CHAPTER 9
SEEPAGE CONTROL IN EARTH FOUNDATIONS

9-1. General. All dams on earth foundations are subject to underseepage. Seepage control in earth foundations is necessary to prevent excessive uplift pressures and piping through the foundation (seepage control in earth abutments is given in Chapter 10). The purpose of the project, i.e., long-term storage, flood control, hydropower, etc., may impose limitations on the allowable quantity of seepage. Generally, siltation of the reservoir with time will tend to diminish underseepage (U. S. Army Engineer Division, Ohio River 1945). Conversely, the use of some underseepage control methods such as relief wells and toe drains may increase the quantity of underseepage (Sowers 1962).

9-2. Selection of Method for Seepage Control. The methods for control of underseepage in dam foundations are horizontal drains, cutoffs (compacted backfill trenches, slurry walls, concrete walls, and steel sheetpiling[1]), upstream impervious blankets, downstream seepage berms, toe drains, and relief wells. To select an underseepage control method for a particular dam and foundation, the relative merits and efficiency of different methods should be evaluated by means of flow nets or approximate methods as described in Chapter 4 and Appendix B, respectively. As shown in table 9-1, the changes in the quantity of underseepage, factor of safety against uplift, and uplift pressures at various locations should be determined for each particular dam and foundation. Since the anisotropy ratio of the foundation has a significant influence on the results of the underseepage analysis, this parameter should be varied as appropriate $\left(\frac{K_H}{K_V} = 1, 10, 25, 100\right)$ to cover the possible range of expected field conditions.

9-3. Horizontal Drains. As mentioned previously in Chapter 8, horizontal drains are used to control seepage through the embankment and to prevent excessive uplift pressures in the foundation. As shown in figure 9-1, the use of the horizontal drain significantly reduces the uplift pressure in the foundation under the downstream portion of the dam. The computation of uplift pressure was illustrated previously in figure 4-15. Figure 9-1 also shows that the horizontal drain increases the quantity of seepage under the dam.

9-4. Cutoffs.

a. Complete Versus Partial Cutoff. When the dam foundation consists of a relatively thick deposit of pervious alluvium, the designer must decide whether to make a complete cutoff (compacted backfill trench, slurry trench, or concrete wall) or allow a certain amount of underseepage to occur under controlled conditions (partial cutoff, upstream impervious blanket, downstream seepage berm, toe trench drain, or relief walls), In some cases, where the alluvium is not very deep or the water is very valuable, it may be obvious

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(1) Steel sheetpiling is not recommended to prevent underseepage but is used to confine the foundation soil and prevent piping.

Table 9-1. Selection of Underseepage Control Method

Method	Changes in Quantity of Underseepage	$F_s = \frac{i_c}{i_e}$ (a)	Uplift Pressure		
			Under Dam	Toe of Dam	Downstream of Dam
Horizontal Drain					
Complete Cutoff	Fill in table for each particular dam and foundation. Vary the permeability anisotropy of the foundation $\left(\frac{K_H}{K_V} = 1, 10, 25, 100\right)$ as appropriate.				
Partial Cutoff					
Upstream Impervious Blanket					
Downstream Seepage Berm					
Toe Drain					
Relief Wells					

(a) Factor of safety against uplift at downstream toe of dam given by the ratio of the critical hydraulic gradient to the vertical component of the exit gradient.

(prepared by WES)

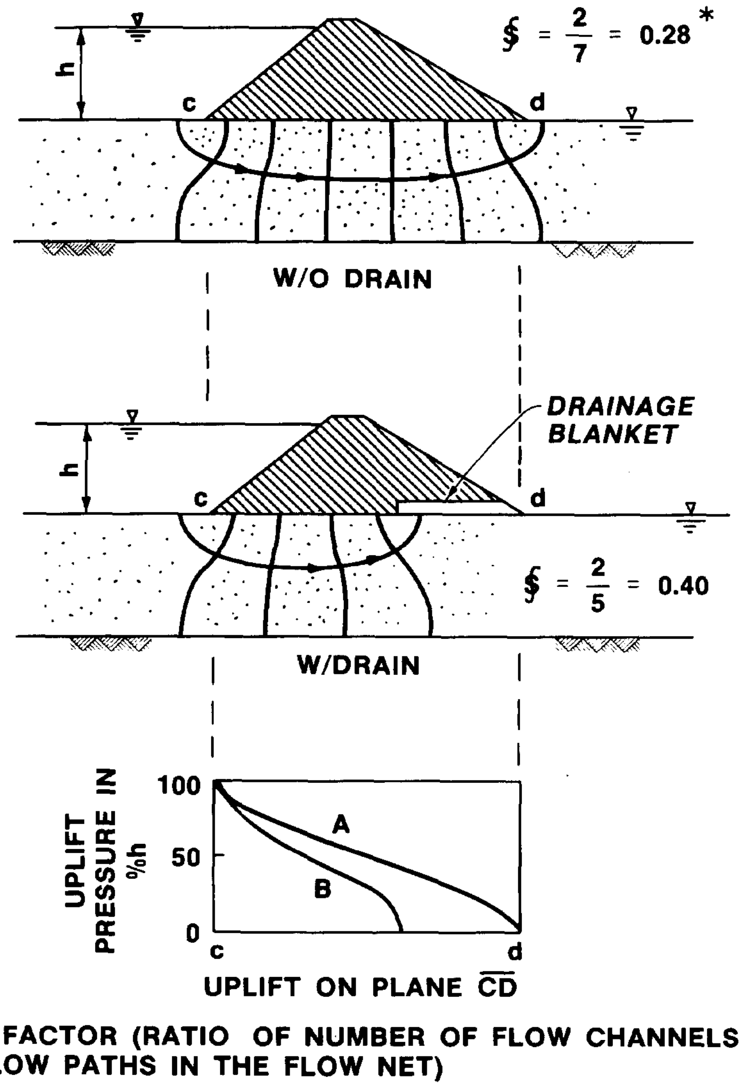


Figure 9-1. Influence of horizontal drain on uplift pressure (prepared by WES)

after relatively little study that a complete cutoff is justified. For example, the hydropower requirements of the Clarence Cannon Dam, Missouri, indicated that a complete cutoff was required to sustain power requirements during periods of little or no rainfall (U. S. Army Engineer District, St. Louis 1969). In many cases, where the cost of a complete cutoff is great and where the amount of underseepage without a complete cutoff is problematical, the decision is not easy. Factors which govern the decision for the type of underseepage control measure to be used are (Sherard 1968):

(1) Economic comparison of the value of the water or hydropower which may be lost versus the cost of the complete cutoff.

(2) The resistance of the foundation alluvium with respect to potential progressive backward erosion of leaks or piping. If the foundation contains layers of fine sand or cohesionless silt, and particularly if these soils are exposed on the surface of the valley floor or walls, a complete cutoff is more desirable than if the foundation is basically gravelly (or even coarse sand). If a large leak develops in a relatively coarse alluvium, in all probability it will be safe against progressive backward erosion, but even a small concentrated leak emerging below the dam in fine cohesionless soils can be hazardous.

(3) If the tailwater conditions are such that ponds of water exist downstream of the dam so that underseepage would emerge underwater and could not be observed, it is desirable to be more conservative in evaluating the need for a complete seepage cutoff.

(4) The amount of silt and clay sized particles in suspension in the river water which contributes to siltation of the reservoir with time and tends to diminish underseepage.

Theory and model tests indicate that it is necessary for a cutoff to penetrate a homogeneous isotropic foundation at least 95 percent of the full depth before there is any appreciable reduction in seepage beneath an earth embankment as shown in figure 9-2 (Telling, Menzies, and Coulthord 1978; and Mansur and Perret 1949). The effectiveness of the partial cutoff in reducing the quantity of underseepage decreases as the ratio of the width of the dam to the depth of penetration of the cutoff increases (see figure 9-2). Partial cutoffs are effective only when they extend down into an intermediate stratum of lower permeability. This stratum must be continuous across the valley foundation to ensure that three-dimensional seepage around a discontinuous stratum does not negate the effectiveness of the partial cutoff.

b. Efficiency of Cutoffs. The effectiveness of the cutoff is assessed either in terms of the flow efficiency (Casagrande 1961)

$$E_q = \frac{Q_o - Q}{Q_o} \tag{9-1}$$

where

E_q = flow efficiency of cutoff

Q_o = rate of underseepage without cutoff

Q = rate of underseepage with cutoff

or head efficiency (Lane and Wohlt 1961)

$$E_H = \frac{h}{H} \tag{9-2}$$

where

E_H = head efficiency of cutoff

h = head loss between points immediately upstream and downstream of the cutoff wall at its junction with the base of the dam

H = head loss across the dam

The head efficiency is more widely used because the field performance may be established from piezometric data taken during construction, before and during initial filling of the reservoir, and subsequently as frequently as necessary to determine changes that are occurring and to assess their implications with respect to safety of the dam, as described in Chapter 13. The flow efficiency may only be approximated since the rate of underseepage without the cutoff cannot be directly established and since it is difficult to measure the rate of underseepage with the cutoff because, except for special cases, only part of the underseepage discharges at the ground surface immediately downstream of the dam (Telling, Menzies, and Simons 1978a and Marsal and Resendiz 1971). The flow efficiency of a compacted backfill partial cutoff in a foundation of permeable soils of moderate thickness overlying an impervious rock is shown in figure 9-3. This figure also illustrates the high seepage gradients that occur along the base of the cutoff and on its downstream face in both the foundation and embankment zones. Suitable filters must be provided to prevent piping of soil at faces A-B-C in figure 9-3a and 9-3b (Cedergren 1977 and Klohn 1979). As shown in figure 9-4, a partial cutoff in a homogeneous isotropic foundation will lower the line of seepage in the downstream embankment somewhat but exit gradients at the downstream toe (as reflected by the distance between the equipotential lines) are reduced only slightly (Cedergren 1973). When the pervious foundation is cut off by a compacted backfill or slurry trench,[1] the rate of underseepage may be estimated by (Ambraseys 1963 and Marsal and Resendiz 1971)

$$\frac{Q_o}{K_o H} = \frac{1}{0.88 + \frac{B}{D} + \left(\frac{K_o}{k} - 1\right)\frac{E}{D}} \qquad (9\text{-}3)$$

where

Q_o = rate of underseepage in m^3/set per running meter of dam

K_o = permeability of the foundation in m/sec

[1] This approach neglects the contribution of the filter cake that forms on the trench walls to the overall slurry trench permeability. When the permeability of the backfill placed in the trench is high, the overall slurry trench permeability will be controlled by the filter cake (D'Appolonia 1980).

H = head of water in the reservoir in m

B = width of the base of the dam in m

D = thickness of the foundation in m

k = permeability of the compacted backfill or slurry trench backfill in m/sec

E = thickness of the cutoff in m

For a concrete wall or steel sheetpiling with defects (openings in the cutoff) the rate of underseepage per unit length of cutoff is given by (Ambraseys 1963 and Marsal and Resendiz 1971)

$$\frac{Q_o}{K_o H} = \frac{1}{0.88 + \frac{B}{D} + \left(\frac{D}{W} - 1\right) \frac{E}{D}} \tag{9-4}$$

where

W = total area of openings in m^2

Figure 9-5 compares the rate of underseepage for an impervious upstream blanket, compacted backfill trench or slurry trench, and concrete wall or steel sheetpiling with defects. As shown in figure 9-5, the rate of underseepage loss is the same for an impervious upstream blanket as for a compacted backfill trench or slurry trench provided $B' = \left(\frac{K_o}{k} - 1\right) E$. If K is relatively high, assuming $\frac{K_o}{k} = 50$ gives B' = 49 E . Such computations allow preliminary cost estimates to be made to determine whether an impervious upstream blanket is preferable over a compacted backfill trench or slurry trench. Figure 9-6 can be used to determine the relative magnitudes of the length of the impervious upstream blanket, the thickness of the compacted backfill trench or slurry trench, or the area of the defects (openings) in the concrete wall or steel sheetpiling that would result in the same rate of underseepage for a given dam.

c. Compacted Backfill Trench. The most positive method for control of underseepage consists of excavating a trench beneath the impervious zone of the embankment through pervious foundation strata and backfilling it with compacted impervious material. The compacted backfill trench is the only method for control of underseepage which provides a full-scale exploration trench that allows the designer to see the actual natural conditions and to adjust the design accordingly, permits treatment of exposed bedrock as necessary, provides access for installation of filters to control seepage and prevent piping of soil at interfaces, and allows high quality backfilling operations to be carried out (U. S. Army Engineer District, St. Louis 1969 and Cedergren 1977).

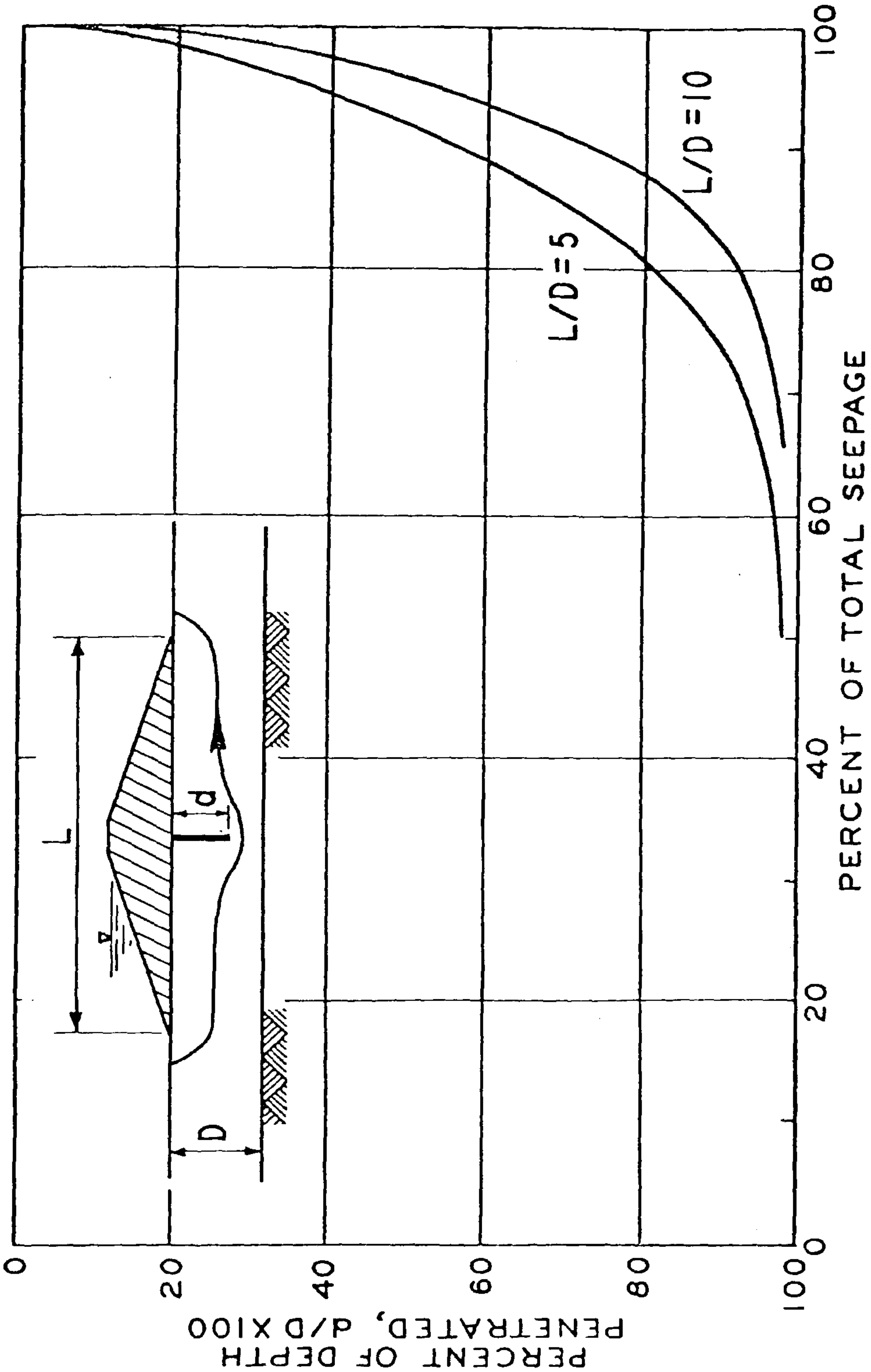
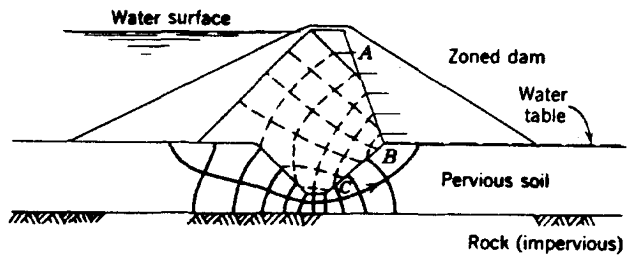
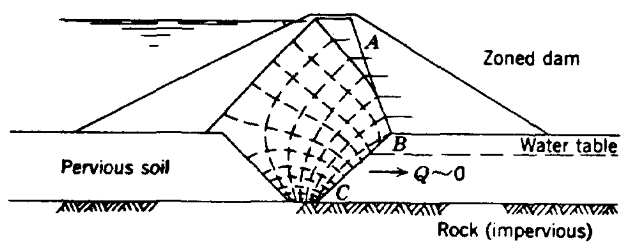


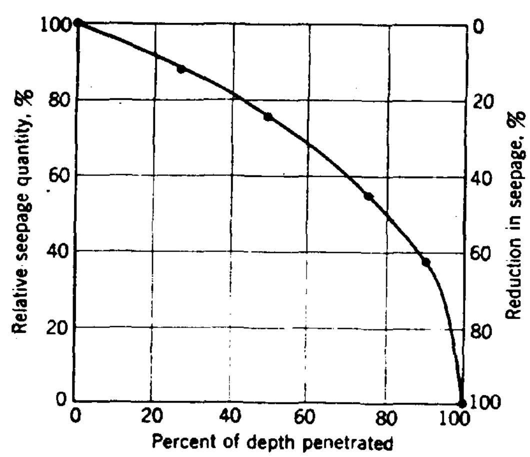
Figure 9-2. Effect of depth of penetration of partial cutoff on seepage reduction for a homogeneous isotropic foundation (prepared by WES)



a. Partial cutoff

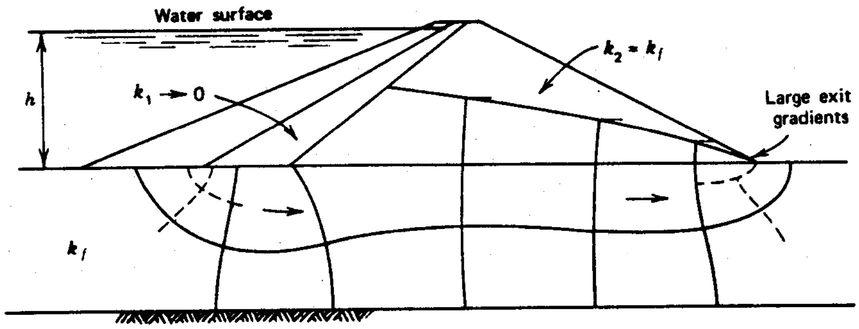


b. Complete cutoff

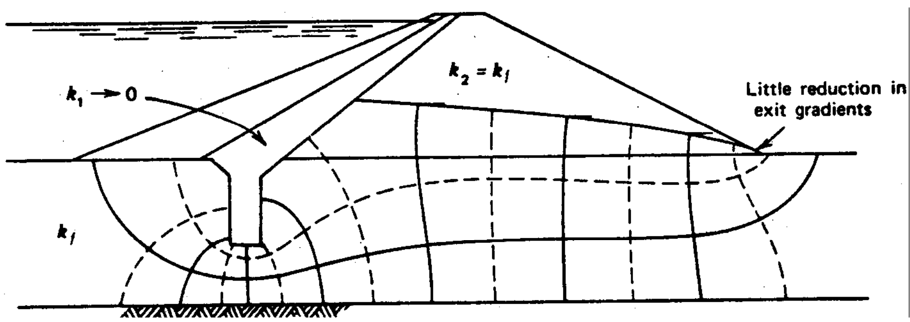


c. Relationship between quantity of seepage and depth of penetration of partial cutoff

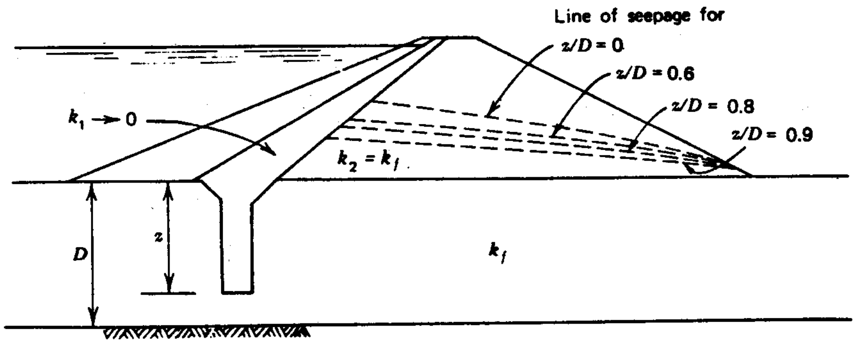
Figure 9-3. Efficiency of a compacted backfill trench partial cutoff in reducing the quantity of underseepage (courtesy of John Wiley and Sons[155])



a. Flow net for no cutoff

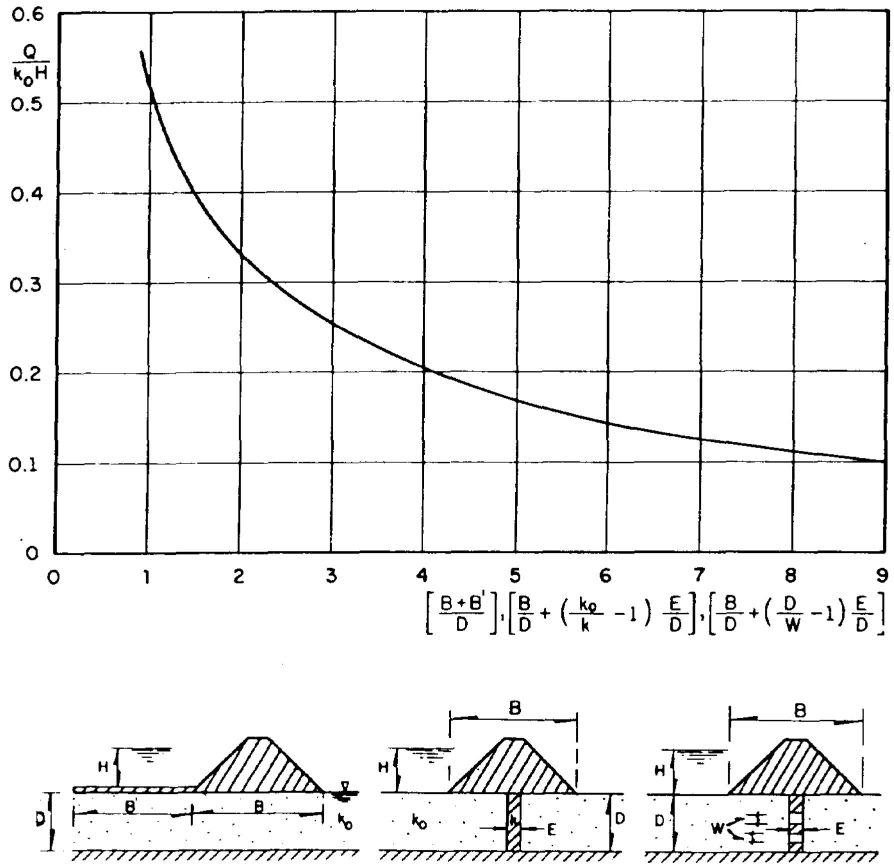


b. Flow net for partial cutoff



c. Position of line of seepage for various values of penetration

Figure 9-4. Effect of depth of penetration of partial cutoff on the height of the line of seepage in the downstream embankment and exit gradient at the toe for a homogeneous isotropic foundation (courtesy of John Wiley and Sons[154])



Impervious upstream blanket

Compacted back fill or slurry trench

Concrete wall or steel sheetpiling

Figure 9-5. Rate of underseepage loss for impervious upstream blanket, compacted backfill trench or slurry trench, and concrete wall or steel sheetpiling with defects (courtesy of American Society of Civil Engineers[218])

Material and compaction requirements are the same as those for the impervious section of the dam (EM 1110-2-1911). When constructing a complete cutoff (see para 9-4a), the trench must fully penetrate the pervious foundation and be carried a short distance into unweathered and relatively impermeable foundation soil or rock. To ensure an adequate seepage cutoff, the base width should be at least one-fourth the maximum difference between the reservoir and tailwater elevations but not less than 20 ft, and should be wider if the foundation material under the cutoff is considered marginal in respect to imperviousness. As previously mentioned (see para 9-4b), high seepage gradients occur along the base of the cutoff and on its downstream face in both the foundation and embankment zones. Suitable filters must be provided (see Appendix D for design of filters) to prevent piping of soil at these interfaces. The trench

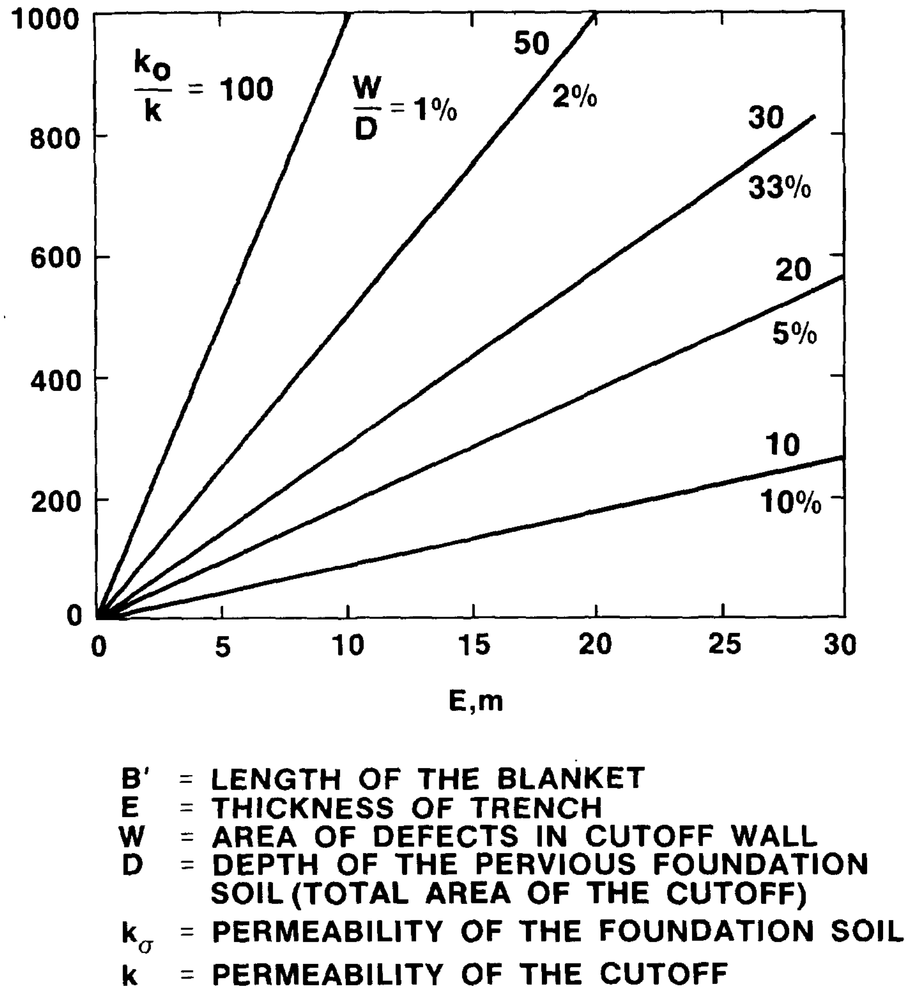


Figure 9-6. Relationships among the length of the impervious upstream blanket, the thickness of the compacted backfill trench or slurry trench, or the area of defects in the concrete wall or steel sheet-piling, for a given rate of underseepage loss (courtesy of American Society of Civil Engineers[218])

excavation must be kept dry to permit proper placement and compaction of the impervious backfill. Dewatering systems of wellpoints or deep wells are generally required during excavation and backfill operations when below groundwater levels (TM 5-818-5). Because construction of an open cutoff trench with dewatering is a costly procedure, the trend has been toward use of the slurry trench method of construction (EM 1110-2-2300 and Cedergren 1977).

d. Slurry Trench.

(1) Introduction. When the cost of dewatering and/or the depth of the pervious foundation render the compacted backfill trench too costly and/or impractical, the slurry trench cutoff may be a viable method for control of

underseepage. Using this method, a trench is excavated through the pervious foundation using a sodium bentonite clay (or Attapulgite clay in saline water) and water slurry to support the sides. The slurry-filled trench is backfilled by displacing the slurry with a backfill material that contains enough fines (material passing the No. 200 sieve) to make the cutoff relatively impervious but sufficient coarse particles to minimize settlement of the trench forming a soil-bentonite cutoff (sometimes called American method). Alternatively, cement may be introduced into the slurry-filled trench which is left to set or harden forming a cement-bentonite cutoff (sometimes called a grouted diaphragm wall or Coulis wall or European method). The slurry trench cutoff is not recommended when boulders, talus blocks on buried slopes, or open jointed rock exist in the foundation due to difficulties in excavating through the rock and slurry loss through the open joints. Where a slurry trench is relied upon for seepage control, the initial filling of the reservoir must be controlled and piezometers located both upstream and downstream of the cutoff must be read to determine if the slurry trench is performing as planned. If the cutoff is ineffective, remedial seepage control measures (see Chapter 12) must be installed prior to further raising of the reservoir pool (KM 1110-2-2300).

(a) History of Use. The first use of the slurry trench method of construction was by the U. S. Army Engineer District, Memphis, in September 1945, to form a partial cutoff along the Mississippi River levee on the Arkansas side of the river just below Memphis, Tennessee (Clay 1976 and Kramer 1946). The idea for the project probably evolved from the use at that time of puddle clay trench cutoffs combined with the use of drilling mud for advancing borings. A paddle wheel mixing device was constructed for making slurry from native clays. Trenches were dug to a 20-ft depth using a trenching machine and to a 35-ft depth using a dragline with a 100-ft boom and 2-cu-yd bucket. Backfill was mixed in windrows at the site from hauled-in clay gravel and native materials and pushed into the trench by a bulldozer when the length of the trench was equal to about twice the trench depth. BG Hans Kramer foresaw the use of the slurry trench method for the construction of cutoffs for earth dams. It is amazing that after 38 years, the technique is still about the same as it was when first developed by the Memphis District. A soil-bentonite cutoff was constructed under the Kennewick Levee adjacent to the Columbia River as part of the McNary Dam Project in Washington by the Walla Walla District in 1952 (Jones 1961). The first application of a soil-bentonite slurry trench cutoff for control of underseepage at a major earth dam was at Wanapum Dam on the Columbia River in Washington in 1959 (La Russo 1963). Subsequently, soil-bentonite cutoffs have been used for control of underseepage at a number of dams as shown in table 9-2. The cement-bentonite slurry trench cutoff was first used to tie into the abutment zones at the Razaza Dam on the Euphrates River in Iraq in 1969 (Soletanche 1969). Subsequently, cement-bentonite cutoffs were installed as remedial seepage control measures through the embankment and foundation of four existing dams in Mexico from 1970 to 1972 (Soletanche 1970, 1971, 1971-1972, 1972). As shown in table 2, the first cement-bentonite cutoff in the United States was constructed at the Tilden Tailings Project to store tailings from the Tilden Mine in Michigan in 1976 (Meier and Rettberg 1978). The first cement-bentonite cutoff constructed at a dam on a river retaining a reservoir in the United States was completed in 1978 at the Elgo Dam (formerly the San Carlos Dam) in Arizona (Anonymous 1978 and Miller and Salzman 1980).

Table 9-2. Comparison of Slurry Trench Cutoffs

Project	Location	Owner	Date Constructed	Foundation Material	Max. Differential Head ft	Cutoff Width ft	Cutoff Max. Depth ft	Cutoff Location	Max. Head / Cutoff Width	Reference
				Soil-Bentonite Slurry Trench						
Kennewick Levee, McNary Dam	Columbia River, Wash.	Corps of Engineers	1952	Sandy or silty gravel with zones of open gravel	15	6	22	Center of dam	2.5	Jones 1961
Wanapum Dam	Columbia River, Wash.	Public Utility District 2, Grant County, Wash.	1962	Sandy gravels and gravelly sands underlain by open work gravels	88	10	80	Center of dam	8.8	La Russo 1963
Wells Dam	Columbia River, Wash.	Public Utility District 1, Douglass County, Wash.	1964	Pervious gravels	70	8	80	Center of dam	8.8	Jones 1967
Yards Creek Lower Reservoir	New Jersey	Public Service Electricity and Gas	1964	Sands, gravels, cobbles, and boulders	55	8	40	Center of dam	6.9	Jones 1967
Comanche Dam - Dike 2	Mokelumme River, Calif.	East Bay Municipal District	1966	Upper stratum of clayey silts, silts, and clayey sands. Lower stratum of sand over zone of gravel	45	8	95	Upstream of toe of dam	5.6	Anton and Dayton 1972
West Point Dam	Chattahoochie River, Ga. and Ala.	Corps of Engineers	1966	Upper stratum of alluvial soil, alternating layers of clay, silt, sand, and gravel. Lower stratum of residual soil, brown silty sand	61	5	60	Upstream toe of dam	12.2	U.S. Army Engr. Dist, Savannah 1968, 1979
Saylorville Dam	Des Moines River, Iowa	Corps of Engineers	1969	Surface zone of sandy clay. Lower stratum of sand and gravel	93	8	65	Upstream toe of dam	11.6	Harza 1965, U.S. Army Engr. Dist, Rock Island 1978
Mill Site Dam	Ferron Creek, Utah	Soil Conservation Service	1969	Sandy gravel	73	8	68	Upstream toe of dam	9.1	Hanson 1972
Plum Creek Dam	Plum Creek, Wis.	Soil Conservation Service	1973	Sands, gravelly sands, silty gravels, silty sands	54	5	40	Center of dam	10.8	Knabach and Dingle 1974
Upper Big Blue Dam	Upper Big Blue River, Ind.	Soil Conservation Service	1978	Sands and gravels	62	4	40	Center of dam	15.5	Bloom, Dynes, Glossett 1979
Addicks Dam[a]	Buffalo Bayou, Tex.	Corps of Engineers	1979	Silty sands	30	3	70	Varies	10.0	U.S. Army Engr. Dist, Galveston 1983 (same for Barker Dam)
Barker Dam[a]	Buffalo Bayou, Tex.	Corps of Engineers	1979	Sandy clay, clayey sand, silty sand	24	3	55	Center of	8.0	
				Cement-Bentonite Slurry Trench						
Tilden Tailings	Tilden Mine, Mich.	Cleveland-Cliffs Iron Company	1976	Sand interspersed with gravel	100[b]	2	80	Upstream toe of dam	50.0	Meier and Rettberg 1978
Elgo Dam (formerly San Carlos Dam)	San Carlos River, Ariz.	San Carlos Apache Tribe	1978	Sands and gravels	66	3	68	Center of dam	22.0	Miller and Salzman 1980

(a) Slurry trench cutoff installed as remedial seepage control for existing dam (see Chapter 12).
(b) Planned maximum differential head when tailings dam is at final height (45 ft in February 1983).

(b) Patents. The soil-bentonite slurry trench cutoff was covered by United States Patent No. 2,757,514 dated August 7, 1956, "Method of Forming an Impermeable Wall in the Terrain," in the name of Harold T. Wyatt. This patent expired in 1973. The cement-bentonite slurry trench cutoff is covered by United States Patent No. 3,759,044 dated September 18, 1973, "Method of Earth Wall Construction Using Cementitious Bentonite Mud," in the name of Claude Caron and Jean Hurtado, both of France, assignor to Soletanche, Paris, France.

(c) Comparison of Soil-Bentonite and Cement-Bentonite Slurry Trench Cutoffs. A comparison of soil-bentonite and cement-bentonite slurry trench cutoffs is given in table 9-3. The soil-bentonite slurry trench cutoff is generally the most economical if the cost of backfill is not prohibitive. For deep cutoffs where the foundation is prone to failure during excavation, the cement-bentonite slurry trench cutoff is more applicable.

(d) Location of Cutoff. Normally, the slurry trench should be located under or near the upstream toe of the dam (EM 1110-2-2300). An upstream location provides access for future treatment provided the reservoir could be drawn down and facilitates stage construction by permitting placement of a downstream shell followed by an upstream core tied into the slurry trench. For stability analysis, a soil-bentonite slurry trench cutoff should be considered to have zero shear strength and exert only a hydrostatic force to resist failure of the embankment (U. S. Army Engineer District, Savannah 1968). If the slurry trench is located under a central core, consolidation of the slurry trench backfill combined with arching of the core material immediately above the slurry trench may result in the opening of a cavity under the dam with possible leakage along the contact. If a central location for the slurry trench is dictated by other factors, some possible benefits are obtained by flaring the top of the trench to provide a transition between the cutoff and the core. Also, the slurry trench can be constructed and allowed to settle before placement of the embankment (Jones 1967 and Jansen 1968). When the groundwater table is located some distance beneath the ground surface, it is usually more economical to excavate a conventional open trench with stable side slopes with the trench bottom a few feet above the highest level of ground water expected during the construction period, as was done at West Point Dam, Alabama and Georgia (U. S. Army Engineer District, Savannah 1968). The bottom of the open trench provides a working level from which the slurry trench may be constructed. Also, this prevents the problem of significant amounts of slurry being lost into the excavated trench above the ground-water table. If the ground-water table is located near the ground surface, compacted impervious fill should be placed in order to raise the level of the slurry trench to maintain the level of slurry in the trench a sufficient distance above the ground-water level (Jones 1967).

(e) Stability of the Trench. In cohesionless soils the penetration of the slurry into the wall of the slurry trench excavation forms a relatively impervious filter cake on which the hydrostatic pressure of the slurry can act. The depth of penetration ranges from 1 to 3 in. in sand, 3 to 6 in. in sand and gravel, and up to 12 in. in gravel, depending on the gradation. The main stabilizing force supporting the slurry trench excavation is the hydrostatic pressure exerted on the trench walls. For a slurry trench excavated in a homogeneous clay, remaining open only for a few days to permit placement of

Table 9-3. Comparison of Soil-Bentonite and Cement-Bentonite Slurry Trench Cutoffs[(a)]

Item	Soil Bentonite Slurry Trench Cutoff	Cement-Bentonite Slurry Trench Cutoff
1. Excavation	Long slope of backfill requires trenching continuously in one direction.	Construction sequence is more flexible to meet site constraints.
	Can accommodate interruptions in construction.	Excavation for each panel should progress uninterrupted so each panel is completed before the slurry begins to set.
	Difficult to work with coarse gravels, with backhoe and/or dragline, which settle out and accumulate on the trench bottom reducing the certainty of a tight seal.	Easier to remove coarse gravels with clam shell mounted on a rigid Kelly bar, from trench trench bottom. Better suited for excavation in areas prone to failure.
	Wider trench required to prevent segregation during backfilling ($\geq$ 3 ft) and prevent piping failure of the slurry trench backfill into the adjacent soil downstream of the cutoff (1 ft width for each 10 ft of differential head)	Narrower trench (2 - 3 ft) may be used.
2. Slurry	Mixture of bentonite and water used to support excavation during trenching. Slurry displaced by backfill is desanded and reused. Upon completion of trench, unused slurry must be disposed of in an environmentally acceptable fashion.	Mixture of cement, bentonite, and water used to support excavation for panel and later sets to form permanent backfill. No disposal or desanding of slurry required.
3. Backfill	Mixture of slurry plus excavated material from trench and/or imported select backfill materials	None (see 2). Therefore not dependent on availabilityor quality of soil for backfill and more suitable for work in confined areas.
4. Properties of completed cutoff:		
a. Permeability	$\approx 10^{-7}$ cm/sec ($\geq$ 1 percent bentonite)	$\approx 10^{-6}$ cm/sec
b. Strength	Little field data available - assumed zero for design purposes.	15 - 20 lbs/sq in. unconfined compressive strength. Will support construction loadings or future structures after several weeks.
c. Consolidation	Some consolidation with time.	No significant consolidation with time.
5. Susceptible to attack by sulphates or other chemicals in groundwater	No	Yes - cement may be.
6. Relative cost	Generally lower if cost of backfill is not prohibitive (see 3)	Generally higher due to cost of cement. More competitive as depth of trench increases due to narrower trench width required.
7. Patent applicable	No (expired 1973)	Yes (United States Patent No. 3,759,044 dated September 18, 1973)

(a) Courtesy of Engineering Construction International, Inc.[132,249], American Society of Civil Engineers[222], [221] backfill material, the factor of safety against instability is (Nash and Jones 1963)

$$F = \frac{4\,c_u}{H(\gamma - \gamma_s)} \tag{9-5}$$

where

F = factor of safety

C_u = undrained cohesion

H = depth of the trench

γ = unit weight of the soil

γ_s = unit weight of the slurry

For a slurry trench excavated in dry cohesionless soil (Nash and Jones 1963)

$$F = \frac{2(\gamma \times \gamma_s)^{0.5} \tan \phi}{\gamma - \gamma_s} \tag{9-6}$$

where ϕ = angle of internal friction.

For a slurry trench excavated in a saturated cohesionless soil with the ground-water table and slurry level in the excavation both at the ground surface (Nash and Jones 1963)

$$F = \frac{2(\gamma' \times \gamma'_s)^{0.5} \tan \phi}{\gamma' - \gamma'_s} \tag{9-7}$$

where

γ' = effective unit weight of the soil

γ'_s = effective unit weight of the slurry

ϕ = effective angle of internal friction

For arbitrary levels of ground water and slurry in cohesionless soil, as shown in figure 9-7a, a slightly conservative (neglects arching effect of short trenches and stabilization of the soil adjacent to the trench face due to slurry penetration and gelation) estimate of the slurry density required to ensure stability of the trench is (Morgenstern and Amir-Tahmasseb 1965)

$$\frac{n^2\gamma_s}{\gamma_w} = \frac{\frac{\gamma}{\gamma_w}\cot\alpha(\sin\alpha - \cos\alpha\tan\phi') + m^2\operatorname{cosec}\alpha\tan\phi'}{\cos\alpha + \sin\alpha\tan\phi'} \qquad (9\text{-}8)$$

where

n = defined in figure 9-7a

γ_s = unit weight of the slurry

γ_w = unit weight of water

γ = unit weight of the soil

α = angle of inclination of the wedge of soil at the point of slipping, in practice assumed to be equal to 45" + $\phi'/2$

ϕ' = effective angle of internal friction

m = defined in figure 9-7a

Equation 9-6 may be solved by use of the nomograph shown in figure 9-7b (Duguid et al. 1971).

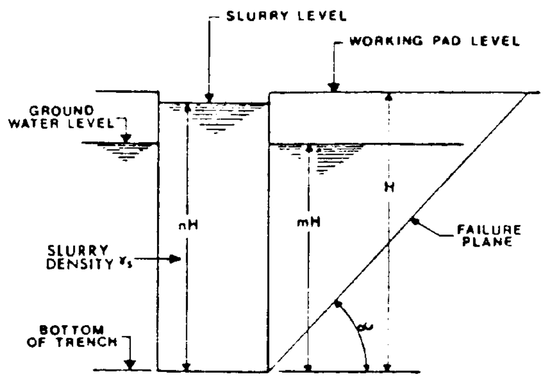
(2) Slurry. The slurry has three basic functions in slurry trench construction (Ryan 1977):

- To hold the trench open and maintain a stable excavation.

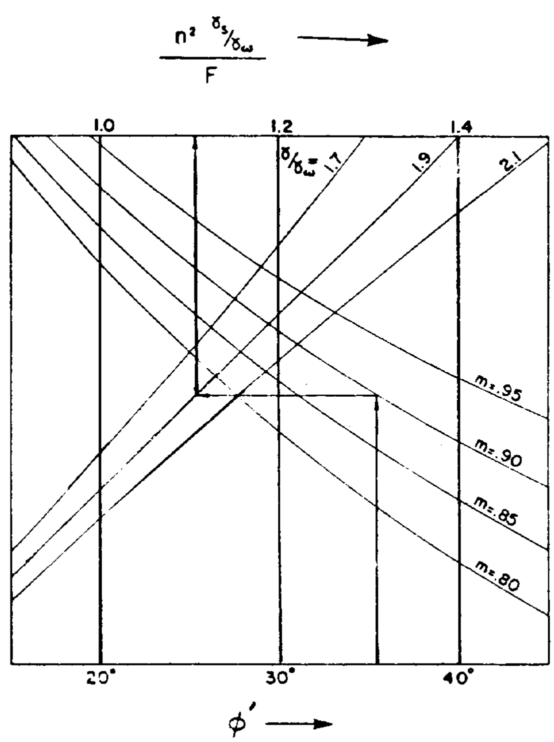
- To be fluid enough to permit passage of the excavating equipment and to allow placement of the backfill (for the cement-bentonite slurry trench, there is no backfill).

- To form a filter cake to enhance the low permeability of the completed trench.

(a) Materials. As a general rule sodium montmorillonite in powder form (Wyoming-type bentonite) is used for slurry trench construction. However, when salt water is present Attapulgite clay is used to avoid flocculation (Spooner et al. 1982). Specifications for both bentonite and Attapulgite are given by the American Petroleum Institute (American Petroleum Institute 1981). Each shipment of bentonite or Attapulgite should be checked for compliance with the specifications. At Saylorville Dam, Iowa, changes in slurry properties were traced to lower quality bentonite which was mined from different beds (U. S. Army Engineer District, Rock Island 19788). No chemically treated



a. Slurry trench typical section



b. Nomograph for slurry trench stability

Figure 9-7. Determination of slurry trench stability in cohesionless soil (courtesy of National Research Council of Canada[168]) bentonite should be used for slurry trench construction. The pH of the water used for mixing with the bentonite should equal 7.0 ± 1.0. Water hardness should not exceed 50 ppm (parts per million). Total dissolved solids should not exceed 500 ppm. The amount of oil, organics, or other deleterious substances should be limited to no more than 50 ppm each (Stanley Consultants, Inc., and Woodward-Clyde Consultants 1977). If the use of poor quality water cannot be avoided, it will require more bentonite and longer mixing times to achieve the desired properties.

(b) Mixing. The methods used to prepare the hentonite slurry vary with project size and layout. Always add the bentonite to the water, never the water to the bentonite. For small jobs the batch system is used where specific quantities of water and bentonite are placed in a tank and mixed at high speeds with a circulationpump or paddle mixer and the slurry is discharged into the trench. Mixing is usually complete in a matter of minutes for the 2- to 5-cu-yd batch produced by this method. The most commonly used method is the flash or Venturi mixer and circulation ponds which is well adapted for bulk handling of large slurry volumes. A flash mixer introduces dry bentonite into a turbulent water jet which discharges into a low speed circulation pond. When Marsh Funnel viscosity readings stabilize, the slurry is stored in a second pond prior to using the trench (Spooner et al. 1982 and D'Appolonia 1980).

(c) Properties. Tests of bentonite quality must be conducted for each rail car or truck load delivered. The minimum acceptable viscosity of a slurry made with the bentonite is 40 seconds Marsh funnel viscosity at 65° F. The fresh slurry shall have a minimum Marsh funnel viscosity of 40 seconds at 65° F and a pH of from 7 to 10, a bentonite content of from 3 to 7 percent by dry weight (depending on the grade of bentonite), a unit weight of from 1.0 to 1.04 g/cm^3 (about 65 lb/ft^3), and the filtrate or water loss shall not be greater than 20 at 100 lb/in.^{2} x cm^3 in 30 minutes (Spooner et al. 1982 and U. S. Army Engineer District, Savannah 1968). The slurry in the trench should be sampled at least twice daily with samples taken from the top of the trench and at 10-ft vertical, 50-ft horizontal intervals along the trench center line. During all stages of construction the minimum acceptable viscosity of the slurry shall be 40 seconds Marsh funnel viscosity at 65° F. The minimum in-trench slurry unit weight is based upon trench stability considerations (see Equation 9-8 and Figure 9-7). The maximum in-trench slurry density is 85/ft^3 to avoid buildup of sediment beyond the slurry capacity to hold it in suspension in the trench during excavation (Clough 1978).

(d) Quality Control Testing. In order to mix and maintain a proper slurry to hold the trench open during excavation and form a filter cake for the soil-bentonite slurry trench, quality control testing must be performed. The property, frequency, standard (if any), and specified value for slurries and their components are given in table 9-4. The quality of the mixing water used can influence the slurry trench characteristics. If the specific values for mixing water quality are not met, the bentonite will flocculate and settle out and not form the filter cake on the sides of the trench. Poor quality mixing water will increase the set time for cement-bentonite slurry trench cutoffs. The bentonite is tested to be sure it will have the minimum viscosity required to keep the soil in suspension. The slurry is tested both after

Table 9-4. Summary of Quality Control Tests for Slurry Trench Cutoff

Category or Material	Property	Frequency	Standard	Specified Value
Mixing Water	pH	Once per month	API RP 13B[a]	7.0 $\pm$ 1.0
	Hardness	Once per month	API RP 13B	$\leq$ 50 ppm
	Total dissolved solids	Once per month	API RP 13B	$\leq$ 500 ppm
	Oil, organics, etc.	Once per month	API RP 13B	$\leq$ 50 ppm each
Bentonite	Viscosity	Each railroad car or truck load	API RP 13A	$\geq$ 40 seconds at 65°F
Bentonite slurry (after mixing)	Viscosity	Twice per day	API RP 13B	$\geq$ 40 seconds at 65°F
	pH	Twice per day	API RP 13A	7 - 10
	Bentonite content	Twice per day	None[b]	3 - 7 percent
	Unit weight	Twice per day	API RP 13B	1.01 - 1.04 g/cm^3
	Filtrate or water loss	Twice per day	API RP 13B	$\geq$ 20 cm^3 at 100 psi in 30 min
Bentonite slurry (in trench)	Viscosity	Twice per day	API RP 13B	> 40 seconds at 65°F
	Unit weight	Twice per day	API RP 13B	minimum[c] - 85 pcf
Cement	Slump	Twice per day	ASTM C-143[d]	2 - 5 in.
Cement-bentonite slurry (after mixing)	Viscosity	Twice per day	API RP 13B	40-50 seconds at 65°F
	pH	Twice per day	API RP 13A	10 - 13
	Bentonite content	Twice per day	None	4 - 7 percent
	Unit weight	Twice per day	API RP 13B	1.03 - 1.04 g/cm^3
	Filtrate or water loss	Twice per day	API RP 13B	$\leq$ 165 cm^3 at 100 psi in 30 min
	Cement/water ratio	Twice per day	None	0.17 - 0.20

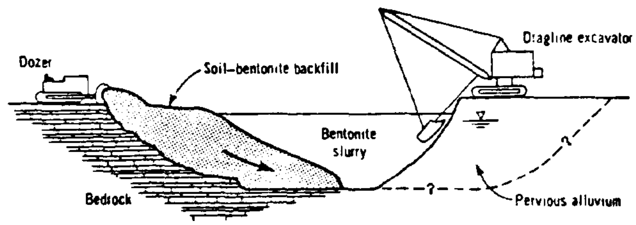
(a) American Petroleum Institute specification.
(b) Can be checked by evaporating the water from a known volume of slurry and weighing the remaining solids
(c) Minimum in trench slurry unit weight is based upon trench stability considerations.
(d) American Society for Testing and Materials Standard.

mixing and in the trench to determine that it is dense enough to stabilize the trench, but not so dense as to cause the backfill to settle too loosely, and that it has sufficient viscosity to maintain cuttings in suspension (Stanley Consultants, Inc., and Woodward-Clyde Consultants 1976).

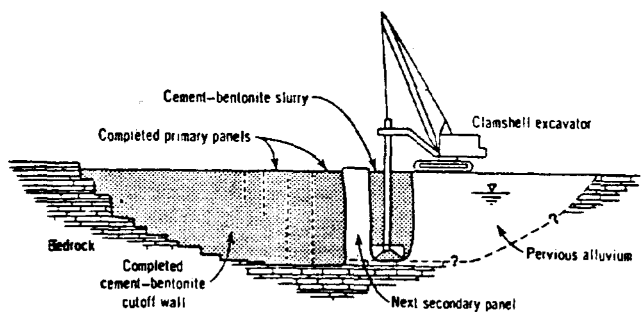
(3) Excavation, Mixing, and Backfilling.

(a) Excavation. The preferred method of trench excavation depends upon the required depth of the slurry trench cutoff, the nature of the subsurface materials, and access to the trench at the ground surface. It is important to ensure that the equipment used can maintain a continuous excavation line to the total depth required. At depths less than about 50 ft, backhoes are generally the most rapid and least costly excavation method. Modified backhoes with an extended dipper stick, modified engine, and counter-weighted frame can excavate to about 80 ft deep. Draglines with weighted ($\geq$ 10,000 lb) buckets, which have been used in the depth range 60 to 80 ft, have been replaced by more efficient extended backhoes. The clamshell bucket can excavate to depths in excess of 150 ft. The clamshell may be mechanically operated attached to a crane or hydraulically operated attached to a Kelly bar. On larger jobs, the backhoe may be used to excavate the first 50 ft followed by a clamshell bucket to excavate the primary and secondary panels to the impervious zone. Regardless of the equipment used, it should be capable of excavating a trench of the desired width in a single pass in order to obtain a fairly consistent trench width. The bucket used should be nonperforated to allow retention and removal of sand particles from the trench. The continuity of the trench is tested by passing the bucket or clamshell of the excavating tool vertically and horizontally along each segment of the trench before it is backfilled. Whatever excavation method is used, it is important that good communications are maintained with the operator of the excavation equipment since abnormalities in the trench excavation are usually noticed first by the equipment operator (D'Appolonia 1980; Spooner et al. 1982; Bloom, Dynes, and Glossett 1979; Case International Company 1982; and Winter 1978). Soil-bentonite slurry trench cutoffs are excavated in a continuous trench as shown in figure 9-8a, while cement-bentonite slurry trench cutoffs are excavated in a continuous trench or in short sections or panels as shown in figure 9-8b. The cement-bentonite slurry begins to harden within 2 to 3 hours after mixing. Alternate panels are excavated under a cement-bentonite slurry and then allowed to partially set. Intervening panels are excavated also under a cement-bentonite slurry and a portion of the initial panel ends are removed to ensure continuity between adjacent panels. Construction delays can cause problems in setup of cement-bentonite slurry trench cutoffs because continued agitation of the cement-bentonite slurry (more than 24 hours) reduces the ability of the cement to set (Spooner et al. 1982).

(b) Bottom Treatment. The aquiclude used for the slurry wall foundations should be continuous, and relatively free of fractures and other pervious zones. The cutoff wall should extend a minimum of 2 ft into clay (or 1 ft into rock) to prevent weathered zones, desiccation cracks, or other geological features from permitting seepage under the cutoff (Spooner 1982). As the trench is excavated, heavier soil particles such as sand and gravel fall to the bottom of the trench. The amount of sand accumulation on the trench bottom depends upon the coarseness of the strata being excavated as well as the



a. Soil bentonite cutoff



b. Cement bentonite cutoff

Figure 9-8. Construction procedure for soil-bentonite and cement-bentonite cutoffs (courtesy of American Society of Civil Engineers[222])

excavation technique used. Although this sand layer may not have a direct effect on trench stability, it may adversely affect the permeability of the slurry trench cutoff wall (Spooner 1982). An air lift pump should be used to remove the sand and gravel particles from the trench bottom prior to backfilling. When the slurry trench is keyed into a soil aquiclude after the trench bottom has been cleaned thoroughly, a minimum of one split-spoon sample shall be taken every 50 ft along the length of the trench to determine if additional excavation is required (Winter 1978).

(c) Backfill Mixing and Placement. A minimum of one day is required between trench excavation and backfilling in order to develop a low permeability filter cake on the trench walls (D'Appolonia 1980). Stockpiled material from the trench excavation and/or material from borrow areas are mixed and blended by windrowing, disc harrowing, bulldozing, or by blading to remove lumps of clay, sand, or gravel. The backfill is then sluiced with slurry (mixing with water shall not be permitted) and just prior to placement has a consistency of a wet concrete with a slump of 5 in. ± 1 in. tested in accordance with ASTM C-143[1] (Winter 1978 and Ryan 1976). The backfill is placed continuously from the beginning of the trench in the direction of the excavation to the end of the trench. Free dropping of the backfill material through the slurry would produce segregation and is not allowed. Depending on the steepness of the excavated slope, it may be necessary to lower the initial backfill to the bottom of the trench with a crane and clamshell bucket until a slope at the angle of repose of the backfill has been formed from the bottom of the trench to the top of the excavation. The toe of the backfill slope is kept to within 50 to 150 ft of the leading edge of the excavation to minimize the open length of the slurry-supported trench while allowing enough space behind the excavation for cleaning the trench bottom. Additional backfill is placed by a bulldozer in such a manner that the backfill enters the trench and slides progressively down the slope of the previously placed backfill and produces a slope ranging from 1V on 5H to 1V on 10H. The slope of the backfill shall be measured with soundings starting at the toe of the backfill in the bottom of the trench and progressing up the backfill slope at 25-ft horizontal intervals. A set of soundings shall be made at least for every 25 ft horizontal advancement of backfill placement. Once the natural slope of the backfill is established during initial placement of the backfill, the slope should remain nearly the same. If the slope, or a portion of the slope, suddenly gets steeper, it could be an indication that sediment is being trapped or that the backfill has a pocket of relatively clean material (slurry not mixed in properly or was washed out). If the slope suddenly gets flatter, it could indicate that a pocket of slurry was trapped in the backfill or that the backfill does not contain sufficient sand or coarser material (Stanley Consultants, Inc., and Woodward-Clyde Consultants 1976; Winter 1978; and Ryan 1976).

(d) Temperature During Construction. The mixing and placing of backfill shall be limited to days when the air temperature is not less than 20" F. Even though the surface of the slurry trench freezes overnight, there will be no difficulty breaking through the surface ice and continuing excavation during the day. Frozen backfill or pieces of ice must never be placed in the trench (U. S. Army Engineer District, Rock Island 1978a and Jones 1967).

(e) Protection of Top of Trench. The top of the completed slurry trench cutoff should be immediately protected with a temporary 2- to 3-ft-thick blanket of moist impervious fill material to prevent drying of the backfill and formation of shrinkage cracks along which paths of seepage could easily develop. The layer is temporary and is removed once the embankment construction is started (Jones 1967; Stanley Consultants, Inc., and Woodward-Clyde Consultants 1977).

(4) Soil-Bentonite Slurry Trench Cutoff.

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(1) American Society for Testing and Materials standard. If a desirable back-filled slope (1V on 5H to 1V on 10H) cannot be maintained in the trench with a 5 in. ± 1 in. slump, the slump may be altered to meet construction conditions. Such was the case at the soil-bentonite slurry trench cutoff constructed at W. G. Huxtable Pumping Plant, Marianna, Arkansas (U. S. Army Engineer District, Memphis 1978).

(a) Design Considerations. The primary design parameters are blowout requirements, permeability, strength, and compressibility. The backfill material must not blow out into the surrounding pervious foundation under the maximum differential hydraulic head that will act on the slurry trench. The permeability is usually sufficiently low ($\sim 10^{-7}$ cm/sec for ≥ 1 percent bentonite) to reduce the seepage through the slurry trench cutoff to an acceptable value. Under most conditions, the only strength requirement for the slurry trench cutoff is to approximate the strength of the surrounding ground. The compressibility of the slurry trench cutoff, once consolidated under its own weight (usually within 6 months after placement), should be compatible with the compressibility of the surrounding ground to minimize differential movement of the dam and resultant stress concentrations in the embankment or its foundation (Ryan 1976 and Xanthakos 1979).

(b) Blowout Requirements. Once the slurry trench is installed, the dam has been constructed, and the reservoir filled, there is a substantial differential head acting on the slurry trench (see table 9-2 for typical values). Depending upon the characteristics of the backfill material and pervious foundation, the hydraulic gradient acting across the slurry trench may be sufficient to cause blowout or piping of backfill material into the surrounding pervious foundation. This is especially critical when the foundation contains openwork gravel where the piping process could result in the formation of channels and cavities that may breach the slurry wall. Based upon laboratory tests conducted on widely graded gravel containing no sand, the blowout gradient ranges from 25 to 35, depending on the properties of the backfill material (La Russo 1963 and Nash 1976). The factor of safety against blowout is

$$F = \frac{i_{allowable}}{i_{actual}} \tag{9-9}$$

where

F = factor of safety against blowout

$i_{allowable}$ = allowable hydraulic gradient from laboratory blowout tests

i_{actual} = actual hydraulic gradient existing on slurry trench

Substituting for the actual hydraulic gradient

$$i_{actual} = \frac{\Delta h}{w} \tag{9-10}$$

where

Δh = maximum differential hydraulic head acting on the slurry trench

w = slurry trench width

and using a factor of safety of 3 and an allowable hydraulic gradient of 30 in equation 9-9 gives

$$w = \frac{\Delta h}{10} \tag{9-11}$$

If the pervious foundation contains openwork gravel, the width of soil-bentonite slurry trench required to prevent blowout failure may be estimated from equation 9-11. Further refinements on the trench width would require conducting laboratory blowout tests (as described by Xanthakos 1979).

(c) Permeability. For design purposes the permeability of the soil-bentonite slurry trench cutoff is based on the backfill material only (Xanthakos 1979). The permeability of the slurry trench is a function of both the filter cake that forms on the trench walls and the backfill material. The contribution of the filter cake and the backfill depends on the relative permeability and thickness of the two materials. The horizontal permeability of the soil-bentonite slurry trench is (D'Appolonia 1980)

$$k = \frac{t_b}{\dfrac{t_b}{k_b} + \dfrac{2t_c}{k_c}} \tag{9-12}$$

where

k = permeability of slurry trench

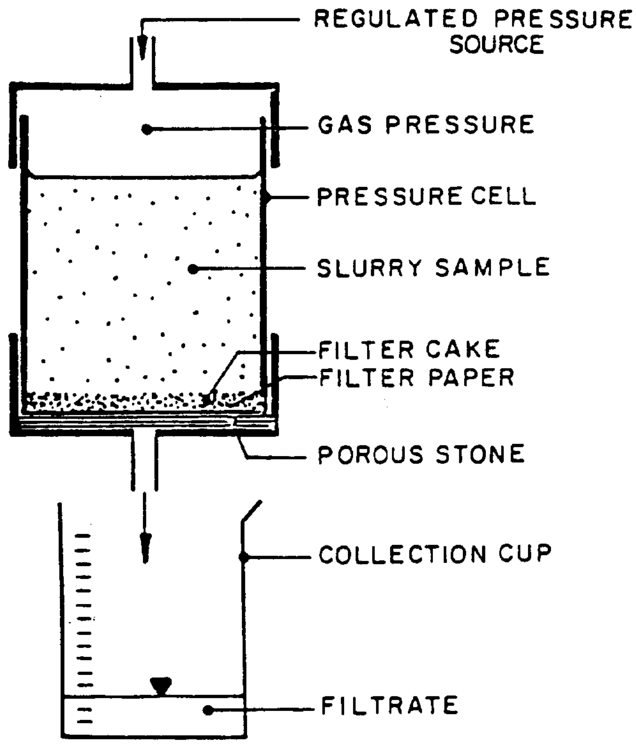
t_b = backfill thickness

k_b = backfill permeability

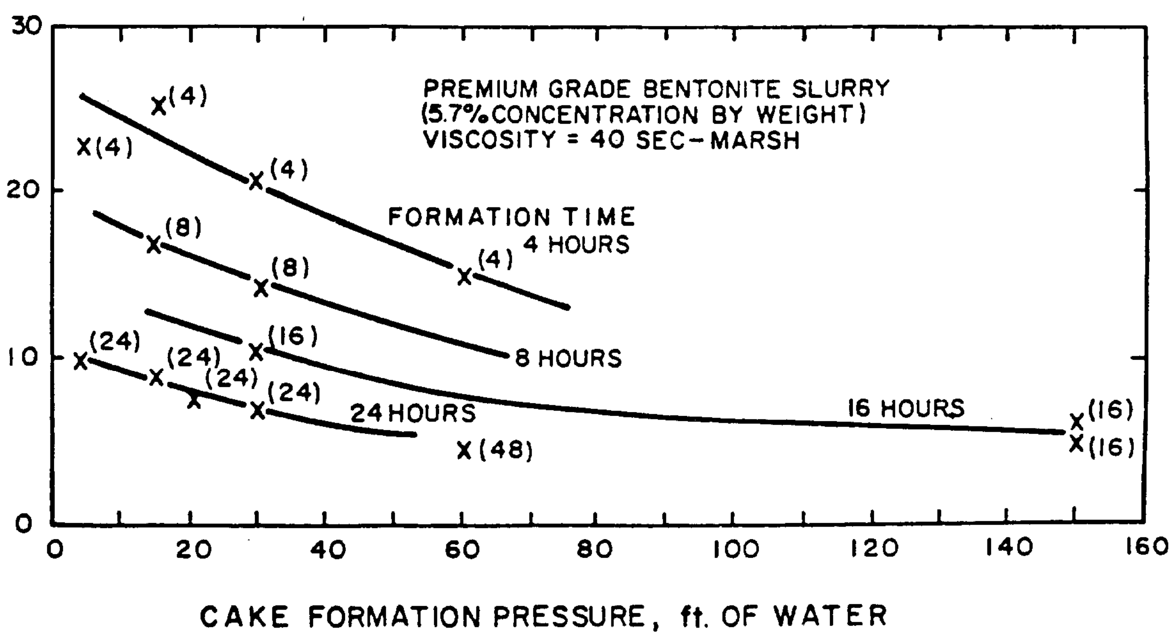
t = filter cake thickness

k_c = filter cake permeability

The permeability of the backfill material can be determined in a laboratory permeability test (EM 1110-2-1906). The thickness of the backfill is selected in design (see figure 9-6). The ratio k_c/t_c is determined from the filter press test (American Petroleum Institute 1982) using various formation cake pressures as shown in figure 9-9a. For a range of practical applications, the ratio k_c/t_c varies from 5×10^{-9}/sec to 25×10^{-9}/sec as shown in figure 9-9b. Figure 9-10 shows the permeability of a soil-bentonite slurry trench cutoff wall 80 cm (about 2-1/2 ft) thick for various values of backfill permeability and ratios of k_c/t_c . As shown in figure 9-10, the slurry trench



a. Schematic of filter press test apparatus



b. Relationship among filter cake permeability, filter cake formation pressure, and time

Figure 9-9. Determination of filter cake permeability (courtesy of American Society of Civil Engineers[163])

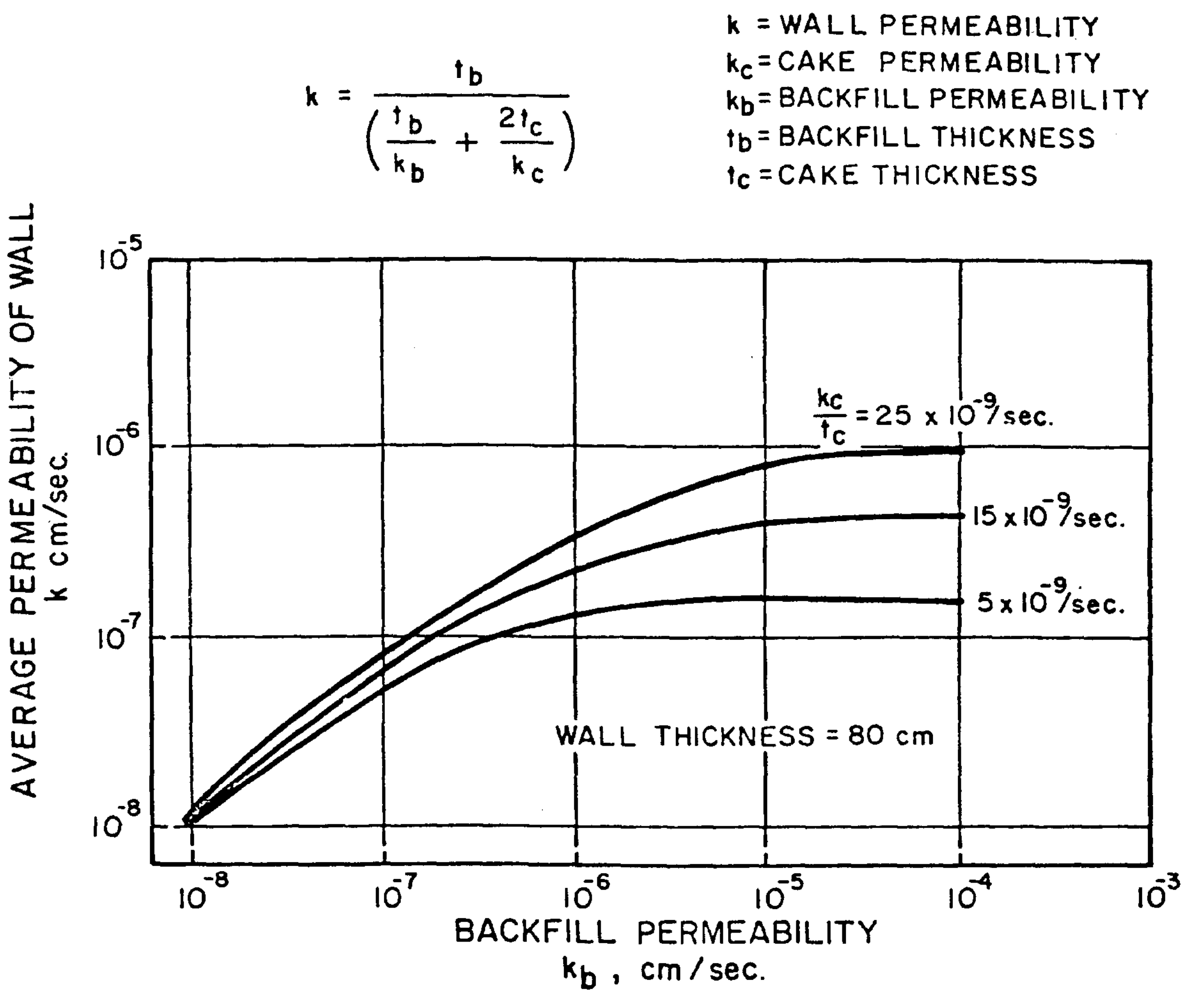


Figure 9-10. Permeability of a soil-bentonite slurry trench cutoff wall 80 cm thick for various values of backfill and filter cake permeability (courtesy of American Society of Civil Engineers[163])

permeability is controlled by the backfill when the backfill permeability is low and by the filter cake when the backfill permeability is high. Also, the slurry trench permeability has an upper limit of about 10^{-6} cm/sec even for very permeable backfill due to the thin low permeability filter cake (D'Appolonia 1980).

(d) Shear Strength. Soil-bentonite slurry trench cutoffs are difficult to sample because of their soft nature and very little data are available on the shear strength of soil-bentonite slurry trench backfill material. For design purposes, in conducting the stability analysis of the embankment and foundation, the shear strength of the backfill material is assumed, to be zero. However, the shear strength of the backfill material does increase with time due to consolidation and thixotropy. At time of placement, the backfill material will stand on a slope ranging from 1V on 5H to 1V on 10H. This improves to about 1V on 2H with time (Ryan 1976 and D'Appolonia 1980). The results of shear strength tests (see table 9-5) on undisturbed samples taken from the soil-bentonite slurry trench at Saylorville Dam, Iowa, show that the undrained shear strength of the slurry backfill about a year after placement was 0.10 to 0.12 tons/sq ft (U. S. Army Engineer District, Rock Island 1978b).

(e) Compressibility. The compressibility of the soil-bentonite slurry trench backfill material depends primarily on the percentage of granular particles in the gradation as shown in figure 9-11. Low permeability and low compressibility are contradictory requirements because the plastic fines required for low permeability result in higher compressibility. Relatively low compressibility results when there is sufficient granular material in the backfill to allow grain-to-grain contact between the granular particles (D'Appolonia 1980).

(f) Mix Design. The gradation of the backfill for the soil-bentonite slurry trench is selected by conducting permeability, shear strength, and compressibility tests on a range of materials including soil to be excavated from the trench. Such a procedure was followed in the mix design for the backfill of the soil-bentonite slurry trench installed for remedial seepage control at Addicks Dam, Texas (U. S. Army Engineer District, Galveston 1977c). The allowable range set on the gradation of the backfill should produce a material which contains enough fines to reduce the seepage through the slurry trench cutoff to an acceptable level and sufficient coarse particles to approximate the strength and compressibility of the surrounding ground. If sufficient fines are not present in material excavated from the trench, borrow sources should be identified or alternatively a higher bentonite content specified for the backfill. If sufficient coarse particles are not present in material excavated from the trench, approved sources should be identified for obtaining natural sound, hard, durable sand and gravel. Crushed limestone, dolomite, or other crushed calcareous materials should not be used. The maximum particle size of the gravel shall be 3 in. and the material should be well graded.

(5) Cement-Bentonite Slurry Trench Cutoff.

Table 9-5. Summary of Shear Strength Data from Post-Construction Testing of Backfill Material from Soil-Bentonite Slurry Trench Cutoff at Saylorville Dam, Iowa[(a)]

Boring No.	Station	Depth ft	Unified Soil Classification	Atterberg Limits LL	PL	PI	Dry Density lb/cu ft	Moisture Content %	Time Elapsed, months: Construction to Sampling	Sampling to Testing	Triaxial Compression Test[(b)] Q: ϕ deg	Q: c tons/sq ft	$\bar{R}$: $\bar{\phi}'$ deg	$\bar{R}$: $\bar{c}'$ tons/sq ft
STU-1	58+90	26.7-28.9	Gravelly clayey sand, SC	33	14	19	118.2	13.3	8.5	5	4.5	0.075		
STU-1	58+90	39.2-41.4	Clayey sand, SC	33	14	19	108.5	17.5	8.5	5	0	0.130		
STU-2	64+00	14.5-16.6	Clayey sand, SC	36	15	21	103.5	21.0	8	5	0	0.075		
STU-1	58+90	49.2-51.4	Clayey sand, SC	41	17	24	107.9	18.9	8.5	19			14.5	0.10
STU-2	64+00	32.0-34.3	Clayey sand, SC	33	14	19	114.1	14.9	8	19			19	0
STU-6	82+90	16.0-18.1	Gravelly clayey sand, SC	34	15	19	120.1	16.9	11.5	7.5	0	0.10		
STU-6	82+90	49.9-51.9	Gravelly clayey sand, SC	37	15	22	113.1	17.7	11.5	7.5	0	0.12		
STU-7	87+80	17.0-19.0	Gravelly clayey sand, SC	32	17	15	113.5	17.6	11	6	0	0.12		
STU-6	82+90	35.4-35.6	Sandy clay, CL	31	18	13	117.0	13.1	11.5	9.5			14.5	0
STU-7	87+80	26.6-28.8	Clayey sand, SC	28	14	14	112.9	19.2	11	11.5			17	0

(a) From U. S. Army Engineer District, Rock Island[106].
(b) Conducted on undisturbed soil specimens 5 in. in diameter by 11 in. high; unconsolidated-undrained (Q) and consolidated-undrained with pore pressure measurements ($\bar{R}$) shear tests.

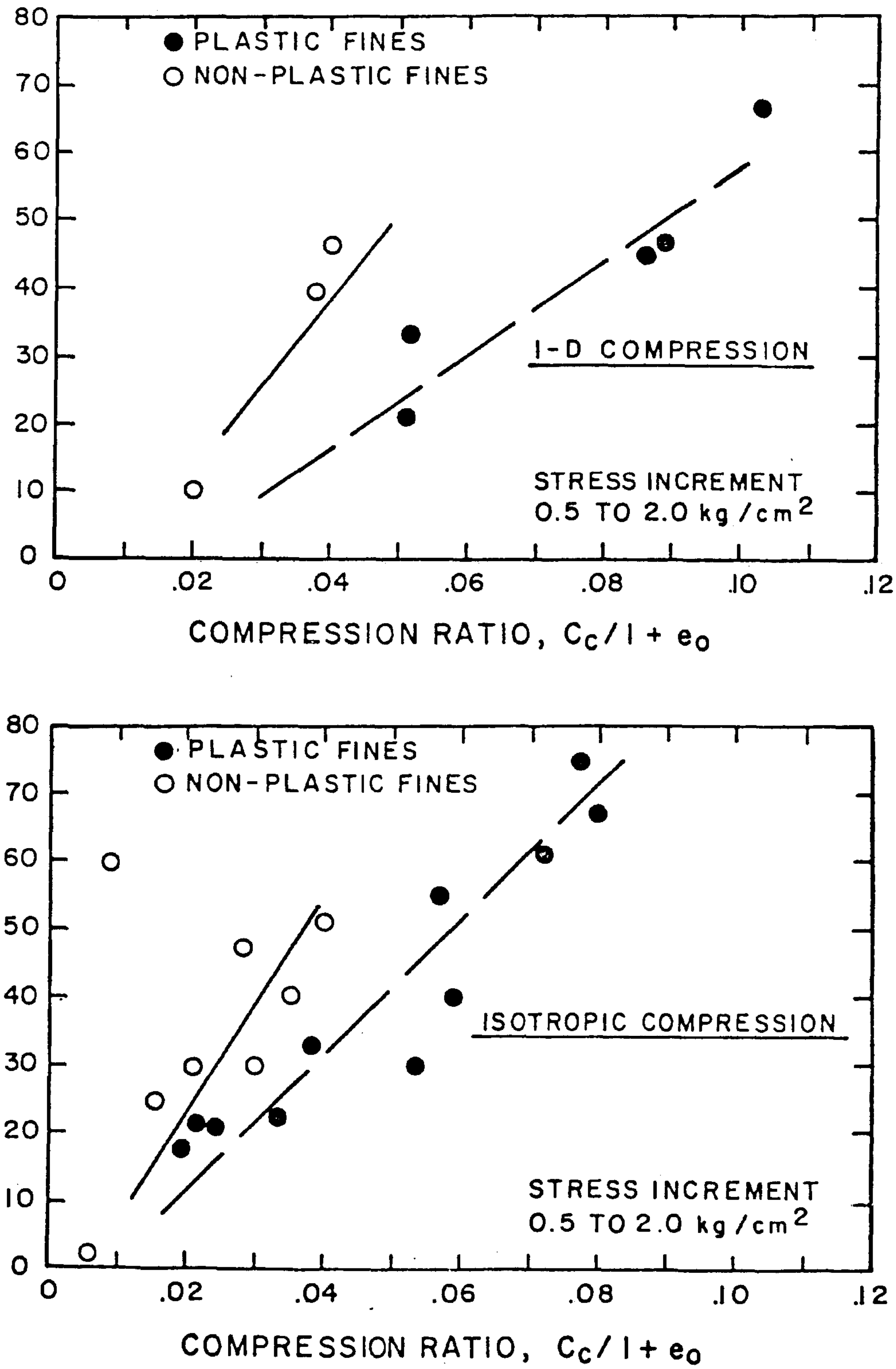


Figure 9-11. Relationship between fines content and compressibility of a soil-bentonite slurry trench backfill (courtesy of American Society of Civil Engineers[163])

(a) General. If backfill for the slurry trench is not available or is prohibitive in cost or if the cutoff is deep and the foundation is prone to failure during excavation, the cement-bentonite slurry trench cutoff may be more applicable (see table 9-3).

(b) Design Considerations. The primary design parameters are continuity, set time, resistance to hydraulic pressure, permeability, shear strength, and compressibility.

(c) Continuity. When cement-bentonite slurry trench cutoffs are constructed in panels rather than in a continuous trench, there is a possibility for unexcavated portions of the trench to remain between the panels. To prevent this the clamshell bucket is moved both vertically and horizontally throughout each slot at the completion of slot excavation. Also, when the connecting area between the initial and subsequent panels is excavated, a portion of the adjacent set panels is removed to ensure that all intervening soil has been excavated (Spooner et al. 1982).

(d) Set Time. The set time is important because of the construction technique employed. After the cement-bentonite slurry in the first set of panels has set, the areas between them can be excavated. A normal cement-bentonite mixture begins to set after a few hours and has a consistency similar to lard after 12 hours. The second day the cement-bentonite slurry can be walked on and final set is normally taken at 90 days (Ryan 1977).

(e) Resistance to Hydraulic Pressure. Once the slurry trench has been completed, the embankment constructed, and the reservoir filled, there is a substantial differential head acting on the slurry trench (see table 9-2 for typical values). The time between completion of the slurry trench and reservoir filling is generally sufficiently long ($\geq$ 90 days) to allow the cement-bentonite slurry trench to develop its final set. The resistance of the cement-bentonite material to withstand gradients comparable to those which will exist in the field should be tested in the laboratory by subjecting intact specimens which have developed full set to hydraulic pressure and measuring the increase (if any) of permeability with time (Spooner et al. 1982 and Jefferis 1981).

(f) Permeability. Although there is some buildup of concentration near the sides of the cement-bentonite slurry trench, the cement-bentonite does not form a low permeability cake. The permeability of the slurry trench is a function of the concentrations of cement, bentonite, sand, and gravel (suspended during the excavation process) in the completed wall (Ryan 1977). The amount of sand and gravel in the cement-bentonite trench cutoff may range from 10 to 60 percent by dry weight, depending on the foundation material and method of construction, and generally increases with depth (Dank 1981). If the trench is excavated under a conventional bentonite slurry which is then replaced by a cement-bentonite slurry, the sand and gravel content will be low (10 to 18 percent was measured on undisturbed samples taken from the San Lorenzo Dam, El Salvador; Dank 1981). Alternatively, if the trench is excavated under a cement-bentonite slurry which is left in the trench to set up and form the cutoff, the sand and gravel content will be relatively high. Also, if the trench is excavated under a cement-bentonite slurry, the slurry loss into the surrounding ground will be higher than normal and in some instances as great as 100 percent of the trench volume (Xanthakos 1979). For design purposes, specimens should be prepared from the cement-bentonite with varying percentages of sand and gravel, cured for 28 days under consolidation pressures existing in the field, and laboratory permeability tests conducted (EM 1110-2-1906).

(g) Shear Strength. Cement-bentonite slurry trench cutoffs are more easily sampled and tested than are soil-bentonite slurry walls. Also, specimens of cement-bentonite may be cast in the laboratory and tested. The results of shear strength tests (see table 9-6) on undisturbed samples taken from the cement-bentonite slurry trench at Tilden Tailings Project, Michigan, show the unconfined compressive strength about 6 months after placement increased with depth ranging from 0.88 to 1.43 tons/sq ft (Dank 1981).

(h) Compressibility. Very little data are available on the compressibility of cement-bentonite slurry trench material (Millet and Perez 1980). The compressibility should decrease with increase in cement to water ratio (provided the bentonite is fully hydrated with water prior to adding the cement) and with increase in sand content (once the concentration of suspended sand and gravel is sufficient to allow grain-to-grain contact between the granular materials).

(i) Mix Design. The cement-bentonite slurry trench mixture consists of water, bentonite, cement, set retarders as necessary, and sand and gravel entering the trench as a by-product of the excavation. The bentonite should be fully hydrated with water prior to adding the cement (Millet and Perez 1980). A retarder of the lignosulphite group may be added in small amounts (0.1 percent) to delay the initial set to avoid hardening of the mix in the panel before the excavation is completed (Xanthakos 1979). When low permeability is required, the bentonite content of the slurry should be increased (in the range from 3 to 6 percent by dry weight). Increased sand and gravel in the slurry will result in an increase in permeability (Dank 1981). The cement, sand, and gravel content are the chief factors in controlling the strength and deformability characteristics of the slurry mix (see table 9-6 and figure 9-12). Generally, the higher the cement to water ratio, the higher the strength, and more brittle (lower failure strain) the cement-bentonite slurry mix (Millet and Perez 1980). By varying the bentonite and cement quantities, flexibility can be designed into the cement-bentonite slurry trench cutoff. This is especially important if the dam is located at a site where strong earthquake shocks are likely. The cement-bentonite slurry mix proportions should be selected by conducting permeability, shear strength, and compressibility tests on a range of materials including soil to be excavated from the trench. Varying proportions of water, bentonite, cement, sand and gravel (representing aggregate entering the trench during the excavation process) should be tested to select a design mix which will reduce the seepage through the slurry trench cutoff to an acceptable level and approximate the strength and compressibility of the surrounding ground.

(6) Failure Mechanisms of Cutoffs

Table 9-6. Summary of Shear Strength Data from Post-Construction Testing of Cement-Bentonite Slurry Trench Cutoff at Tilden Tailings Project, Michigan[a]

Boring No.	Depth ft	Dry Density lb/cu ft	Sand Content By Volume %	Moisture Content %	Time Elapsed Construction to Sampling months	Unconfined Compressive Strength tons/sq ft	Triaxial Compression Test[b] S Ø' degrees	Triaxial Compression Test[b] S c' tons/sq ft
1	6.0	26.5	2.5	271.6	6	0.92		
1	10.7	29.4	11.6	151.4	6	0.94		
1	15.9	32.1	13.2	141.0	6	0.97		
1	21.0	37.0	11.4	138.8	6	1.02		
1	31.0	37.5	6.2	129.0	6	1.02	15.5	0.99
1	41.0	40.0	--	115.8	6	--	24.5	0.66
2	6.9	21.4	0.0	236.0	6	0.88		
2	10.8	43.0	6.2	201.0	6	1.00		
2	16.0	38.3	11.4	134.5	6	1.14	29.6	0.36
2	21.0	39.0	13.4	117.1	6	1.30		
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

(a) From Northwestern University

(b) Conducted on undisturbed soil specimens; consolidated-drained (S) shear test

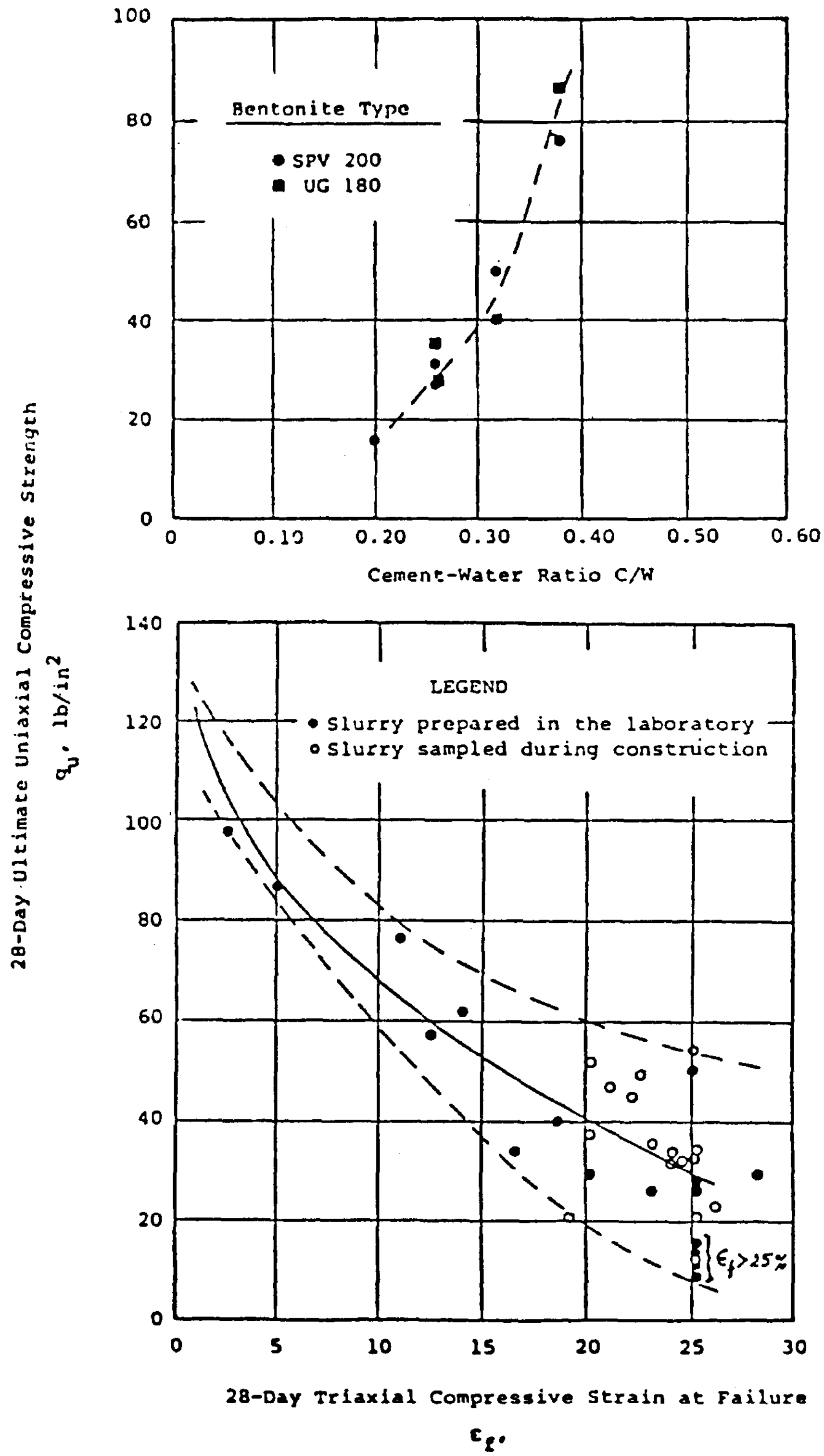


Figure 9-12. Strength and deformation characteristics of cement-bentonite slurries (courtesy of American Society of Engineers[223])

(a) Introduction. Several mechanisms can affect the functioning of slurry walls and cause failure. Failure may occur during excavation of the trench, upon first filling of the reservoir with the resulting rise in differential head acting across the slurry wall, or at some future time due to adverse chemical substances in the soil and ground water. Specific failure mechanisms include trench collapse, gaps (or windows) in the slurry wall, inadequate aquiclude key-in, blowout or piping of backfill material into the surrounding pervious foundation, and chemical destruction of the slurry wall (Spooner et al. 1982).

(b) Trench Collapse. Trench collapse is caused by instability of the trench walls during excavation and before backfilling (for soil-bentonite slurry trench) or setup (for cement-bentonite slurry trench). Causes of trench collapse include failure to maintain the minimum differential head between the top of the slurry and the top of the ground water and/or too low unit weight of slurry (see figure 9-7). Drop in the slurry level in the trench may be caused by contact with gravel, fissures, etc., during excavation, while rise in the ground-water level may be caused by surface runoff into cracks adjacent to the trench, particularly following heavy rainfall. Too low unit weight of the slurry may be caused by the cessation of agitation by excavation equipment over the week end (Spooner et al. 1982). Such a set of circumstances contributed to the collapse of a portion of one wall of a soil-cement bentonite slurry trench at Duncan Dam in Canada (Duguid et al. 1971).

(c) Gaps (or Windows) in the Slurry Wall. Trench collapse or improper placement of backfill can create gaps (or windows) which result in zones of higher permeability as well as variations in wall thickness and strength (Spooner et al. 1982). The continuity of the trench should be tested before backfilling by passing the bucket or clamshell of the excavating tool vertically and horizontally along each segment of the trench. As mentioned previously, for soil-bentonite slurry trenches irregularities in the backfill slope are indications that pockets of clean material (slurry not mixed in properly or was washed out) or slurry were trapped in the backfill or that the backfill does not contain sufficient sand or coarse material.

(d) Inadequate Aquiclude Key-In. As discussed previously, inadequate aquiclude key-in will permit seepage under the cutoff. Inadequate key-in can result from variations in trench depth, insufficient aquiclude penetration, trench collapse, or the presence of boulders (Spooner et al. 1982).

(e) Blowout or Piping of Backfill Material. As mentioned previously, blowout or piping of backfill material into the surrounding pervious foundation is especially critical for soil-bentonite slurry trenches when the foundation contains openwork gravel. The required width of the slurry trench to prevent blowout (factor of safety of 3) in openwork gravel may be estimated from Equation 9-11.

(f) Chemical Destruction of the Slurry Wall. Chemical substances in the foundation soil and ground water can affect the durability of the slurry wall once it is constructed. If salt water is present in the construction area, appapulgite may be used instead of bentonite. Permeation of a soil-bentonite or cement-bentonite slurry wall by polluted ground water generally leads to increased permeability. The bentonite may become entrained in the solution and the cement may become slurry solubilized as a solution channel is created through the slurry wall and into the foundation. Chemicals may also prevent the slurry from forming an adequate filter cake along the sides of the soil-bentonite slurry trench (Spooner et al. 1982). Where polluted ground water is present, long-term permeability tests should be conducted using the specific soil-bentonite backfill materials or cement-bentonite mix from the site permeated by the actual pollutant in designing the slurry trench cutoff (Spooner et al. 1982 and D'Appolonia 1980).

(7) Instrumentation and Monitoring.

(a) Introduction. Whenever a slurry trench is used for control of underseepage, the initial filling of the reservoir must be controlled and instrumentation monitored to determine if the slurry trench is performing as planned. If the slurry trench cutoff is ineffective, remedial seepage control measures must be installed prior to further raising of the reservoir pool (EM 1110-2-2300).

(b) Parameters of Interest. There are two parameters of interest with regard to slurry trench cutoffs for control of underseepage. These are the drop in piezometric head from upstream to downstream across the trench during reservoir filling and the differential settlement between the top of the slurry trench and the overlying compacted embankment material.

(c) Efficiency of Slurry Trench Cutoff. To evaluate the head efficiency (see equation 2) of the slurry trench cutoff, the head loss is determined between points immediately upstream and downstream of the slurry trench cutoff wall at its junction with the base of the dam. The head loss is established from piezometer readings taken during construction, before and during initial filling of the reservoir, and subsequently as frequently as necessary to determine changes that are occurring and to assess their implications with respect to safety of the dam (see Chapter 13). Equal numbers of piezometers are normally placed on each side of the slurry trench cutoff. Piezometers should be installed at two or more locations along the length of the slurry trench depending upon the foundation conditions at the site. Pneumatic piezometers installed upstream and downstream of the soil-bentonite slurry trench at West Point Dam, Alabama and Georgia, showed a near-horizontal piezometric surface existed across the trench prior to filling the reservoir. Piezometer readings taken after reservoir filling indicated a drop in piezometer head from upstream to downstream across the slurry trench, confirming the effectiveness of the cutoff (U. S. Army Engineer District, Savannah 1979). Open-tube piezometers installed upstream and downstream of the soil-bentonite slurry trench at Addicka Dam, Texas, indicated a drop in piezometric head across the slurry trench (U. S. Army Engineer District, Galveston 1983).

(d) Differential Settlement. The differential settlement between the top of the slurry trench and the overlying compacted embankment material is important because a separation of materials in this region could result in piping at the interface between the embankment and the foundation. Settlement plates placed on top of the soil-cement bentonite slurry trench at West Point Dam, Alabama and Georgia, indicated a uniform total settlement of approximately

0.5 ft throughout the trench. Excavation of a portion of the trench prior to filling the reservoir showed no void between the slurry trench backfill and the overlying compacted fill (U. S. Army Engineer District, Savannah 1968 and 1979).

e. Concrete Wall.

(1) Introduction. When the depth of the pervious foundation is excessive ($\leq$ 150 ft) and/or the foundation contains cobbles, boulders, or cavernous limestone, the concrete cutoff wall may be an effective method for control of underseepage. Using this method, a cast-in-place continuous concrete wall is constructed by tremie placement of concrete in a bentonite slurry supported trench. Two general types of concrete cutoff walls, the panel wall and the element wall have been used, as shown in figures 9-13 and 9-14, respectively. Since the wall in its simpler structural form is a rigid diaphragm, earthquakes could cause its rupture; therefore, cutoff walls should not be used at a site where strong earthquake shocks are likely (U. S. Army Engineer District, Pittsburgh 1965).

(2) History of Use. Conventional (excavated without bentonite slurry) concrete cutoff walls were widely used prior to 1925. Since they require about the same excavation and dewatering as compacted backfill trenches and the wall itself is far more expensive than compacted soil, the popularity of conventional concrete cutoff walls has declined (Sowers 1962 and Sherard et al. 1963). The method of excavating trenches supported by bentonite for the construction of cast-in-place concrete cutoff walls was used for the first time in 1951 at the Volturno-Garigliano hydroelectric plant on the Volturno River at Venafro, near Naples, Italy (Veder 1963, Veder 1975 and Franke 1954). Since the 1950's, concrete cutoff walls constructed by tremie placement of concrete in a bentonite slurry supported trench have been used for projects throughout the world. The deepest concrete cutoff wall to date was constructed at Manicouagan 3 Dam in Quebec, Canada, in 1972, where two parallel concrete walls, 2 ft thick and 10 ft apart, extended 430 ft deep (Anonymous 1972). A comparison of concrete cutoff walls constructed at Corps of Engineers dams is given in table 9-7.

(3) Sequence of Construction and Location of Wall. Normally the embankment is constructed first, followed by the concrete cutoff wall located upstream of the toe of the dam as was done at Kinzua Dam (formerly Allegheny Dam) and tied into the core of the dam with an impervious blanket (U. S. Army Engineer District, Pittsburgh 1965). The upstream location minimizes the possibility of compressive failure of the concrete cutoff wall due to negative skin friction as the foundation settles under the weight of the embankment (as would occur is the cutoff wall is located under the center of the dam). Constructing the embankment first, followed by the concrete cutoff wall, minimizes the possibility of rupture of the concrete cutoff wall due to horizontal movement of the foundation as the embankment is constructed. For remedial seepage control of existing dams (see Chapter 12) where it is not practical to draw down the reservoir and primary consolidation of the foundation has been complete, a central location for the concrete cutoff wall may be feasible.

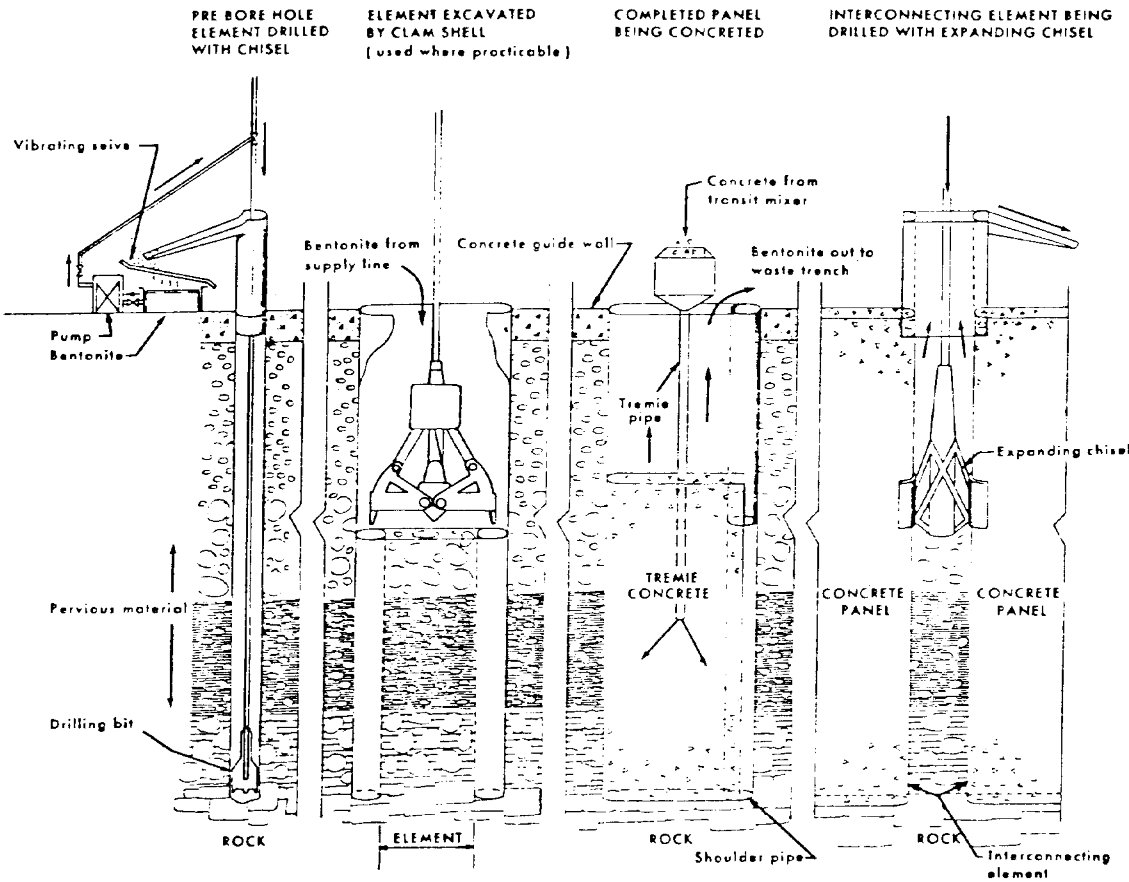


Figure 9-13. Construction procedure for concrete cutoff wall at Kinzua Dam (formerly Allegheny Dam), Pennsylvania (after U. S. Army Engineer District, Pittsburgh[103])

(4) Design Considerations. The primary design parameters are permeability, strength, and compressibility.[1] The permeability is usually sufficiently low ($=10^{-10}$ cm/sec for water-cement ratio of 0.6) to reduce the seepage through the concrete cutoff wall to an acceptable value (Xanthakos 1979). The concrete cutoff wall is generally stronger (>3,000 psi compressive strength) than the surrounding foundation soil and introduces a heterogeneous zone (in the form of a rigid diaphragm) in the foundation. The compressibility of the concrete cutoff wall is sufficiently low that the wall is essentially rigid with respect to the surrounding foundation soil (Xanthakos 1979).

(a) Permeability. For workable concrete mixes used in concrete cutoff walls (see table 9-8), the permeability increases rapidly for water-cement ratios higher than 0.5. For a concrete mix with maximum coarse aggregate size

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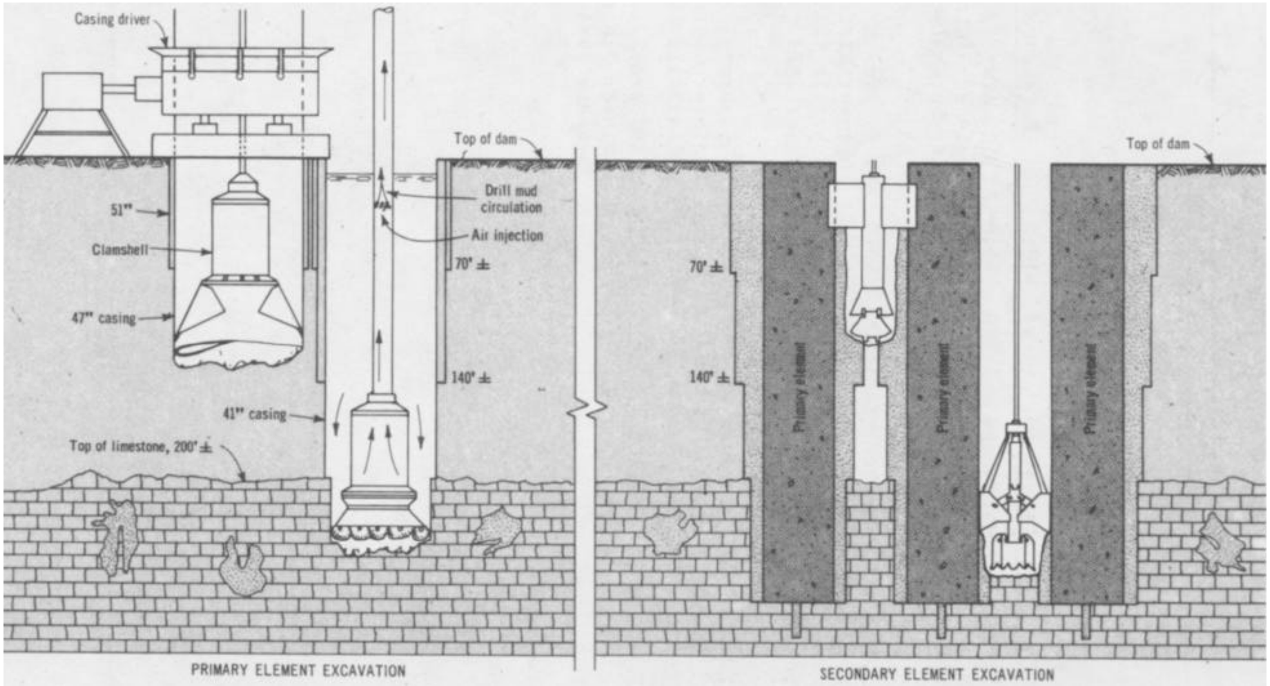
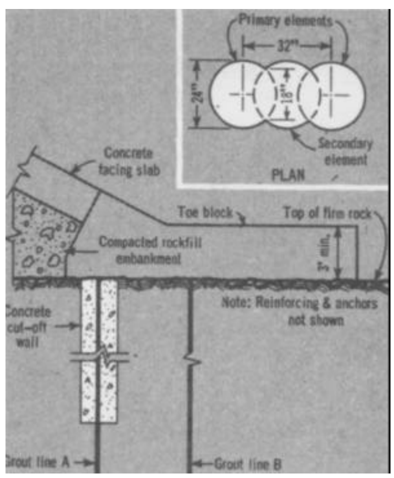
[1] The workability of the concrete, discussed under Mix Design, is of primary importance with respect to tremie placement of the concrete.



a. Excavation procedure for primary and secondary elements



b. Interlocking of primary and secondary elements

Figure 9-14. Construction procedure for concrete cutoff wall at Wolf Creek Dam, Kentucky (courtesy of American Society of Civil Engineers[160])

Table 9-7. Comparison of Corps of Engineers Concrete Cutoff Walls

				Cutoff						
Project	Location	Date Con-structed	Embankment-Foundation Material	Max. Dif-fer-ential Head ft	Width ft	Max. Depth ft	Location	Max. Devi-ation from Ver-tical[a]	Max. Head / Cutoff Width	Reference
Kinzua (formerly Allegheny) Dam	Allegheny River Pennsylvania	1964	Silts, sands and gravels, boulders	140	2.5	185	Upstream of toe of dam	1:370	56.0	U. S. Army Engineer District, Pittsburgh 1965
Wolf Creek Dam[b]	Cumberland River Kentucky	1978	Embankment (200 ft); cavernous limestone (100 ft)	187	2.0	300	Center of dam	1:600	93.5	U. S. Army Engineer District, Nashville 1975; Fetzer 1979
Mill Creek Dam	Mill Creek Washington	1981	Silt overlying conglomerate	51	2.0	170	Upstream toe of dam	1:100	25.5	U. S. Army Engineer District, Walla Walla 1980a,b
Clemson Lower Diversion Dam[b]	Hartwell Lake Georgia and South Carolina	1982	Embankment (50 ft); silty sand and sand (35 ft)	44	2.0	85	Center of dam	1:100	22.0	U. S. Army Engineer District, Savannah 1981

(a) Deviation from vertical in any direction at any depth.
(b) Concrete cutoff installed as remedial seepage control for existing dam (see Chapter 12).

Table 9-8. Comparison of Concrete Mix Proportions for Corps of Engineers Concrete Cutoff Walls

Project	Cement[a] lb/cu yd	Fly Ash lb/cu yd	Water lb/cu yd	Water/Cement + Fly Ash	Aggregate lb/cu yd		28 Day Compressive Strength lb/sq in.		
					Coarse	Fine	Slump in.	Specified	Obtained
Kinzua (formerly Allegheny Dam)	564	0	292	0.52	1,652	1,345	7	3,000	3,843
Wolf Creek Dam	564	123	275	0.40	1,619	1,369	6.5-7.5	3,000	4,685
Mill Creek Dam	299	134[b]	260	0.60	1,628	1,587	8	1,500	1,970
Clemson Lower Diversion Dam	507	110	334	0.54	1,540	1,310	60.9.0	1,300	--

(a) Weights are saturated surface dry.
(b) Fly ash percentage replaced by solid volume was raised from 35 to 45 before wall was completed.

of 3/4 in. and a water cement ratio of 0.6, the permeability is usually lower than 10^{-10} cm/sec (Xanthakos 1979). The permeability of a concrete cutoff wall is influenced by cracks in the finished structure and/or by void spaces left in the concrete as a result of honeycombing or segregation (see Equation 9-4 and figure 9-5). The joints between panels are not completely impermeable but the penetration of bentonite slurry into the soil in the immediate vicinity of the joint usually keeps the flow of water very small (Hanna 1978). Measured head efficiency for concrete cutoff walls from piezometric data generally exceeds 90 percent (Telling, Menzies, and Simons 1978b). At Kinzua Dam (formerly Allegheny Dam), the measured head efficiency was 100 percent, i.e., the head just downstream of the concrete cutoff wall was of the magnitude established by vertical seepage through the upstream connecting blanket (Fuquay 1968).

(b) Strength. The compressive strength for concrete cutoff walls is generally specified to exceed 3,000 lb/sq in. (see table 9-8). Therefore, the concrete cutoff wall is generally stronger than the surrounding foundation soil. The most important factor influencing the strength of the concrete is the water-cement ratio. The concrete's fluidity, i.e., ability to travel through the tremie and fill the excavation, also depends upon the water-cement ratio. Too low a water-cement ratio would decrease flowability and increase compressive strength. Too high a water-cement ratio would promote segregation. A good balance is achieved with a water-cement ratio near 0.5 which results in a 28-day compressive strength exceeding 3,000 lb/sq in. (see table 9-8). Cement continues to hydrate and concrete continues to increase in compressive strength, at a decreasing rate, long after 28 days (Winter and Nilson 1979).

(c) Compressibility. The concrete cutoff wall is essentially rigid and has low compressibility compared to the surrounding foundation soil. The modulus of elasticity for concrete cutoff walls may be approximated from (Winter and Nilson 1979)

$$E_c = 33W^{3/2}\sqrt{F'_c} \qquad (9\text{-}13)$$

where

E_c = modulus of elasticity in lb/sq in.

W = unit weight of concrete in lb/cu ft

F'_c = compressive strength of concrete in lb/sq in.

(5) Mix Design. In addition to strength, workability is an important requirement for the concrete mix. The mix must not segregate during placement. Too high a water-cement ratio or too low a cement content (with a good water-cement ratio) will tend to segregate. Natural well rounded aggregate increases flowability and allows the use of less cement than an angular

manufactured aggregate. Since the concrete is poured into the trench through tremie pipes and displaces the bentonite slurry from the bottom of the excavation upward, the concrete must have a consistency such that it will flow under gravity and resist mixing with the bentonite slurry. Admixtures may be used as required to develop the desired concrete mix characteristics. Fly ash is often used to improve workability and to reduce heat generation. The unique problems inherent at each project require studies to develop an adequate concrete mix (Holland and Turner 1980). Some typical concrete mixes used in Corps of Engineers concrete cutoff walls are given in table 9-8. The placement techniques used for the concrete are of equal importance in assuring a satisfactory concrete cutoff wall.

(6) Excavation and Placement of Concrete. Temporary guide walls are constructed at the ground surface to guide the alignment of the trench and support the top of the excavation. Typically, a cross section, 1 ft wide and 3 ft deep, is sufficient for most concrete cutoff walls. In order to ensure continuity between panels and provide a watertight joint to prevent leakage, an appropriate tolerance is placed on the maximum deviation from the vertical (see table 9-7). The same general requirements apply to the slurry used to keep the trench open for concrete cutoffs. As stated previously, two general types of concrete cutoff walls, the panel wall, and the element wall have been used. The panel wall is best suited for poorly consolidated materials and soft rock can be installed to about a 200-ft depth. The element wall has the advantage of greater depth (430 ft deep at Manicouagan 3 Dam in Quebec, Canada), better control of verticality, the ability to penetrate hard rock using chisels and/or nested percussion drills, and the protection of the embankment with casing when used for remedial seepage control. However, the element wall is more costly and has a slower placement rate than the panel wall. Both types of concrete cutoff walls open short horizontal sections of the embankment and/or foundation at a time, which limits the area for potential failure to a segment that can be controlled or repaired without risking catastrophic failure of the project. The concrete cutoff wall penetrates the zone(s) of seepage with a rigid, impermeability barrier capable of withstanding high head differentials across cavities with no lateral support. The concrete must be placed at considerable depth through bentonite slurry in a continuous operation with as little contamination, honeycomb, or segregation as possible. The bottom of the excavation must be cleaned so that a good seal can be obtained at grade. Fresh bentonite slurry is circulated through the excavation to assist in the cleaning and lower the density of the slurry to allow the concrete to displace the slurry easier once placement begins. The tremie procedure used to place the concrete is straightforward in theory and yet often in practice causes more problems with the final quality of the concrete cutoff wall than any other factor. The tremie system consists of a hopper, tremie pipe, and a crane or other lifting equipment to support the apparatus. The hopper should be funnel shaped and have a minimum capacity of 0.5 cu yd. The size of the tremie pipe depends upon the size of aggregate used in the concrete mix. For 3/4-in. maximum diameter coarse aggregate, a 10-in.-diam tremie pipe

should be used.[1] The dry tremie is placed in the hole with a metal plate and rubber gasket wired to the end of the tremie. The tremie pipe is lifted, breaking the wires and allowing the concrete flow to begin. Concrete is added to the hopper at a uniform rate to minimize free fall to the surface in the pipe and obtain a continuous flow. The tremie apparatus is lifted during placement at a rate that will maintain the bottom of the pipe submerged in fresh concrete at all times and produce the flattest surface slope of concrete that can practically be achieved. The flow rate (foot of height per hour) and surface slope of the concrete shall be continuously measured during placement with the use of a sounding line. A sufficient number of tremies should be provided so that the concrete does not have to flow horizontally from a tremie more than 10 ft. As soon as practical, core borings should be taken in selected panels through the center of the cutoff wall to observe the quality of the final project. Unacceptable zones of concrete such as honeycombed zones, segregated zones, or uncemented zones found within the cored panels or elements should be repaired or removed and replaced. One means of minimizing such problems at the start of a job is to require a test section in a noncritical area to allow changes in the construction procedure to be made early in the project (Hallford 1983; Holland and Turner 1980; and Gerwick, Holland, and Komendant 1981).

(7) Treatment at Top of Concrete Cutoff Wall. As mentioned previously, normally the concrete cutoff wall is located under or near the upstream toe of the dam and tied into the core of the dam with an impervious blanket. If a central location for the concrete cutoff wall is dictated by other factors, the connection detail between the top of the concrete cutoff wall and the core of the dam is very important. Generally, the concrete cutoff wall extends upward into the core such that, the hydraulic gradient at the surface of the contact does not exceed 4 (Wilson and Marsal 1979). Various precautions (see figure 9-15) have been taken to prevent the top of the concrete cutoff wall from punching into the core of the dam and causing the core to crack as the foundation settles on either side of the rigid cutoff wall under the weight of the embankment. The bentonite used at the connection between the concrete cutoff wall and the core of the dam (see figure 9-15) is intended to create a soft zone to accommodate differential vertical settlements of the core around the concrete cutoff wall. Also, saturation of the bentonite is intended to produce swelling which will provide for a bond between the core and the concrete cutoff wall to prevent seepage (Radukic 1979).

(8) Failure Mechanisms of Concrete Cutoff Walls. Several mechanisms can affect the functioning of concrete cutoff walls and cause failure. As mentioned previously, the wall in its simpler structural form is a rigid diaphragm and earthquakes could cause its rupture. For this reason concrete cutoff walls should not be used at a site where strong earthquake shocks are

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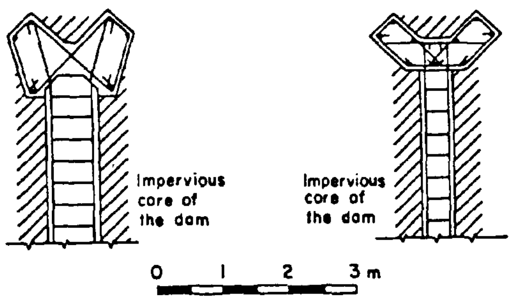
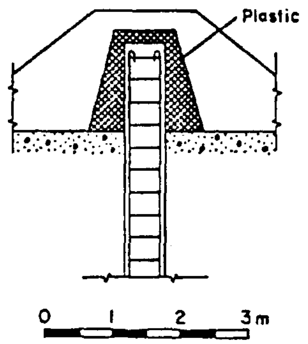
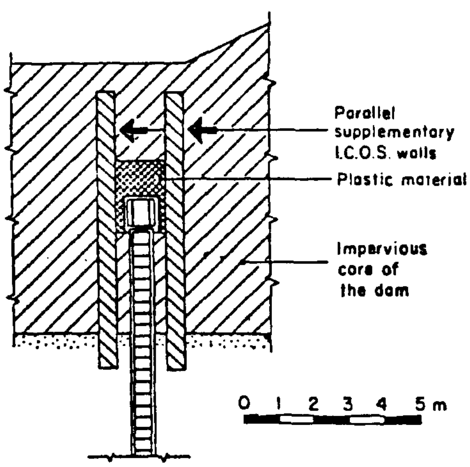
[1] At Wolf Creek Dam concrete problems (areas of segregated sand or coarse aggregate, voids, zones of trapped laitance, and honeycombed concrete) occurred for tremie-placed 26-in.-diam cased primary elements. This must be considered in future projects which involve tremie-placed elements of small cross-sectional areas (Holland and Turner 1980).



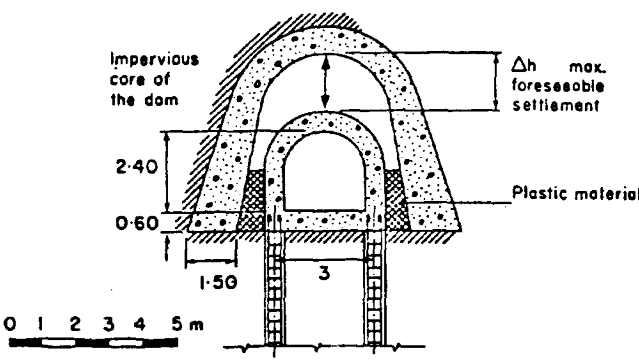
a. Forked connection



b. Plastic impervious cap



c. Piston connection



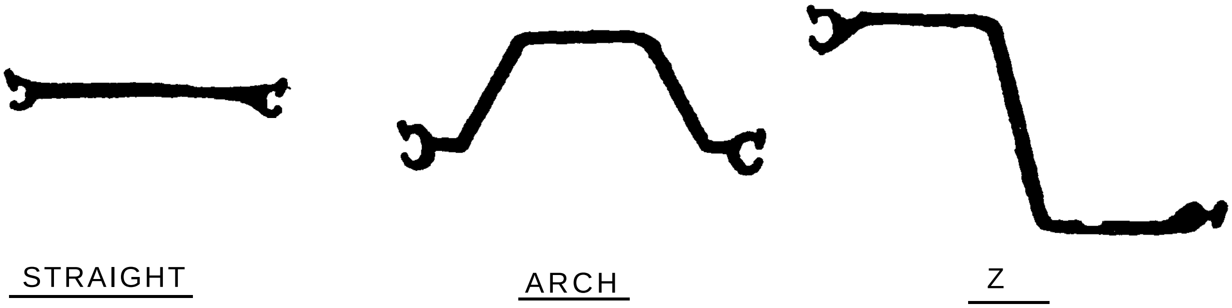
d. Double wall connection

Figure 9-15. Connections between concrete cutoff wall and core of dam (courtesy of ICOS[182]) likely. Concrete cutoff walls located under or near the toe of the dam are subject to possible rupture from horizontal movements of the foundation soil during embankment construction. This effect can be minimized by constructing the dam embankment prior to the concrete cutoff wall. As mentioned previously, concrete cutoff walls located under the center of the dam are subject to possible compressive failure due to negative skin friction as the foundation settles under the weight of the embankment. The probability of this occurring would depend upon the magnitude of the negative skin friction developed at the interface between the concrete cutoff wall and the foundation soil and the stress-strain characteristics of the concrete cutoff wall. Also, as previously mentioned, a centrally located concrete cutoff wall may punch into and crack the overlying core material unless an adequate connection is provided between the concrete cutoff wall and the core of the dam.

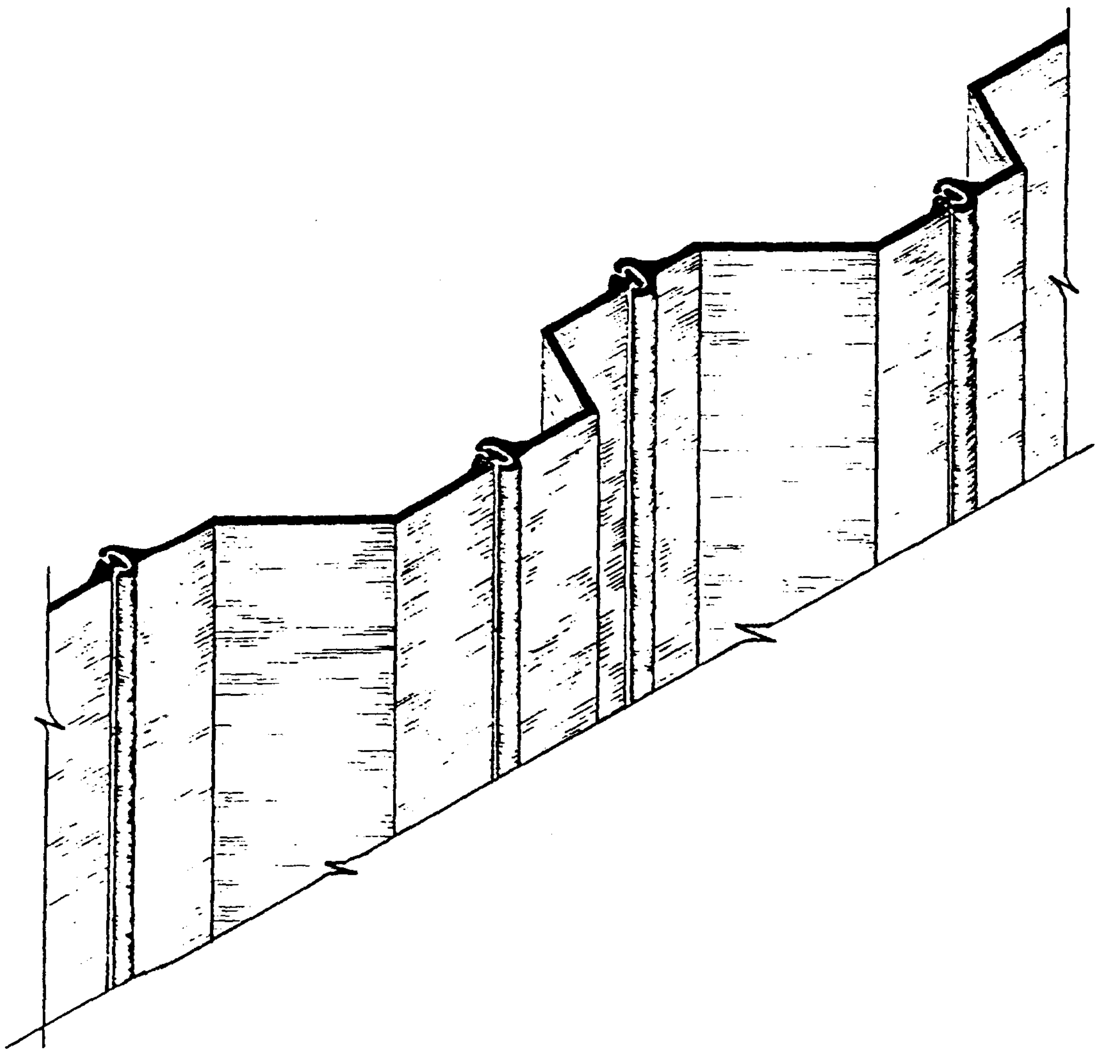
(9) Instrumentation and Monitoring. Whenever a concrete cutoff wall is used for control of underseepage, the initial filling of the reservoir must be controlled and instrumentation monitored to determine if the concrete cutoff wall is performing as planned. If the concrete cutoff wall is ineffective, remedial seepage control measures must be installed prior to further raising the reservoir pool. When the embankment is constructed first, followed by the concrete cutoff wall located upstream of the toe of the dam, as was done at Kinzua (formerly Allegheny Dam), the parameters of interest are the drop in piezometric head from upstream to downstream across the concrete cutoff wall, differential vertical settlement between the upstream impervious blanket and the top of the concrete cutoff wall, and vertical and horizontal movement of the concrete cutoff wall due to reservoir filling. If a central location for the concrete cutoff wall is dictated by others factors, the parameters of interest are the drop in piezometric head from upstream to downstream across the cutoff wall, differential vertical settlement between the core of the dam and the top of the concrete cutoff wall, and vertical and horizontal movement of the concrete cutoff wall due to construction of the embankment and reservoir filling. Instrumentation data should be obtained during construction, before and during initial filling of the reservoir, and subsequently as frequently as necessary to determine changes that are occurring and to assess their implications with respect to the safety of the dam (see Chapter 13). The head efficiency for concrete cutoff walls is evaluated in the same manner as described previously for slurry trench cutoffs. As previously mentioned, measured head efficiency for concrete cutoff walls generally exceeds 90 percent.

f. Steel Sheetpiling.

(1) Introduction. Steel sheetpiling is rolled steel members with interlocking joints along their edges. Sheetpiling is produced in straight web, arch web, and Z sections in a graduated series of weights joined by interlocks to form a continuous cutoff wall as shown in figure 9-16. Steel sheetpiling is not recommended for use as a cutoff to prevent underseepage beneath dams due to the low head efficiency. Steel sheetpiling is frequently used in conjunction with concrete flood control and navigation structures to confine the foundation soil to prevent it from piping out from under the structure (EM 1110-2-2300 and Greer, Moorhouse, and Millet 1969).



a. Sections



b. Interlocking of sections

Figure 9-16. Steel sheetpiling installation (from U. S. Army Engineer Waterways Experiment Station[57])

(2) History of use. Steel sheetpiling was first used by the Corps of Engineers to prevent underseepage at Fort Peck Dam, Montana (U. S. Army Engineer District, Omaha 1982). The steel sheetpiling, driven to Bearpaw shale bedrock with the aid of hydraulic spade jetting, reached a maximum depth of 163 ft in the valley section (see table 9-9). An original plan to force grout into the interlocks of the steel sheetpiling was abandoned during construction as impractical. Steel sheetpiling was used as an extra factor to prevent piping of foundation soils at Garrison Dam, North Dakota (U. S. Army Engineer District, Omaha 1964). At Garrison Dam, underseepage control was provided for by an upstream blanket and relief wells and the contribution of the steel sheetpiling to reduction of underseepage was neglected in the design of the relief wells. Steel sheetpiling and an upstream blanket were installed for control underseepage at Oahe Dam, South Dakota. Relief wells were installed for remedial seepage control to provide relief of excess hydrostatic pressures developed by underseepage (U. S. Army Engineer District, Omaha 1961).

(3) Efficiency of Steel Sheetpiling Cutoffs. The efficiency of steel sheetpiling cutoffs is dependent upon proper penetration into an impervious stratum and the condition of the sheeting elements after driving. When the foundation material is dense or contains boulders which may result in ripping of the sheeting or damage to the interlocks (see figure 9-17), the efficiency will be reduced (Guertin and McTigue 1982). Theoretical studies indicate that very small openings in the sheeting ($\leq$ 1 percent of the total area) will cause a substantial reduction in the cutoff efficiency (from 100 to 10 percent efficiency) as shown in figure 9-18 (Ambraseys 1963). The measured head efficiency for steel sheetpiling cutoffs installed at Corps of Engineers dams is given in table 9-9. The effectiveness of the steel sheetpiling is initially low, only 12 to 18 percent of the total head was lost across the cutoff as shown in table 9-9. With time, the head loss across the steel sheetpiling increased to as much as 50 percent of the total head. This increase in effectiveness is attributed to migration of fines and corrosion in the interlocks and reservoir siltation near the dam.

9-5. Upstream Impervious Blanket.[1]

a. Introduction. When a complete cutoff is not required or is too costly, an upstream impervious blanket tied into the impervious core of the dam may be used to minimize underseepage. Upstream impervious blankets should not be used when the reservoir head exceeds 200 ft because the hydraulic gradient acting across the blanket may result in piping and serious leakage. Downstream underseepage control measures (relief wells or toe trench drains) are generally required for use with upstream blankets to control underseepage and/or prevent excessive uplift pressures and piping through the foundation. Upstream impervious blankets are used in some cases to reinforce thin spots in natural blankets. Effectiveness of upstream impervious blankets depends upon their length, thickness, and vertical permeability, and on the stratification and permeability of soils on which they are placed (EM 1110-2-2300, Barron 1977 and Thomas 1976).

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[1] The blanket may be impervious or semipervious (leaks in the vertical direction).

Table 9-9. Comparison of Corps of Engineers Steel Sheetpiling Cutoff Walls

Project	Location	Date Constructed	Foundation Materials	Max Depth ft	Location	Purpose	Head Efficiency of Cutoff, % Initial	Head Efficiency of Cutoff, % With Time	Reference
Ft. Peck Dam	Missouri River, Montana	1940	Gravels, sands, and silts	163	Center of dam	Prevent underseepage	12	30[a]	Lane and Wohlt 1961
Garrison Dam	Missouri River, North Dakota	1954	Sand with gravel lenses	110	Upstream portion of dam	Prevent piping	18	50[b]	U. S. Army Engineer District, Omaha 1964
Oahe Dam	Missouri River, South Dakota	1958	Sands, gravels, and silts	75	Upstream portion of dam	Prevent piping	16	22[c]	U. S. Army Engineer District, Omaha 1961

(a) After 17 years (relief wells were installed in 1942 as remedial underseepage control)
(b) After 10 years (upstream blanket and relief wells were constructed prior to impoundment)
(c) After 4 years (upstream blanket constructed prior to impoundment and relief wells installed in 1962 as remedial underseepage control)

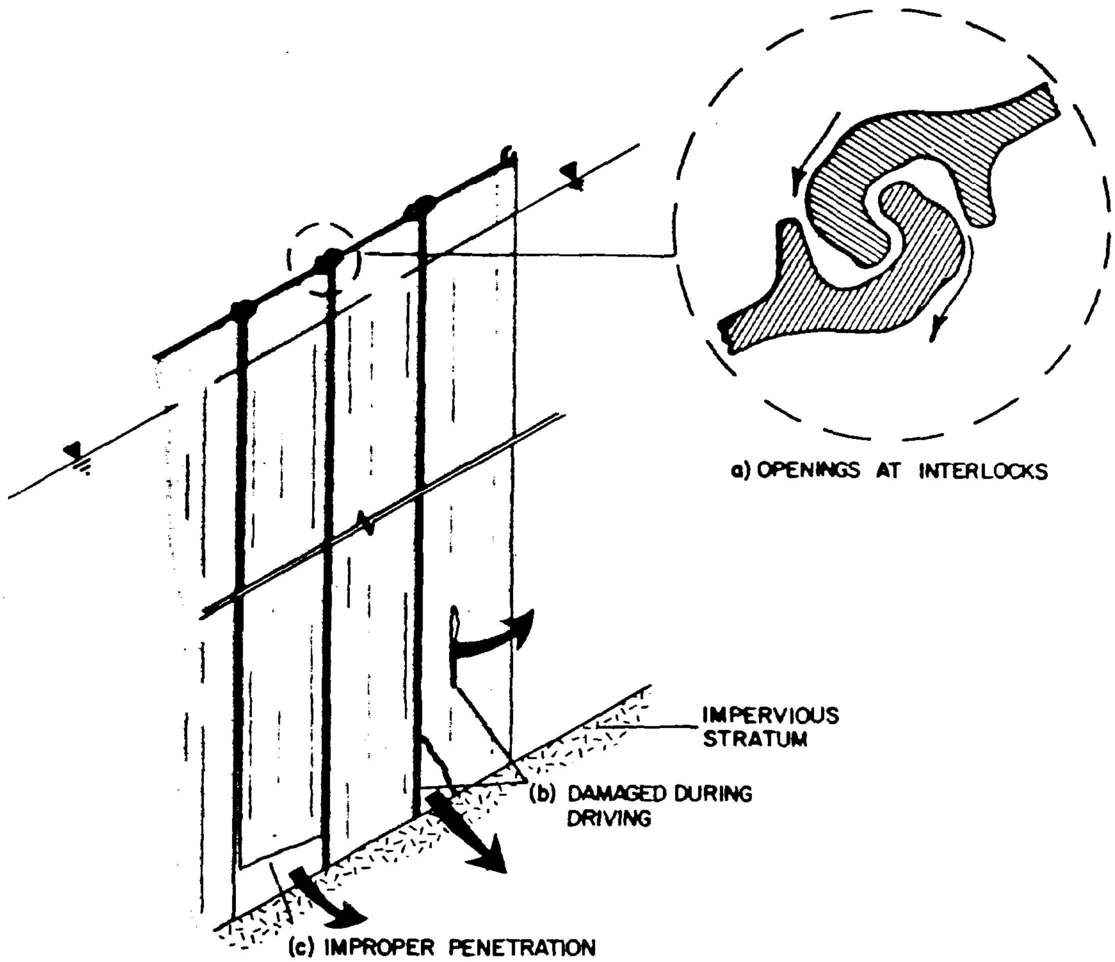


Figure 9-17. Sources of leakage associated with steel sheetpile cutoffs (from U. S. Department of Transportation[41])

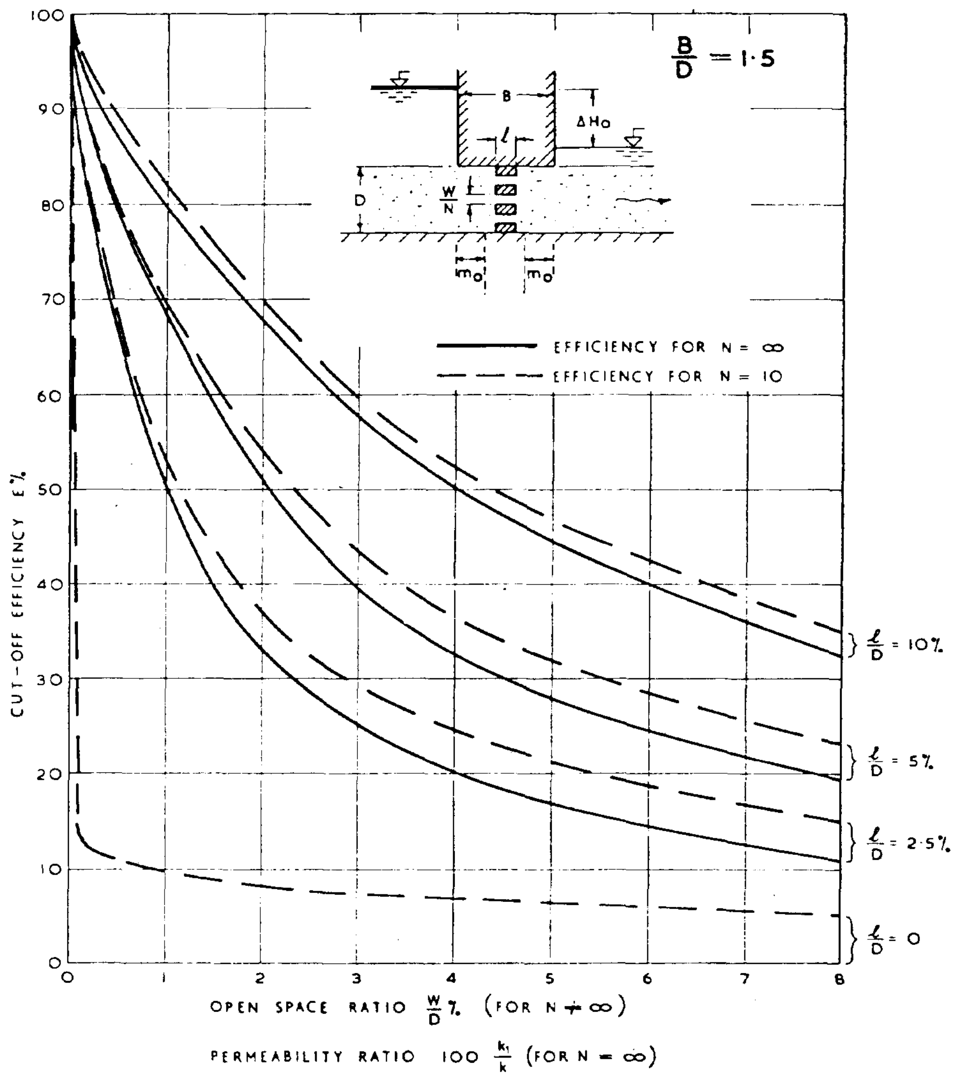


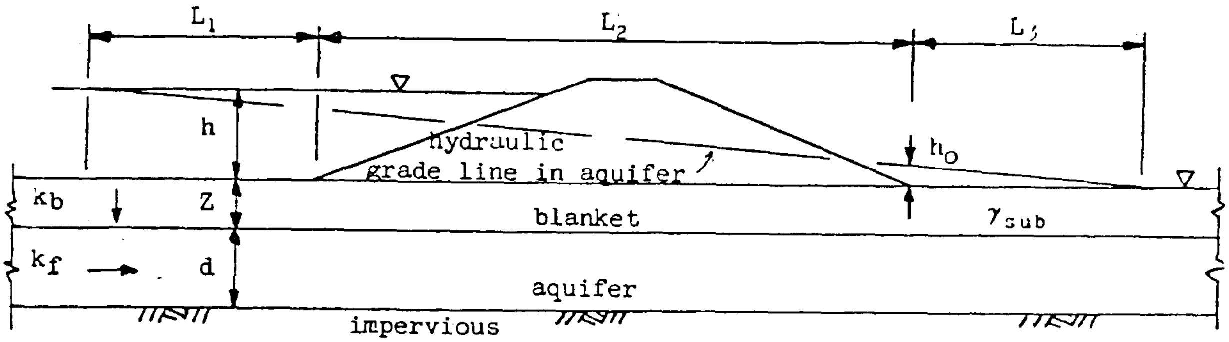
Figure 9-18. Cutoff efficiency versus open space ratio for imperfect cutoffs (courtesy of Butterworths, Inc.[129])

b. Design Considerations. In alluvial valleys, frequently soils consist of fine-grained top stratum of clay, silt, and silty or clayey sand underlain by a pervious substratum of sand and gravel. As stated previously, the top stratum or blanket may be impervious or semipervious (leaks in the vertical direction). The substratum aquifer or pervious foundation is generally anisotropic with respect to permeability so the flow is horizontal. For this condition, shown in figure 9-19, the basic assumptions for the design of upstream impervious blankets are:

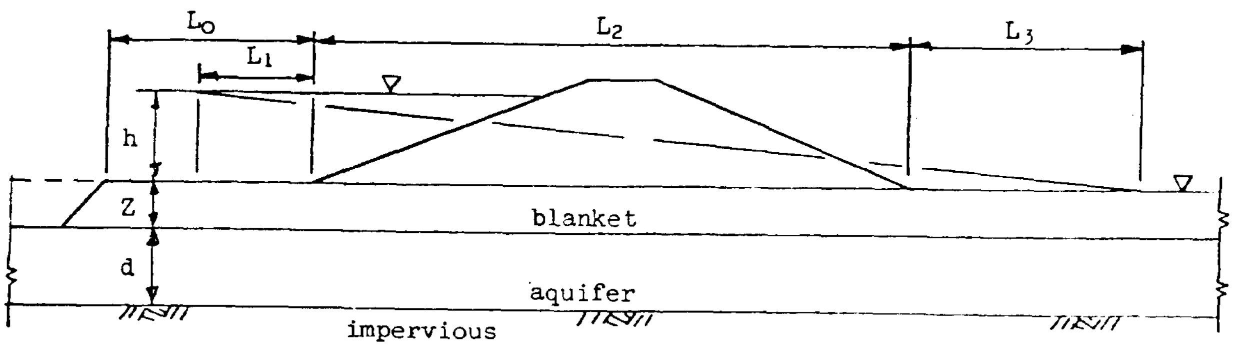
(1) Flow through the blanket is vertical.

(2) Flow through the pervious foundation is horizontal.

(3) All flows are laminar and steady state.



a. Continuous blanket and aquifer



b. Discontinuous upstream blanket, continuous aquifer

L_1 = Effective length of upstream natural blanket

L_2 = Length of embankment base

L_3 = Effective length of downstream natural blanket

L_o = Length of discontinuous upstream blanket

h = Net head to dissipate

Z = Thickness of natural blanket

d = Thickness of aquifer

k_b = Permeability coefficient of blanket

k_f = Permeability coefficient of aquifer

γ_{sub} = Submerged unit weight of blanket

h_o = Pressure head under blanket at downstream toe of dam

h_c = Critical head under blanket at downstream toe of dam

F_h = Factor of safety relative to heaving at downstream toe

γ_w = Unit weight of water (63.4 pcf)

q_f = Rate of discharge through aquifer with unit length normal to the section

Figure 9-19. Upstream impervious blanket (from U. S. Department of Agriculture[72])

(4) The dam (or core of a zoned embankment) is impervious.

(5) Both the blanket and substratum have a constant thickness and are horizontal.

When the top stratum or pervious foundation consists of several layers of different soils, they must be transformed into a single stratum with an effective thickness and permeability (see procedure given in U. S. Army Engineer Waterways Experiment Station 1956a). For the upstream impervious blanket shown in figure 9-19, the effective length of the upstream blanket is

$$L_1 = \sqrt{(k_f/k_{bR})Z_{bR}d} \tag{9-14}$$

where

L_1 = effective length of upstream blanket

k_f = horizontal permeability of pervious foundation

k_{bR} = vertical permeability of upstream blanket

Z_{bR} = thickness of upstream blanket

d = thickness of pervious foundation

The effective length of the downstream blanket is

$$L_3 = \sqrt{(k_f/k_{bL})\ Z_{bL}d} \tag{9-15}$$

where

L_3 = effective length of downstream blanket

k_{bL} = vertical permeability of downstream blanket

Z_{bL} = thickness of downstream blanket

Upstream blankets should be designed so that under maximum reservoir conditions the pressure head under the blanket at the downstream toe of the dam and the rate of discharge through the pervious foundation are acceptable. The pressure head under the blanket at the downstream toe of the dam (see figure 9-19) is

$$h_o = \frac{hL_3}{L_1 + L_2 + L_3} \tag{9-16}$$

where

h_o = pressure head under the blanket at the downstream toe of the dam

h = net head to dissipate

L_2 = length of impervious core or dam base

The critical pressure head under the blanket at the downstream toe of the dam is

$$h_c = \frac{Z_{bL}\gamma_{sub}}{\gamma_w} \tag{9-17}$$

where

h_c = critical pressure head under the blanket at the downstream toe of the dam

γ_{sub} = submerged unit weight of downstream blanket soil

γ_w = unit weight of water

The factor of safety against uplift or heaving at the downstream toe of the dam is

$$F_h = \frac{h_c}{h_o} \tag{9-18}$$

where F_h is the factor of safety against uplift or heaving at the downstream toe of the dam. Generally dams are designed without relying upon natural downstream blankets because it is difficult to assure the continuity and the existence of the blanket throughout the life of the structure. Also, downstream seepage control measures (relief wells or trench drains) are generally used with upstream blankets to reduce uplift or heaving at the downstream toe of the dam. However, for the exceptional case where the dam is designed with a natural downstream blanket and with no downstream seepage control measures (relief wells or trench drains), upstream blankets should be designed so that the factor of safety against uplift or heaving at the downstream toe of the dam is at least 3. The rate of discharge through the pervious foundation per unit length of dam (see figure 9-19) is

$$q_f = \frac{k_f hd}{L_1 + L_2 + L_3} \tag{9-19}$$

where q_f is the rate of discharge through the pervious foundation per unit length of dam. The acceptable rate of discharge or underseepage depends upon the value of the water or hydropower lost, availability of downstream right-of-way, and facility for disposal of underseepage. The following procedure is used to determine the length of an upstream blanket when there is a downstream blanket present (see figure 9-19b):

(a) Determine L_1 from equation 9-14 using a conservative value of k_f/k_{bR}, i.e., the highest probable ratio.

(b) Determine L_3 from equation 9-15 using a conservative value of k_f/k_{bL}, i.e., the highest probable ratio.

(c) Determine h_o, h_c, and F_h from equations 9-16, 9-17, and 9-18, respectively. If $F_h < 3.0$, the blanket thickness of the upstream blanket may be increased, the permeability of the upstream blanket material may be decreased by compaction, or downstream seepage control measures may be used.

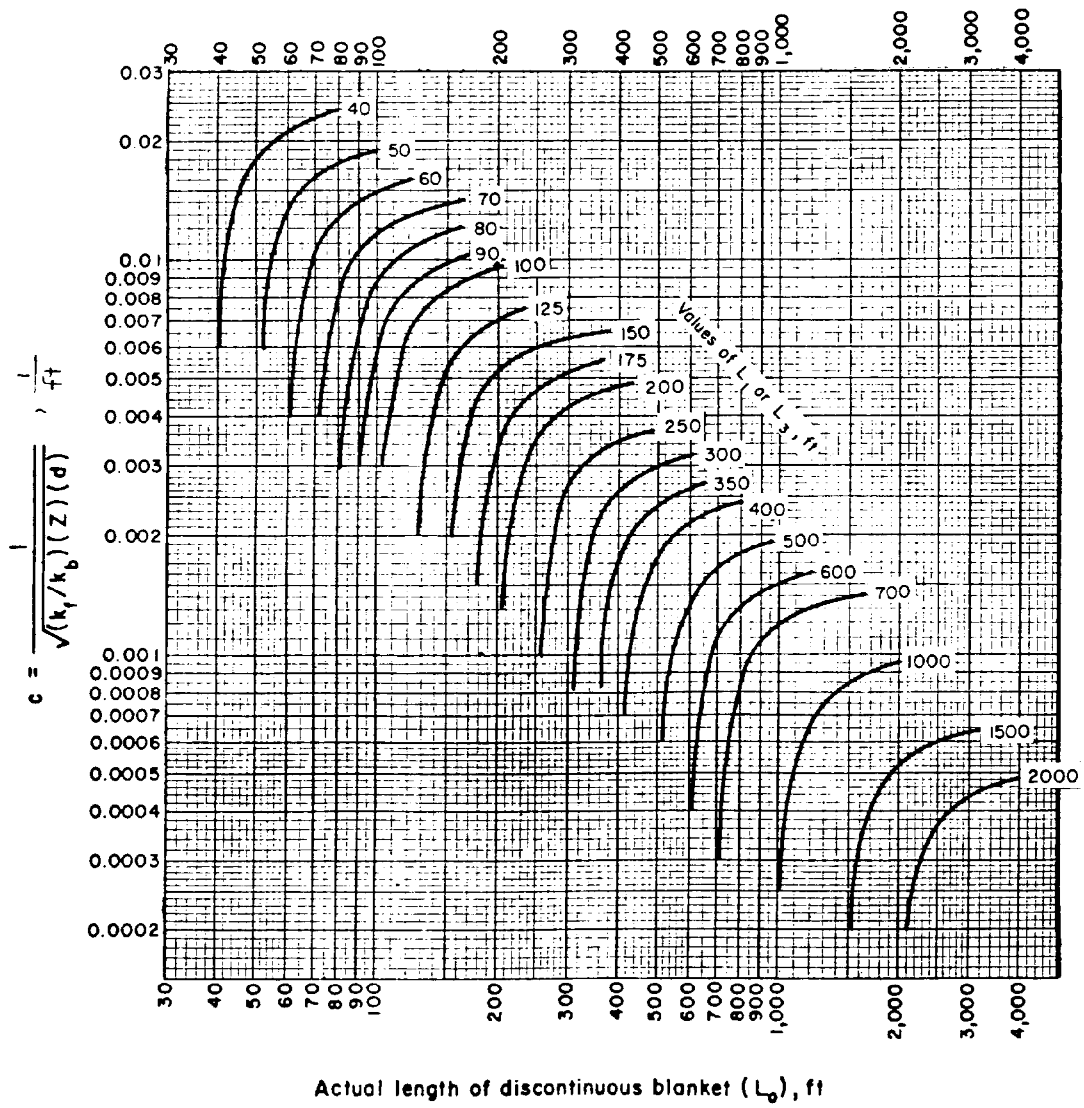
(d) Determine the rate of discharge through the pervious foundation per unit length of dam from equation 9-19. If the rate of discharge is excessive, a reduction can be obtained by increasing the thickness of the upstream blanket or reducing the permeability of the upstream blanket material by compaction. When these methods are used, steps 1 to 4 are repeated before going to step 5.

(e) If the rate of discharge is acceptable, calculate the factor

$$c = \frac{1}{\sqrt{(k_f/k_{bR})Z_{bR}d}} \tag{9-20}$$

where c has the units of 1/ft.

(f) Enter figure 9-20 with c and L_1 and obtain L_o, which is the distance from the upstream toe of a homogeneous impervious dam or the impervious core section of a zoned embankment to where a discontinuity in the upstream blanket will have no effect on the uplift at the downstream toe of the dam or rate of discharge through the pervious foundation. This is the point beyond which a natural blanket may be removed in a borrowing operation. Also, L_o would represent the distance upstream from the toe of the dam to which a streambed should be blanketed to ensure the continuity of a natural upstream blanket. If there is no downstream blanket the pressure head under the blanket at the downstream toe of the dam will be zero (see equation 9-16)



$$L_{1,3} = \frac{e^{2cL_o} - 1}{c(e^{2cL_o} + 1)}$$

Figure 9-20. Effective lengths of upstream and downstream impervious blankets (from U. S. Department of Agriculture[72])

and the following procedure is used to determine the length of the upstream blanket:

- Assume several values of L_o (length of the upstream blanket from the upstream toe of a homogeneous impervious dam or the impervious core section of a zoned embankment).

- Calculate c from equation 9-20 using the design thickness and permeability rates for the constructed blanket and pervious foundation. Note that c has units of 1/ft.

- Enter figure 9-20 with the assumed values of L_o and the calculated values of c to obtain the corresponding value of L_1 for each assumed value of L_o.

- Calculate q_f from equation 9-19 ($L_3 = 0$ for no downstream blanket) using the values of L_1 obtained from figure 9-20.

- Plot q_f versus L_o. The curve will indicate a rapid decrease in q_f with increasing values up to a point where the curve flattens out indicating an optimum length. The upstream blanket can be terminated at any point where the desired reduction in rate of discharge through the pervious foundation per unit length of dam is achieved (Talbot and Nelson 1979).

c. Materials and Construction. At sites where a natural blanket of impervious soil already exists, the blanket should be closely examined for gaps such as outcrops of pervious strata, streambeds, root holes, boreholes, and similar seepage paths into the pervious foundation which, if present, should be filled or covered with impervious material to provide a continuous blanket to a distance L_o from the upstream toe of the dam. Also, as previously stated, upstream borrow areas should be located greater than the distance L_o from the upstream toe of the dam so as not to reduce the effectiveness of the natural blanket. Figure 9-21 shows the influence of gaps in the upstream blanket on relative seepage and uplift at the toe of the dam. That portion of the upstream blanket placed beneath the embankment to tie into the impervious core should be composed of the same material and compacted in the same manner as the core. Upstream of the embankment, the blanket is constructed by placing impervious soil in lifts and compacted only by movement of hauling and spreading equipment, or to whatever additional extent is necessary for equipment operation. Exposed clay blankets can shrink and crack after placement. If such cracks penetrate the blanket, they will reduce the effectiveness of the blanket. Thus it may become necessary to sprinkle the surface of the blanket to help retain moisture until a permanent pool is impounded. In higher reaches of abutments which are infrequently flooded by the reservoir, a thicker blanket may be required so that cracks will not fully penetrate the blanket. In colder climates, the blanket thickness should be increased to account for the loosening of the upper part of the blanket by frost action which substantially increases the permeability.

d. Reservoir Siltation. For some reservoirs, appreciable siltation occurs which may both increase the thickness of and lengthen the upstream blanket. Although the siltation may reduce the rate of discharge through the pervious foundation with time, it is not a factor to be counted upon in design because the upstream blanket must function adequately following initial filling of the reservoir prior to the occurrence of siltation.

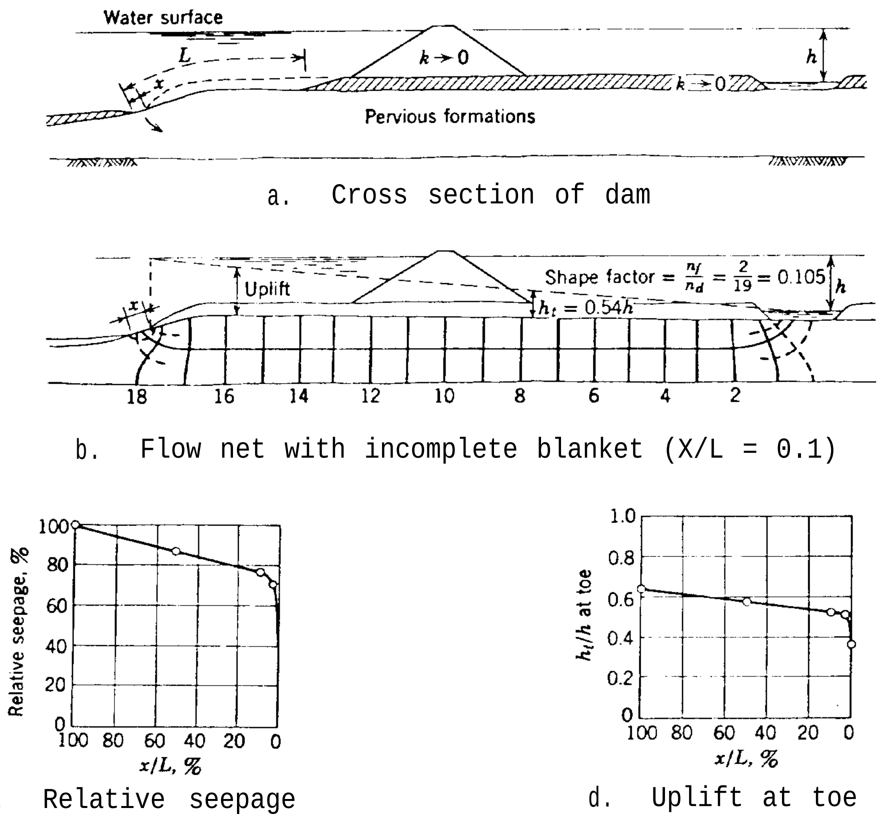


Figure 9-21. Effect of gap in upstream blanket on relative seepage and uplift at toe (courtesy of John Wiley and Sons[155])

9-6. Downstream Seepage Berms.

a. Introduction. When a complete cutoff is not required or is too costly, and it is not feasible to construct an upstream impervious blanket, a downstream seepage berm may be used to reduce uplift pressures in the pervious foundation underlying an impervious top stratum at the downstream toe of the dam. Other downstream underseepage control measures (relief wells or toe trench drains) are generally required for use with downstream seepage berms. Downstream seepage berms can be used to control underseepage efficiently where the downstream top stratum is relatively thin and uniform or where no top stratum is present, but they are not efficient where the top stratum is relative thick and high uplift pressures develop. Downstream seepage berms may vary in type from impervious to completely free draining. The selection of the type of downstream seepage berm to use is based upon the availability of borrow materials and relative cost of each type.

b. Design Considerations. When the top stratum or pervious foundation consists of several layers of different soils, they must be transformed into a single stratum with an effective thickness and permeability (see procedure given in U. S. Army Engineer Waterways Experiment Station 1956a). Where a downstream natural blanket is present, the downstream seepage berm should have a thickness so that the factor of safety against uplift or heaving at the downstream toe of the dam is at least 3 and width so that the factor of safety against uplift at the downstream toe of the seepage berm is at least 1.5. Formulas for the design of downstream seepage berms where a downstream natural blanket is present are given in figure 9-22. If there is no downstream natural blanket present, the need for a downstream seepage berm will be based upon Bligh's creep ratio.

$$C_B = \frac{X_1 + L_2 + X}{h} \tag{9-21}$$

where

c_B = Bligh's creep ratio

X_1 = effective length of upstream blanket

L_2 = length of dam base

X = width of downstream seepage berm

h = net head on dam

Minimum acceptable values of Bligh's creep ratio are given in table 9-10. If the creep ratio is greater than the minimum value, a downstream seepage berm is not required.[1] If the creep ratio is less than the minimum value, the width of the downstream seepage berm should be made such that the creep ratio is above the minimum value shown in table 9-10. The thickness of the downstream seepage berm at the toe of the dam will be determined so that the factor of safety against uplift or heaving at the downstream toe of the dam is at least 3. The pressure head beneath the downstream seepage berm at the landside toe of the levee is

$$h_o = \frac{h(X + 0.43d)}{X_1 + L_2 + X + 0.43d} \tag{9-22}$$

where

h_o = pressure head under the seepage berm at the downstream toe of the dam

[1] A downstream seepage berm may be required to correct other problems such as excessive seepage gradients under the dam (could be detected by checking the rate of underseepage).

d = thickness of pervious foundation

X_1 = effective length of upstream natural blanket (taken equal to 0.43d where no upstream natural blanket exists)

The rate of discharge through the pervious foundation per unit length of dam is

$$q_f = \frac{k_f hd}{X_1 + L_2 + X + 0.43d} \tag{9-23}$$

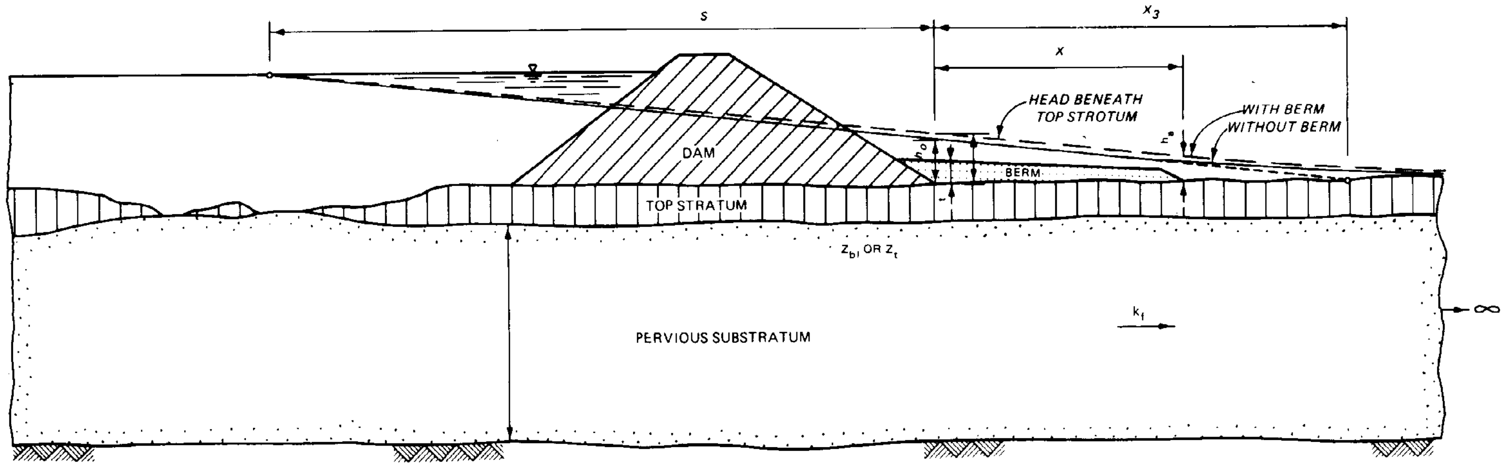
where

q_f = rate of discharge through the pervious foundation per unit length of dam

k_f = horizontal permeability of pervious foundation

As stated previously, the acceptable rate of discharge or underseepage depends upon the value of the water or hydropower lost, availability of downstream right-of-way, and facility for disposal of underseepage. Downstream seepage berms should have a minimum thickness of 10 ft at the dam toe and a minimum thickness of 5 ft at the berm toe. The computed thickness of the berm should be increased 25 percent to allow for shrinkage, foundation settlements, and variations in the design factors. Downstream seepage berms should have a slope of 1V on 50H or steeper to ensure drainage (U. S. Army Engineer Waterways Experiment Station 1956a).

c. Materials and Construction. As previously stated, the selection of the type of material used to construct the downstream seepage berm is based upon the availability of borrow materials and relative cost of each type. A berm constructed of impervious soil should be composed of the same material as the impervious core. That portion of the downstream impervious seepage berm placed beneath the embankment to tie into the impervious core should be compacted in the same manner as the core. Downstream of the embankment, the impervious seepage berm is constructed by placing impervious soil in lifts and compacting only by movement of hauling and spreading equipment, or to whatever additional extent is necessary for equipment operation. Semipervious material used to construct downstream seepage berms should have an in-place vertical permeability equal to or greater than that of the upstream natural blanket and are compacted in the same manner as described previously for impervious material. Material used in a sand berm should be as pervious as possible, with a minimum in-place vertical permeability of 100×10^{-4} cm per sec. Downstream seepage berms constructed of sand should be compacted to an average in-place relative density of at least 85 percent with no portion of the berm having a relative density less than 80 percent. As proper functioning of a downstream seepage berm constructed of sand depends upon its continued perviousness, it should not be constructed until after the downstream slope of the earth dam has



NOTATIONS

$C = \sqrt{\dfrac{k_{bL}}{k_f Z_{bL} d}}$

γ'_z = SUBMERGED UNIT WEIGHT OF TOP STRATUM

γ'_t = SUBMERGED UNIT WEIGHT OF BERM

γ_w = UNIT WEIGHT OF WATER

h_o = HEAD AT LANDSIDE TOE OF DAM WITHOUT BERM = $\dfrac{H x_3}{S + x_3}$

h'_o = HEAD AT LANDSIDE TOE OF DAM WITH BERM

i_o = ALLOWABLE UPWARD GRADIENT AT LANDSIDE TOE OF DAM

i_1 = ALLOWABLE UPWARD GRADIENT AT TOE OF BERM

h_a = ALLOWABLE HEAD AT TOE OF BERM = $i_1 z_t$

X = REQUIRED BERM WIDTH

t = REQUIRED THICKNESS OF BERM AT TOE OF DAM

k_t = VERTICAL PERMEABILITY OF BERM

F = FACTOR OF SAFETY AGAINST UPLIFT AT TOE OF DAM

Q_B = FLOW INTO BERM PER FT OF DAM

FORMULAS

IMPERVIOUS BERM ($k_t = 0$)

$$X_I = x_3\left(\frac{H}{h_a} - 1\right) - s$$

$$h'_o = H\left(\frac{x_3 + X}{S + x_3 + X}\right)$$

$$t = \frac{h'_o - Z_t\left(\dfrac{\gamma'_z}{F\gamma_w}\right)}{1 + \dfrac{\gamma'_t}{F\gamma_w}}$$

USE $F \geq 1.5$

SEMIPERVIOUS BERM ($k_t - k_{bL}$)

$$X_{SP} = \frac{-A + \sqrt{A^2 - 24(2 + r)\left(1 + SC - \dfrac{H}{h_a}\right)}}{2C(2 + r)}$$

WHERE IN:

$A = 6 + 3SC(r + 1)$

$r = \dfrac{i_o}{i_1}$

$$h'_o = h_a\left[1 + CX + \left(\frac{2 + r}{6}\right)(CX)^2\right]$$

$$t = \frac{h'_o - i_o Z_t}{1 + i_o}$$

SAND BERM

$$X_S = \frac{1}{3}\left(X_p + 2x_{SP}\right)$$

$$h'_o = h_a\left[1 + CX + \left(\frac{2 + r}{6}\right)(CX)^2\right]$$

$$t = \frac{h'_o - Z_t\left(\dfrac{\gamma'_z}{F\gamma_w}\right)}{1 + \dfrac{\gamma'_t}{F\gamma_w}}$$

PERVIOUS BERM WITH COLLECTOR

$$X_p = x_3 \,\mathrm{LOG}_e\left(\frac{h_o^1}{h_a}\right)$$

$$h'_o = h_o = \frac{Hx_3}{S + x_3}$$

$$t = \frac{h'_o - Z_t\left(\dfrac{\gamma'_z}{F\gamma_w}\right)}{1 + \dfrac{\gamma'_t}{F\gamma_w}}$$

$$Q_B = \frac{k_f dH}{S + x_3}\left(1 - e^{\frac{-X}{x_3}}\right)$$

Figure 9-22. Design of downstream seepage berms where a downstream natural blanket is present (from U.S. Army Engineer Waterways Experiment Station[120])

Table 9-10. Minimum Bligh's Creep Ratios for Dams Founded on Pervious Foundations[a]

Material	Minimum Bligh's Creep Ratio
Very fine sand or silt	18
Fine to medium sand	15
Coarse sand	12
Fine gravel or sand and gravel	9
Coarse gravel including cobbles	

[a] From U. S. Army Engineer Waterways Experiment Station[120]

become covered with sod and stabilized so that soil particles carried by surface runoff and erosion will not clog the seepage berm. If it is necessary to construct the downstream seepage berm at the time the earth dam is built or before it has become covered with sod, an interceptor dike should be built at the intersection of the downstream toe of the dam and the seepage berm to prevent surface wash from clogging the seepage berm. A free-draining downstream seepage berm is one composed or random fill overlying horizontal sand and gravel drainage layers with a terminal perforated collector pipe system (U. S. Army Engineer Waterways Experiment Station 1956a).

9-7. Relief Wells.

a. Introduction. When a complete cutoff is not required or is too costly, relief wells installed along the downstream toe of the dam may be used to prevent excessive uplift pressures and piping through the foundation. Relief wells increase the quantity of underseepage from 20 to 40 percent depending upon the foundation conditions. Relief wells may be used in combination with other underseepage control measures (upstream impervious blanket or downstream seepage berm) to prevent excessive uplift pressures and piping through the foundation. Relief wells are applicable where the pervious foundation has a natural impervious cover. The well screen section (see figure 9-23), surrounded by a filter if necessary, should penetrate into the principal pervious stratum to obtain pressure relief, especially where the foundation is stratified. The wells, including screen and riser pipe, should have a diameter which will permit the maximum design flow without excessive head losses but in no instance should the inside diameter be less than 6 in. Filter fabrics should not be used in conjunction with relief wells (see Appendix D). Even in nearly homogeneous stratum, a penetration of less than 50 percent results in significant rise in pressure midway between adjacent wells, or requires close spacing. Relief wells should be located so that

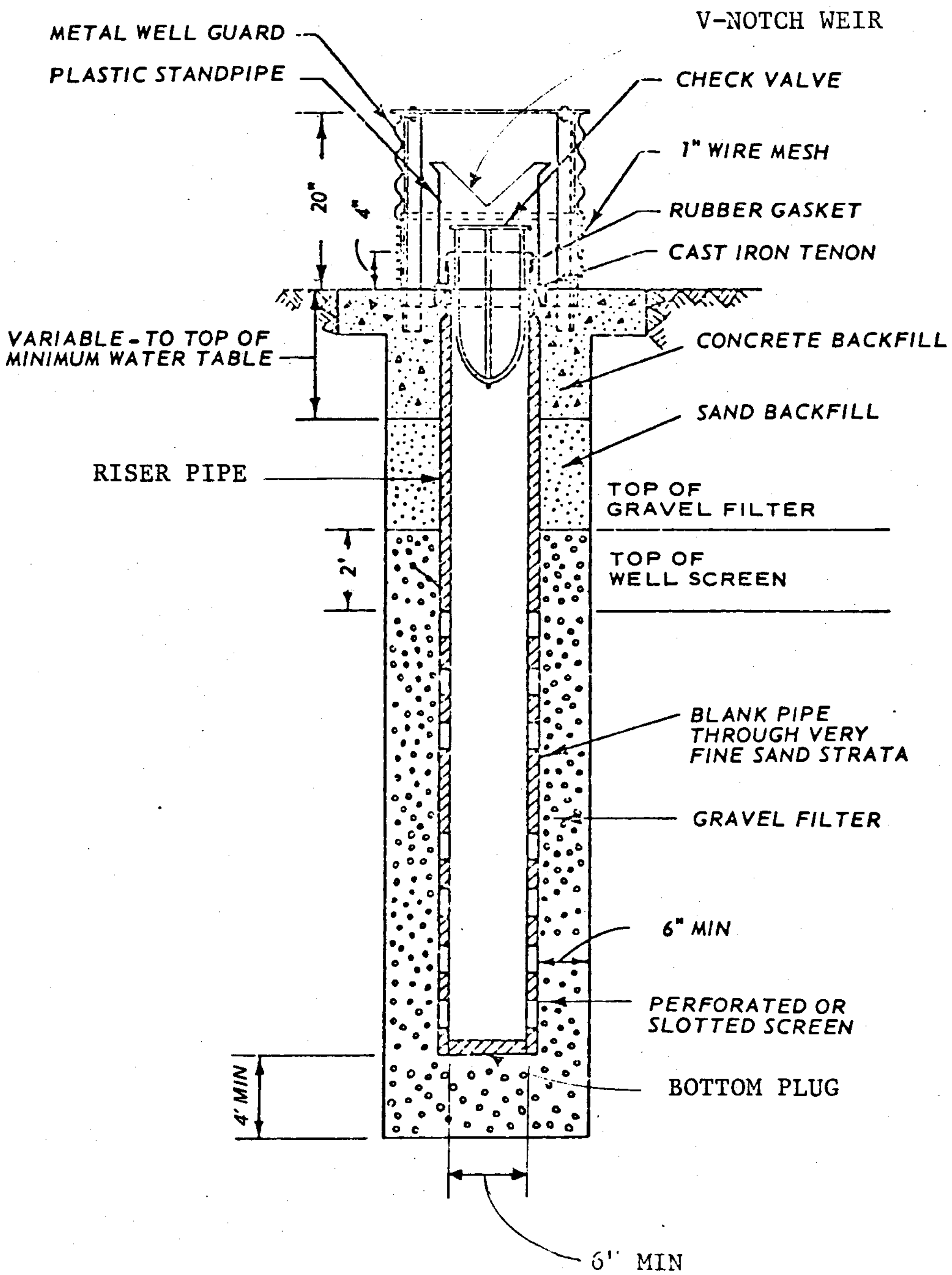


Figure 9-23. Typical relief well (after EM 1110-2-1913)

their tops are accessible for cleaning, sounding for sand, and pumping to determine discharge capacity. Relief wells should discharge into open ditches or into collector systems outside of the dam base which are independent of toe drains or surface drainage systems. Experience with relief wells indicates that with the passage of time the discharge of the wells will gradually decrease due to clogging of the well screen and/or reservoir siltation. A comprehensive study of the efficiency of relief wells along the Mississippi River levee showed that the specific yield of 24 test wells decreased 33 percent over a 15-year period. Incrustation on well screens and in gravel filters was believed to be the major cause (Montgomery 1972). Therefore, the amount of well screen area should be designed oversized and a piezometer system installed between the wells to measure the seepage pressure, and if necessary additional relief wells should be installed (EM 1110-2-2300, U. S. Army Engineer Waterways Experiment Station 1956a, Singh and Sharma 1976).

b. History of Use. The first use of relief wells to prevent excessive uplift pressures at a dam was by the U. S. Army Engineer District, Omaha, when 21 wells were installed from July 1942 to September 1943 as remedial seepage control at Fort Peck Dam, Montana. The foundation consisted of an impervious stratum of clay overlying pervious sand and gravel. Although a steel sheetpile cutoff was driven to shale, sufficient leakage occurred to develop high hydrostatic pressure at the downstream toe that produced a head of 45 ft above the natural ground surface. This uplift pressure was first observed in piezometers installed in the pervious foundation. The first surface evidence of the high hydrostatic pressure came in the form of discharge from an old well casing that had been left in place. Since it was important that the installation be made as quickly as possible, 4- and 6-in. well casings, available at the site, were slotted with a cutting torch and installed in the pervious stratum with solid (riser) pipe extending to the surface. Wells were first spaced on 250-ft centers and later intermediate wells were installed making the spacing 125 ft. The hydrostatic pressure at the downstream toe was reduced from 45 to 5 ft and the total flow from all wells averaged 10 cu ft per sec (U. S. Army Engineer District, Omaha 1982). The first use of relief wells in the original design of a dam was by the U. S. Army Engineer District, Vicksburg, when wells were installed during construction of Arkabutla Dam, Mississippi, completed in June 1943. The foundation consisted of approximately 30 ft of impervious loess underlain by a pervious stratum of sand and gravel. The relief wells were installed to provide an added measure of safety with respect to uplift and piping along the downstream toe of the embankment. The relief wells consisted of 2-in. brass wellpoint screens 15 ft long attached to 2-in. galvanized wrought iron riser pipes spaced at 25-ft intervals located along a line 100 ft upstream of the downstream toe of the dam. The top of the well screens was installed about 10 ft below the bottom of the impervious top stratum. The well efficiency decreased over a 12-year period to about 25 percent primarily as a result of clogging of the wells by influx of foundation materials into the screens and/or the development of corrosion or incrustation. However, the piezometric head along the downstream toe of the dam, including observations made at a time when the spillway was in operation, has not been more than 1 ft above the ground surface except at sta 190+00 where a maximum excess hydrostatic head of 9 ft was observed (U. S. Army Engineer Waterways Experiment Station 1958). Since these early installations, relief wells have been used at many dams to prevent excessive uplift pressures and piping through the foundation.

c. Design Considerations.

(1) General. The factors to be considered in determining the need for and designing a relief well system include characteristics of the landside top stratum; permeability, stratification, and depth of the pervious foundation in which. seepage is to be controlled; net head acting on the dam; dimensions of the relief well system being considered; allowable factor of safety with respect to uplift at the downstream toe of the dam; and allowable rate of discharge through the pervious foundation. Some factors, like the net head acting on the dam, can be determined with good accuracy. Other factors like permeability and stratification are more difficult to assess. The design of the relief wells should be based on the best estimate of permeability values and then subjected to a sensitivity analysis using several values of permeability to ensure that the adopted design is adequate to intercept seepage and lower uplift pressures to the required extent allowing for the likelihood that the values of permeability used in design lake precision (Kaufman 1976). The area between the dam abutments is divided into reaches where geologic and soil conditions are assumed uniform within the reach (see figure 9-24). Generally, the design procedure for relief wells consists of determining the head which would exist along the downstream toe of the dam without relief wells, comparing this head to that desired for a given factor of safety, and designing a relief well system to reduce the head to the desired value. There is no unique solution because there is an infinite number of well systems (radius, penetration, spacing, etc.) which reduce the head to the given value. The objective is to select one which is economical, has reasonable dimensions, and can produce the desired results. Usually the designer selects the radius and penetration and then determine the required spacing of the well system. This becomes an iterative procedure wherein the designer assumes a value of well spacing, computes the head between wells and repeats this for several trial spacings until a spacing is found that produces the desired head along the downstream toe of the dam. The cost of the well system is determined and then a design can be prepared for a different penetration to determine if some economy can be achieved by changing the penetration of the system. Fully penetrating relief wells are often used in aquifers up to about 75 ft thick. For larger depths of pervious strata, it is usually more economical to have well systems with 50 percent or greater penetration at closer spacing. The equations for relief well design depend upon the values of the source of seepage and seepage exit length as shown in figure 9-25. The source of seepage is assumed to be a line source parallel to the well system and the dam axis. The location of the source of seepage depends upon the thickness and vertical permeability of any natural top stratum upstream of the dam and any impervious blanket constructed upstream of the dam, the permeability and thickness of the pervious foundation, and the presence of any borrow pits and/or major erosion features which reduce the thickness of the top stratum (see procedure to evaluate the source of seepage given in U. S. Army Engineer Waterways Experiment Station 1956a). The value of the seepage exit (X_3 in figure 9-25) depends upon the thickness and permeability of the top stratum downstream from the toe of the dam, the thickness and permeability of the pervious substratum,

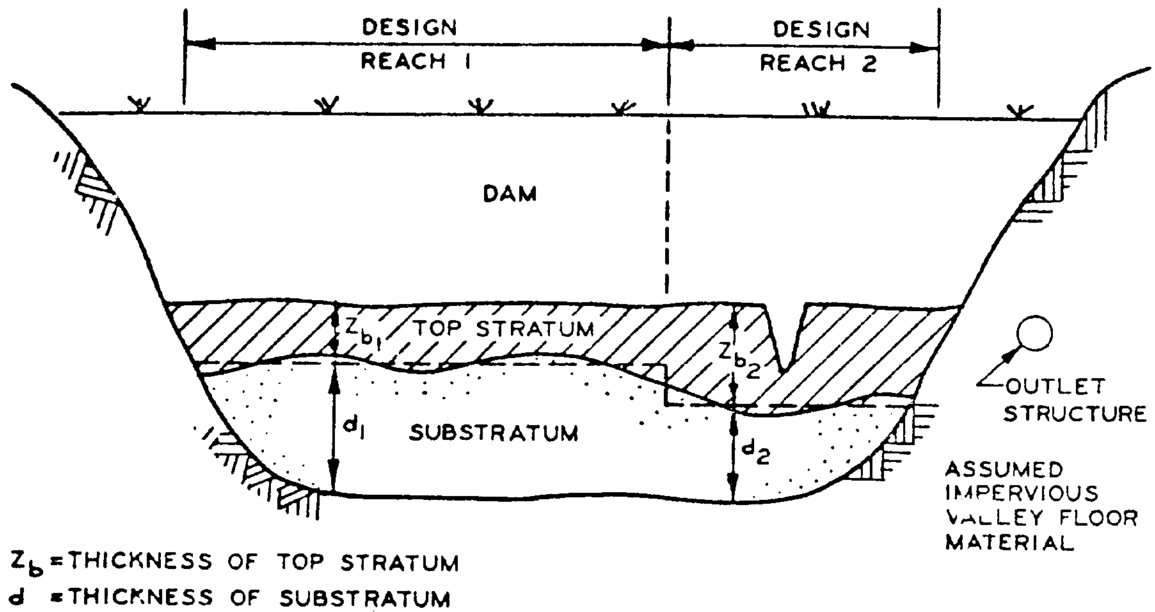


Figure 9-24. Profile of typical design reaches for relief well analysis (prepared by WES)

and the presence of any geologic features and/or man-made features which would result in an open or blocked seepage exit. The procedure for computation of the seepage exit distance, rate of discharge through the pervious foundation per unit length of dam, and pressure head without relief wells is given in figure 9-26. Generally relief wells have diameters of 6 to 18 in. and screen lengths of 20 to 100 ft, depending on the requirements. Some types of screens used for wells are slotted or perforated steel pipe, perforated steel pipe wrapped with steel wire, slotted wood stave pipe, and slotted plastic pipe. Riser pipe usually consist of the same material as the screen but does not contain slots or perforations. The open area of a well screen should be sufficiently large to maintain a low entrance velocity ($\leq$ 0.1 ft per sec) at the design flow in order to minimize head losses across the screen and reduce the incrustation and corrosion rates. The entrance velocity is calculated by dividing the expected or desired yield of the relief well by the area of openings in the screen (Johnson Division, Universal Oil Products Co. 1972). Filter packs around relief wells are usually 6 to 8 in. thick and must meet the criteria specified in Appendix D. Head losses within the relief well system consist of entrance head loss, friction head loss in the screen and riser pipes, and velocity head loss as shown in figure 9-27. The effective well radius is that radius which would exist if there were no hydraulic head loss into the well. For a well without a filter, the effective well radius is one-half the outside diameter of the well screen. Where a filter has been placed around the well, the effective well radius is the outside radius of the well screen plus one-half of the thickness of the filter.

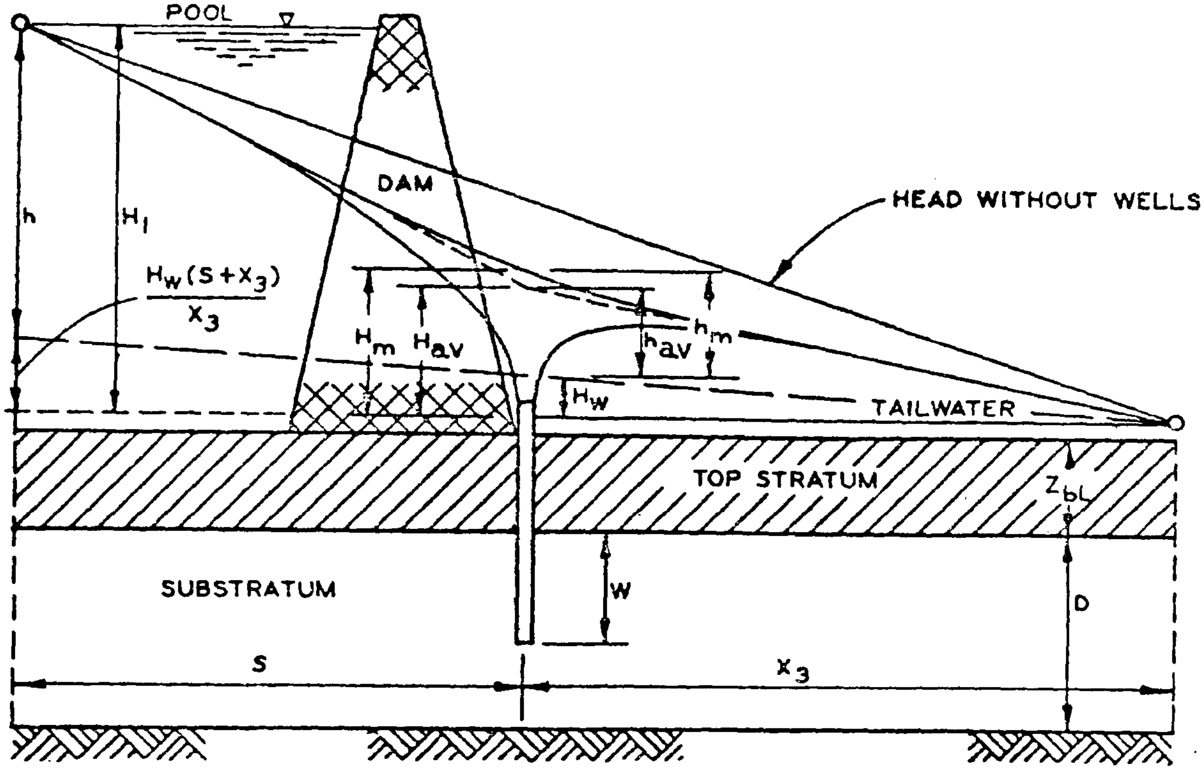


Figure 9-25. Design of relief wells (prepared by WES)

(2) Effective Well Penetration. In a stratified foundation, the effective well penetration usually differs from that computed from the ratio of the length of well screen to the total thickness of the aquifer. The procedure for determining the required length of well screen to achieve an effective penetration in a stratified aquifer is as follows. Each stratum of the pervious foundation is first transformed into an isotropic layer (Leonards 1962)

$$d = d'\sqrt{\frac{k_H}{k_V}} \qquad (9\text{-}24)$$

where

d = transformed layer thickness

d' = actual layer thickness

k_H = horizontal permeability of layer

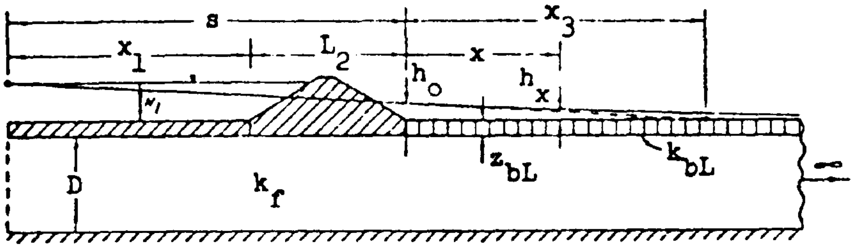
k_V = vertical permeability of layer

Seepage per unit length of dam....$Q_s = f k_f H_1 = k_f H_1 \dfrac{D}{s + x_3}$

Head beneath top stratum of downstream toe of dam............$h_o = H_1 \dfrac{x_3}{s + x_3}$

A factor........................ $c = \sqrt{\dfrac{k_{bL}}{k_f \cdot z_{bL} D}}$

CASE 1, $L_3 = \infty$

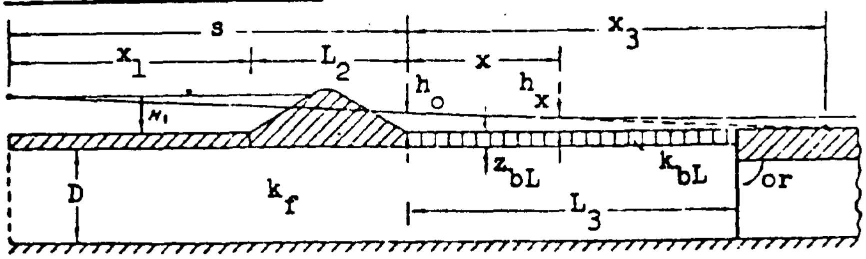


$$x_3 = \frac{1}{c}$$

$$h_x = h_o \, \varepsilon^{-cx}$$

$(\varepsilon = 2.718)$

CASE 2, BLOCKED EXIT

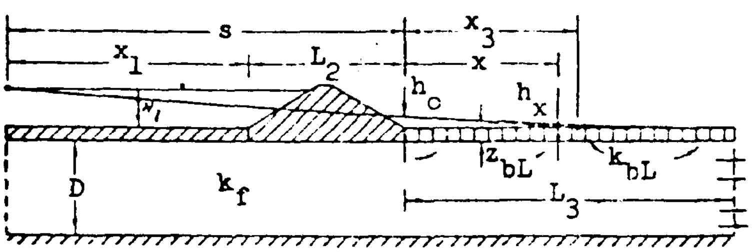


$$x_3 = \frac{1}{c \tanh(cL_3)}$$

$$h_x = h_o \frac{\cosh \cdot c(L_3 - x)}{\cosh(cL_3)}$$

$$h_{x=L_3} = \frac{h_o}{\cosh (cL_3)}$$

CASE 3, OPEN EXIT



$$x_3 = \frac{\tanh(cL_3)}{c}$$

$$h_x = h_o \frac{\sinh c(L_3 - x)}{\sinh (cL_3)}$$

$$h_{x=L_3} = 0$$

NOTE: x_1 can be computed from formulas for x_3 by inserting the length of upstream blanket L_1 for L_3 in the appropriate expression when upstream conditions are similar to the above downstream conditions.

Figure 9-26. Computation of rate of discharge and pressure heads for semi-pervious downstream top stratum and no relief wells (from U. S. Army Engineer Waterways Experiment Station[120])

TOTAL HYDRAULIC HEAD LOSS IN A WELL (H_w) IS

$$H_w = H_e + H_s + H_r + H_v$$

WHERE H_e = ENTRANCE HEAD LOSS (SCREEN AND FILTER) ESTIMATE FROM CURVE a.

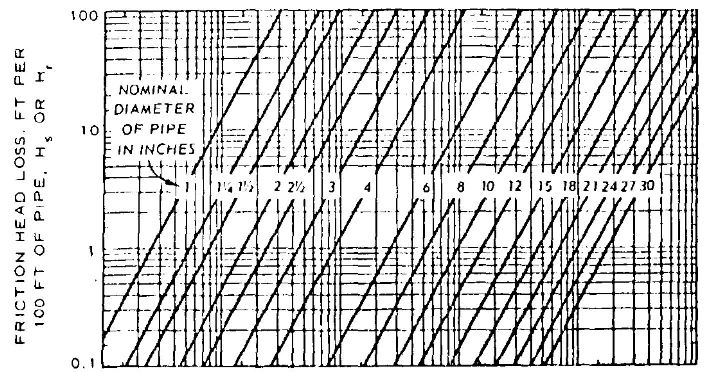
H_s = HEAD LOSS IN SCREENED SECTION OF WELL: ESTIMATE FROM CURVE b FOR A DISTANCE OF ONE-HALF THE SCREEN LENGTH.

H_r = HEAD LOSS WITHIN THE RISER AND CONNECTIONS. ESTIMATE FROM CURVE b.

H_v = VELOCITY HEAD LOSS. ESTIMATE FROM CURVE c.

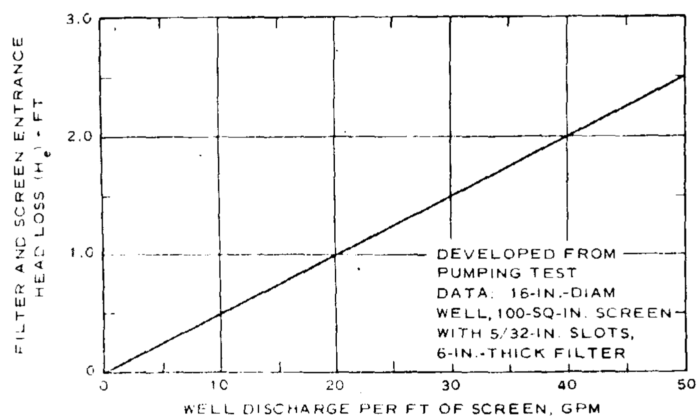
THE VALUE OF H_w MUST BE SUBTRACTED FROM THE COMPUTED VALUE OF h_w TO OBTAIN THE LIFT OR WATER LEVEL IN A WELL.

TYPE OF PIPE	C	$(100/C)^{1.85}$
STEEL (NEW)	125	0.67
STEEL (AVG CONDITION)	110	0.83
WOOD STAVE RISER	130	0.71
WOOD STAVE SCREEN	100	1.00
CORRUGATED METAL	70	1.92

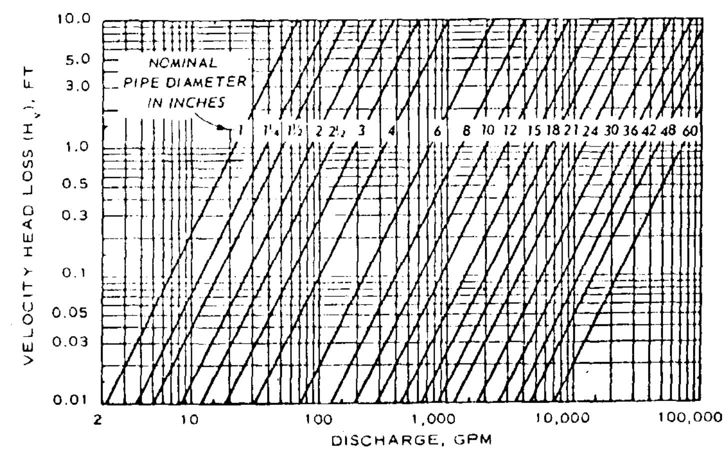


BASED ON HAZEN-WILLIAMS EQUATION WITH C = 100; MULTIPLY LOSSES BY $(100/C)^{1.85}$ FOR VALUES OF C OTHER THAN 100

(b)



(a)



(c)

Figure 9-27. Hydraulic head loss in relief wells (after Leonards[205])

The transformed permeability of each layer of the pervious foundation is

$$\bar{k} = \sqrt{k_H k_V} \tag{9-25}$$

where $\bar{k}$ is the transformed permeability of layer. The thickness of the transformed, homogeneous, isotropic pervious foundation is

$$D = \sqrt{\Sigma(d' k_H)\Sigma(d'/k_V)} \tag{9-26}$$

where D is the thickness of pervious foundation. The effective permeability of the transformed pervious foundation is

$$k = \sqrt{\frac{\Sigma(d' k_H)}{\Sigma(d'/k_V)}} \tag{9-27}$$

where k is the effective permeability of transformed pervious foundation. The effective well screen penetration into the transformed pervious foundation is

$$W = \frac{\sum_{o}^{\bar{w}} d' k_H}{k} \tag{9-28}$$

where

w = effective well screen penetration into transformed pervious foundation

$\bar{w}$ = actual well screen length

The percent penetration of the well screen into the transformed pervious foundation is

$$\frac{W}{D'}, \% = \frac{100 \sum_{o}^{\bar{w}} d' K_H}{kD} = \frac{100 \sum_{o}^{\bar{w}} d' K_H}{\sum_{o}^{\bar{D}} d' k_H} \qquad (9\text{-}29)$$

where $\bar{D}$ is the actual pervious foundation thickness.

(3) Factor of Safety. The factor of safety against uplift or heaving at the downstream toe of the ham, based upon the critical gradient, is

$$F_h = \frac{i_{CR}}{i_o} = \frac{\gamma_{sub}/\gamma_w}{h_a/Z_{bL}} = \frac{\gamma_{sub}/\gamma_w}{\gamma_w \; h_a} \qquad (9\text{-}30)$$

where

F_h = factor of safety against uplift or heaving at the downstream toe of the dam

i_{CR} = critical upward hydraulic gradient under the top stratum at the downstream toe of the dam

i_O = allowable upward hydraulic gradient under the top stratum at the downstream toe of the dam

γ_{sub} = submerged unit weight of downstream top stratum soil

h_a = allowable pressure head under the top stratum at the downstream toe of the dam

Z_{bL} = thickness of downstream top stratum

γ_w = unit weight of water

The factor of safety against uplift or heaving at the downstream toe of the dam provided by the relief well system should be at least 1.5[1].

(4) Infinite and Finite Relief Well Systems. Formulas for the design of relief wells are based on the assumption that the flow is laminar,

[1] Relief wells should be designed to reduce the excess head to zero to prevent upward seepage from occurring beneath the downstream top stratum.

artesian, and continuous and that a steady-state condition exists. Relief well systems are considered to be infinite or finite in length. The term infinite is applied to a system of wells that conforms approximately to the following idealized conditions:

(a) The wells are equally spaced and identical in dimensions.

(b) The pervious foundation is of uniform depth and permeability along the entire length of the system.

(c) The effective source of seepage and the effective line of downstream exit are parallel to the line of wells.

(d) The boundaries at the ends of the relief wells are impervious, normal to the line of wells, and at a distance equal to one-half the well spacing beyond the end wells of the system.

If these conditions exist, the flow to each well and the pressure distribution around each well are uniform for all wells along the line. Therefore, there is no flow across planes centered between wells and normal to the line, hence no overall longitudinal component of flow exists anywhere in the system. The term infinite is applied to such a system because it may be analyzed mathematically by considering an infinite number of wells; the actual number of wells in the system may be from one to infinity. Normally, a line of relief wells below a dam extending entirely across a valley and terminating at relatively impervious valley walls should be designed as an infinite line. A finite system of wells in any system that does not approximate the idealized condition for the infinite system. Whenever a major and abrupt change in the character of the system such as penetration or well spacing might result in an appreciable component of flow parallel to the line of wells, the use of design procedures for finite systems will be used (see U. S. Army Corps of Engineers 1963).

(5) Drawdown to Infinite Line of Fully Penetrating Relief Wells with Impervious Top Stratum. Where the flow to an infinite line of fully penetrating relief wells is from an infinite line source and the top stratum is assumed to be completely impervious,[1] as shown in figure 9-28. The drawdown produced by an equivalent continuous slot is

$$H - h_e = \frac{Q_w L}{kDa} \tag{9-31}$$

[1] Also applicable-when the top stratum is semipervious provided the well system is located in a drainage ditch and the head is kept below the ground surface on the downstream side of the dam to prevent any seepage upward through the top stratum. Under these conditions, the downstream top stratum acts as if it is impervious.

where

$H - h_e$ = drawdown produced by flow from continuous slot

Q_w = discharge from equivalent continuous slot

L = distance from line source of seepage to wells

k = effective permeability of transformed pervious foundation

D = thickness of transformed pervious foundation

a = well spacing

However, an additional head occurs because of converging flow at the wells. This head loss is a function of well flow, well spacing and penetration, well radius, and thickness and permeability of the pervious foundation. For fully penetrating wells

$$\Delta h_w = \frac{Q_w}{2\pi kD} \ln \frac{a}{2\pi r_w} \tag{9-32}$$

where

Δh_w = head loss at well due to converging flow (see figure 9-28)

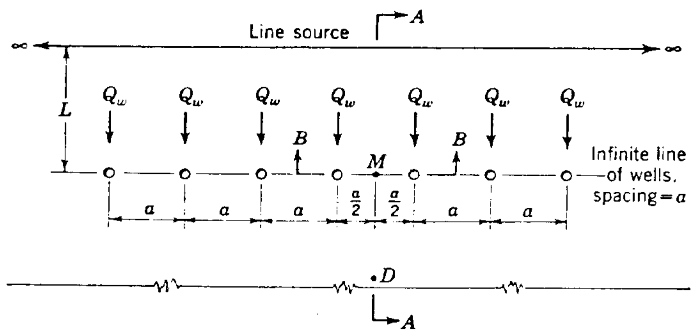
r_w = effective radius of well (outside radius of well screen plus one-half of the thickness of the filter)

The total drawdown at the well, neglecting hydraulic head losses in the well, is that at the slot plus that due to the well

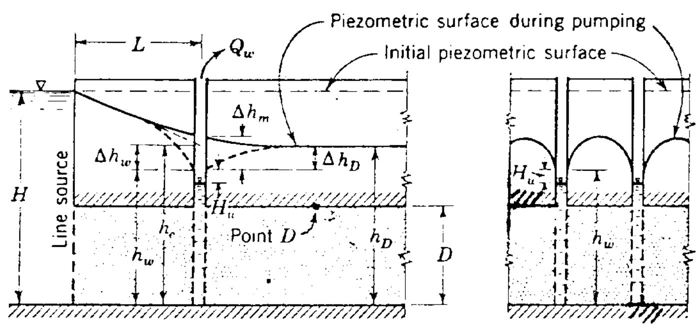
$$H - h_w = H - h_e + \Delta h_w \tag{9-33}$$

Substituting equations 9-31 and 9-32 into equation 9-33

$$H - h_e = \frac{Q_w L}{kDa} + \frac{Q_w}{2\pi kD} \ln \frac{a}{2\pi r_w} \tag{9-34}$$



a. Plan view of wells



b. Section A-A

c. Section B-B

Figure 9-28. Flow to an infinite line of fully penetrating relief wells from an infinite line source of seepage (after Leonards[205])

The head midway between wells will exceed the head at the well by

$$\Delta h_m = \frac{Q_w}{2\pi kD} \ln \frac{a}{2\pi r_w} \tag{9-35}$$

where Δh_m is the head increase midway between wells. The drawdown midway between wells is

$$H - h_m = \frac{Q_w}{kDa} - 0.11 \frac{Q_w}{kD} \tag{9-36}$$

At a distance downstream from the well system, the head will exceed that at the well by

$$\Delta h_D = \frac{Q_w}{2\pi kD} \ln \frac{a}{2\pi r_w} \tag{9-37}$$

where Δh_D is the head increase downstream of wells. The drawdown downstream of the wells is

$$H - h_D = \frac{h_D - h_w}{\frac{a}{2\pi L} \ln \frac{a}{2\pi r_w}} \tag{9-38}$$

(6) Drawdown to Infinite Line of Partially Penetrating Relief Wells with Impervious Top Stratum. For an infinite line of partially penetrating relief wells where the flow is from an infinite line source and the top stratum is assumed to be impervious (or semipervious as previously described) the head loss at the partially penetrating well due to converging flow is

$$\Delta h_w = \frac{Q_w \theta_{avg}}{kD} \tag{9-39}$$

where θ_{avg} is the average uplift factor (obtained from figure 9-29). The total drawdown at the partially penetrating well, neglecting hydraulic head losses in the well, is that at the slot plus that due to the well

$$H - h_w = \frac{Q_w}{kD}\left(\frac{L}{a} + \theta_{avg}\right) \tag{9-40}$$

The head midway between partially penetrating wells will exceed the head at the well by

$$\Delta h_m = \frac{Q_w \theta_m}{kD} \tag{9-41}$$

where

θ_m is the midpoint uplift factor (obtained from figure 9-29). The drawdown midway between partially penetrating wells is

$$H - h_m = \frac{Q_w}{kD}\left(\frac{L}{a} + \theta_{avg} - \theta_m\right) \tag{9-42}$$

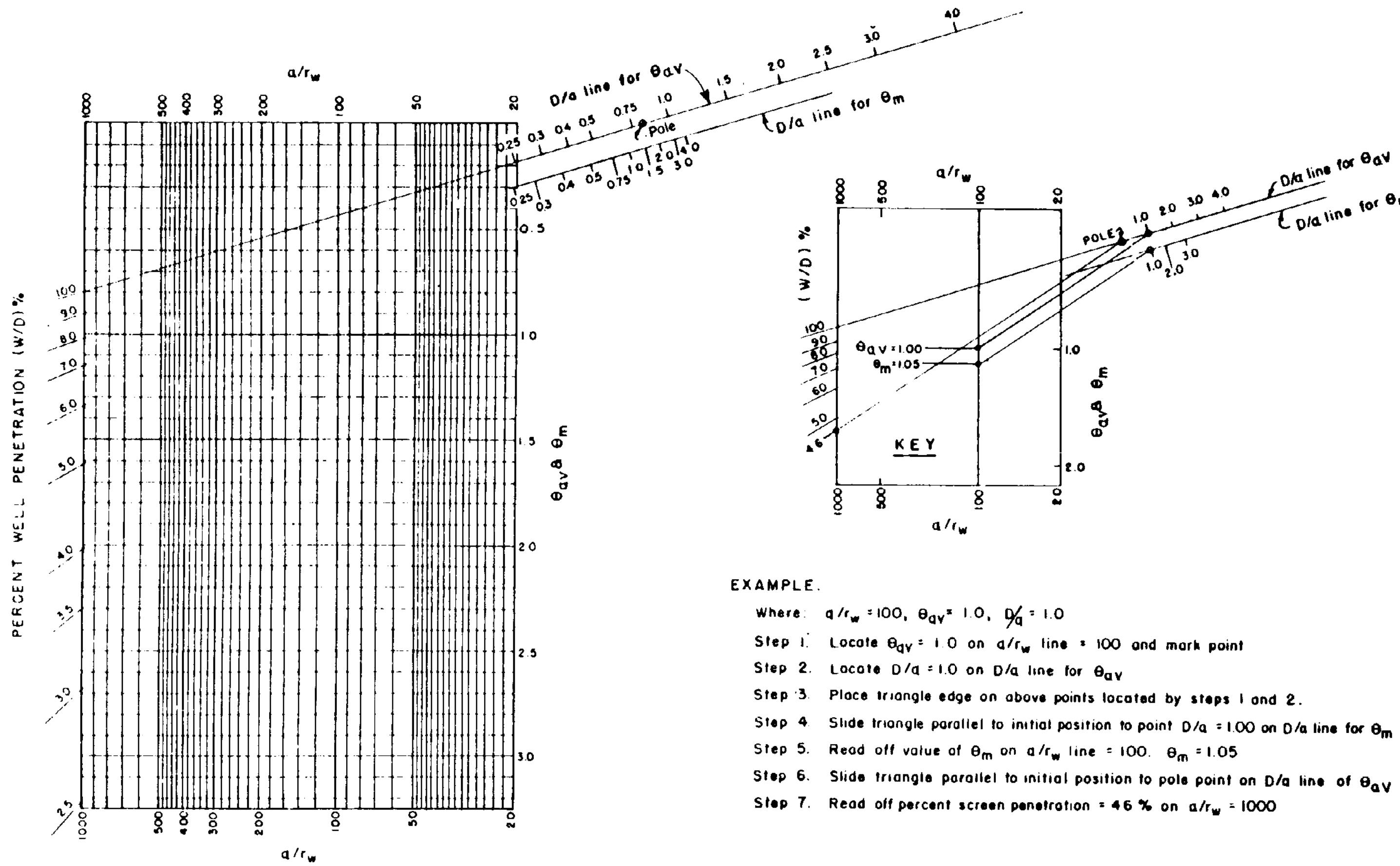


Figure 9-29. Nomograph for obtaining uplift factors for design of partially penetrating relief wells (from U.S. Army Engineer Waterways Experiment Station[120])

(7) Drawdown to Infinite Line of Relief Wells with Semipervious Top Stratum. Where the top stratum is semipervious, the need for relief wells is evaluated by determining the piezometric grade line without relief wells using blanket formulas given in figure 9-26. As stated previously, the factor of safety against uplift or heaving at the downstream toe of the dam, as determined from equation 9-30, should be at least 1.5. If relief wells are required, the spacing for an infinite line of relief wells for a given penetration is determined using a procedure of successive trials and the nomograph given in figure 9-29. The required well spacing is affected by hydraulic head losses in the well which are estimated from figure 9-27. The procedure for computing the well spacing is as follows:

(a) Compute the allowable pressure head under the top stratum at the downstream toe of the dam from

$$h_a = \frac{\gamma_{sub} Z_{bl}}{\gamma_w F_h} \tag{9-43}$$

where

h_a = allowable pressure head under the top stratum at the downstream toe of the dam

γ_{sub} = submerged unit weight of downstream top stratum soil

Z_{bl} = thickness of downstream top stratum

γ_w = unit weight of water

F_h = factor of safety against uplift or heaving at the downstream toe of the dam

(b) Assume that the net head in the plane of the wells equals the allowable pressure head under the top stratum at the downstream toe of the dam and compute the net seepage gradient toward the well line

$$\Delta M = \frac{h - H_{avg}}{S} - \frac{H_{avg}}{X_3} \tag{9-44}$$

where

ΔM = net seepage gradient toward the well line

h = net head acting on the dam

H_{avg} = net head in the plane of the wells

S = distance from line of relief wells to effective source of seepage entry (see procedure in U. S. Army Engineer Waterways Experiment Station 1956a)

X_3 = distance from line of relief wells to effective seepage exit (see procedure in figure 9-26)

Setting $H_{avg} = h_a$ in equation 9-44 gives

$$\Delta M = \frac{h - h_a}{S} - \frac{h_a}{X_3} \qquad (9\text{-}45)$$

(c) Assume a well spacing and compute the flow from a single well

$$Q_w = \frac{h k_f D}{\frac{S}{a} + \left(\frac{S + X_3}{X_3}\right)\theta_{avg}} = a\,\Delta M k_f D \qquad (9\text{-}46)$$

where

Q_w = flow from a single well

k_f = effective permeability of transformed pervious foundation

D = transformed thickness of pervious foundation

a = well spacing

θ_{avg} = average uplift factor (obtained from figure 9-29)

(d) Estimate the total hydraulic head loss in the well from figure 9-27.

(e) Compute the net average head in the plane of wells above the total head loss in the well including elevation head loss (see figure 9-25) from

$$h_{avg} = H_{avg} - H_w \qquad (9\text{-}47)$$

where

h_{avg} = net average head in the plane of wells above the total head loss in the well including elevation head loss

H_{avg} = net head in the plane of wells

H_w = total head loss in the well including elevation head loss

(f) Substitute values obtained from ΔM and h_{avg} from equation 9-45 and 9-47, respectively, and solve for the average uplift factor

$$\theta_{avg} = \frac{h_{avg}}{a\Delta M} \tag{9-48}$$

where θ_{avg} is the average uplift factor.

(g) Find θ_{avg} from figure 9-29 using the values of a used in equation 9-48 and the corresponding a/r_w and D/a values.

(h) The first trial well spacing is that of value a for which θ_{avg} from step (f) equals θ_{avg} from step (g).

(i) Find θ_m from figure 9-29 for the first trial well spacing and the corresponding values of a/r_w and D/a .

(j) If $\theta_{avg} > \theta_m$, repeat steps (c) to (i) using the first trial well spacing in lieu of the spacing originally used in step (c), and determine the second trial well spacing. This procedure should be repeated until relatively consistent values of a are obtained on two successive trials. Usually the second trial spacing is sufficiently accurate.

If in step (j), $\theta_{avg} > \theta_m$, a modified procedure is used for the second trial:

(k) Assume $H_m = h_a$ and compute Q_w from equation 9-46 using the value of AM obtained in step (b) and the first trial well spacing from step (h).

(1) Estimate H_w from Q_w of step (k) and figure 9-27.

(m) Compute the net head beneath the top stratum midway between the wells above the total head loss in the well including elevation head loss (see figure 9-25) from

$$h_m = H_m - H_w \tag{9-49}$$

where

h_m = net head beneath the top stratum midway between the wells above the total head loss in the well including elevation head loss

H_m = net head beneath the top stratum midway between the wells

H_w = total head loss in the well including elevation head loss

(n) Using θ_{avg} and θ_m from steps (h) and (i), respectively, compute h_{avg} from

$$h_{avg} = \frac{\theta_{avg}}{\theta m} \tag{9-50}$$

where h_{avg} is the net head in the plane of wells.

(o) Using H_w and h_{avg} from steps (1) and (n), respectively, and compute H_{avg} from

$$H_{avg} = H_w + h_{avg} \tag{9-51}$$

(p) Compute ΔM from equation 9-44 using H_{avg} from step (o).

(q) Using h_m and ΔM from steps (m) and (p), respectively, compute θ_m for various values of a from

$$\theta_m = \frac{h_m}{a\Delta M} \tag{9-52}$$

where θ_m is the midpoint uplift factor.

(r) Find θ_m from figure 9-29 for the values of a used in step (q) and the corresponding a/r_w and D/a values.

(s) The second trial well spacing is that value of a which θ_m from step (q) equals θ_m from step (r).

(t) Find θ_{avg} from figure 9-29 for the second trial well spacing and the corresponding values of a/r_w and D/a .

(u) Determine the third trial well spacing by repeating steps (k) to (t) using the second trial well spacing in lieu of the spacing originally assumed in step (k), and in step (n) using the values of θ_m and θ_{avg} from steps (s) and (t), respectively, instead of those from steps (h) and (i). This procedure should be repeated until relatively consistent values of a are obtained on two successive trials. Normally, the third trial is sufficiently accurate.

(8) Drawdown to Finite Line of Relief Wells. In a short, finite line of relief wells, the heads midway between wells exceed those for an infinite line of wells both at the center and near the ends of the well system as shown in figure 9-30. Note that the pressures between wells, or midpoint pressures, are lower at the center of the well system and gradually increase towards the end of the line. With an infinite line of wells, the heads midway between wells are constant along the entire length of the well line. Numerous well systems may be fairly short, and for these it will be necessary to reduce the well spacing computed for an infinite line of wells so that heads midway between wells will not be more than the allowable pressure head under the top stratum at the downstream toe of the dam. The ratio of the head midway wells at the center of finite well systems to the head between wells in an infinite line of wells, for various well spacings and seepage exit lengths, is given in figure 9-31. The spacing of relief wells in a finite line should be the same as that required in an infinite line of wells to reduce the head midway between wells to h_a divided by the ratio of H_{m_n}/H_{m_∞} from figure 9-31. In any finite line of wells of constant penetration and spacing, the head midway between wells near the ends of the system exceeds that at the center of the system. Thus at the ends of both short and long well systems, the relief wells should generally be made deeper to provide additional penetration of the pervious substratum so as to obtain the same head reduction as in the central part of the well line. The above-mentioned procedures for designing finite relief well systems, although approximate, are usually sufficient. More exact, but more complex, procedures are available (see U. S. Army Corps of Engineers 1963).

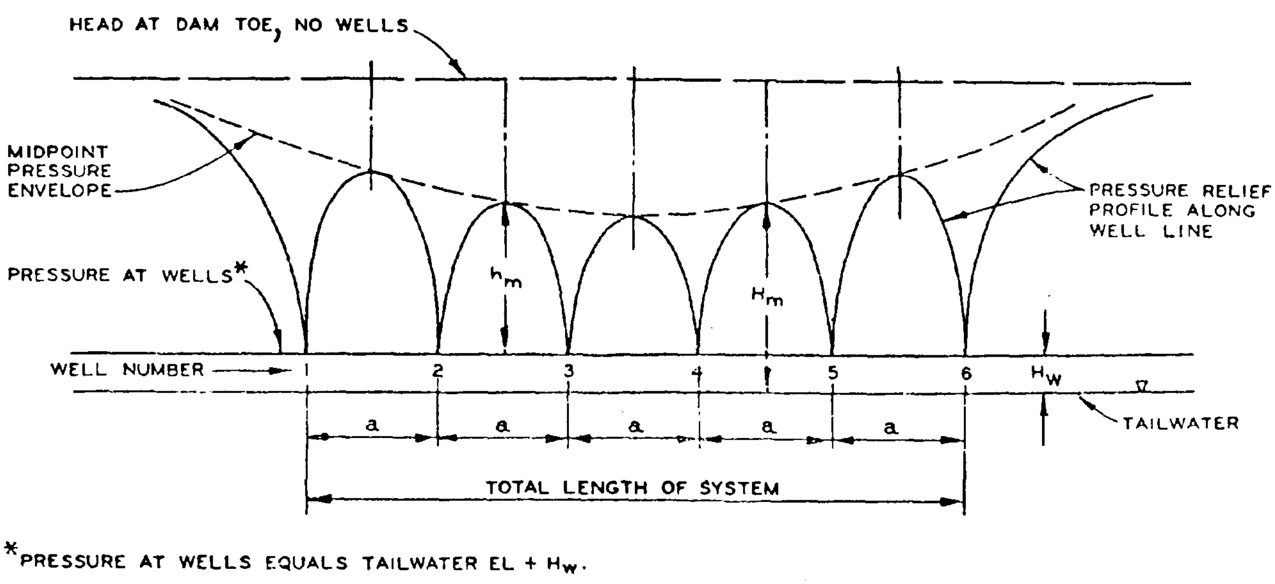


Figure 9-30. Variation of pressure relief along a finite line of relief wells (after EM 1110-2-1905)

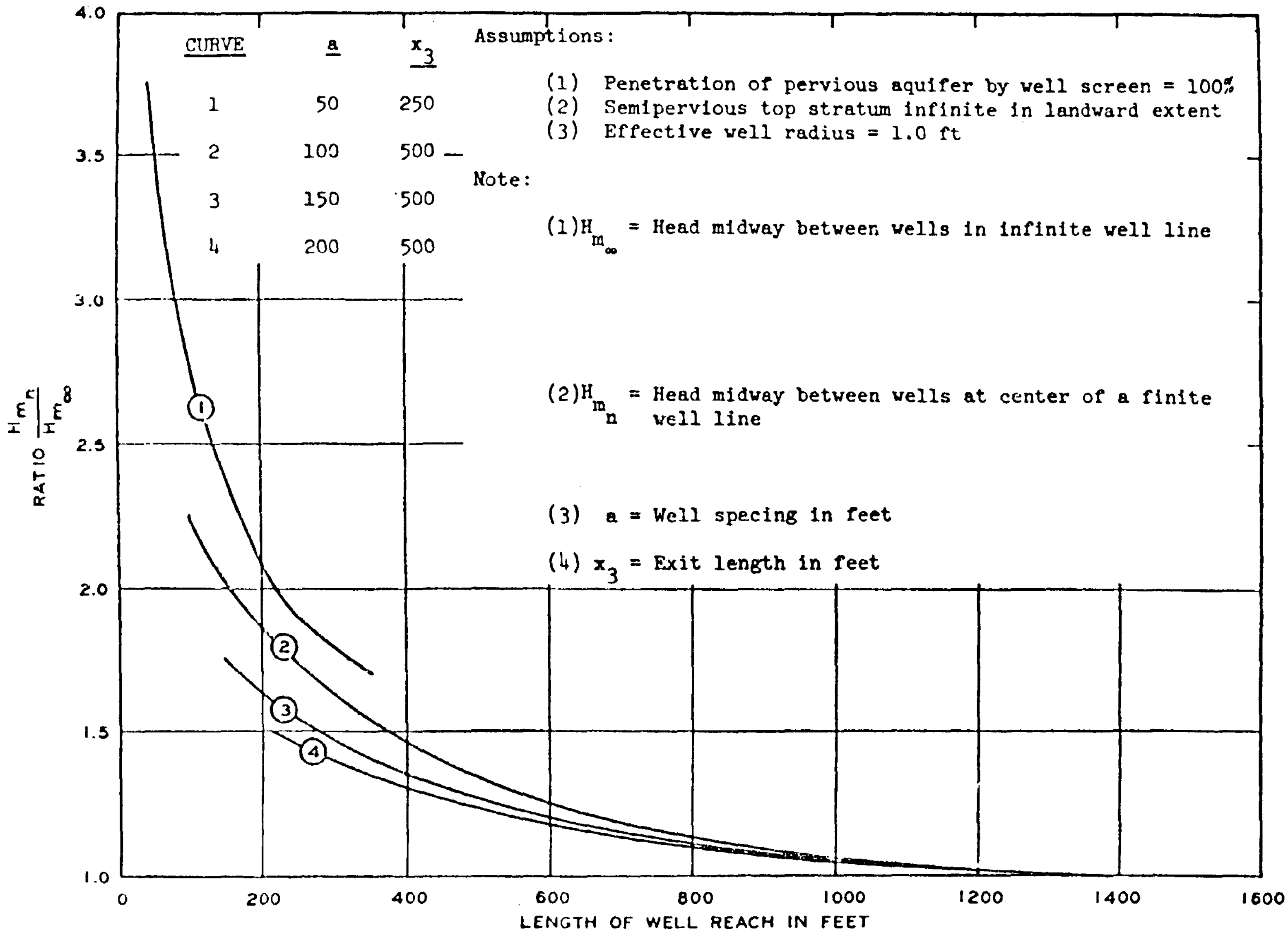


Figure 9-31. Ratio of head midway between relief wells at center of a finite well system to head midway between wells in an infinite system (from U.S. Army Engineer Waterways Experiment Station [120])

d. Installation. While the specific materials used in the construction of relief wells and methods of installation differ, relief wells are basically very similar. They consist of a boring to facilitate the installation, a screen or slotted pipe section to allow the entrance of ground water, a filter to prevent entrance and ultimate loss of foundation material, a riser pipe to conduct the water to the ground surface, a check valve to prevent backflooding and entrance of foreign material detrimental to the installation, backfill to prevent recharge of the formation by surface water, a bottom plug to prevent inflow of soil,[(1)] a V-notch weir at the top of the relief well to facilitate measurement of flow, and a cover and some type of barricade protection to prevent vandalism and damage to the top of the well by maintenance crews, livestock, etc. (see figure 9-23). Following development of the relief well, a pumping test should be conducted to determine the specific yield of the well and the amount of sand infiltration. Information from the pumping test is used to determine the acceptability of the well and for evaluating any changes in performance or loss of efficiency with time. Procedures for installation, development, and pumping tests are given in EM 1110-2-1913. A guide specification for relief wells is available.

e. Monitoring. As mentioned previously, the discharge of relief wells gradually decreases with time due to clogging of the well screen and/or reservoir siltation. Piezometers should be installed between relief wells to determine the seepage pressure in the main pervious strata. Relief wells should be sounded for sand and pumped to determine their discharge capacity under varying reservoir levels (see Chapter 13). A trend toward fall in relief well discharge accompanied by a fall in piezometric levels indicates a decrease in underseepage due to reservoir siltation and is favorable. However, a decrease in relief well discharge accompanied by a rise in piezometric levels indicates clogging of the relief wells and immediate rehabilitation and/or replacement of the wells or installation of additional wells is required (Singh and Sharma 1976). The operation, maintenance, and rehabilitation of relief wells is discussed in Chapter 14.

9-8. Trench Drain.

a. Introduction. When a complete cutoff is not required or is too costly, a trench drain may be used in conjunction with other underseepage control measures (upstream impervious blanket and/or relief wells) to control underseepage. A trench drain is a trench generally containing a perforated collector pipe and backfilled with filter material (see figure 9-32). Trench drains are applicable where the top stratum is thin and the pervious foundation is shallow so that the trench can penetrate into the aquifer. The existence of moderately impervious strata or even stratified fine sands between the bottom of the trench drain and the underlying main sand aquifer will render the trench drain ineffective. Where the pervious foundation is deep, a trench drain of practical depth would only attract a small portion of underseepage, and detrimental underseepage would bypass the drain and emerge downstream of the drain, thereby defeating its purpose. Trench drains may be used in

(1) For partially penetrating relief wells, the bottom plug should be such that future screen extension will be possible,

conjunction with relief well systems to collect seepage in the upper pervious foundation that the deeper relief wells do not drain. If the volume of seepage is sufficiently large, the trench drain is provided with a perforated pipe. A trench drain with a collector pipe also provides a means of measuring seepage quantities and of detecting the location of any excessive seepage (U. S. Army Engineer Waterways Experiment Station 1956a, EM 1110-2-1911, EM 1110-2-1913, and Cedergren 1977).

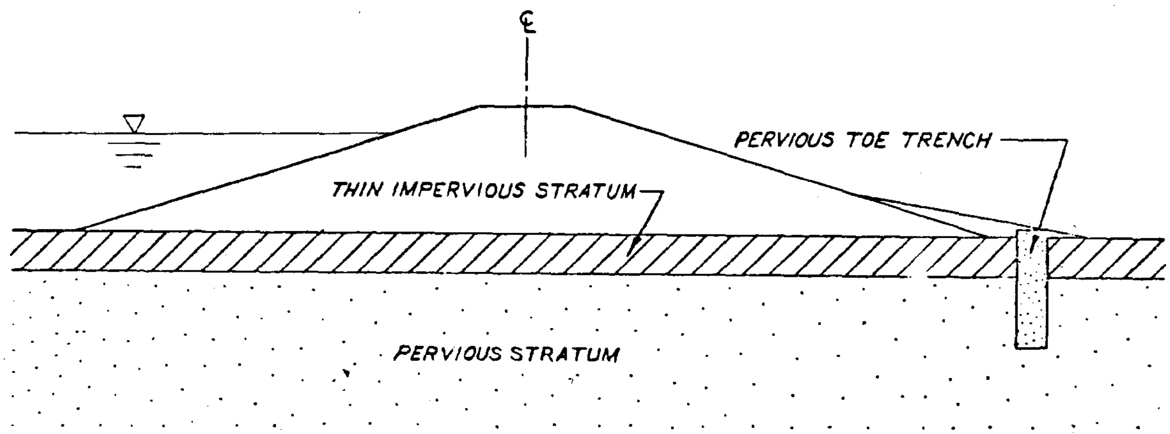
b. Location and Geometry. Trench drains are generally located at the downstream toe of the dam as shown in figures 9-32a and 9-32c, but are sometimes located beneath the downstream slope of the dam as shown in figure 9-32b. Trench geometry will depend on the volume of expected underseepage, desired reduction in uplift pressure, construction practicalities, and the stability of the material in which the trench is to be excavated. Trenches with widths as small as 2 to 6 ft have been used. However, narrow trench widths require special compaction equipment (EM 1110-2-1913).

c. Design Considerations. The maximum head at the base of an impervious top stratum downstream of a toe trench drain overlying a homogeneous, isotropic, pervious foundation may be computed from figure 9-33. The distance to the source of seepage may be evaluated using the procedure given in U. S. Army Engineer Waterways Experiment Station 1956a. If the pervious foundation is stratified, it is transformed into an isotropic layer, as described previously (see Equations 9-24 to 9-27) prior to using figure 9-33. The' factor of safety against uplift or heaving at downstream toe of the dam provided by the trench drain should be at least 1.5. If the downstream top stratum is semipervious, seepage into the trench, and the maximum head landward of the trench, will be somewhat less than that computed from figure 9-33 giving a slightly conservative design. When there is no downstream top stratum, seepage flow into the trench can be estimated from a flow net analysis (U. S. Army Engineer Waterways Experiment Station 1956a).

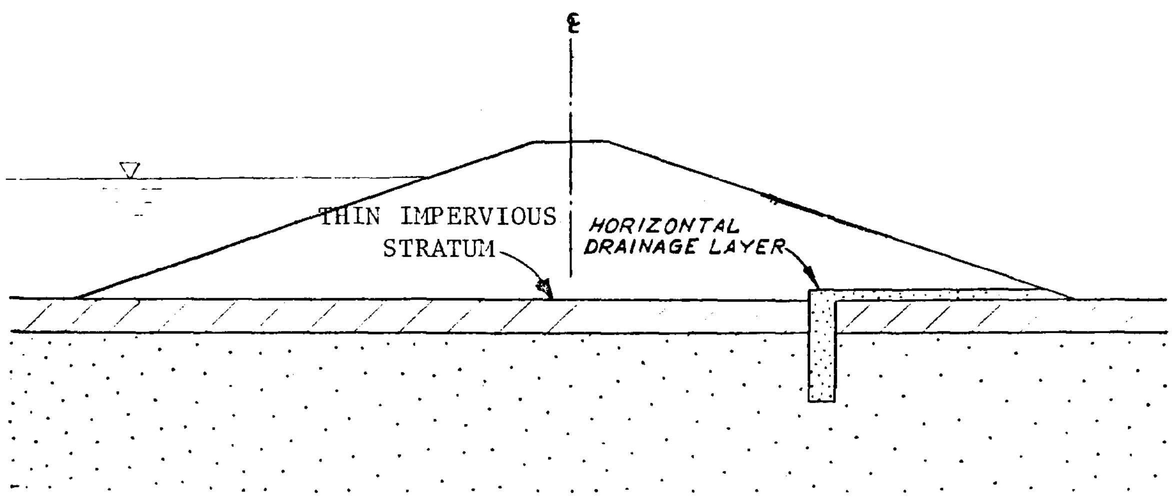
d. Construction. A trench drain usually contains a perforated pipe, surrounded by filter gravel, and backfilled with sand as shown in figure 9-34. Materials in trench drain must satisfy the filter gradation criteria given in Appendix D. As filter materials are placed, they must be protected from contamination resulting from inwash that might occur during a rainfall. The same control procedures are used for trench drains as those used in construction of pervious fill in the main embankment (EM 1110-2-1911).

9-9. Concrete Galleries. Internal reinforced concrete galleries have been used in earth and rockfill dams built in Europe, for grouting drainage, and monitoring of behavior. Galleries have not been constructed in embankment dams built by the Corps of Engineers to date. Some possible benefits to be obtained from the use of galleries in earth and rockfill dams are as follows (Sherard et al. 1963):

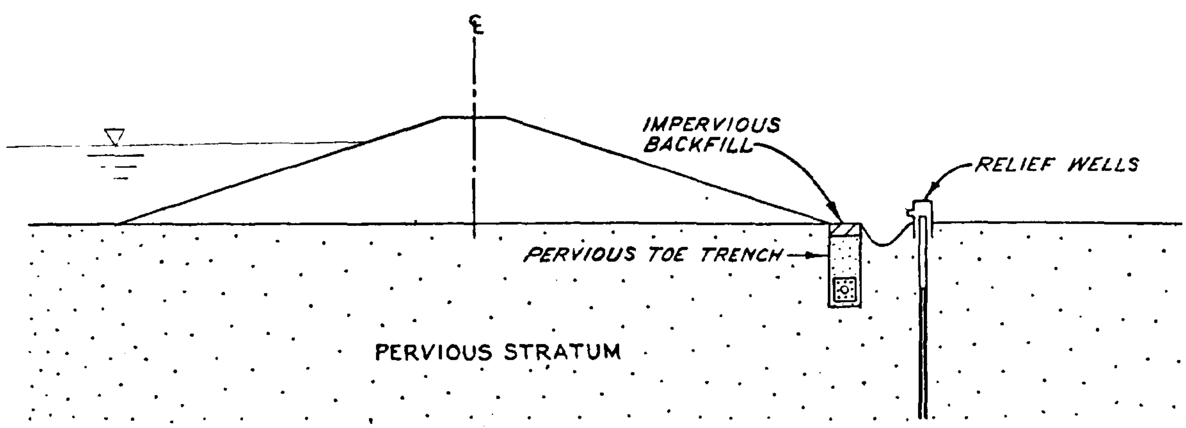
a. Construction of the embankment can be carried out independently of the grouting schedule.



a. Trench drain at downstream toe of dam



b. Trench drain under downstream slope of dam



c. Trench drain used in conjunction with relief wells

Figure 9-32. Trench drains to control underseepage (from EM 1110-2-1913)

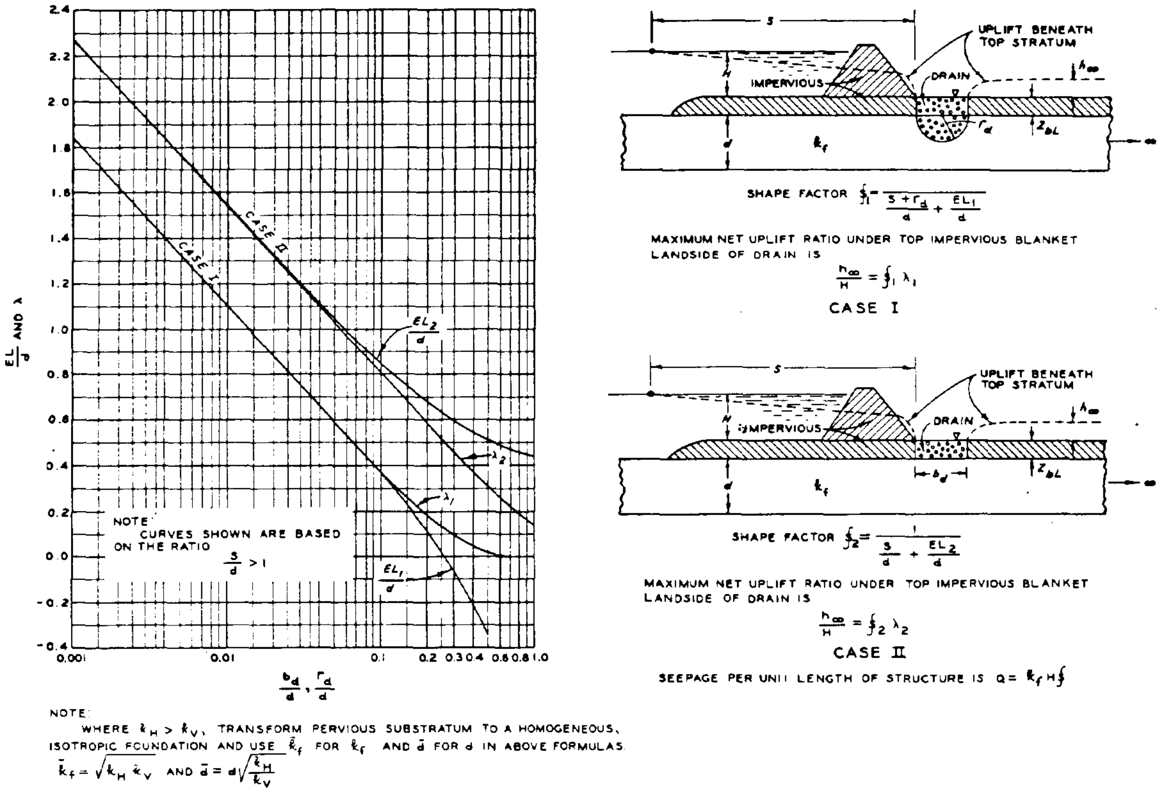


Figure 9-33. Design of toe trench drains for homogeneous, isotropic, pervious foundation, and for an impervious downstream top stratum (from U. S. Army Engineer Waterways Experiment Station[120])

b. Drain holes drilled in the rock foundation downstream from the grout curtain can be discharged into the gallery and observations of the quantities of seepage in these drain holes will indicate where foundation leaks are occurring.

c. Galleries provide access to the foundation during and after reservoir filling so that additional grouting or drainage can be installed, if required, and the results evaluated from direct observations.

d. The additional weight of the overlying embankment allows higher grout pressures to be used.

e. Galleries can be used to house embankment and foundation instrumentation outlets in a more convenient fashion than running them to the downstream toe of the dam.

f. If the gallery is constructed in the form of a tunnel below the rock surface along the longitudinal axis of the dam, it serves as an exploratory tunnel for the rock foundation.

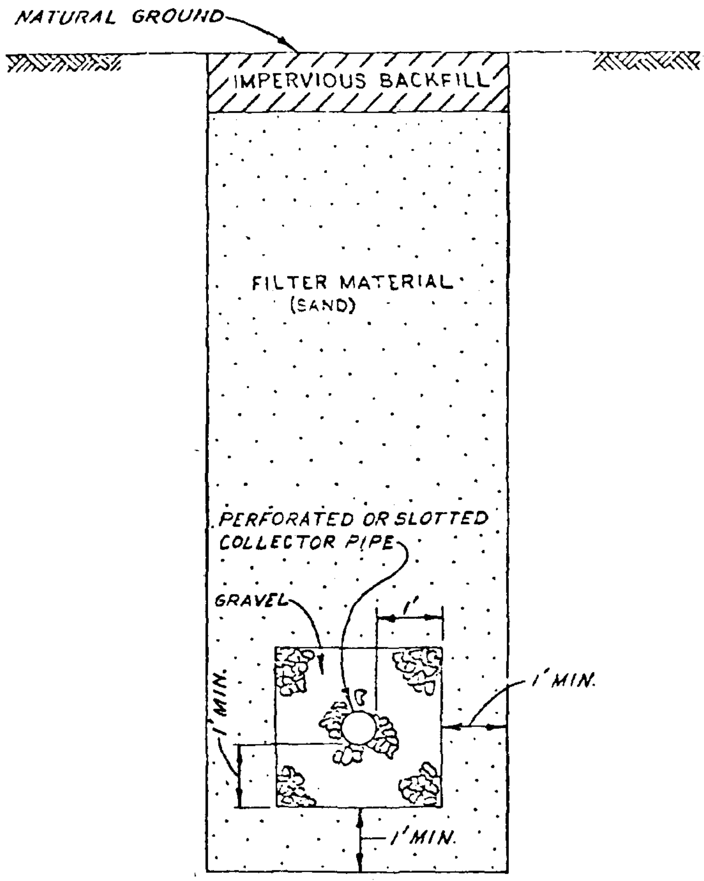


Figure 9-34. Trench drain with collector pipe (from EM 1110-2-1913)

The minimum size cross section recommended for galleries and access shafts is 8 by 8 ft to accommodate drilling and grouting equipment. A gutter located along the upstream wall of the gallery along the line of grout holes will carry away cuttings from the drilling operation and waste grout from the grouting operation. A gutter and system of weirs located along the downstream wall of the gallery will allow for determination of separate flow rates for foundation drains (EM 1110-2-3502, and Blind 1982).