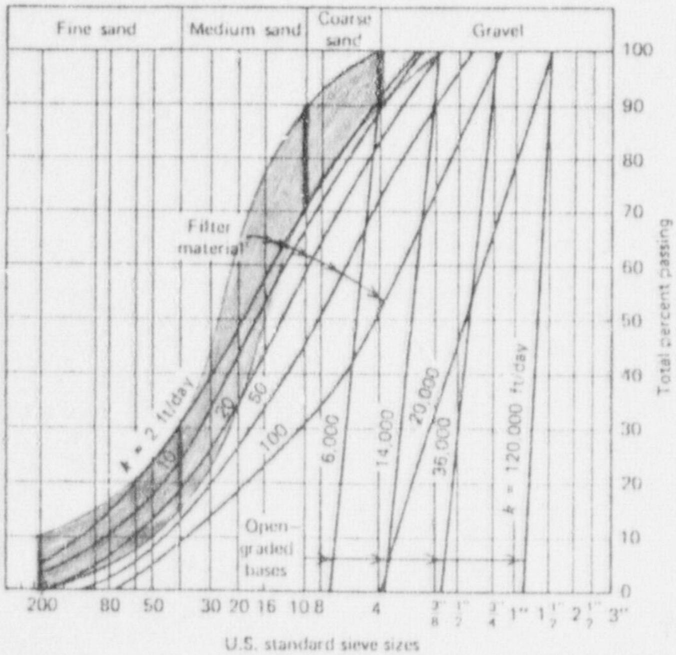
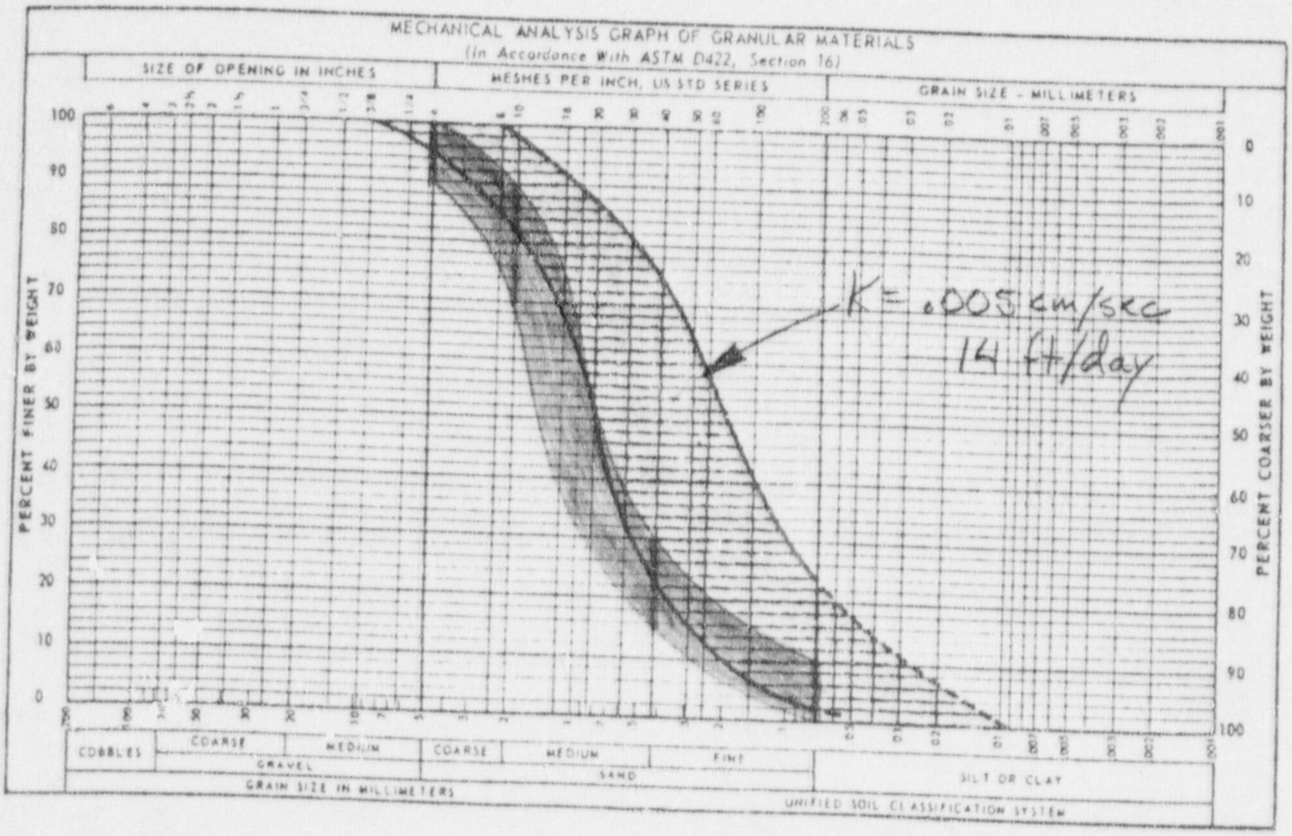


FIG. 6.9 Typical gradations and permeabilities of several open-graded aggregates and several filter materials. (FHWA *Guidelines for the Design of Subsurface Drainage Systems for Highway Structural Sections*, Figure No. 4).

GRAIN SIZE DISTRIBUTION
COARSE TAILING



SUPPLEMENT NO. 1
EBASCO SERVICES INCORPORATED

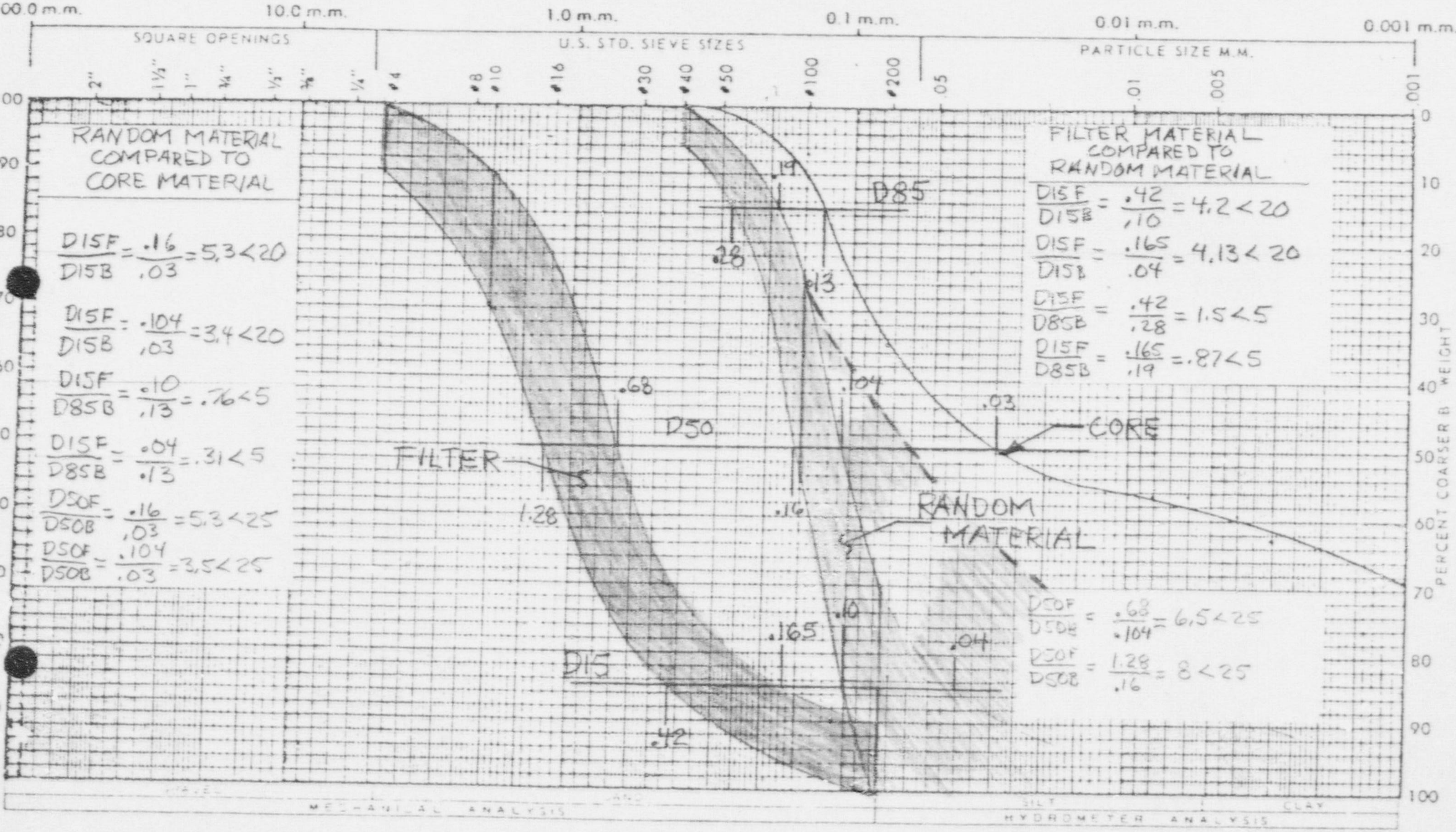
ERIE MINING COMPANY
GRAIN SIZE DISTRIBUTION SUMMARY PLOT COARSE TAILING FIGURE 5

REV. 12-1-77

JOB NO. E78-1089

PARTICLE SIZE DISTRIBUTION CURVE

PROJECT Stability Analysis - Tailings Embankment
Nose Rock Uranium Mine & Milling Project LOCATION McKinley County, New Mexico



CURVE	SAMPLE	LIQUID LIMIT	PLASTICITY INDEX	ACTIVITY	UNIFIED SOIL CLASSIFICATION	LAB NO.
	Bulk Sample Test Boring	36	15	.48	CL	89-133

SERGENT HAUSKINS & BECKWITH

FILTER CRITERIA

RANDOM – D_{15} MIN = 0.007

D_{85} MIN = 0.19

FILTER – D_{15} MAX = 0.28

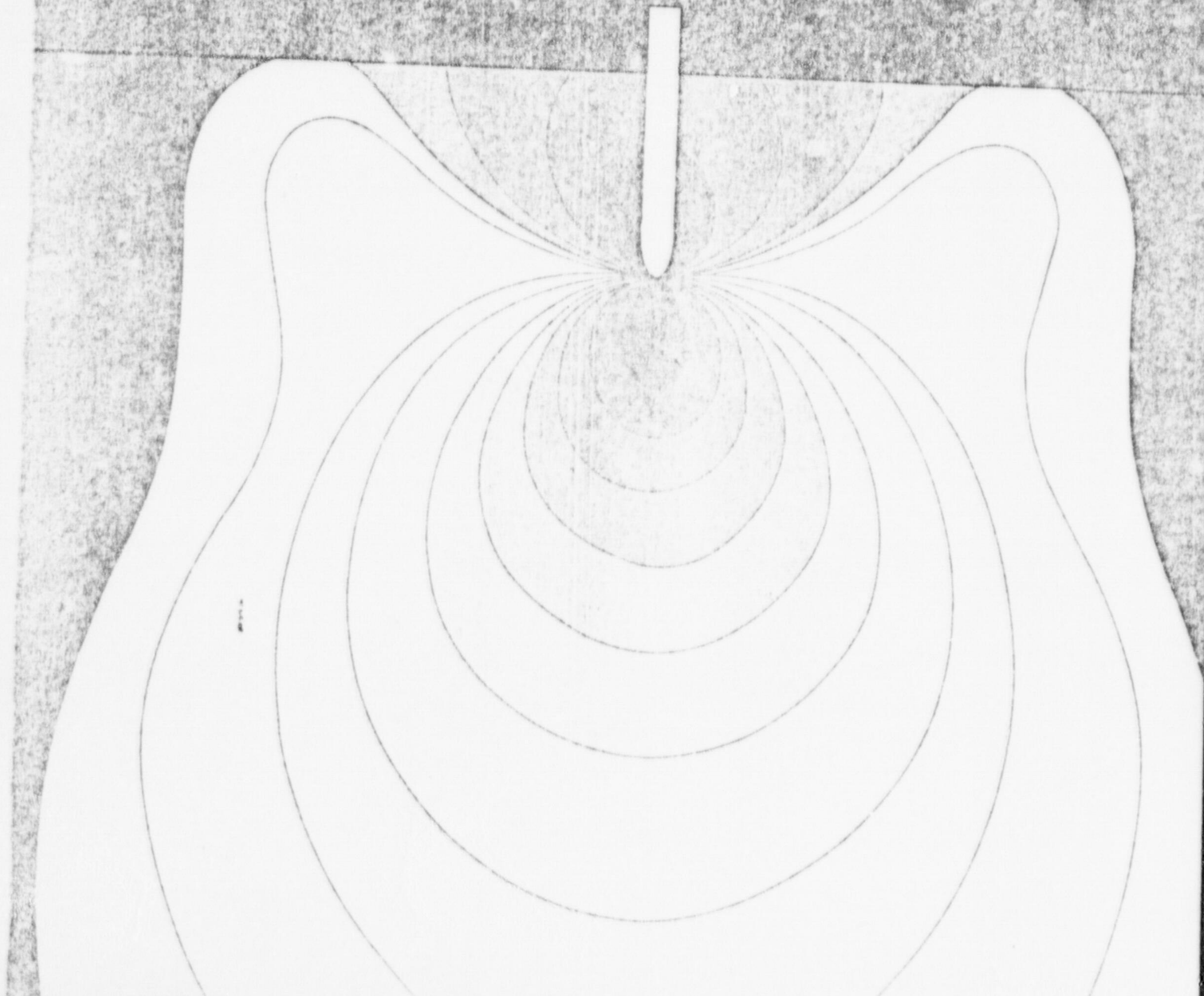
$$\frac{D_{15F}}{D_{85B}} = \frac{0.28}{0.19} = 1.47$$ CRITERIA < 5

$$\frac{D_{15F}}{D_{15B}} = \frac{0.28}{0.007} = 40$$ Criterial 5 – 40

Foundation Engineering Handbook

EDITED BY

Hans F. Winterkorn and Hsai-Yang Fang



can develop. In designing and constructing filters and drains for groundwater and seepage control, permeable materials are often placed over seepage exit surfaces to allow seepage to discharge freely while preventing the erosion of soft, cohesionless formations, which could lead to piping failures. If the basic criteria and concepts presented in section 6.2 are always fulfilled in filters and drains, structures can operate permanently in safety without danger of seepage failures. But, when these criteria are ignored or improperly used, serious damage or failure can occur. Also, if the designers do not make adequate studies of probable seepage conditions, they may not know what kind of drainage system is really needed, where it should be located, its required size, or its detailed requirements.

An example of not taking physical factors and laws into account is the placement of macadam bases for roads on silt, sand, and other fine-grained subgrades, and the clogging that almost always takes place. Another example is the "French drain", placed in soft, erodible soils without filter protection. Many field engineers believe that coarse one-sized stone or gravel makes excellent drainage material, although in reality such materials provide no filter protection to fine-grained soils. A small earth dam in central California failed because the field maintenance men placed 3-inch-diameter "drain rock" over sand boils that developed at the downstream toe. Fine foundation soil washed out through the coarse rock, unnoticed, until one night the dam failed quite suddenly when an underground cavity reached the reservoir side of the dam's foundation.

A common deficiency of drains for structures is inadequate water removal caused by the use of "pervious" drainage materials that are in reality relatively impervious. Countless structures and pavements have been damaged or destroyed because water pressures built up in drainage layers that were only slightly more permeable than the soils they were supposed to drain.

Roadbeds and airfields throughout the world are deteriorating prematurely because the "pervious" bases that are supposed to be protecting them from groundwater and seepage and infiltered water do not have sufficient permeability to remove the water, and the pavements are forced to carry traffic while in a completely flooded state.

6.2 DESIGN OF FILTERS AND DRAINS

1. Basic Considerations

As noted in section 6.1, the design and construction of all important civil engineering works involving the control of groundwater and seepage by drainage requires careful geological and soils surveys, field and laboratory tests, and interpretation and analysis of overall conditions, to determine what kinds of drainage systems are needed and where they can best be located, and to establish criteria on which to develop realistic designs. Although natural earth formations can in some cases enhance drainage, or minimize drainage problems, the arrangements of natural formations can also aggravate the problems of obtaining good drainage.

In most cases, the drainage of engineering works is accomplished by artificial devices, such as blanket drains over seepage exits, intermittent line drains or trench drains, vertical relief wells, horizontal drains with small-diameter perforated or slotted pipes, tunnels, galleries, etc. Most drainage systems make use of porous filter aggregates to collect the water and conduct it to outlets, often with the aid of perforated or slotted pipes. If the formations being drained are firm, nonerodible rocks, drainage may be obtained simply by drilling small-diameter drain wells which feed the water to exits or to galleries. However, if the water-bearing materials are soft, erodible formations of soil or rock, the porous aggregate drainage materials must hold the erodible materials firmly in place, while freely allowing the water to escape. This is in keeping with the basic purpose of any filter: to allow the passage of a fluid or gas while separating out solid matter. To ensure complete filter protection to erodible materials, porous aggregate drain layers in contact with the soil must not have any continuous openings large enough for the passage of the soil particles. Thus, *filters generally must be relatively fine-grained*. In addition, every drainage system must be capable of freely discharging all of the groundwater and seepage that reaches it, under relatively small hydraulic gradients and small excess hydraulic head. Therefore, whenever appreciable quantities of water must be removed, drainage systems must contain porous aggregates of relatively high permeabilities. Thus, *filters generally must be relatively coarse-grained*. Here are the two basic, but conflicting requirements of porous aggregate filters and drains: they must be fine enough to hold erodible materials in place, but they must also be coarse enough to discharge all of the water that reaches them.

How can a single type of filter aggregate in a single-layer drain ever do both of these jobs properly? If the quantities of water being discharged are relatively minute, a single-layer drain of carefully specified, washed filter aggregate might be adequate; but if any appreciable quantity of water has to be removed, *graded filter* drains (using separate fine-grained layers for filters, and coarse-grained layers to conduct the water) are nearly always required (Lovering, 1960; Cedergren, 1962, 1967; Cedergren and Lovering, 1968; Winterkorn, 1967).

2. Piping Prevention

Some of the road and dam builders of times past seem to have understood the need for drains to provide filter protection of fine-grained soils, but this need was not universally understood. For example, some of the early road builders who understood soil behavior placed a thin layer of dry stone screenings on soil subgrades before placing open-graded "macadam" bases (Hewes and Ogelsby, 1954). Others not so wise placed coarse stone directly on fine-grained soil subgrades with the result that within a short time the stone worked down into the wet soil, became clogged, poorly drained, and low in strength. For many decades after about 1800, roadbuilders made use of the "French drain", which was a trench at the edge of the road backfilled with large-sized gravel or boulders. When these drains were used in stiff clays and other nonerodible formations, they often served as relatively good conveyors of water for long periods of time. Unfortunately, however, the French drains were often constructed in wet, erodible sands and silts without filter protection, with the result that they frequently became clogged with the fine soil, and the adjacent roads deteriorated from lack of subsurface drainage.

Before the development of modern filter criteria, some dam builders constructed drains with successively coarser layers of stone or gravel, placing the finer materials in contact with the soil, and progressively coarser materials toward the centers of the drains. Creager et al. (1945) describe the Tabeaud Dam in California, which was constructed in 1902 with a rock drain having two progressively coarser filter zones between the foundation soil and the rock drain.

Until the past few decades, earthwork design was considered more of an art than a science, with the result that many efforts failed because fundamental factors were not understood or taken into consideration. With the development of the rational and experimental approach of soil mechanics, earthwork design has become more of a science than an art. Bertram (1940), with the advice of Terzaghi and Casagrande, conducted laboratory filter experiments at the Graduate School of Engineering, Harvard University, to test filter criteria that had been suggested by Terzaghi. Bretram's work led to the following widely used criteria for designing filters:

$$\frac{D_{15}\text{ (of filter)}}{D_{85}\text{ (of soil)}} < 4 \text{ to } 5 < \frac{D_{15}\text{ (of filter)}}{D_{15}\text{ (of soil)}} \qquad (6.1)$$

The left half of Eq. 6.1, a fundamental criterion for the prevention of piping through filters, may be stated as follows:

Piping criterion: The 15 percent size (D_{15}) of a filter material must be not more than four or five times the 85 percent 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. 6.1 may be stated as follows:

Permeability criterion: The 15 percent size (D_{15}) of a filter must be at least 4 or 5 times the 15 percent size (D_{15}) of a protected soil. This requirement will generally ensure that filter layers will be several times more permeable than adjacent soils, but does not always guarantee adequate hydraulic conductivity in drains, as will be outlined in section 6.2.3.

Many researchers since Bertram have made experimental and theoretical studies of filter behavior and criteria (Cedergren, 1967b). The U. S. Army Corps of Engineers (1941) and the U. S. Bureau of Reclamation (Karpoff, 1955) have done considerable work with filter criteria and filter materials. Many theoretical studies have been made of the behavior of soils in relation to filters and fine filter materials in relation to coarse materials. If a filter layer satisfies the left half of Eq. 6.1 *in every part*, it is virtually impossible for piping to occur, even under extremely large hydraulic gradients. Some design organizations place additional restrictions on filter materials. For example, the U. S. Bureau of Reclamation limits the maximum size of filter aggregates to 3 inches in order to minimize segregation and bridging of large particles during placement.

The U. S. Army Engineers (1955) normally limits the piping ratio to 5, and also uses the following criterion:

$$\frac{50\text{ percent size of filter material}}{50\text{ percent size of protected soil}} \leq 25 \qquad (6.2)$$

If a protected soil is a plastic clay, the U. S. Army Engineers (1955a) allows much higher piping ratios than required by Eq. 6.1, as indicated by the following:

> "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 (D_{60} to D_{10}) of not greater than 20 will be required."

If crushed stone is used, the U. S. Army Engineers recommends limiting the piping ratio (D_{15} of filter to D_{85} of soil) to less than 5. The safe ratio is usually checked for important works by performing filter tests in the laboratory with materials representative of those to be used in the construction.

Both the Corps and the Bureau also require that the grain-size curves of filters and protected soils be somewhat parallel to each other. This is the objective of Eq. 6.2.

If perforated or slotted pipes are used in drain wells, drainage blankets, line drains, etc., no unplugged ends should be allowed, and the filter materials surrounding the pipes must have gradations that are compatible with the sizes of the holes or slots. The following range in criteria is commonly allowed by large designers of filters and drains for earth dams:

$$\frac{D_{85}\text{ size of filter material}}{\text{hole width or diameter}} > 1 \text{ or } 2 \qquad (6.3)$$

When the filter criteria described above are satisfied *in every part of a filter or drain*, piping cannot occur under even extremely large hydraulic gradients. As pointed out in section 6.1.3, adequate specifications and careful construction are required if works *as they are constructed* are to be completely safe from piping troubles.

When wellpoints and deep pumped wells are used for the dewatering of saturated soils, experience has shown that somewhat less stringent criteria sometimes may be used in selecting filter materials to install in "sand casings" around the slotted or perforated well pipes or wellpoint screens or slotted pipes. Under some conditions, experienced installers have been able to use slots at least as wide as the largest soil particles with relatively little loss of soil through the openings. Slow development of the dewatering wells and wellpoints by experienced persons is important to the success of these installations. Since the plugging of individual wells or wellpoints for dewatering is not likely to have the serious consequences of failures of permanent relief wells and drains for hydraulic structures, smaller factors of safety often can be tolerated. Wide departures from the recognized filter criteria are not recommended, however, even for these temporary usages.

3. Discharge Requirements of Drains

General Besides functioning permanently without becoming clogged by infiltration or allowing piping of the adjacent soil, drains must also be capable of removing, with small head and small hydraulic gradients, all of the water that reaches them. Although the knowledge of designing drains for discharge requirements has been available for decades (Darcy 1850; Creager, et al., 1945a), only recently have serious efforts been made to apply this knowledge to the design of drainage systems for roads, earth dams, reservoirs, etc. (Lovering, 1960; Cedergren, 1962, 1967; Lovering and Cedergren, 1968). Engineers have long had an intuitive appreciation of the inherent capabilities of coarse stone as conductors of water (see section 6.2.2). Field engineers and construction and maintenance people are still to be found who believe that "drain rock," composed entirely of 2-inch-diameter or larger particles, is a "universal" drainage material, suitable for every drainage problem. But, as noted in section 6.1.3, when such coarse materials have been used for draining fine sands, silts, and other erodible soils *without filter protection*, serious infiltration and piping troubles have developed. Bad experiences with coarse, unprotected rock and cobbles in "French drains" and "macadam" bases, together with the development of the rational and experimental filter criteria described in section 6.2.2, led to a swing of the pendulum to the other extreme of using nothing for highway drainage except sandy gravel blends of fine filter aggregates. The blends that have been so popular in recent years usually contain so many fines that they are incapable of draining roadbeds, even for minor seepage conditions.

Ironically, many road builders have been willing to specify fine-grained blends of sand and gravel for roadbed drainage (one of the most difficult kinds of drainage), whereas agriculturalists prefer not to use beach sand for the bedding of plants, because it is "too impervious to drain." Yet, most beach sands are considerably more permeable than the sand and gravel blends that have been used for roadbed drainage!

Butterfield (1964) describes the material used for the core of the Howard A. Hanson Dam in Washington as being a blend of sand and gravel with a minimum requirement of 3% of material passing the No. 200 sieve. This dam is relatively watertight, even though *the most impervious part* was made of the same basic class of material that highway engineers have been trying to use to drain roadbeds! Many examples could be cited of not only roads, but earth dams, levees, reservoirs, and other structures which have had serious troubles because of lack of drainage resulting from the practice of *not designing for discharge needs.*

Serious misconceptions have existed in the minds of many individuals as to the capabilities of various kinds of porous aggregate "filter materials" for use in drains and "pervious" bases for roads and airfields. Many pavements that have been constructed on granular subbases extending across the full widths of shoulders have failed permaturely because they actually were poorly drained. Water gets into roadbeds much faster than it can get out, and becomes trapped within structural sections, even in roads built on high fills. Consequently, roads all over the world are being forced to carry traffic while they are in a completely flooded state even though the designers and builders thought they were building well-drained roads.

Much of the problem of draining roadbeds and certain other types of structures can be attributed to the fact that *permeability*, the engineering property controlling rate of flow of water in porous media, is the most widely varying property of engineering materials. Other engineering properties, such as unit weight and shearing strength, vary over minute ranges when compared with permeability. Open-work gravels and highly permeable filter aggregates can have coefficients of permeability *5 to 10 billion times* those of fat clays. These wide ranges in the value of a property are almost beyond human comprehension, and tend to obscure the true nature of seepage through soils and porous aggregate drains. Variations in the hydraulic gradients that cause flow of water in porous media magnify the possible spread of seepage behavior by an additional factor of at least 100, giving an overall possible variation of *a trillion times*! It must be obvious that depending on "intuition" or "rule of thumb" methods to solve drainage problems and to select drainage materials can be extremely misleading.

Analyzing flow in drains To apply the rational and experimental methods of soil mechanics to the analysis of seepage conditions in drains, it is first necessary to look for all possible sources of water that may enter a drain, and have to be removed. Then, it is necessary to consider the hydraulic conditions within the drain and develop a design that will ensure sufficient hydraulic conductivity or transmissibility to remove the water without excessive buildup of head in the drain. Seepage within the surrounding water-bearing soils and within the drain can be analyzed with flow nets as described in section 6.5; however, practical approximate solutions can also be obtained with Darcy's law as outlined below.

A common form for Darcy's law for flow in porous media is

$$q = kiA \tag{6.4}$$

In Eq. 6.4, q is the seepage quantity in unit time flowing through a porous material having a coefficient of permeability k, under a hydraulic gradient i in the direction of flow, through a cross-sectional area A normal to the direction of flow. With Darcy's law it is possible to estimate quantities of seepage that can flow from the soil and other formations that contribute inflow to a drain, and also to analyze seepage within a drain. Reasonable estimates of *inflow* quantities from the various sources can be made provided reasonable values can be assigned to the following:

(a) The *average* or *effective* permeabilities of the formations feeding water to a drain (see section 6.5.3 for a description of well pumping tests for determining field permeabilities).

(b) The average hydraulic gradients causing flow in the formations bringing water to a drain.

(c) The average cross-sectional areas of the media through which water is flowing toward a drain.

After estimating probable maximum rates of seepage from all known sources, it is then necessary to analyze the hydraulic conditions within the drain. When using Darcy's law to study seepage in drains, the factors of importance become more revealing if the equation is rearranged in the following form:

$$\frac{q}{i} = kA \tag{6.5}$$

In Eq. 6.5, the term q is the seepage quantity for which a drain is being designed. Usually the total estimated inflow rate should be multiplied by a factor of at least 5 or 10 to provide a reasonable margin of safety to take care of errors in evaluating permeabilities of water-bearing formations and other uncertainties in the seepage estimates. The *allowable* gradient i in a drain is selected by the designer as the maximum gradient he considers desirable or safe to ensure the required level of protection needed to safeguard the structure being designed. The gradient i in a drain is often restricted by the geometry and orientation of the cross section. For example, in a vertical drain in a dam (Fig. 6.2), i often can be about 1.0, whereas in a drain under a roadbed (Fig. 6.3) or in a horizontal blanket drain in a dam (Fig. 6.8) i is often limited to around 0.01 to 0.05, depending upon conditions.

The ratio of q/i in Eq. 6.5, which is equal to the product of $k \times A$ of a drain, may be defined as the minimum allowable *conductivity* or *transmissibility* of a drain. Having properly estimated q/i (allowing an adequate factor of safety), it is then only necessary to design a drain with the most satisfactory and economical combination of area A and permeability k to provide the desired conductivity. One procedure is to select a permeability representing an available filter material of a desired type and calculate the required thickness A. Alternately, a practical thickness of filter can be selected, and the minimum required permeability determined.

Frequently, the most practical design for a specific project is influenced either by minimum practical construction thicknesses, or by availability of materials. In almost every case involving the removal of appreciable quantities of

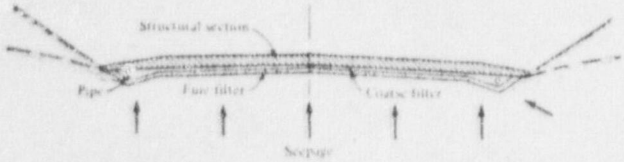


Fig. 6.6 Layered drain under a pavement, for control of groundwater seepage and surface infiltration.

water from roadbeds, airfield pavements, reservoir drains, and many other kinds of structures, it will be found that graded filters or multiple-layer drains are more satisfactory and economical than single-layer drains constructed of blends of sand and gravel (see Fig. 6.6).

The Economics of Aggregate Drains One of the primary objectives of engineers is to design the most economical structures satisfying any given set of requirements. When comparing various alternate types of drains for engineering works, relative *costs* and relative *benefit/cost* factors should be taken into account. By examination of Eq. 6.5, it is evident that drain transmissibility (a benefit) varies directly with drain area A and drain permeability k. Whenever drain transmissibility is important to engineering projects, two basic questions ought to be asked: (a) How does the cost of drain transmissibility vary with the cross-sectional area A of drains? and (b) How does the cost of drain transmissibility vary with drain permeability k?

Obviously, if drain transmissibility kA is to be increased by increasing the area or thickness of a drain, the cost will increase essentially in direct proportion to thickness, since doubling the number of cubic yards of filter material in a drain virtually doubles the cost of the drain. In contrast, the permeabilities of filters often can be increased hundreds or thousands of times, often at little or no increase in cost per cubic yard, and sometimes at less unit cost. For example, plant-processed, washed blends of sand and fine gravel with permeabilities of 10 to 20 ft/day have been produced and supplied in the western United States (1973) for prices up to $12.00 and more per cubic yard. On the other hand, "bird's-eye" roofing gravel, pea gravel, and one-sized crushed rock or gravel in sizes up to 1 inch, with permeabilities from 3000 to 100 000 ft/day, have been supplied for $3.00 to $5.00 per cubic yard. Thus, materials with permeabilities of many thousands of feet per day are being supplied for the same or less cost per cubic yard than sand and gravel blends with permeabilities of 10 and 20 ft/day or less. Consequently, increasing the coarseness and permeability of drainage aggregates can increase drain transmissibility hundreds or thousands of times, often at reduced cost.

When pea gravel and coarser one-sized, highly permeable filter aggregates are used in drains, the piping criterion (Eq. 6.1) should always be used to be sure that there is no danger of clogging or piping. This criterion will usually require the placement of a fine filter layer between the coarse filter material and fine soils, as in Figs. 6.2, 6.3, and 6.6, which requires use of "graded filters." When graded filters are used, the outer fine filter layers usually provide very little of the total conductivity of a drain. In subsequent comparisons of the *benefit/cost* factors of single-layer drains and graded-filter drains, the conductivity of the outer filter layers of graded filters is assumed to be zero. Thus, the *benefit/cost* factors in Table 6.1 were calculated on the assumption that half of the total aggregate in the graded filters is in the highly permeable conducting layer and half is in the fine filter layers. Table 6.1 shows that the potential

TABLE 6.1. COMPARISON OF BENEFIT/COST FACTORS OF SEVERAL SINGLE-LAYER AND GRADED-FILTER DRAINS.

Kind of drain	Thickness of conducting part (feet)	Permeability of conducting part (ft/day)	kA	Relative cost	Relative benefit/cost factors
Single-layer	2	10	20	1	1
	4	10	40	2	1
	6	10	60	3	1
	2	20	40	1	2
	2	40	80	1	4
Graded-filter	1	1000	1000	1*	50
	2	1000	2000	2*	50
	2	5000	10 000	2*	250
	2	10 000	20 000	2*	500
	2	100 000	200 000	2*	5000

*Assumes that 50% of total quantity of filter material is in the conducting part of the drain.

Note: This study assumes that drains are flowing full, and that $q = kiA$.

benefit/cost factors of single-layer drains do not change with thickness, but the *benefit/cost* factors of graded filter drains can be hundreds or thousands of times greater than those of the single-layer drains. The conclusion that must be reached is that *when appreciable amounts of water must be removed by drains, single-layer drains almost never can be justified, either from a water-removing standpoint, or from a cost standpoint.*

If the actual water-removing needs of drains can only be approximated (which often is the case), graded-filter or multilayered drains can provide much greater water-removing capabilities than single-layer drains, and at less dollar cost. Consequently, the widespread usage of graded-filter drains would result in far fewer drainage failures than would occur with single-layer drains.

A comparison of the potential economic benefits of various classes of permeable materials, *as conductors of seepage*, can be made on the basis of the cost of conveying an arbitrary quantity of water a given distance. Any conveyor or conductor of a material or substance can be rated in terms of the cost of moving a given amount of material over a given distance. Thus, in earthwork it is customary to use the term *station-yard*, and in freight hauling the cost may be expressed for the *ton-mile*. Similarly, the water-conducting capabilities of drainage aggregates of various permeabilities can be compared on the basis of any convenient units. Such a comparison can be made (Cedergren and Lovering, 1968) with the aid of Darcy's law, $q = kiA$, by multiplying the right-hand side of the equation by unity (L/L), as follows:

$$q = kiA \quad \frac{L}{L}$$

Hence,

$$q = \frac{kiV}{L}$$

in which V is the volume of filter material having a cross-sectional area A and a length L. It then follows that,

$$V = \frac{qL}{ki} \tag{6.6}$$

In Eq. 6.6, V is the volume of filter material needed to conduct seepage quantity q a distance L under hydraulic gradient i in a material with a coefficient of permeability k. In most subsurface drains, both q and i will vary from point to point, but a reasonable comparison of the theoretical capabilities of various drainage materials as *conveyors of seepage* can be made for the assumption that both of these factors are relatively constant.

Using Eq. 6.6, the costs of both single-layer drains and graded-filter (multiple-layer) drains are compared in Fig. 6.7 on the basis of conducting 1 gpm of seepage a distance of 100 feet. Aggregates are assumed to cost \$5.00/yd^3, in place. In the range of filter permeabilities of less than about 40 ft/day, it is assumed that single-layer drains are used; but for filter permeabilities over 40 ft/day, graded filter drains are required. The costs of the graded filter drains in Fig. 6.7 are based on the assumption that only half of the total cubic yards of filter materials in graded-filter drains is in the conducting part of these drains.

Referring to Fig. 6.7, this theoretical comparison of drains shows that it would cost over \$10 000 to conduct 1 gpm a distance of 100 feet with a single-layer drain of filter material with a permeability of 10 ft/day, discharging seepage under a hydraulic gradient of 0.02. In contrast, 1 gpm can be moved 100 feet for about \$30 by a graded-filter drain having a core of coarse pea gravel with a permeability of 10 000 ft/day, under the same hydraulic gradient.

Using granulometric principles to examine strength and permeability characteristics of granular bases for roads, Hans Winterkorn (1967) concluded that desirable strength and permeability characteristics can be expected of mineral aggregates of relatively large dimensions, and of a single size, or a very narrow range of sizes. Winterkorn's work gives additional proof of the desirability of using graded-filter drains with internal cores of high permeability where large amounts of groundwater and seepage have to be removed from roadbeds, or other engineering structures.

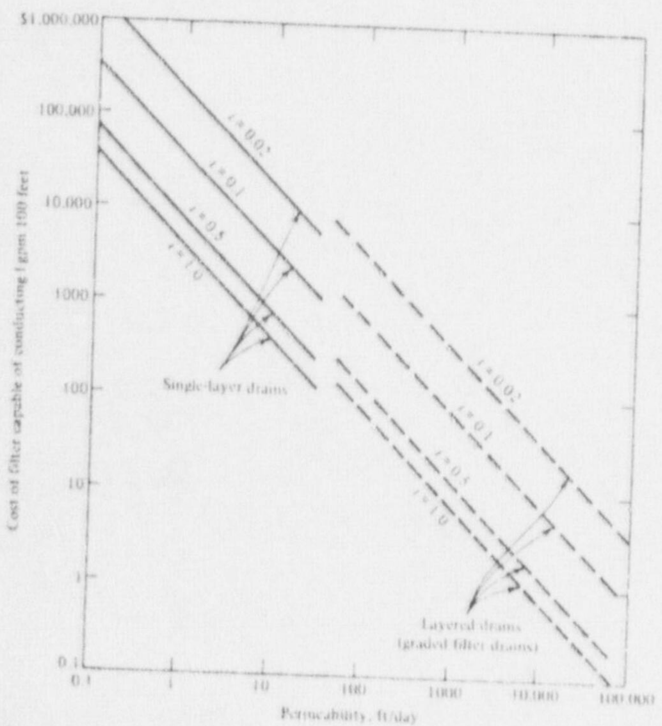


Fig. 6.7 Cost of filter aggregate per seepage unit (1 gpm conducted 100 ft). (After Cedergren and Lovering, *Highway Research Record*, No. 215, HRB, 1960, p. 3.)

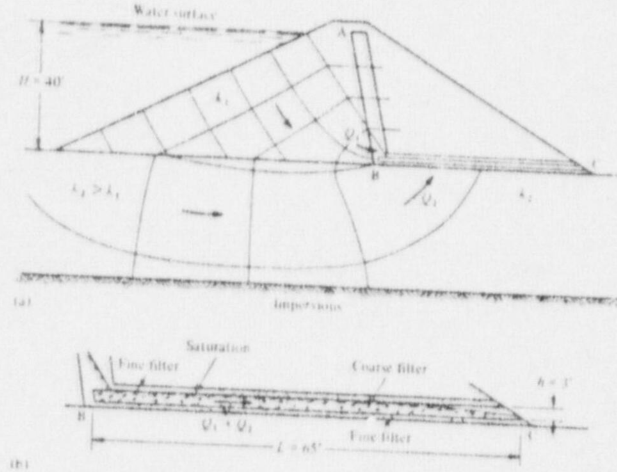


Fig. 6.8 Designing a drain for conductivity. (a) Cross section; (
horizontal drain blanket (enlarged).

4. Examples of Designing Drains for Conductivity

Earth Dam on Pervious Foundation The earth dam in Fi
6.8 is assumed to have a relatively impervious core with
coefficient of permeability of 0.1 ft/day, and its found
tion is assumed to have a permeability 40 times greater,
4 ft/day. The dam has a vertical interceptor drain betwee
A and B, and a horizontal outlet blanket drain from B to

Using the flow net to determine seepage through th
dam and foundation, the total rate (which must be remove
by the horizontal part of the drain) is estimated to b
4 ft/day (40 ft)(1.25/4) = 50 ft^3/day per linear foot of da
and drain. Allowing a factor of safety of 10, the conducti
ity of the horizontal drain blanket should be (Q_1
Q_2)(10) = (50 ft^3/day)(10) = 500 ft^3/day.

If it is assumed that the maximum desirable head in th
horizontal blanket drain in the dam in Fig. 6.8 should n
exceed 3 feet, as shown, the allowable average hydrauli
gradient in the horizontal blanket drain is 3 ft/65 ft
0.046; then the minimum required transmissibility o
horizontal part of the drain is approximately 500/0.0
11 000 ft^3/day. If the conducting layer in this drain is pe
gravel with a coefficient of permeability of 5000 ft/day
the required thickness of this layer is 11 000/5000 = 2.2 f
Other classes of permeable materials might be considere
and their thicknesses determined; however, the desig
shown in Fig. 6.8b, with a 3-ft-thick core of material wit
k = 5000 ft/day would more than satisfy the stipulated di
charge needs of this drain.

Roadbed in Wet Cut with Springs Assume that a highwa
is to be constructed in a deep cut in which pervious joint
in the bedrock cross the full width of the roadbed, and pro
duce a localized inflow of 10 gpm, equally distributed ove
a width of 50 ft, as shown in Fig. 6.9. A rate of 10 gpm i

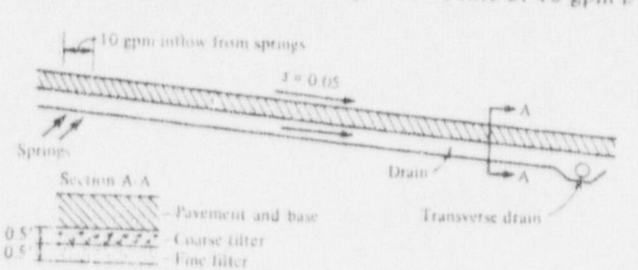


Fig. 6.9 Design of an underdrain to remove inflow from springs under a roadbed in hilly terrain.

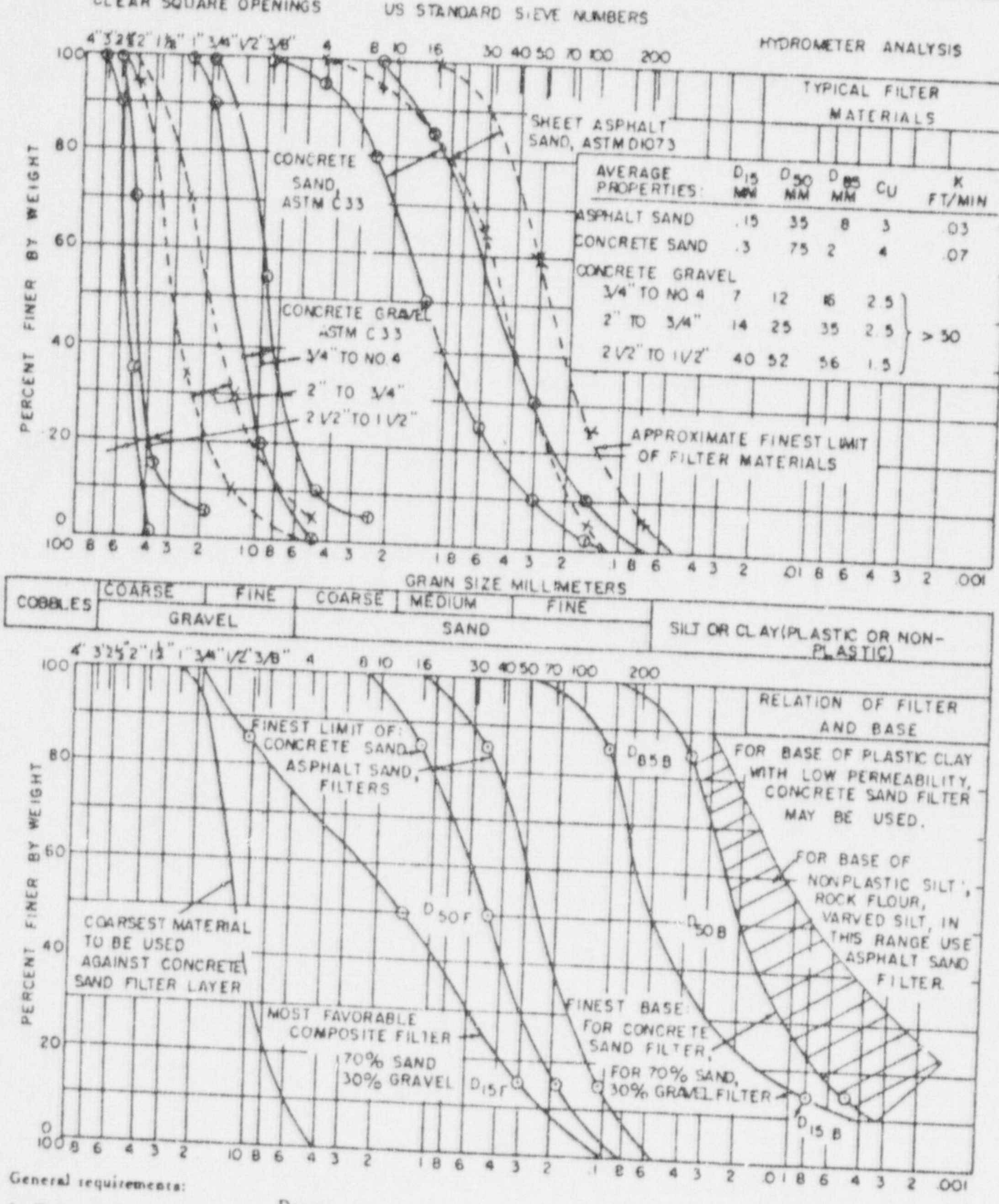
DEPARTMENT OF THE NAVY
FACILITIES ENGINEERING COMMAND

DESIGN MANUAL

SOIL MECHANICS, FOUNDATIONS, AND EARTH STRUCTURES

NAVFAC DM-7
March 1971
(Including Change 1)

DEPARTMENT OF THE NAVY
NAVAL FACILITIES ENGINEERING COMMAND
200 STOVALL STREET
ALEXANDRIA, VA. 22332



General requirements:

1. To avoid head loss in filter: $\frac{D_{15\,F}}{D_{15\,B}} > 4$, and permeability of filter must be large enough to suffice for the particular drainage system.
2. To avoid movement of particles from base: $\frac{D_{15\,F}}{D_{85\,B}} < 5$, $\frac{D_{50\,F}}{D_{50\,B}} < 25$, $\frac{D_{15\,F}}{D_{15\,B}} < 20$

   For very uniform base material ($C_u < 1.5$): $D_{15\,F}/D_{85\,D}$ may be increased to 6

   For broadly graded base material ($C_u > 4$): $D_{15\,F}/D_{15\,B}$ may be increased to 40
3. To avoid movement of filter in drain pipe perforations or joints:

   $D_{85\,F}$/slot width > (1.2 to 1.4) $D_{85\,F}$/hole diameter > (1.0 to 1.2)
4. To avoid segregation filter should contain no sizes larger than 3".
5. To avoid internal movement of fines, filter should have no more than 5% passing No. 200 sieve.

FIGURE 8-8
Design Criteria for Protective Filters

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DESIGN OF SMALL DAMS

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out a positive cutoff trench. Figure 131(B) illustrates the recommended design for a zoned dam with an impervious core larger than "Minimum Core B." The reverse slope of the impervious core (fig 131 (B)) is a device used to:

(1) Reduce the length of the downstream pervious shell.
(2) Facilitate construction of the downstream pervious shell if material excavated from the cutoff trench is used.
(3) Reduce the volume of embankment, as shown in figure 131(B).

The dashed outline in figure 131(B) indicates the drainage blanket that would be required if the reverse slope were not used.

The horizontal drainage blanket shown in figure 131(C) must satisfy three requirements:

(1) Gradation must be such that particles of soil from the foundation and the overlying embankment are prevented from entering the filter and clogging it.
(2) Capacity of the filter must be such that it adequately handles the total seepage flow from both the foundation and the embankment.
(3) Permeability must be great enough to provide easy access of seepage water in order to reduce seepage uplift forces.

Requirements for gradation and permeability are closely related and are discussed below.

A minimum drainage blanket thickness of 3 feet is suggested to provide unquestionable capacity for seepage flows.

Multilayer filters for small earthfill dams should in general be avoided; they are more efficient but add to the cost of filter construction. In cases where large seepage quantities must be handled, it has been demonstrated [39] that multilayer filters can provide an economical solution.

If the overlying pervious zones in (A) and (B) of figure 131 are sand-gravel similar in gradation to the sand-gravel of the foundation, there is no danger of flushing of particles of the foundation into the embankment, and no special filters are required. If these zones are constructed of rockfill, a filter must be provided so that the finer foundation material is not carried into the voids of the rockfill.

If sufficient quantities of filter material are available at reasonable cost, it usually will be found economical to provide thicker layers than described above rather than to process material to meet the exact requirements for thin filter design, as subsequently described. The thicker the layer, the greater the permissible deviation from the filter requirements given, especially in the requirement of parallelism of gradation curves between filter and base.

The rational approach to the design of filters is generally credited to Terzaghi [17]. Considerable experimentation has been performed by the Corps of Engineers [18] and the Bureau of Reclamation [19]. Several somewhat different sets of criteria are given by these authorities. The following limits are recommended to satisfy filter stability criteria and to provide ample increase in permeability between base and filter. These criteria are satisfactory for use with filters of either natural sand and gravel or crushed rock and for filter gradations which are either "uniform" or "graded":

(1) $\dfrac{D_{15} \text{ of the filter}}{D_{15} \text{ of base material}} = 5$ to 40, provided that the filter does not contain more than 5 percent of material finer than 0.074 mm. (No. 200 sieve)

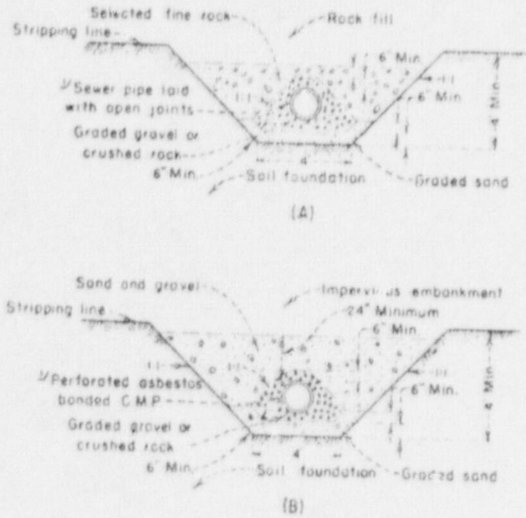
(2) $\dfrac{D_{15} \text{ of the filter}}{D_{85} \text{ of base material}} = 5$ or less

(3) $\dfrac{D_{85} \text{ of the filter}}{\text{Maximum opening of pipe drain}} = 2$ or more

(4) The grain-size curve of the filter should be roughly parallel to that of the base material.

In the foregoing, D_{15} is the size at which 15 percent of the total soil particles are smaller; the percentage is by weight as determined by mechanical analysis. The D_{85} size is that at which 85 percent of the total soil particles are smaller. If more than one filter layer is required, the same criteria are followed; the finer filter is considered as the "base material" for selection of the gradation of the coarser filter.

In addition to the limiting ratios established



1/ Drain pipes can be either vitrified clay, concrete, or asbestos bonded corrugated metal pipe (C.M.P.). The vitrified clay and concrete pipes can be either plain (laid with open joints) or perforated (laid with closed joints); if asbestos bonded C.M.P. is used it should be perforated and laid with closed joints.

Figure 133. Typical toe drain installations. 288-D-2484

for adequate filter design, the 3-inch particle size should be the maximum utilized in a filter to minimize segregation and bridging of large particles during placement of filter materials. Also, in designing filters for base materials containing gravel particles, the base material should be analyzed on the basis of the gradation of the fraction smaller than No. 4.

It is important to compact filter material to the same density as that required for construction of sand-gravel zones in embankments, as given in appendix G. Care must be used in placing filter materials to avoid segregation. The construction of thin filter layers requires proper planning and adequate inspection during placement. In many cases, it is possible that concrete sand used in the spillway, outlet works, or appurtenant structures may be used as filter material. This reduces costs by eliminating any special blending requirements.

The following is an example (see fig. 132) of a typical design which would be applicable for thin filters, such as those shown around the toe drains in figure 133.

Given:

Average gradation curve of foundation soil shown on figure 132, with $D_{15}=0.006$ mm. and $D_{85}=0.10$ mm.

Openings in drainpipe, ½ inch.

To find:

Gradation limits of filter materials.

Procedure:

(1) Lower limit of D_{15} of filter$=5\times0.006=$ 0.03 mm.

(2) Upper limit of D_{15} of filter=the *smaller* of the values: $40\times0.006=0.24$ mm., and $5\times0.10=0.50$ mm.; *use 0.24 mm.*

To meet conditions (1) and (2) and the criterion of parallelism, sand shown as F_1 in figure 132 was selected. For F_1, $D_{15}=$ 0.14 mm. and $D_{85}=2.4$ mm. This material is too fine to place adjacent to a pipe with ½-inch openings, since the requirement is for D_{85} of the filter to be at least $2\times\frac{1}{2}=1$ inch; hence, a second filter layer of gravel or crushed rock is required.

(3) Lower limit of D_{15} of gravel is 5×0.14 $=0.70$ mm.

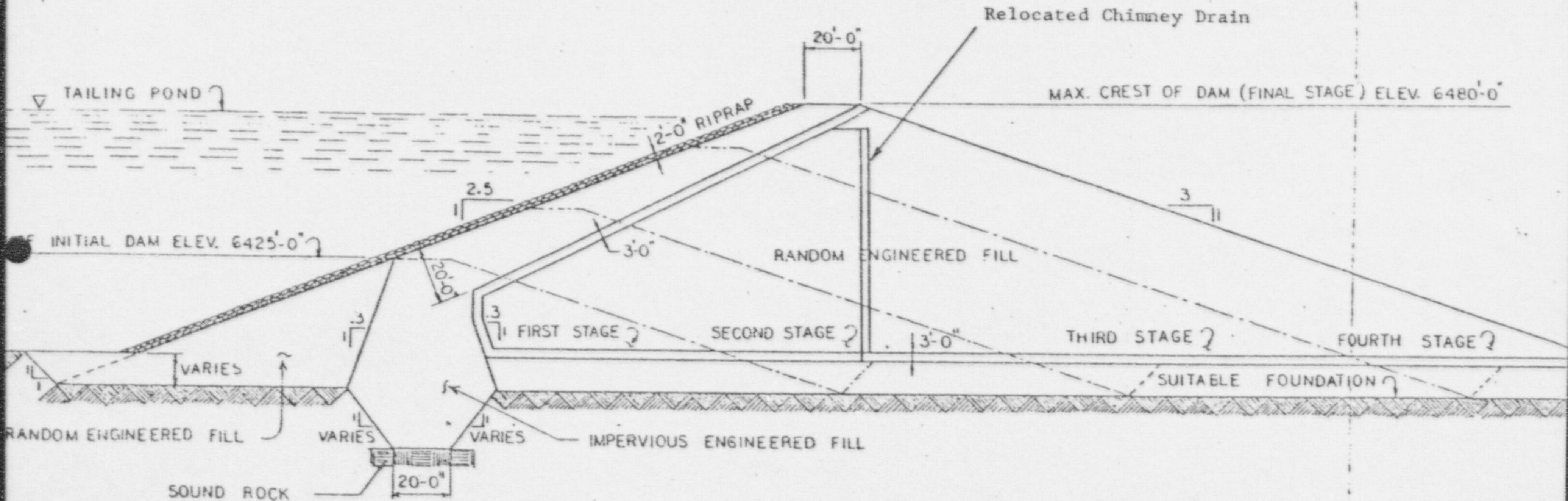
(4) Upper limit of D_{15} of gravel=the *smaller* of the values: $40\times0.14=5.6$ mm., and $5\times2.4=12.0$ mm.; *use 5.6 mm.*

(5) Least D_{85} of gravel$=2\times\frac{1}{2}=1$ inch$=$ 25.4 mm.

To meet conditions (3), (4), and (5) and the criterion of parallelism, the gravel shown as F_2 in figure 132 was selected.

The lower and upper limits of the D_{15} for F_1 and for F_2 as well as the lower limit for D_{85} for F_2 are shown on figure 132.

(j) *Toe Drains and Drainage Trenches.*—Toe drains are commonly installed along the downstream toes of dams in conjunction with horizontal drainage blankets in the position shown in figure 131. Beginning with smaller diameter drains laid along the abutment sections, the drains are progressively increased in size, the lines of maximum diameter being placed across the canyon floor. The purpose of these drains is to collect the seepage discharging from the embankment and foundation and lead it to an outfall pipe which discharges into either the spillway or outlet works stilling basin or into the river channel below the dam. Pipes rather than French drains are used to insure adequate capacity to carry seepage flows. Toe drains are also used on impervious foundations to insure that any seepage that may come through the foundation or the embankment is collected and that the ground-



TYPICAL DAM SECTION AT MAXIMUM HEIGHT

SCALE: 1:40

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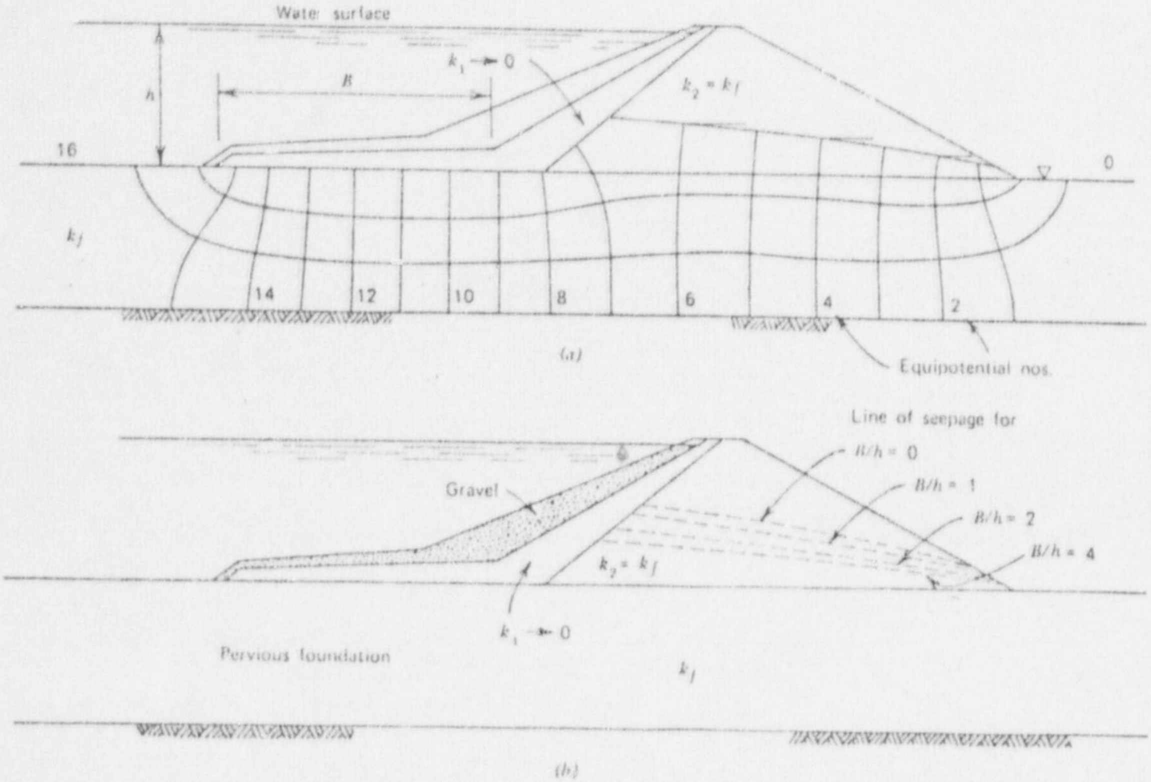


Fig. 15. Effect of upstream blanket on position of line of seepage. (*a*) Typical flow net ($B/h = 2$). (*b*) Position of line of seepage for various values of B/h.

Relief wells frequently offer the most economical, positive control over underseepage (Turnbull and Mansur, 1961a; Middlebrooks and Jervis, 1947). However, in some cases deep slurry trenches or other thin cutoffs may offer the most practical, economical solution (Cedergren, 1967).

Summary

In the preceding paragraphs a few of the many seepage-reducing methods in widespread use have been described. Seepage-reducing methods depend on the introduction of relatively impervious elements at the upstream side, or well within the cross section of a dam, its foundation, and its abutments. These impervious elements restrict the area through which seepage can occur, lengthen the seepage path, or reduce the permeability. Several flow-net studies presented in this paper show that seepage-reducing methods must be almost perfectly efficient if they are to greatly increase stability and control underseepage. Since perfection is not easily attained, seepage-reducing methods often are not sufficient alone, and they usually should be combined with some form of drainage.

4. DRAINAGE METHODS

Design of Drains

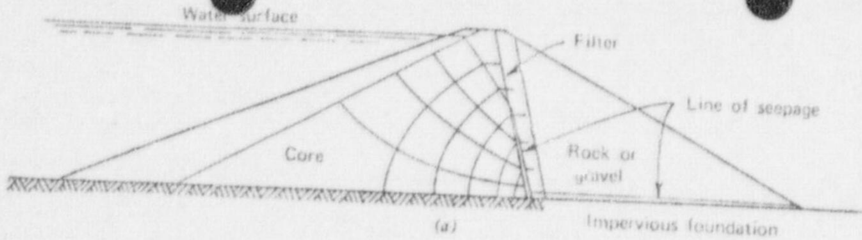
Drainage methods depend on the introduction of *highly permeable* discharge elements into the cross section. Filters for drains must be designed to prevent piping, as described in Section 2, and drains must permit discharge of the seepage water without excessive head loss.

As stated in Section 2, the following criterion is used for ensuring that filters will be more pervious than the soils they protect:

$$\frac{D_{15}\text{ (of filter)}}{D_{15}\text{ (of soil)}} > 4 \text{ or } 5 \qquad (2)$$

If the 15% size (D_{15}) of a filter is 4 or 5 times the 15% size (D_{15}) of the protected soil, the permeability of the filter generally will be at least 10 to 20 times that of the soil. This ensures that the head loss in the filter generally is not significant.

Equation 2 is a suitable criterion for providing adequate permeability in drainage situations similar to those shown in Fig. 17, in which flow is *across* the narrow dimension of the filter into considerably



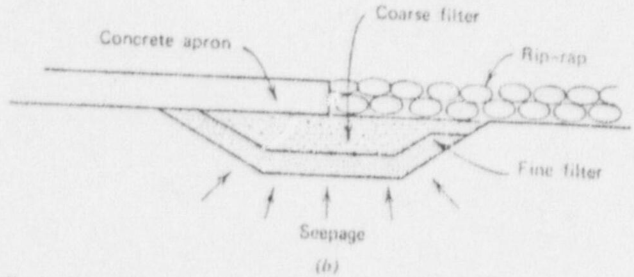


Fig. 17. Examples of filters with large discharge gradients and areas. (a) Earth dam with internal transition. (b) Toe drain for overflow weir.

more pervious materials which remove the water. However, in some cases, such as horizontal drains in earth dams, water must flow through the drain under relatively small hydraulic gradients and through comparatively small cross-sectional areas, as in the horizontal outlet drain in Fig. 18. In such cases, Eq. 2 does not necessarily ensure adequate discharge capacity, and the drain should be designed as a hydraulic conductor, capable of removing at least several times the anticipated seepage quantities. When drains are designed and built with ample discharge capacity, the line of seepage does not rise above the drain zones (Fig. 18).

The required minimum permeability and thickness of a drain can be estimated with flow nets or with Darcy's law, as described in the following paragraphs. In principle, one could sketch flow nets to determine, by trial and error, the necessary dimensions to ensure adequate discharge capacity of the drains. Figure 19 is an illustration of one such flow net in which all of the flow channels are compressed within the boundaries of the drains.

Since drains usually are very slender in proportion to the overall dimensions of an earth dam, it becomes tedious and difficult to construct accurate flow nets within the drains themselves. More practical, although somewhat approximate procedures are described in the following paragraphs.

The total combined *quantity* of seepage q from all sources that must discharge through a drain can be evaluated from a flow net analysis in which it is assumed that the drains have an infinite permeability. For a horizontal drain, the designer should ensure that the line of seepage does not rise to the top of the drain.

For a given permeability of the drain material, the required thickness h_3 for a given value of k_3 can be computed using the following formula, which is derived for laminar flow (i.e., on the basis of Darcy's law):

$$q = \frac{k_3 h_3^2}{2L_3} \tag{6}$$

Equation 6 was derived by Dupuit (1863) on the basis of simplifying assumptions and was shown by Charny (1951) to be the analytically exact solution for flow

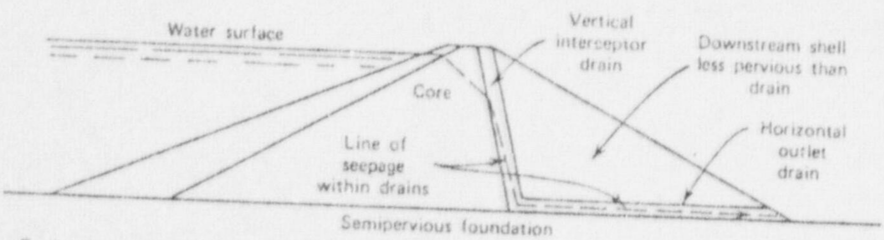


Fig. 18. Earth dam with internal drain designed to prevent line of seepage from rising above top of drain.

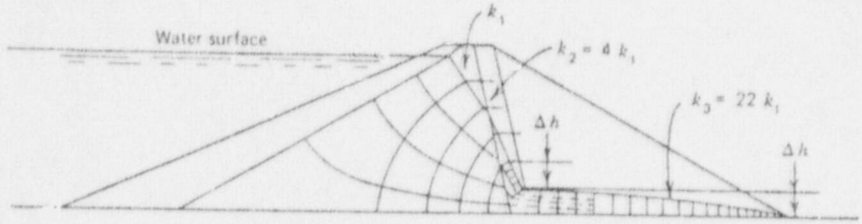


Fig. 19. Flow net for core and drains.

through the section shown in Fig. 20*b* (see Lo, 1969, p. 7-3). It is a very good approximation for the type of drain shown in Fig. 20*a*, because the section has such a small h_3/L_3 ratio that the geometry of the entrance and discharge faces has an insignificant effect.

If flow in a drain reaches a turbulent or semi-turbulent state, discharge rates will be less than those estimated on the basis of Darcy's law, and the required thickness of the drains will be larger than calculated.

The minimum required permeability of a vertical interceptor drain such as shown in Fig. 19 can be estimated with Darcy's law if the quantity of water to be removed can be predicted with reasonable accuracy. Thus in Fig. 20*c*, the discharge capability of the vertical drain is $q_2 = k_2 i_2 A_2$, and by rearranging the terms, $k_2 = q_2/i_2 A_2$. Since for this nearly vertical drain, i_2 can be taken as h_2/L_2, which is nearly unity, the minimum required drain permeability $k_2 = q_2/A_2$.

Frequently the minimum thickness of drains depends on practical placement considerations such as the steepness of the surfaces on which a drain is to be constructed and on the costs of using spreader boxes or movable forms in placing materials in narrow drains as compared to the cost of placing larger quantities of materials in wider drains by inexpensive placing and spreading methods. If there is a possibility that a dam may undergo shear because of displacements along an underlying fault, the drains should be made conservatively wide to reduce the probability that they might be completely severed.

Pervious Downstream Shells

At many dam sites abundant quantities of at least two different materials with significantly different permeabilities are available. In such cases, a pervious material is placed downstream of a less pervious material, frequently with a narrow transition between. Figure 17*a* is a cross section through such a *zoned* dam, which rests on an impervious foundation and has a thick impervious core. The line of seepage in the downstream portion is very low, and seepage has a negligible effect on the stability of the downstream slope, which is the ideal condition in zoned earth dams. Large, well-drained masses of earth in the downstream parts of dams have inherently large resistance to failure not only under normal static conditions, but also under shocks caused by earthquakes.

It has been previously seen (Figs. 11*c*, 12*c*, 14*c*, and 15*b*) that when earth dams are constructed on semipervious or pervious foundations, the line of seepage can rise substantially above the base of the dam, greatly lowering stability and increasing seepage problems.

Zoning alone may not be sufficient to control seepage through an earth dam if either of the following conditions exist:

1. The permeability of the downstream zone is not at least 100 times greater than that of the im-

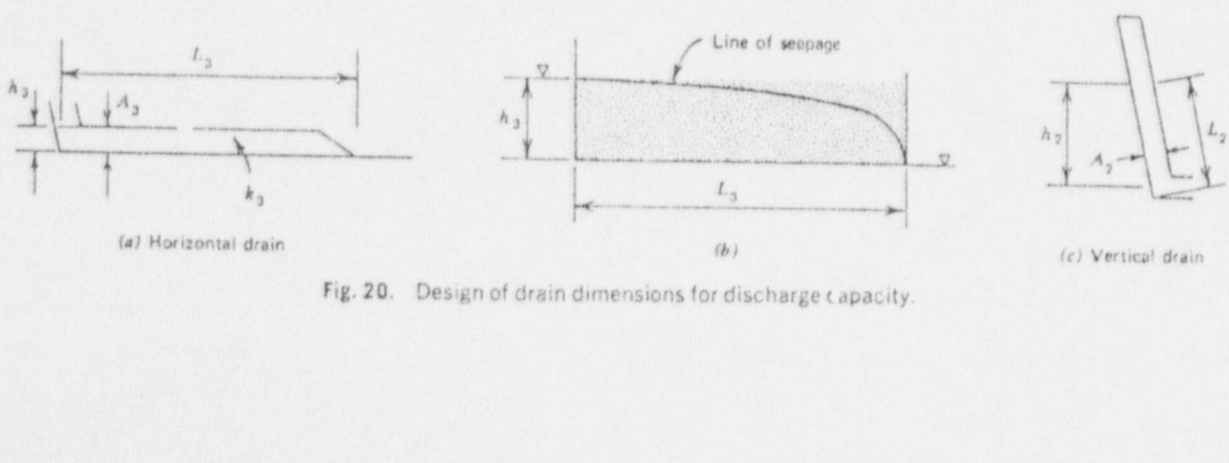


Fig. 20. Design of drain dimensions for discharge capacity.