CHAPTER 12
REMEDIAL SEEPAGE CONTROL

12-1. General Considerations. This chapter assumes that a seepage problem with an existing structure has been identified and defined by methods discussed in Chapter 13, or by other observations. The next step is to decide on a remedy, design and install the remedial measure, and monitor its performance to determine if the problem has been satisfactorily addressed. Several factors, including consequences of continued detrimental seepage, the geotechnical environment (embankment, foundation, abutment), and economy, will determine the type and degree of remedial seepage control. Some of the more critical consequences include:

a. Breaching of the embankment or loss of support to structural members due to piping.

b. Breaching of the embankment from slope instability induced by loss of material and/or strength due to seepage.

c. Loss of significant amounts of reservoir water.

d. Maintenance problems or loss of useful areas due to seepage on the downstream slope or areas downstream of the embankment.

12-2. Remedial Methods.

a. Factors Affecting Choice of Methods, Several methods of reducing undesirable seepage are discussed in this chapter; most have been previously addressed in Chapters 8-11, which described methods and appropriate settings for each. The remedial designer, while possibly having more advanced technology available than the original designer, must work with existing conditions. The embankment and its foundation, abutments, and seepage control measures may form a complicated structure through which seepage occurs. This can make precise detection and remedial control difficult or impossible. Remedial action may range from continued or additional monitoring to rebuilding or abandonment of the dam. Choice of remedial method(s) will depend on several factors, which include:

(1) Geotechnical environment.

(2) Risk.

(3) Degree of correction required.

(4) cost.

b. Effects of Methods on Other Structure Elements. The remedial designer must also consider the interplay of the remedial measures with other dam elements. For example:

(1) Effect of excavation for drains, cutoff trenches, slurry trenches, etc., on embankment stability.

(2) Difficulty of tying remedial measure to existing seepage control elements.

(3) Possibility of hydraulic fracturing when grouting.

c. Monitoring. In all cases, pre- and post-remedial monitoring of seepage is essential to determine the effectiveness of remedial action. Since Chapters 8-11 describe control measures in detail, this chapter will just point out primary considerations in the choice of remedial measures and give examples of their use. These examples are provided for general guidance only, since efficient use of remedial measures is very dependent upon geotechnical characteristics of the particular site's as-built configuration, reservoir uses, and pool history.

12-3. Storage Restriction. The most direct method to alleviate a seepage problem is to lower the reservoir and restrict pool levels in order to stop or reduce seepage and its effects. This is often done during problem identification. If piezometer and seepage quantity measurement devices are in place at this time, the effect of this remedy will be experimentally determined. Considerations in storage reduction include:

a. Reduction of downstream inundation area and level should breaching occur.

b. Effects of pool lowering on water supply, flood control, power generation, navigation, recreation, and environment.

Normally, lowering and restriction of the reservoir pool is not an acceptable long-term solution, but this depends on restriction levels and purpose of the reservoir. Care must be taken in lowering the reservoir since rapid drawdown can lead to instability of the upstream slope. Of course, risk of upstream slope failure would normally be a preferred alternative to breaching of the dam and release of a full reservoir.

12-4. Grouting. Grouting is a common, long-used remedy for seepage. Its effectiveness is dependent upon being able to rather specifically locate the leaking area and fill the culprit openings without damage to the embankment. Possible damage includes cracking of impermeable cores or other impermeable areas of the embankment, foundation, or abutments, and clogging of drains. If grouting results in sealing of the foundation just downstream of or beneath the downstream portion of the dam, uplift pressures may increase beneath the embankment or seepage may be forced up into the downstream portion of the embankment. Pore pressure instrumentation should be in place to monitor such changes before grouting begins. This must be considered in design of remedial controls. Because of the many variables in grouting, it is highly desirable to have an experienced contractor and field engineer. In many cases, post-grout drilling may be warranted to determine if the grout has thoroughly penetrated the desired area. Information about grout properties and grouting is given in Chapters 9 and 11. Several case histories follow which provide general examples.

a. 95-ft-High Earthfill Dam (Ley 1974). Upon initial filling, an earthfill dam with a foundation and abutments of volcanic tuffs and breccias exhibited leakage at one of the downstream embankment-abutment contacts and out onto the downstream slope. Inspection revealed leakage from open fractures in the volcanic rock and drains were installed in the areas of seepage. The seepage was stable for a number of years. Subsequent evaluation of the embankment for seismic safety resulted in a need to reduce foundation seepage quantities and piezometric levels within the embankment. A grouting program employed a low viscosity chemical grout in order to penetrate any permeable layers in the embankment where seepage might be occurring. Grout holes were split-spaced for 120 ft along the dam crest from the left abutment. If significant circulation water was lost during drilling, the hole was grouted. Initial spacing was 12 ft with 14 of 23 holes taking low pressure grout (0-5 psi at the collar of the hole). This low pressure was to prevent embankment heave. Most of the take was well into the left abutment with holes spaced as close as 2-1/2 ft and being deepened in stages and further grouted. Gel time varied from 2 to 18 minutes and final depth of holes varied from 24 to 84 ft. Total take was 3,200 gal with seepage being reduced 90 percent, but with little reduction in piezometric levels. Grouting can reduce seepage quantities significantly but still not alleviate high piezometric pressures, particularly in tight or fine-grained materials since any continuous void or pore space can transmit upstream heads.

b. 140-ft-High Earth Dam (Ley 1974). In the left abutment, gypsum had apparently formed in the bedding planes and fractures of folded and faulted shale and siltstone. After water was impounded, leakage, carrying dissolved gypsum, occurred from the abutment. Settlement and gradual increase in seepage also indicated that gypsum was being removed from the formation. Built in 1915, the dam underwent a grouting program from 1930-1933, resulting in placement of about 35,000 cu ft of grout in a series of holes along the dam crest, the left abutment, and at the bottom of the hill forming the left abutment. This program reduced seepage quantities by 75 percent. Approximately 30 years later, over 32,000 cu ft of cement-bentonite grout (colored with iron oxide to distinguish from previously placed grout) was placed in 137 holes to again reduce seepage and replace material removed by solution. Cores indicated good penetration with most seams from hairline to 1/8 in. thick. Seepage was greatly reduced. Other geologic materials may also be dissolved when subjected to seepage. In a similar manner, silt and clay in limestone cavities may also be removed by seepage. Grouting may only be a temporary solution to a seepage problem if solution of a soluble foundation continues after grouting.

c. 70-ft-High Earthfill Dam (Ley 1974). Seepage of 130-140 gal/minute was discovered downstream and attributed to foundation leakage. Installation of drains downstream of the dam allowed collection, metering, and return of water to the reservoir. Drilling for a grouting program, undertaken some years later to reduce seepage losses, revealed loose, sugarlike, decomposed granite 30-40 ft below the dam foundation. The grouting was unsuccessful in reducing seepage. Subsequently, the bottom and right side of the reservoir were covered with an impervious blanket of 40 tons of bentonite mixed with native material. After mixing of the bentonite with native soil to a depth of 3 in., the surface was rolled with a rubber-tired roller. Surface drainage provisions prevented runoff from eroding the blanket during partial pool. Seepage, after blanketing, decreased 50 percent. In this case, attacking the seepage problem further upstream (at the reservoir) proved more efficient than trying to seal an underlying seepage path.

d. Fontenelle Dam (Gebhart 1974). A 165-ft zoned embankment, Fontenelle Dam, almost failed when a leak of up to 20 cu ft/second developed at the downstream contact with the right abutment. Much of the embankment was eroded before drawdown was effective in stabilizing the embankment. Fortunately, outlet capacity allowed lowering the reservoir 3-4 ft per day. The source of leakage was not specifically determined, but an extensive grouting of foundation rock (calcareous sandstone, siltstone, and carbonaceous shale) was successful in preventing a recurrence of the problem. A 90- by 140-ft cement grout blanket was placed upstream from the original grout cap. Grout curtains were extended beneath the dam beyond the abutments. Over 200,000 cu ft of grout was used.

e. Hills Creek Dam (Jenkins and Bankofier 1972). Hills Creek Dam, constructed by the Portland District, has a maximum height of 338 ft and consists of a central impervious core with gravel and rock shells. Minor seepage occurred near the left abutment during first filling, but decreased with time. Seepage markedly increased in extent and volume after 6 years of normal operation. Vertical drains placed in the downstream shell as an initial remedial measure were not effective in lowering water levels in the downstream shell and seepage continued to increase. An investigation to determine the seepage source continued during the remedial action. Initially it was thought that leakage was through the upstream blanket into the foundation and abutment, but further observations indicated flow was through the core or core-foundation contact. Grouting , which injected 4,500 sacks of cement, most in a 1-1/2:1 mix at zero psi, resulted in elimination of almost all seepage. Four 42-in. bucket auger holes, as well as several smaller borings, were drilled to inspect grouting of the core and foundation. The main source of seepage was at a point along the core foundation contact where a haul road had crossed the abutment. Twelve years later, seepage is still negligible. Frequently, the source of seepage is not obvious. The engineer must consider all possibilities and, after choosing and installing a remedial measure, try to understand what post-remedial monitoring is indicating. The extent of the engineer's knowledge of the foundation, embankment materials, and construction history will greatly influence the accuracy of his analysis of the seepage problem. Often available foundation and construction information will not be adequate and further geotechnical investigation will be required.

12-5. Upstream Impervious Blanket.

a. If it is determined that sealing of the reservoir bottom and sides immediately upstream of the embankment will be useful in reducing undesirable seepage quantities and pressures beneath the embankment, an upstream impervious blanket may be employed. If successful and economically feasible, this is one of the most efficient measures since the source of water, the reservoir, is controlled upstream of the embankment and its foundation. This generally requires removal of reservoir water, though some small reservoirs have been sealed by placement of materials through water. Sources of fine-grained material and, in some cases, filter materials are required. The impervious materials are usually placed on the reservoir bottom. If sloped areas such as the reservoir sides of upstream embankment slope are to be sealed, consideration must be given to protection against wave attack and erosion from runoff. Additionally, fine-grained materials placed on the upstream embankment slope may be removed during drawdown because of low saturated strength and high saturated weight. If seepage can also go through the upstream portion of the embankment and then into the foundation an upstream blanket will be less effective and another remedy may be necessary, e.g., cutoff beneath dam, figure 12-1. The nature of reservoir bottom materials must be considered. Any large voids must be filled with a stable material such as compacted soil,

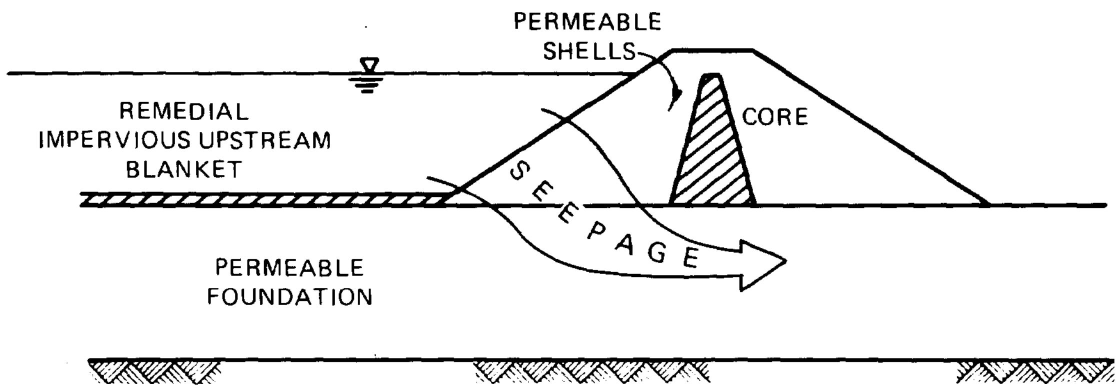

Figure 12-1. Possible problem if existing and remedial seepage control measures are not properly coordinated (prepared by WES)

stabilized soil, concrete etc. High gradients will likely exist through the blanket during high reservoir levels, particularly close to the embankment. It may be necessary to place a filter material before placing the blanket to prevent piping of the blanket material into the foundation. The extent of the blanket is determined by analysis and will depend on several factors, including extent of desired decrease in seepage quantities and pressures and blanket material available (quantity and permeability) (EM 1110-2-1913 and Barron 1977). Man-made liners have provided a seal for reservoirs with pervious foundations when fine-grained materials were not economically available. They are usually rather expensive, require relatively smooth surface for placement, and coverings (normally soil) to protect them from puncture in stressed areas and deteriorating exposure to sunlight. Joining of sections is one of the most critical and difficult aspects of man-made liners. Field seams, especially under difficult field conditions and with other than highly experienced personnel, can be an appreciable source of leakage. Quality control of seaming should be strict. One example of the use of an impervious upstream blanket was given in paragraph 12-3c; another is provided below:

b. An impervious upstream blanket connected to a sloping impervious core was placed during the construction of Tarbela Dam on the Indus River in Pakistan (Lowe 1978). The blanket material consisted of sandy silt mixed with a sandy silt angular boulder gravel. The blanket lay over an alluvium of cobble gravel choked with fine sand. The blanket, which was to increase the length of seepage path and not necessarily to reduce seepage quantities, met the piping criteria, D_{15} (alluvium) $\leq 5D_{85}$ (blanket), Appendix D. As the reservoir emptied after first filling, several sinkholes and cracks were noted in the blanket. Sinkholes ranged from 1 to 15 ft in diameter and 4 to 6 ft in depth. It was felt that uneven settlement during the first reservoir filling caused tension and compression cracks in the blanket which allowed considerable seepage into the underlying sand-choked gravel. In areas where the sand was less dense, the seepage moved the sand down to form a layer in the lower part of the gravel. This created open work gravel just beneath the blanket, and fines from the blanket moved into and through this open layer forming the sinkholes. Sinkholes were filled with filter material and mounded over with blanket material. Typically, the blanket mounds were approximately 15 ft high and extended 30 to 35 ft beyond the sinkhole edge. After filling of the reservoir, sinkholes were located by side-scan sonar and filled with a mixture of filter material and silt from self-propelled bottom dump barges. Each sinkhole generally received 50 barge loads of material. Sinkholes continued to be discovered and covered over another 3-4 years after the initial remedial action. Siltation on the reservoir blanket and filling of sinkholes have reduced seepage about one half.

12-6. Downstream Berm. Berms control seepage by increasing the weight of the top stratum so that the weight of the berm plus top stratum is sufficient to resist uplift pressure. If of low permeability, they will reduce seepage, but increase uplift pressures beneath the downstream toe of the dam since they force seepage to exit further downstream of the dam. If pervious, they must be designed as a filter or with an underlying filter to prevent upward migration of line particles from the foundation materials beneath them. Again, a seepage analysis must be made to determine the resisting load required of the berm. Downstream slope stability of the embankment will normally increase because of the resistance to sliding provided by the berm. Huntington District has employed berms as remedial measures at several flood control dams in the Muskingum River flood control system (Coffman and Franks 1982). Similarity of the embankments and environments allowed a standard remedial action for several of the dams at the downstream embankment toe. A 3- to 7-ft-thick pervious blanket of appropriate length is placed over the soft seepage areas at the downstream toe. This adds weight and provides a working platform for installation of relief wells at points of excessive seepage.. Another example of a stability berm is given in the Addicks and Barker Dams example, paragraph 12-7a.

12-7. Slurry Trench Cutoff. Two major technical considerations in the use of slurry trenches as remedial seepage control measures are (a) the effect on stability of the embankment due to excavation of the trench and the presence of a vertical plane of relatively weak soil (in the case of a soil-bentonite backfill) and (b) tying the slurry trench to other existing or proposed seepage control measures. If a competent upstream blanket exists, the trench may be placed upstream of the embankment and tied to the blanket or may be placed through the dam and any pervious substratum if stability requirements are met. A cement-bentonite backfill may be placed in panels or a concrete wall may be placed in separately excavated elements if an open trench and the relatively weak soil-bentonite backfill are unacceptable because of stability risks. The following experiences with slurry trenches provide general examples of this cutoff type as a remedial measure.

a. Addicks and Barker Dams, Houston, Tex. (U. S. Army Engineer District, Galveston 1977a; U. S. Army Engineer District, Galveston 1977b; U. S. Army Engineer District, Galveston 1983). Completed in the late 1940's, Addicks and Barker Dams are rolled earth embankments providing flood control in the Houston, Texas, area, with respective maximum heights of 48.5 and 36.5 ft above streambed. Neither normally impound water except in periods of rainfall. The embankments contain some silts and sands, foundations have silt and sand layers, and upstream borrow areas expose the foundation permeable layers. At the time of construction, these conditions were not considered, significant because of the large discharge capability and short detention time of the reservoirs. Residential and commercial development of the downstream local area caused several changes in operating conditions which increased detention time and made the effect of seepage more critical. These included restriction of discharge rates and construction of drainage channels on non-Federal land within 200-300 ft downstream of the center line of the dams which expose the pervious portion of the foundation. Erosion of the drainage channel slopes on the side of the channel nearest the dam and boils in the channel bottom during times of low reservoir impoundment indicated the potential for dangerous seepage conditions during high reservoir levels. Downstream piezometers also indicated a quick response to changes in reservoir levels. This example describes remedial actions at Addicks Dam; actions at Barker Dam were similar. Several remedial measures were considered:

(1) Downstream drainage blanket and stability berm - rejected due to requirement for additional right-of-way and Government responsibility for maintenance of local interest's drainage ditch.

(2) Downstream drainage blanket, stability berm, and relief well system and downstream slurry trench - (relief wells between embankment toe and slurry trench) very positive control (blanket and berm control embankment seepage while wells and slurry trench control underseepage), but very costly, long-term well maintenance required, and all seepage forces would be directed at the embankment toe.

(3) Same plan as (2) except slurry trench replaced with steel sheet pile cutoff - same reasoning as (2) except sheet pile would greatly increase cost.

(4) Slurry trench cutoff through embankment and foundation - a very positive, controlled cutoff for embankment and foundation; no maintenance; all work on Government property; less costly and quicker than other alternatives. For most of the remedial work, alternative (4) was chosen, though for selected lengths of the embankment where they were the best alternative, alternative (1) was used and some relief wells were placed. With a maximum depth of 64 ft and width of 3-5 ft, the slurry trench penetrated 2-4 ft into a relatively impervious clay underlying the pervious foundation materials. Figure 12-2 provides a general cross section of the design. The trench was placed 10-20 ft upstream of the embankment center line with equipment working from a platform established by degrading the upper portion of the embankment. Cemented materials, present in some portions of the excavation, were broken by dropping a 10-ton percussion tool on the cemented layers. Portions of the trench collapsed but were successfully reexcavated. Additionally, small (3-in. diameter) tunnels were encountered in the upstream side of the trench but were

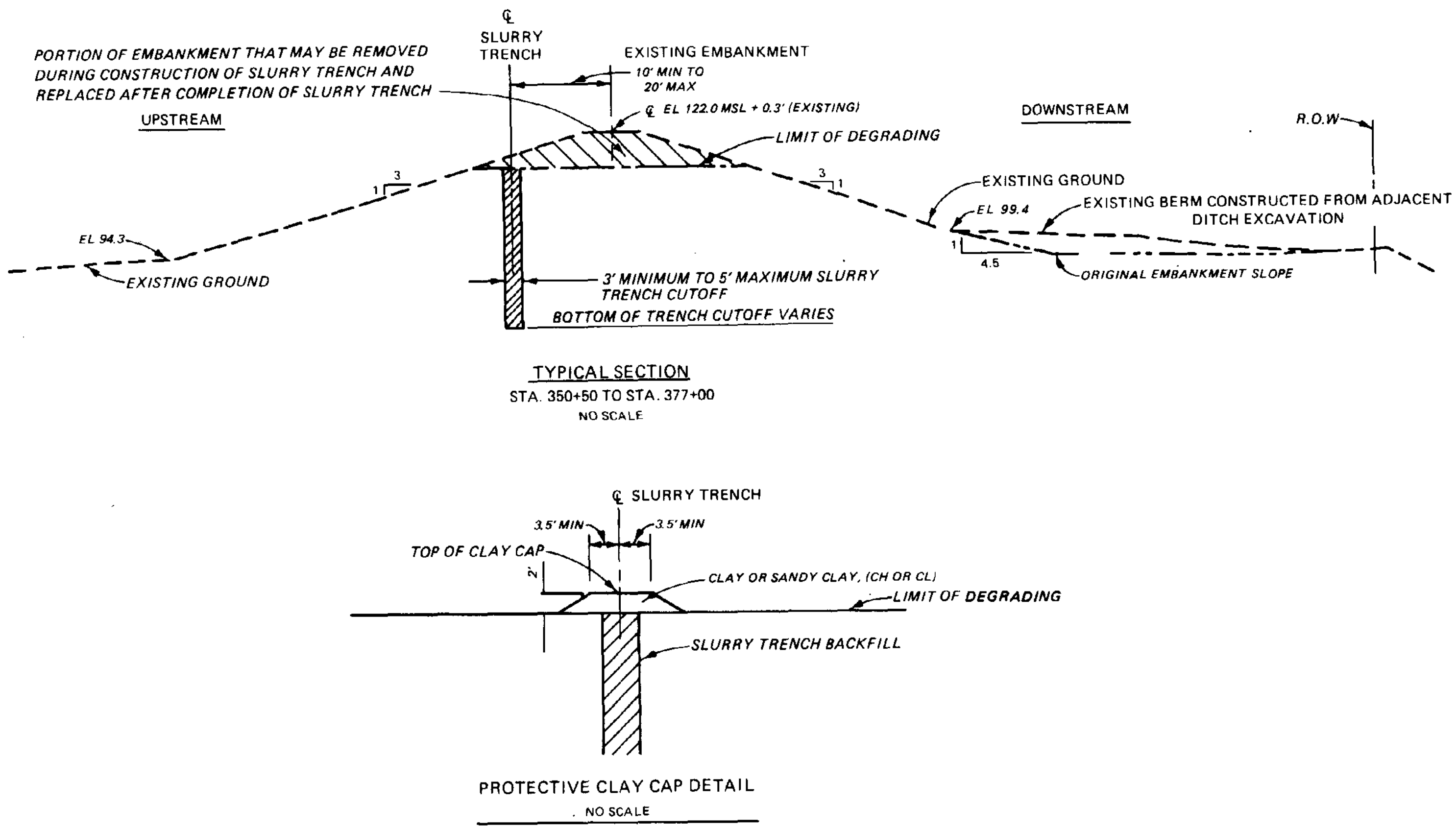

Figure 12-2. Addicks Dams remedial slurry trench for embankment and foundation seepage control (from U.S. Army Engineer District, Galveston[83])

plugged with cloth. Backfill gradation is shown in table 12-1. For some portions of the project, percent passing for the No. 200 sieve were 15-30 percent. Backfill mixing and transport to the trench were conducted in several ways. Some backfill was batched dry, placed in concrete trucks with slurry added, then mixed and transported to the trench. The higher fines content backfill in some cases proved too sticky to mix in trucks. Mixing was conducted on the ground next to the trench but occasionally excess fines were picked up from the working surface. A concrete mixing pad was used as an alternative though wear from the mixing equipment destroyed the concrete. Excess or unsatisfactory material was deposited in old borrow areas upstream of the embankment to reduce underseepage. In one area, a slurry trench located at the upstream toe of the embankment provided underseepage control while a downstream berm provided embankment stabilization. The berm of sandy clay had permeability characteristics similar to the embankment and provided a 1V on 8H slope. Several of the discharge conduits which suffered from seepage and piping were resealed, after cleaning, with ethafoam backer rods and a polyurethane sealant. Where the sealant would not adhere to the concrete, joints were talked with oakum soaked with a grouting compound. Well screens were placed in weep holes to prevent loss of soil, and relief wells with submersible pumps were installed. For certain portions of Barker Dam, use of an upstream clay blanket and a downstream stability berm (1V on 8H) was more cost effective than a slurry trench. There was intermittent surface exposure of pervious foundation materials and a source of CH materials for the blanket was available within the reservoir. Prior to placement of the blanket, ponded water and soft surface materials were removed. The blanket was placed in 8-in. layers and compacted with tamping rollers at natural moisture content.

Table 12-1. Backfill Mix for Slurry Trench, Addicks Dam[a]

Sieve Size or Number (U. S. Standard)	Percent Passing by Weight
3 in.	100
1-1/2 in.	95 to 100
3/4 in.	80 to 100
No. 4	55 to 100
No. 10	40 to 80
No. 40	18 to 45
No. 200	10 to 25

[a] From U.S. Army Engineer District, Galveston.[85]

Though not yet severely tested, the control measures have performed satisfactorily based on the following observations:

(a) Foundation downstream piezometers do not respond to reservoir levels experienced so far.

(b) Phreatic surface has been raised upstream of the slurry trench.

(c) No embankment seepage, but there have been no significant pools.

(d) Settlement plates indicate no significant settlement of the slurry trench. Though the restored embankment has cracked in the area of the trenches, inadequate compaction of the embankment fill is considered the cause.

b. Wolf Creek Dam, Ky. (Fetzer 1979). Constructed in the 1940's, Wolf Creek Dam is a 200-ft-high combination earthfill and concrete dam founded on limestone containing shale and solution cavities. During excavation of a 10-ft-wide cutoff trench, several interconnected solution cavities were discovered in the limestone. These were backfilled for a short distance with impervious material, and a 50-ft-deep single-line grout curtain was placed beneath the bottom of the cutoff trench. In 1967, muddy flow was observed in the tailrace, a small sinkhole developed near the downstream toe, and wet areas existed near the downstream toe. In 1968, a larger sinkhole developed (13 ft wide, 10 ft deep) and drilling revealed solution features running perpendicular and parallel to the dam axis. It was concluded that reservoir water was passing beneath the cutoff trench. Grout lines were placed along the dam axis near the embankment-concrete contact and downstream of this area. During 1971-1972, an overall assessment of the seepage problem was made since the remedial grouting had only addressed about 200 ft of the 4,000-ft embankment portion of the dam. A diaphragm concrete cutoff wall was considered the best solution because it could be installed without draining the reservoir, a very costly operation due to reservoir use. Explorations , which included borings spaced on 3.1-ft centers along the axis of the wall (parallel to the dam axis), defined the depth and length of the wall. Depth was 10 ft below the lowest indication of solution activity (maximum depth 278 ft) and length was 2,239 ft. In 1974, a request for technical proposals resulted in seven proposals with two acceptable. In the second stage, a bid invitation was issued and an award was made for a wall in the area of the switchyard and 989 ft of the wall along the dam axis. The award in 1975 was followed by a second competition and an award in 1977 for the remaining 1,250 ft of the axis wall. The wall consists of alternate cylindrical primary elements and connecting secondary elements installed using bentonite slurry, figure 9-14. Primary elements are 2.17-ft-diam steel casings filled with tremied concrete (see table 9-8 for mix proportions). Weak cement grout fills the volume between excavation walls and the casing. A 25-ft-deep core hole was drilled beyond the bottom of each primary element to explore for cavities and was pressure-tested and grouted prior to the placement of a closed-end primary casing. The primary element was required to set for a minimum of 20 days before excavation of the secondary element which is also filled with tremied concrete. Frequent piezometer readings (as often as every 4 hours) were made during construction to determine the hydraulic condition of the embankment and foundation and warn of any potentially critical seepage conditions. Excavation and drilling were closely monitored to observe any drill rod drops or mud losses. Sealers and reserve mud, constantly on hand, provided for emergencies. Grout takes around the primary casings and volume of concrete used in the secondary elements were closely monitored as was the embankment in general. Efficient management of a large number of observations was necessary to determine the current condition of the dam. The lack of major losses of slurry, grout, or concrete during construction was probably due to the densely spaced borings and grouting done during the earlier exploration program. Wall construction, completed in 1979, took approximately 4 years and two construction contracts. Subsequent piezometric levels indicate the wall is a successful seepage barrier.

c. Camanche Dike 2, California (Anton and Dayton 1972). One of several earthfill dikes containing Camanche Reservoir, Dike 2, is a zoned earth embankment about 70 ft high founded on alluvium containing an upper 20-ft strata of clayey sand underlain by layered silty-to-fine uniform sand stratum. The underlying sand stratum varies in permeability with its lower portion containing gravel. Original construction involved extending the core horizontally to the upstream toe and discing and compacting the top of the alluvium to 1,000 ft upstream of the dike axis. This was expected to provide acceptable underseepage conditions, while providing the option of tying an upstream cutoff through the alluvium to the core if operating underseepage conditions were intolerable. After reservoir filling, underseepage proved extensive and flowed over downstream property. Lowering of the reservoir reduced the seepage and revealed holes in the compacted alluvium upstream of the embankment. Several seepage control methods, including upstream impervious blanket, grout curtain, relief wells, downstream drains, sheet piles, and others, were considered. Evaluation of the options resulted in choosing an upstream slurry trench. This method provided a positive cutoff, minimized piping potential, allowed retention of a partial reservoir, and was the least expensive of positive cutoff methods. Placed 50 ft upstream of the upstream berm toe, the 1,660-ft-long slurry trench, 8 ft wide, extended through the alluvial materials to a maximum depth of 95 ft. Backfill specifications required a 4-in. slump and a gradation as shown in Table 12-2. An 8- to 11-ft-deep sandy clay blanket protected by a 1-ft-thick cobble and gravel cover connected the slurry trench to the horizontal core extension. Excess slurry was blended into the top portion of the blanket to decrease blanket permeability. Since slurry trench placement, downstream piezometers reflect decreased influence of reservoir levels with only very small seepage flows at high, prolonged reservoir levels. Two potential sources of seepage are the somewhat pervious bedrock formation which the slurry trench is keyed into and the sandy clay blanket connection between the slurry trench and extended core. Connection of the 8- to 11-ft-deep blanket to the slurry trench after placement of the slurry was difficult and may allow reservoir leakage into the alluvium. Placement of the blanket or a partial thickness prior to slurry trench construction was recommended. This would provide a platform for construction of the slurry trench and allow a more secure attachment of the trench to the blanket. This procedure has been standard practice on many subsequent slurry trench projects.

12-8. Relief Wells. Though Chapter 9 describes design and installation of relief wells, additional factors must be considered when relief wells are used for remedial seepage control. Relief wells can relieve excessive uplift and potential piping when pervious layers are overlain by relatively impervious strata by providing controlled release of relatively large volumes of water. Relief wells, as compared with cutoffs, allow loss of reservoir water and

Table 12-2. Backfill Mix for Slurry Trench, Camanche Dike 2 [a]

Sieve Size or Number (U. S. Standard)	Percent Passing by Weight
6 in.	100
3 in.	80 to 100
3/4 in.	60 to 100
No. 4	40 to 80
No. 30	20 to 60
No. 200	10 to 30

[a] Courtesy of American Society of Civil Engineers.[135]

require proper handling of discharge flows and periodic maintenance. Flooding and erosion from well discharges must be prevented. Wells may be installed quickly with a minimum of downstream right-of-way and, in many cases, without reducing reservoir levels. If high uplift is present, boring and installation may be difficult requiring extra measures to keep the hole open and stable until the screen and filter are installed.

12-9. Drainage of Downstream Slope. Seepage emerging on or at the toe of the downstream slope will normally be controlled by one of the methods previously mentioned, Expedient installation of filter materials and a toe drain can help prevent piping of embankment and foundation materials and may increase embankment stability, but will not normally reduce seepage quantities. If seepage is confined to a small area or areas, horizontally drilled drains may help control the problem (Royster 1977). Horizontal drains of slotted pipe normally do not have a filter envelope and would generally be used for "nuisance" seepage or as an expedient measure until a more permanent solution could be installed.