Chapter 3: Summaries of Case Histories

This chapter presents summaries of 30 case studies on the performance of seepage barriers in dams that were reviewed for this study. Information was collected on additional dams that were not included in this study because either the data was too sparse to make a useful assessment of the dam performance or the seepage barrier had never had a differential hydraulic head across it, and thus the barrier was never tested. The materials reviewed consist of published papers, unpublished reports, correspondence, and monitoring data. Each of the case history summaries includes a description of the dam and seepage barrier, site geology and dam material properties, instrumentation and monitoring program, and an assessment of the long-term performance of the seepage barrier and dam.

Wolf Creek Dam

Background

Wolf Creek Dam is managed by the U.S Army Corps of Engineers, Nashville District and retains Lake Cumberland, the largest United States reservoir east of the Mississippi River. The dam was constructed between 1938 and 1953 with a construction hiatus during World War II. An aerial photograph of the dam is presented on Figure 3-1 and a plan of the dam is presented on Figure 3-2. The dam is a 5,736 foot-long combination homogenous earth fill embankment and concrete gravity structure with a maximum height of 258 feet (USACE 2005, Zoccola et al. 2006). A cross section through the earthen portion of the dam and its foundation is presented on Figure 3-3.

The foundation conditions at the dam site consist of an approximately 40-foot thick alluvial deposit that rests on top of limestone of the Liepers and Catheys formations (Zoccola et al. 2006, USACE 2005, AMEC 2004). The alluvium is up to 50 feet thick and consists of silty sand near the base fining upwards to sandy clay and silty clay near the top of the deposit. The Liepers formation is approximately 100 feet thick and
contains an extensive interconnected system of solution cavities. Most of the solution cavities are partially or completely filled with soil. The Catheys formation underlies the Liepers formation and has experienced a much lower degree of solutioning activity.

Figure 3-1  Aerial Photograph of Wolf Creek Dam (After Zoccola et al. 2006, with permission from USSD)

Figure 3-2  Plan of Wolf Creek Dam (After USACE 1973, with permission from USACE)

Many of the developed solution voids in the Liepers Formation follow two primary joint orientations. One of these joint sets trends roughly parallel to the axis of the dam with
Foundation preparation for the earth embankment portion of the dam was minimal (USACE 2005, Zoccola et al. 2006). Most of the alluvium remains in place except for a small portion of the embankment adjacent to the concrete structure. A minimal cutoff trench was constructed beneath the upstream slope of the dam by removing soil from a large solution feature, constructing a single-line grout curtain from the base of the trench, and backfilling with minimally compacted earth fill. The location of the cutoff trench is presented in the cross section in Figure 3-3 and in plan view in Figure 3-4. Poorly compacted fill was placed along both sides of a narrow central zone of compacted soil, and large caves, vertical bedrock walls, overhangs, and solution voids branching off from the cutoff were left untreated. A photograph of the construction of a portion of the cutoff trench is presented in Figure 3-5. A drainage blanket was constructed below the embankment downstream of the cutoff trench (see Figures 3-3 and 3-4). Based on the lack of flows emanating from the drainage blanket, it was judged to be largely ineffective in draining the embankment (USACE 2005).
Foundation preparation for the concrete gravity portion of the dam was more extensive. The concrete monoliths are founded on bedrock near the contact between the Catheys and Liepers formations (USACE 2005).

The embankment fill consists of compacted clays, sandy clays, and clayey sands derived from the alluvium upstream and downstream of the dam. In addition to the main
embankment fill, a random fill berm was constructed along the downstream toe. The random fill berm consists of a mixture of soil types and in some areas is coarser grained than the embankment fill (USACE 2005).

**History of Seepage Problems and Mitigation**

In 1967 and 1968 evidence of serious internal erosion was observed at the dam (USACE 2005). In 1967, a sinkhole developed near the right toe of the embankment. In 1968, two more sinkholes developed in the random fill at the left end of the embankment. One of the 1968 sinkholes extended through the random fill and alluvium to the top of bedrock 40 feet below. In addition to the sinkholes wet areas near the downstream toe and muddy flows into the tailrace were observed (Fetzer 1979, Simmons 1982, USACE 2005).

In response to the observations described above, an emergency grouting and exploration program was performed from 1968 to 1970. Approximately 290,000 cubic feet of grout was placed in the heavily solutioned bedrock beneath the left end of the embankment where it wraps around the end of the concrete section. Between 1970 and 1975, a grout line was added along the crest of the embankment from the concrete section to the right abutment. The holes for this grout line were drilled on 3.1 foot centers and water pressure test results and grout takes were recorded for each hole (Fetzer 1979, USACE 2005).

While the grouting program was credited with saving the dam, a board of consultants recommended that a more permanent solution be constructed (USACE 1974, 2005). As a result, the board of consultants recommended construction of a deep concrete seepage barrier extending from the top of the dam, through the Leipers Formation and into the top of the Catheys Formation. The board recommended the cutoff be constructed for the entire length of the earthen embankment.
Design and Construction of Seepage Barrier

The grout-take data from the cutoff trench grouting during initial construction and the grouting below the dam crest provided valuable information on the conditions of the bedrock and alluvium. A plot of grout volumes placed along the alignment of the original cutoff trench is presented on Figure 3-6. Plots of the grout volumes in the alluvium and bedrock placed from the crest of the dam between 1970 and 1975 are presented in Figures 3-7 and 3-8, respectively. In order to make the data in Figures 3-7 and 3-8 more readable the data presented are a running average of grout takes over 10 adjacent grout holes for each location. It should be noted that the relatively low grout takes from station 35+00 to Station 37+00 are likely due to grouting that had taken place in this area a few years before.

Figure 3-6   Grout volume takes from the base of the original cutoff trench at Wolf Creek Dam (after AMEC 2004, with permission from USACE)
A review by the US Army Corps of Engineers of the grout takes versus depth beneath the dam crest identified areas of extensive solutioning near the top of the Leipers formation and two locations where the extensive solutioning extended to the top of the Catheys formation. The remainder of the formation was assessed to have far less extensive solutioning. Based on this assessment and taking into consideration the desire to reduce
the cost of the project, it was decided to reduce the horizontal and vertical extent of the proposed seepage barrier from that recommended by the board of consultants (Fetzer 1979, USACE 2005). The resulting limits of the constructed seepage barrier are presented on Figure 3-9. The constructed barrier is 2250 feet long extending from the concrete section of the dam to Station 57+50 leaving the right 1,700 feet of the dam without a barrier. The constructed barrier extends into the Cathays Formation in two short sections and extends only through the highly solutioned upper portion of the Leipers Formation for the remaining length.

![Figure 3-9 Limits of the constructed seepage barrier in Wolf Creek Dam (after Zoccola et al. 2006, with permission from USSD)](image)

The barrier was constructed in a two-phase process using steel-encased drilled pier primary elements that were connected with secondary bi-concave elements as shown on Figure 3-10 (Couch 1977, Fetzer 1979, Holland et al. 1982, USACE 2005, Zoccola et al. 2006). The primary elements were spaced at 4.5-foot centers and installed through the embankment and alluvium using a stepped down casing. Once into the bedrock, the primary elements were drilled to the design elevation. After the primary elements were drilled, borings were extended from the base of each of the primary elements 20 to 40 feet into the underlying bedrock. The borings were pressure tested to check the soundness of the underlying rock, and grouted. The primary elements were cased for their full height with permanent 26-inch diameter steel casing, and filled with tremmied concrete (Holland et al. 1982). The stepped-down casings through the embankment and
alluvium were removed. The secondary elements were excavated using an excavator guided by the 26-inch steel casing of the primary elements.

Figure 3-10  Plan and section of primary and secondary elements of seepage barrier constructed in Wolf Creek Dam (after USACE 2005, with permission from USACE)

Following construction of the secondary elements, 43 of the 498 total secondary elements were cored to observe the quality and continuity of the concrete backfill (AMEC 2004). Twenty-eight of these cores reached the base of the element while the remaining cores penetrated the side of the element before reaching the base. Of the 43 elements cored, aggregate segregation or honeycombing was encountered in 36 of the elements, or about 84 percent of the elements. The worst of the segregation appeared to occur at the base of the elements. Of the cores that reached the base of the elements, about 46 percent showed poor contact between the concrete and the bedrock (AMEC 2004). The
segregation encountered in other parts of the elements consisted mainly of an absence of course aggregate in the concrete.

Instrumentation

Following the seepage incidents in 1967 and 1968 the USACE installed water pressure and deformation monitoring devices on the dam (USACE 2005). The monitoring devices installed since 1968 consist of piezometers, inclinometers, and settlement monuments. In addition to the water level measurements in the piezometers, a water temperature survey was performed in 2004.

Piezometers. Between 1968 and 1979, when the construction of the seepage barrier was completed, over 300 open standpipe piezometers were installed to monitor water levels in the embankment, alluvium and foundation bedrock (USACE 2005). The piezometers were recorded at varying intervals throughout the construction of the grout curtains and seepage barrier. These piezometers showed that the grouting program initiated after the seepage incidents in 1967 and 1968 was effective in reducing the elevated piezometric pressures in the downstream portion of the embankment adjacent to the tie-in with the concrete section of the dam (Fetzer 1979, USACE 2005, Bradley et al. 2007). However, the piezometer readings following the installation of the seepage barrier indicated very little change in the piezometric pressures measured just after the grouting program (USACE 2005, Bradley et al. 2007). After the completion of the seepage barrier the number of piezometers monitored was reduced to about 100, although additional piezometers have been added since then.

In 2004, AMEC Earth & Environmental, Inc. (2004) reviewed the records of a total of 195 piezometers that have been monitored on or around Wolf Creek Dam at various times between 1984 and 2004. Eighty-nine of the piezometers were deleted from the data base due to lack of data, unreliable data, or being located beyond the area of interest. Piezometric levels from the remaining piezometers were plotted versus time to observe changes in the seepage regime over time. AMEC (2004) calculated the average linear
trends of 82 piezometers that had sufficient data from which to calculate a trend line. A histogram showing the distributions of the resulting changes in piezometric head from 1984 to 2004 for piezometers located in and below the main dam embankment and for piezometers located in the switchyard area is presented in Figure 3-11.

The data on Figure 3-11 indicate that a vast majority of the piezometers located below the dam embankment show small increases in piezometric level between 1984 and 2004. Most of the piezometric trends indicate an increase of 0 to 7 feet while a few of the piezometers indicate rises as much as 21 feet. In the switchyard area, located adjacent to the tailrace, seven out of the eight piezometers indicated drops in the piezometric levels on the order of 10 to 16 feet. These trends indicate that there has been an overall change in the seepage regime through the embankment at Wolf Creek Dam since the installation of the seepage barrier. The decrease in piezometric level in the switchyard areas (downstream of the dam) is likely a consequence of a decrease in the seepage resistance in this area resulting in little drop in hydraulic head from the piezometers to the tailrace.

Data from the piezometers in which water elevations in excess of elevation 630 feet were
recorded in 2004, and those showing an increase in piezometric level greater than 10 feet over the past 20 years, are presented in Table 3-1. The locations of these piezometers are shown on Figure 3-12. Also shown on Figure 3-12 are the reaches of the original cutoff trench where high grout takes were recorded and the reaches under the dam crest where high grout takes were recorded in the alluvium and the bedrock between 1970 and 1975 (see Figures 3-6 to 3-8).

Table 3-1  Wolf Creek Dam piezometers that recorded piezometric levels above elevation 630 in 2004 or experienced an increase in piezometric head of greater than 10 feet between 1984 and 2004 (based on data from AMEC 2004)

<table>
<thead>
<tr>
<th>Letter</th>
<th>Piezometer Number</th>
<th>Station Along Dam Axis</th>
<th>Piezometer Tip Elevation (ft.)</th>
<th>Values from Linear Trends of Piezometric Date – 1984 to 2004</th>
<th>2004 Piezometric Elevation (ft.)</th>
<th>Change in Head 1984 to 2004 (ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>WP-72E</td>
<td>35+50</td>
<td>605</td>
<td>708</td>
<td>16.8</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>D-322</td>
<td>35+95</td>
<td>574</td>
<td>653</td>
<td>11.7</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>D-268E</td>
<td>36+10</td>
<td>605</td>
<td>686</td>
<td>20.5</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>DC-252R</td>
<td>36+80</td>
<td>593</td>
<td>631</td>
<td>5.1</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>D-321</td>
<td>37+00</td>
<td>574</td>
<td>638</td>
<td>4.4</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>WA-53E</td>
<td>42+40</td>
<td>635</td>
<td>636</td>
<td>-1.5</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>WP-87</td>
<td>50+52</td>
<td>598</td>
<td>626</td>
<td>11.0</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>WA-38</td>
<td>52+92</td>
<td>633</td>
<td>634</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>D-289</td>
<td>55+41</td>
<td>630</td>
<td>639</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>J</td>
<td>D-290</td>
<td>55+99</td>
<td>624</td>
<td>634</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>WA-11</td>
<td>56+93</td>
<td>625</td>
<td>639</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td>L</td>
<td>D-301</td>
<td>57+48</td>
<td>620</td>
<td>637</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>M</td>
<td>WP-93</td>
<td>59+50</td>
<td>658</td>
<td>658</td>
<td>-0.2</td>
<td></td>
</tr>
<tr>
<td>N</td>
<td>D-308</td>
<td>62+48</td>
<td>653</td>
<td>661</td>
<td>-2.2</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3-12 shows clusters of the piezometers from Table 3-1 at each end of the seepage barrier with a few piezometers located in between. The piezometers from Stations 35+00 to 37+00, close to the tie-in with the concrete section of the dam, include those with the highest piezometer readings and the highest head increases over the past 20 years (Piezometers A, B and C). The tips of Piezometers A and C are located within the embankment while the remaining piezometer tips are located in bedrock. The piezometers below the right end of the seepage barrier (at Station 57+50) indicate moderately-high to high piezometer levels but very little change in piezometric level over
the last 20 years. All of the piezometer tips in this location are located within the bedrock or close to the bedrock/alluvium interface. It is also worth noting that Piezometer G, which indicated an 11 foot rise in head over the last 20 years, is located directly downstream from a location where high grout takes were recorded in the alluvium and rock below the dam crest.

Figure 3-12  Locations of piezometers from Table 3-1 and reaches of high grout takes from original cutoff wall grouting and 1970 to 1975 grouting

Settlement Monuments. Surface monuments were installed along the dam crest in 1981 to monitor embankment settlement. Small settlements have been recorded along the entire dam crest since construction of the seepage barrier. An anomalously large amount of settlement has been recorded at Station 37+00 where 0.15 feet of settlement was recorded from 1981 to 1997 and another 0.15 feet of settlement was recorded from 1997 to 2004 (USACE 2005). Settlement monuments adjacent to the one at Station 37+00
indicate the depression gradually decreases over a distance of 200 to 300 feet to the levels experienced in other locations along the dam crest.

The settlement at Station 37+00 is significant in two ways. First, the settlement readings indicate that the rate of settlement is increasing, taking 16 years for the first 0.15 feet of settlement and only 4 years for an additional 0.15 feet of settlement. Second, the location of the settlement coincides with the general area where the piezometers recording the highest piezometric levels and the greatest increases in piezometric levels over the last 20 years are located. This is also an area where high grout takes were recorded in soft alluvial and embankment materials and is close to where the solution feature used for the original cutoff wall was located.

In addition to the recorded settlements in the embankment, members of a peer review panel discovered evidence of movement of Monolith 37, the last monolith of the concrete section of the dam (Bradley et al. 2007). The evidence consisted of a gap in the upstream portion of the joint between Monolith 37 and the adjacent monolith, and evidence of compression in the downstream portion of the joint. This indicates that the upstream side of the monolith has moved toward the embankment and is further evidence of softening and movement in the embankment concrete section interface.

Inclinometers. A total of 13 inclinometers were installed in the seepage barrier between 1976 and 1979 (USACE 2005). Three additional inclinometers were installed just downstream of the barrier in 1980. The inclinometers all show a similar pattern of lateral displacement in the upstream-downstream orientation. At depth, the inclinometers showed downstream movement in the alluvium and embankment fill. Near the top of the inclinometers, upstream movement was detected. Maximum deflections in this direction were on the order of 0.2 feet. Movements parallel to the dam crest were erratic and were of much lower magnitude with maximum deflections less than 0.1 feet.

The downstream movement is likely result of the buildup of water pressures on the upstream side of the barrier and an associated decrease in water pressure on the
downstream side. The upstream movement indicated at the top of the barrier may be due to saturation of previously unsaturated soils on the upstream side of the barrier. The saturation may result in compression of the upstream embankment soils resulting in an upstream movement.

**Temperature Survey.** A survey of the water temperature in the piezometers was performed by AMEC Earth and Environmental in 2004 (AMEC 2004). The survey identified 6 areas where the piezometer water temperatures were several degrees cooler than in the surrounding piezometers. Two of these locations were close to the transition between the embankment and concrete portions of the dam, three were located by the switchyard adjacent to the tailrace, and one was located beyond the toe of the dam at Station 46+50. These indicate areas of relatively rapid seepage, bringing in cooler reservoir water.

**Borings.** Because of the indicators of renewed seepage problems at the dam, 12 borings were drilled in the embankment using the resonant sonic drilling technique (USACE 2005). Eight of the borings were located in the general area of the interface between the embankment and the concrete section. Six of the borings encountered soft, wet zones within the embankment and alluvium. The soft zones were limited to 7 feet or less in thickness with the exception of one boring that encountered 16 feet of soft, wet clay. Most of the soft, wet clay was encountered near the contact between the embankment fill or alluvium and the bedrock. While most of the soft clay layers were encountered around the interface with the concrete section, a soft clay layer was also encountered in a boring at Station 56+25, close to the right end of the seepage barrier.

**Assessment of Seepage Barrier Performance**

There are numerous indicators that the seepage barrier installed between 1975 and 1979 is not performing as intended. High water levels and rising water levels in some downstream piezometers, several areas with anomalously cold piezometer water temperatures, settlement of the dam crest, and soft zones in the embankment and
alluvium indicate zones of seepage through the barrier. Additional indicators of unsatisfactory performance include the following (USACE 2005, AMEC 2004, Zoccola et al. 2006, Bradley et al. 2007):

- Numerous wet areas have developed beyond the toe of the dam,
- Several areas have been identified where the surface of the embankment is depressed relative to the surrounding embankment surface,
- The cable tunnel located near the downstream toe of the embankment has experienced seepage and cracking since the mid 1980s, and
- Observations indicate that the amount of seepage from the riverbank downstream of the dam has increased, and the bank is experiencing slope instability.

Based on an assessment of the locations where (1) piezometers are reading the highest piezometric levels, (2) piezometers have experienced a large rise in piezometric level over the past 20 years, (3) piezometers have anomalously low water temperatures, (4) anomalously high settlements of the embankment crest were measured, and (5) soft zones in the embankment and alluvium were encountered, three areas have been identified where there is strong evidence that significant seepage is occurring through or under the barrier. These areas include (1) the portion of the embankment adjacent to the end of the concrete section of the dam (approximate Stations 35+00 to 37+00), (2) the area around Station 50+50, and (3) the area around the right end of the seepage barrier (approximate Stations 55+50 to 62+50). A discussion of each of these areas is presented below. It should be noted that there may be other areas where large amounts of seepage through or around the barrier are occurring that have not been detected by the monitoring instruments.

Station 35+00 to 37+00. In the portion of the embankment, adjacent to the concrete section, nearly every indicator of seepage related distress was observed. There are also two characteristics of this area that represent opportunities where seepage may have bypassed the barrier and, in doing so, removed soil particles and adversely affected the long term performance of the barrier. The first characteristic is the potential for gaps in the interface between the seepage barrier and the concrete portion of the dam resulting
from construction difficulties caused by the 1 horizontal to 10 vertical camber of the end of the concrete section. The second characteristic is the existence of extensive solution voids in the bedrock that trend from the end of the original cutoff trench, beneath the dam crest, and below the switchyard area. Both of these characteristics, working either independently or together, may have lead to the performance deterioration in this location.

The mechanism that appears to be in effect in this location is the removal of soil infill from the solution voids in the Liepers Formation. While grouting has been performed in this area, the grout cannot displace the soil infilling, leaving the solution cavities filled partially with soil and partially with grout. Water backing up behind the seepage barrier and grouted section results in high hydraulic gradients that act on the soil-filled voids and results in piping into an open solution cavity downstream. Where these eroded cavities come in contact with the overlying alluvium and embankment fill, sloughing of the alluvium or fill into the void results in the soft areas observed in the borings drilled in 2002 and 2003.

The potential gap between the seepage barrier and the concrete dam section may expedite the mechanisms discussed in the preceding paragraph, but has a lesser potential to be the sole source of the observed distress. If the potential gaps existed solely within the low permeability embankment fill, it is likely that the fill would retard the seepage to the point that erosive velocities would not develop in the gaps. Furthermore, a pathway for the removal of eroded particles might not be available. However, because the gap likely extends down into voids in the bedrock, it represents a pathway where the erosion can propagate upward from the solution voids and accelerate upward migration of the distress. The gap is also important since the seepage barrier extends down to the Catheys Formation in this area. With the barrier cutting off most of the major solution voids, the gap may act as the major pathway for seepage and removal of soil particles.

Station 50+50. At this location a single piezometer indicated a moderate piezometric head in the bedrock and a head increase of 11 feet over 20 years. The location of the
piezometer is just downstream from a location where high grout takes were recorded during the 1970 to 1975 grouting program below the dam crest. It is likely that the same erosion of solution void soil infill mechanism that was described for the preceding area is also responsible for the deterioration of the seepage performance in this location. However, because the barrier did not extend completely through the Liepers Formation the likelihood that continuous pathways for seepage and soil particle removal is significant.

**Stations 55+50 to 62+50.** In this area, located at the end of seepage barrier, moderately-high to high water levels in the bedrock were recorded in six piezometers. However, none of these piezometers has shown a significant increase in head between 1984 and 2004. It is possible that the high piezometric levels are due to seepage flow around the end of the barrier. Most of the seepage flow is likely occurring in the alluvium and bedrock. The lack of change in these piezometers over time is likely attributable to either (1) the hydraulic gradients flowing around the end of the seepage barrier not being high enough to cause significant erosion, or (2) the solution voids in this area not being large or extensive enough to cause large increases in seepage due to erosion of soil infill.

**Conclusions.** Based on the above discussions, it is concluded that the most likely mechanism responsible for the redevelopment of seepage problems after the construction of the seepage barrier is the erosion of soil infill in the solution voids below the barrier. It seems likely that there may be additional areas where seepage has redeveloped through previously grouted solution voids but remain undetected because no piezometer was located where the excessive seepage could be detected.

There are likely also defects in the seepage barrier caused by construction defects or cracking as a result of barrier deflection. These defects are not thought to represent a significant percentage of the seepage through the dam due to the low permeability of the embankment fill. Exceptions to this may be gaps at the interface between the barrier and the concrete section or defects that occur adjacent to highly permeable alluvial layers or solution voids in the bedrock.
Upper and Lower Clemson Diversion Dams

Background

The Upper and Lower Clemson Diversion Dams are managed by the U.S Army Corps of Engineers Savannah District. The dams are located approximately 2,000 feet from each other and they protect against the inundation of parts of Clemson University by Hartwell Lake (the primary dam for Hartwell Lake is Hartwell Dam located roughly 20 miles to the south). The dams are similar in design and cross section, constructed on similar foundations, and have similar heights and crest elevations. They were both completed in 1961 as homogenous rolled earth fill embankments containing chimney drains and horizontal drainage blankets (USACE 1987a, 1989). Both dams are approximately 55 feet high with a crest elevation of 680 feet. A profile representative of the cross section of both dams and their foundations is shown on Figure 3-13. The fill material consists mostly of silty sand (SM) with lesser amounts of clay (CL and CH). The full pool elevation of 660 feet is controlled by a spillway on Harwell Dam. There are no spillways or outlet works associated with Upper or Lower Clemson Diversion Dams.

Figure 3-13  General profile of Upper and Lower Clemson Diversion Dams (modified from USACE 1983, 1987, 1989, with permission from USACE)

The embankments are located across the old channel alignment of the Seneca River and rest on foundations of alluvium (USACE 1981a, 1987, 1989). The alluvium consists of interbedded layers of clean sand and gravel (SP and GP), silty and clayey sand (SM and SC), and low-plasticity silt (ML). The thickness of the alluvium varies from about 15 feet to 30 feet. In general, the upper portion of the alluvium is dominated by silty sand, clayey sand, and silt while the lower portion of the alluvium is dominated by deposits of
clean sand and gravel. The alluvium is underlain by granite gneiss bedrock that varies from intensely weathered near the alluvial contact to fresh with depth. The depth of bedrock weathering varies from a few feet to over 80 feet.

According to the construction documents (USACE 1981a, 1983, 1987, 1989) 15-foot wide foundation keyway trenches were constructed through the alluvium in both dam foundations. The trenches were reported to be located 45 feet upstream of the dam centerlines and extended into the weathered bedrock.

History of Seepage Problems and Mitigation

**Lower Clemson Diversion Dam.** Evidence of seepage problems at Lower Clemson Diversion Dam appeared during the first filling of the reservoir in 1962 when a large wet area developed at the downstream toe near the right abutment (USACE 1987a). In June 1962 ten relief wells were installed to the base of the alluvium in this area to relieve excess pore pressure and in September 1962 an additional seven relief wells were constructed in the area of the old Seneca River Channel. Piezometers indicate the relief wells had little effect on the piezometric levels in the area. In late 1963 a sand berm was placed along the downstream toe of the right groin of the embankment. Locations of the wet area, boils, and various mitigations are shown in Figure 3-14.

In 1963 a single-line grout curtain was constructed over a 900 foot section of the dam near the right abutment as shown on Figure 3-14. Grout takes in the alluvium were much higher than expected, totaling over 18,000 cubic feet of grout. The grouting program was terminated after showing little or no change in the piezometric levels or seepage observations (USACE 1987a).

In 1966 a large boil appeared at the toe of the sand berm in an area above the old Seneca River channel. Muddy water was exiting the boil at a rate of approximately 70 gpm. The pipe for the boil was excavated a distance 30 feet into the dam embankment and backfilled with filter material. In 1967 and 1968, the existing blanket drain in the area of
the old Seneca River channel was extended 100 feet out from the toe of the dam and a berm was constructed over the drain extension (USACE 1987a).

In 1979, following a period of high reservoir levels, several boils developed at the toe of the dam. In response, a 225-foot long French drain system was installed consisting of graded stone wrapped in filter fabric. Because the muddy flows from this French drain never cleared, the drain was replaced with a drain consisting of sand and gravel around a well screen (USACE 1987a).

Upper Clemson Diversion Dam. Evidence of seepage problems at Upper Clemson Diversion Dam appeared soon after the first filling of the reservoir in 1962 (USACE 1989a). In 1963 several boils formed at the toe of the central portion of the dam. These boils were mitigated by excavating the pipes (seepage tunnels through the soil) near the ground surface and installing a trench drain consisting of a perforated steel pipe and filter material. Additional boils formed in 1964 in the same location as 1963 and in the right groin area. Mitigation for the additional boils consisted of the removal of the pipe in the trench drain and installation of eight relief wells extending to the base of the alluvium. The relief wells were installed between December 1964 and November of 1965. Locations of the boils and relief wells are shown in Figure 3-15.
Between 1965 and 1977 additional areas of concentrated seepage and wet areas were encountered and mitigated with sand and gravel filters and trenches constructed near the ground surface. In 1966, low areas and ditches downstream of the toe were filled with sand dredged from the lake to act as a filter to seepage into these areas. In 1977, a 40-foot-long by 6-foot-deep by 4-foot-wide interceptor drain with a 6-inch pipe and two-staged filter was constructed below the toe of the dam in the area where boils first developed in 1963. Continued occurrence of wet areas after the interceptor drain construction prompted the performance of detailed subsurface investigations to determine a course of action to deal with the seepage problems. These investigations and analyses were performed from 1980 through 1982 and resulted in the recommendation that a positive cutoff wall be constructed beneath the dam centerline and a drainage system be constructed downstream of the dam to collect any additional seepage water.

**Design and Construction of Seepage Barrier**

The seepage barriers for both dams were designed to act as cutoffs for seepage through sand and gravel layers in the alluvium. The Lower Clemson Diversion Dam seepage barrier was constructed in 1982 (USACE 1987a) and the Upper Clemson Diversion Dam
seepage barrier was constructed between 1983 and 1984 (USACE 1989a). The barriers were installed from the dam crest over the entire lengths of the dams and were embedded a minimum of 5 feet into the weathered bedrock as shown on Figure 3-13. Because the abutments of the dams are constructed on weathered bedrock, the resulting seepage barriers extend through the full depth and width of the alluvial deposits. The Lower Clemson Diversion Dam barrier is a total of 2,580 feet long and has a maximum depth of 91 feet. The Upper Clemson Diversion Dam barrier is a total of 2,150 feet long and has a maximum depth of 91 feet.

The barriers were excavated using a slurry wall panel technique that utilized a hydraulic clamshell excavating device installed at the end of an 8-inch square Kelly bar (called the “Kelly Clam” by the contractor) (USACE 1987a, 1989). The wall was constructed in 25-foot-long panels. Primary panels were constructed first in every other panel location along the alignment. Following primary panel excavation, 2-foot diameter shoulder piles were placed at the ends of the primary panels to provide a smooth surface for concrete placement. Secondary panels were constructed between the primary panels after they had been backfilled and partially cured. After excavation of the secondary panels, the shoulder piles placed during the construction of primary panels were removed and the secondary panel backfilled.

The barriers were backfilled with concrete with a specified 28-day strength of 3,000 psi and a slump of 6 to 9 inches (USACE 1987a, 1989). Prior to concrete placement the bottoms of the panels were cleaned with an airlift pump and the slurry in the panel was desanded. The concrete was placed through 10-inch tremie pipes. A retrievable traveling plug “go-devil” was placed ahead of the first concrete to prevent mixing of the concrete and slurry.

Following curing of the concrete, some of the panels were cored to observe the quality of the concrete (USACE 1987a, 1989). In general, the quality of the concrete was excellent. Some segregation of the aggregate was occasionally observed. In the upper elevations of the panels, small areas containing “worm holes” where observed. These “worm holes”
were thought to be formed by the expulsion of bleed water from the concrete. No such structures were observed in the deeper portions of the panels. At the base of the panels 0.1 to 0.3 feet of flocculated slurry were encountered. The flocculated slurry was plastic and had a greasy feel. In one core, the contact between two panels was encountered. The contact was tight (one-sixteenth of an inch aperture), well-fitting, and contained a seam of bentonite.

Instrumentation

The dams have been monitored with an extensive array of piezometers since the investigations preceding seepage barrier construction. Prior to these investigations the dams were monitored by less extensive piezometer arrays.

Lower Clemson Diversion Dam. Prior to 1980 the dam was monitored with 14 piezometers installed in the dam in 1962, shortly after completion of construction. By 1981 all but four of the original 14 piezometers had been abandoned. During the subsurface investigations in 1979 and 1980, 41 piezometers were installed. Most of the recent piezometers are located along four cross sections aligned perpendicular to the dam axis. A cross section through Station 8+90 (Cross Section LC-BB) is presented in Figure 3-16 and shows the typical arrangement of piezometers along the three cross sections. These piezometers have been monitored from the time they were constructed to the present (USACE 1981a, 1987).

In addition to the piezometers, five inclinometers have been installed on the dam (USACE 1987a). Three inclinometers were installed prior to the seepage barrier and located 30 feet downstream from the dam centerline. Two inclinometers were installed in core holes within the barrier in 1982.

Upper Clemson Diversion Dam. Prior to 1980 the dam was monitored with one set of three piezometers installed in the dam in 1963, shortly after completion of construction. During the subsurface investigations in 1980, 1981 and 1983, a total of 39 piezometers
were installed. Nine of these piezometers had to be abandoned during construction of the seepage barrier, but were reconstructed at similar locations at the completion of construction. Most of the recent piezometers are located along three cross sections aligned perpendicular to the dam axis. A cross section through Station 6+00 (Cross Section UC-AA) is presented in Figure 3-17 and shows the typical arrangement of piezometers along the three cross sections. These piezometers have been monitored from the time they were constructed to the present (USACE 1987, 2002).

![Cross Section LC-BB showing the typical arrangement of piezometers along the four monitoring cross sections of Lower Clemson Diversion Dam](image)

In addition to the piezometers a total of four inclinometers have been installed on the dam (USACE 1989a). Two inclinometers were installed in 1983 prior to the seepage barrier construction on the downstream bench of the dam. Two additional inclinometers were installed in core holes within the barrier in 1984 and 1985.

**Assessment of Seepage Barrier Performance**

Comparison of piezometer readings before and after the construction of seepage barriers indicates that the barriers had considerable influence on the downstream piezometric
surfaces. In Lower Clemson Diversion Dam, the observed drop in piezometric level varied from two to 15 feet. The drop was most pronounced in Profile LC-BB where the alluvium was comprised mostly of sand and least pronounced in Profile LC-AA where the alluvium consisted predominantly of fine grained soils. In Upper Clemson Diversion Dam, the observed drop in downstream piezometer levels varied from two to nine feet following the installation of the barrier. The piezometric surface change was most pronounced in the piezometers located nearer the abutments and less effect was observed along the central portion of the dam. In both dams the drop in piezometric level was greatest in the piezometers located closest to the dam axis and decreased in the piezometers located beyond the dam toe. Comparisons of the pre-barrier and post-barrier piezometric levels measured in the alluvium at nearly full reservoir levels are presented on Figures 3-6 and 3-7.

The piezometer data collected since the construction of the seepage barrier indicates that the behavior of the piezometers has not changed significantly since the installation of the barrier. Annual fluctuations observed in some of the measured piezometric levels measured are generally tied to the pool level in the reservoir and were observed to be following the same trends as they did shortly after seepage barrier construction. Thus,
based on the piezometer data, it can be assessed that the performance of the seepage barrier has not changed significantly since construction.

The inclinometers installed on the downstream benches of both dams indicate movements of up to 2.0 inches, with one inclinometer indicating movement upstream and the other three moving downstream. The movement has occurred mostly in the upper 10 to 15 feet and has been attributed mainly to construction disturbance (USACE 1987a, 1989). The inclinometers installed in the seepage barriers indicated movements between 5/8 and 3/4 of an inch with the maximum movements being in the upstream direction and toward the right abutment.

Seepage and deformation analyses performed on these dams are presented in Chapter 4, along with an overall assessment of the performance of the seepage barriers that encompasses both the information presented above and the results of the analyses.
Figure 3-19  Comparison of pre-barrier and post-barrier piezometric levels in the alluvium of Upper Clemson Diversion Dam at Station 6+00

**Wister Dam**

**Background**

Wister Dam is managed by the U.S Army Corps of Engineers, Tulsa District. The dam was completed in 1949 and is a homogenous rolled earth fill embankment with a primary function of flood control and recreation. A general site plan and profile of the dam and its foundation are shown in Figure 3-20. The dam crest is at an elevation of 527.5 feet and the maximum height of the embankment is 98 feet. The outlet works, consisting of two 14-foot by 16-foot semi-elliptical conduits, are located beneath the embankment near the right abutment. The outlet works are regulated by lift gates located on a gate tower on the upstream side of the embankment. The spillway is located off the embankment about ½ mile to the east and has a crest elevation of 502.5 feet. The total length of the embankment is about 5,700 feet (USACE 1986a, 1988b).

The dam is constructed across the flood plane of the Poteau River on a foundation consisting of up to 38 feet of alluvium (average thickness of 28 feet) overlying sandy shale bedrock (USACE 1986a). The alluvium consists predominantly of sandy clayey
silt with occasional deposits of sand and gravel up to two feet thick at the base of the deposit. The alluvium thins to only a few feet in the Poteau River Channel.

Figure 3-20 Plan and profile of Wister Dam (after Erwin and Glen 1992, with permission from ASCE).

The bedrock underlying the dam consists of moderately hard silty shale with thin sandstone lenses and occasional sandstone beds (USACE 1986a). The bedding planes dip northward, toward the left abutment, at an angle of about 25 degrees, thus, the strike of the bedrock runs roughly perpendicular to the axis of the dam.
The embankment material consists of silty clay and clayey silt with minor amounts of clayey sand (USACE 1986a, 1988a). The embankment material was found to be dispersive in nature following a seepage and internal erosion incident that occurred on the first filling of the reservoir (see discussion below). Finger drains were installed at the interface of the embankment and the alluvial overburden under the downstream three quarters of embankment footprint. The finger drains were constructed in 2.5-foot deep by 2.5-foot wide trenches spaced at 20-foot intervals and backfilled with coarse gravel without filter material. Berms were constructed of crushed shale material on the upstream and downstream toes of the embankment.

History of Seepage Problems and Mitigation

In January 1949, when the dam was in the final stages of construction, a flood event occurred that resulted in water being impounded behind the dam to an elevation of 494 feet. Two days following the high water level of this event, severe seepage and internal erosion was observed on the downstream face of the dam. An emergency drawdown of the reservoir level through the outlet works prevented a breach of the dam. Following drawdown it was observed that the inlet of the piping was at an elevation of 485 feet, only 9 feet below the highest elevation that the reservoir attained during the flood (Sherard 1986, Erwin and Glen 1982). The exit points of the piping were located at an elevation of about 475 feet, or about 10 feet lower than the inlet. The inlet and exit points of the piping are shown on Figure 3-21.

Sherard (1986) assessed that the cause of the seepage was a combination of hydraulic fracturing and the existence of dispersive clays. The hydraulic fracturing was able to occur under such a low hydraulic head (9 feet at the inlet) due to low stresses in the embankment caused by differential settlement and bridging across the Poteau River channel in the dam foundation. The internal erosion progressed rapidly due to the dispersive nature of the embankment soils that was not well understood at the time of the near failure.
To mitigate the observed seepage problems, the USACE initiated a program consisting of cement grouting in the embankment and foundation, mud grouting in the embankment, installation of a sheet pile seepage barrier down to bedrock, and construction of a filter and drain system at the downstream toe (USACE 1986a). Cement grout was placed in the embankment in places were drilling fluid was lost during the drilling of the grout holes for foundation grouting. A grout curtain was constructed to a depth of 30 feet into bedrock. A total of 3,176 sacks of cement were pumped into the embankment and another 4,475 sacks were pumped into the foundation. The mud grouting program, started from the centerline of the embankment, placed 32,140 cubic feet of mud grout before the operation was halted because it was causing hydraulic fracturing of the embankment. The sheetpile wall was constructed from the top of the upstream berm along a 1120-foot length spanning the affected area. The sheetpiles were intended to be driven to bedrock, however, driving problems left many windows in the wall.

In 1979, 10 relief wells were installed to provide relief for high uplift pressures that were observed in the bedrock during high pool elevations (USACE 1986a). Based on pump tests, only four of the relief wells appear to be effective in relieving pressures. The remaining six wells showed moderate to little response to the reservoir pool elevation. This variation in performance between the wells indicates there are zones of bedrock below the dam having significantly higher permeability than in other areas.
Design and Construction of Seepage Barrier

Other than the high piezometric pressures that prompted the installation of the relief wells in 1979, no major seepage problems were encountered with the dam since the original incident in 1949. However, continuing erosion problems associated with the dispersive clay on the downstream face of the embankment, high piezometric pressures in the embankment, and the lack of a second line of defense against piping failure (such as a chimney drain and filter) prompted an investigation into the potential for seepage failure of the embankment. Based on the results of the investigation, the USACE concluded the following (USACE 1988a):

1. There are a significant number of sand zones in the alluvium and bedrock foundation,
2. The strength of the embankment and alluvium is lower than expected,
3. There are a significant number of cracks within the embankment, many of which were wet, and
4. The high piezometric levels at the toe of the dam are the result of leakage through cracks and seams in the embankment and alluvium.

Based on these conclusions, the USACE designed a mitigation plan consisting of (1) a slurry wall seepage barrier constructed from the upstream berm and extending five feet into the bedrock, (2) an upstream seepage blanket constructed between the top of the seepage barrier and the crest of the dam, (3) a filter blanket constructed on the downstream slope of the dam, and (4) a lime-treated surface on the downstream face of the dam, as shown on shown on Figures 3-22 and 3-23.
Figure 3-22  Details of the Wister Dam filter blanket (after Erwin and Glen 1992, with permission from ASCE)
Figure 3-23  Details of the Wister Dam seepage barrier (after USACE 1988b, with permission from ASCE)
Construction of the seepage barrier was completed between February 1990 and January 1991 using a hydrofraise excavating device 35 feet high, with four hydraulically-driven cutter wheels at the base. Prior to excavation, three-foot high guide walls were constructed to help align the hydrofraise. Each pass of the hydrofraise excavated a bite 7.2 feet long and 2 feet wide. Primary panels, 18 feet in length and two feet wide, were constructed first. One pass was made at each end of the primary panels first and then the remaining portion of the panel was excavated in a third pass of the hydrofraise, maintaining overlap with both of the first pass excavations. A distance of 6.5 feet was left between the primary panels to be excavated as a secondary panel. After the primary panels were backfilled with plastic concrete and partially cured, the secondary panels were excavated, cutting into the primary panel on both sides of the cut to form joints against sound concrete.

The panels were backfilled using a plastic concrete mixture designed to have a compressive strength of 600 psi and a modulus of elasticity of 2,300,000 psi (USACE 1988a). The average compressive strength of the concrete averaged nearly 900 psi, resulting in a much less ductile wall than anticipated. Primary panels were backfilled using two tremie pipes while the secondary panels were backfilled using one tremie pipe.

**Instrumentation**

The instrumentation and monitoring program at Wister Dam includes the reading of piezometers, toe drain flow measurements, and surveying pins that monitor crest settlements and horizontal deflections (USACE 2002a). A total of 38 piezometers were installed in 1988 prior to construction of the seepage barrier. All 38 of these piezometers where monitored until 1999 when it was decided that it was no longer necessary to monitor 26 of the original piezometers. The remaining 12 piezometers were supplemented with eight new piezometers in 1999 to make a total of 20 piezometers presently being monitored. Of the 20 piezometers, four are located below the dam crest,
11 are located below the downstream berm, and five are located below the downstream toe. All of the piezometers measure pressures in the foundation alluvium and bedrock.

Seepage from the toe drain has been measured since 1993 at three outfall locations below the dam toe. The water collected by the toe drain system is from a number of sources including: through and under seepage collected in the filter blanket beneath the downstream face of the dam, flow from relief wells, flow from the finger drains installed during the original dam construction, and surface runoff infiltrating the system (USACE 2002a). A plot of readings from the three drain outlets and the reservoir levels is shown in Figure 3-24.

Iron survey pins located along the entire downstream crest shoulder have been monitored since 1992. No significant settlement has been observed since the installation of the pins.

**Assessment of Seepage Barrier Performance**

Piezometer readings before and after the construction of the seepage barrier indicate that the barrier had an influence on the downstream piezometric surface. Readings from selected piezometers are shown on Figure 3-25. The observed change in downstream piezometer levels were as much as 15 feet lower at high reservoir pool levels after completion of the cutoff than at similar pool levels prior to barrier construction. The effects of the seepage barrier are most pronounced in the piezometers located along the dam crest and the effects decrease in the downstream direction.

Observations of piezometer levels over time since the installation of the seepage barrier indicate that, with a few exceptions, no appreciable changes in piezometer behavior appear to be occurring. The piezometers react to the reservoir elevation to varying degrees and no significant change in this reaction has been observed. The exceptions to this behavior are observed in three piezometers (PZEL07, PZEL14, and PZEL25) that recorded sudden rises or drops in piezometric level that remained for several years before returning to levels near their original readings. Plots of these piezometers are shown in
Figure 3-24  Drain outlet flow measurements and reservoir pool elevation for Wister Dam, 1992 to 2005
Figure 3-26. In at least one case (PZEL07) the drop in piezometric level was coincidental with a very high pool level in the reservoir. This behavior may be due to the opening and subsequent plugging of seepage pathways through bedrock joints.

Figure 3-25  Selected Wister Dam piezometer readings, 1988 to 2004

Review of the measurements of toe drain outflow over time indicates there has been change in the behavior of the components feeding into this system over time. Figure 3-24 shows that the flows from the north outlet have increased steadily over time. A less pronounced increase can also be observed from the middle outlet. The 2002 Periodic Inspection Report (USACE 2002a) attributes the high flows in the north outlet to infiltration of surface water along the dam abutment.

Other explanations of the increasing flows may be increased flow from one of the relief wells tied into the drain system or increased flow in the foundation. This increase in flow may be due to the removal of infilling from bedrock joints along the seepage path between the reservoir and a relief well. Increased foundation flows may be attributable to development of seepage pathways through the course-grained layers of the foundation.
Notes:  
A) Two-year drop in level of Piezometer 07.  
B) Four-year rise in level of Piezometer 14.  
C) Three-year period of increased response to reservoir pool level.

Figure 3-26  Wister Dam piezometer readings showing periods of anomalous piezometric level, 1988 to 2004

**Fontenelle Dam**

**Background**

Fontenelle Dam is managed by the U.S Bureau of Reclamation and is located on the Green River southeast of La Barge, Wyoming. The dam was completed in 1964 and is a 139-foot high (crest elevation 6,516 feet) zoned earthfill embankment with a crest length of 5,450 feet (USBOR 2002a). An uncontrolled concrete spillway is located on the right abutment. There are three outlet works, a river outlet at the base of the embankment near the center of the dam, the east canal near the left abutment, and the west canal near the right abutment. The dam has a power plant adjacent to the toe of the dam (USBOR 2002a). A plan of the dam is shown in Figure 3-27.
A cross section of the dam is shown on Figure 3-28. The embankment is made up of zones consisting of three soil types (USBOR 2002a). Zone 1 is the low permeability core and consists of selected clay, silt, sand and gravel compacted in 6-inch layers. Zone 2 is the shell material and consists of sand, gravel and cobbles compacted in 8-inch layers. Zone 3 is the miscellaneous materials obtained from the excavations and consists of random material compacted in 8-inch layers. The upstream slope of the dam is inclined at 3 horizontal to 1 vertical and the downstream slope is inclined at 2 horizontal to 1 vertical. The upstream slope is armored with a 5-foot thick layer of riprap. An 80-foot wide cutoff trench extending through the alluvium to the top of the bedrock was constructed beneath the core section of the dam. A single line of grout holes extending a maximum of 110 feet into bedrock was constructed along a line 30 feet upstream of the crest centerline.

The foundation consists of a thin layer of alluvium overlying interbedded calcareous sandstone, siltstone, shale and minor beds of limestone (USBOR 2002a). The bedrock is
layered in beds ranging from less than a foot to several tens of feet in thickness. The bedding dips at very low angles (less than 2 degrees) in a direction between downstream and toward the left abutment. Minor amounts of gypsum were encountered in some core holes drilled during site exploration and travertine deposits were encountered in several core holes in the left abutment.

Jointing in the bedrock consists of (1) bedding plane joints in the siltstone and shale layers, (2) near vertical tectonic joints crossing all rock types, and (3) near vertical stress-relief joints in the sandstone layers. The stress-relief joints are the result of the loss of lateral confinement in the walls of the canyon. These joints often run the entire thickness of the sandstone layer and continue laterally for considerable distances. The stress-relief jointing often results in open joints that are roughly parallel to the abutments (USBOR 2002a).

History of Seepage Problems and Mitigation

Seepage problems became evident at Fontenelle Dam when the dam nearly failed during first filling in September 1965 (Bellport 1967, Murray and Browning 1984, USBOR 2002a). In May through July 1963 three episodes of sloughing occurred in the backfill adjacent to the spillway chute. The sloughing was attributed to seepage through bedrock joints and was mitigated with drainage pipes. On September 3, 1965, when the reservoir was about 2 feet below the spillway crest, a wet spot was observed at mid-height on the embankment about 100 feet from the spillway chute. By the next morning, the wet spot had developed into a cone-shaped hole emitting a flow of about 21 cubic feet per second. Approximately 10,500 cubic yards of material had eroded from the embankment. Emergency measures were taken to lower the reservoir through the outlets, and rock was dumped from the dam crest into the hole.

Although the dumping of rock into the hole appeared to slow the erosion, the hole continued to enlarge, and on the afternoon of September 6 an area of the dam crest about 20 feet in diameter collapsed and dropped about 33 feet into the hole. This collapse
exposed bedrock on the side of the abutment and water was observed emitting from the joints in the rock. No further sloughing occurred after this collapse. The reservoir was lowered at a rate of 4 feet per day to an elevation of 6458 feet (48 feet below the spillway crest). Observation of the collapse area indicated that the flow had come through open joints in the bedrock and had eroded the adjacent Zone 1 material (Bellport 1967, Murray and Browning 1984, USBOR 2002a). A photograph of the collapsed area is presented in Figure 3-29.

The near failure was attributed to (1) inadequate grouting of the abutment, (2) soluble material or soil fill in the joints inhibiting grouting, (3) the lack of slush grouting and dental concrete in the abutment, and (4) the steep and irregular abutment leading to a poor embankment-abutment contact and possible cracking due to differential settlement (Bellport 1967, Murray and Browning 1984, USBOR 2002a). The area of the near failure was reconstructed between September 1965 and December 1966 by extensive remedial grouting of the dam foundation and repair of the embankment using similar materials to the original construction.

The dam appeared to perform well until late June 1982 when a noted increase in seepage detected in the weir monitoring the left abutment. In October 1982 two additional seeps were detected and wet areas downstream of both abutments appeared to be migrating closer to the embankment. Settlement measurement points indicated that there was an acceleration of the rate of settlement near the left abutment.

In the fall of 1982 electrical self potential surveys and water quality testing were performed (Murray and Browning 1984, USBOR 2002a). The self potential survey delineated four major anomalies through which water appeared to be emerging and flowing downstream. The most pronounced of these anomalies was where the embankment contacts the left abutment. The water quality testing compared the amount of dissolved solids in the reservoir water versus the seepage water and concluded that large quantities of dissolved salts were being removed by the seepage water.
In July 1983 sediment was reported to have filled the bottom 12 feet of an open standpipe piezometer located at the downstream toe of the dam near the left abutment (Murray and Browning 1984). This observation, along with the observations and measurements discussed above, prompted the Bureau of Reclamation to limit reservoir operations and to institute an extensive monitoring system.

From 1983 to 1984 an extensive exploration program was conducted in response to the observed increased seepage (Murray and Browning 1984, USBOR 2002a). This investigation indicated the presence of soft zones in the Zone 1 portion of the
embankment adjacent to the left abutment. Sampling of these zones indicated they consisted of loose sandy material with much of the fines removed. It was concluded that internal erosion had occurred along the contact between the Zone 1 material and the foundation, and that there was a high potential for continued embankment piping. The most likely mechanism for initiating the piping was water flowing through bedrock joints in direct contact with the Zone 1 material. It was further concluded that the Zone 2 material does not meet filtering criteria for the Zone 1 material in all cases.

Based on the findings of the investigation, the conclusion was made that a positive cutoff was necessary to reduce the risk of piping. It was concluded that a concrete seepage barrier was the best alternative.

**Design and Construction of Seepage Barrier**

The seepage barrier at Fontenelle Dam was designed to provide a barrier to seepage flow through the uppermost and highly fractured portions of the bedrock and the lower portions of the embankment that had been compromised by the internal erosion described above. The design called for a 2-foot wide concrete barrier extending approximately 40 feet into the bedrock below the dam and extending approximately 200 feet laterally into each abutment. The barrier was constructed in two phases. Phase I consisted of two test sections constructed in the abutments between 1985 and 1986. The test sections were 661 and 644 feet long and were located in the right and left abutments, respectively. Phase II consisted of a section of seepage barrier connecting the two test sections and two additional sections that extended the barrier past the test sections into the abutments. Phase II was constructed from 1987 to 1989 (USBOR 1994, 2002).

Both phases of the seepage barrier were constructed using a rockmill (hydrofraise) tool approximately 8 feet long and two feet wide. The barrier was excavated in bentonite slurry-filled panels. After excavation, the slurry was desanded and concrete was placed using the tremie method. Primary panels were 20 to 30 feet long and were connected with 7-foot long secondary panels. The 7-foot long panels allowed for 6 inches of
concrete to be removed from each of the adjacent primary panels to create a good bond between the panels (USBOR 1994, 2002).

A special construction challenge was encountered where the river outlet works (ROW) was located at the base of the embankment. No barrier was constructed in the dam directly above and below the outlet structure. Instead, soil-bentonite panels were constructed on the upstream and downstream sides of the barrier adjacent to the concrete structure and in front of the transverse walls of the outlet to plug any potential seepage paths that could develop through construction joints between the barrier and the concrete structure. Where the concrete seepage barrier panels came in contact with the outlet structure, the structure concrete was cleaned with wire brushes to remove any embankment materials. The bedrock beneath the ROW was grouted using a five-row grout curtain.

Instrumentation

Seepage at the dam has been assessed and monitored since 1982 using survey monuments, inclinometers, weirs, and piezometers (USBOR 1994, 2002). A description of each of these elements and a summary of their observed behavior is described below.

Survey Monuments. Survey monuments have been monitored on the embankment and the top of the seepage barrier since seepage barrier construction (USBOR 1994, 2002). One-hundred-twenty-four embankment measurement points are located in four lines across the embankment. One line is located upstream of the crest and three are located on the downstream slope. Seventeen survey measurement points are located on the top of the seepage barrier.

Inclinometers. Six inclinometers were installed in the seepage barrier shortly after construction and have been monitored since that time (USBOR 1994, 2002).
Weirs. Seepage flows from the dam and foundation have been collected and measured using five weirs and a flume since 1983. Three V-notch weirs collect seepage from below the right abutment and two V-notch weirs and a trapezoidal flume measure seepage from the left abutment.

Piezometers. Three types of piezometers have been used to monitor the piezometric levels in the dam since 1982: open-standpipe piezometers, standpipes with retrievable electrical vibrating-wire transducers, and non-retrievable electrical vibrating-wire transducers (USBOR 1994, 2002). Fifty-two open standpipes are located with piezometer tips in the foundation beneath the embankment, in the alluvial channel below the dam, and in the abutments. Thirty-six retrievable vibrating-wire piezometers are located either in the rock foundation or in the Zone 1 portion of the embankment no closer than 11 feet from the foundation interface. Fifteen non-retrievable vibrating-wire piezometers are embedded in locations (1) upstream of the river outlet, (2) below the river outlet, and (3) below the concrete apron upstream of the spillway crest.

Assessment of Seepage Barrier Performance

Deformation. Deformation measurements on the embankment surface since the construction of the seepage barrier have measured small deformations that are uniform and without anomalous trends (USBOR 2002a). Most measured deformations are less than 0.1 foot.

Five of the six inclinometer readings detected similar deflection patterns (USBOR 2002a). Shortly after the completion of the barrier and refilling of the reservoir, these five inclinometers detected bow-shaped downstream deformations between the top of bedrock and the top of the barrier. The maximum displacement in these piezometers is on the order of 0.02 feet. Since this initial deformation, additional deformations have not been of significant magnitude. The sixth inclinometer is located near the left abutment where the embankment height is significantly less. No significant deformations have been measured in this inclinometer.
Seepage Flow. The measurement of seepage flow is usually a good indicator of seepage barrier performance because it generally encompasses seepage from a large area rather than the discrete zones monitored by individual piezometers. The seepage monitoring devices at Fontenelle Dam indicate that the seepage barrier was effective in reducing the seepage flows after the barrier construction, but give mixed results for the long-term performance of the barrier. The weirs in the right abutment indicate the seepage barrier was effective in reducing seepage flows in this area. A comparison of seepage flows from 1984 to 1992 indicates a 30 percent decrease in flows following construction of the barrier. Similarly, in the left abutment flows reduced from around 50 gpm to just a trace of flow over the same time period.

Long-term data from the right abutment weirs since the barrier construction indicate a decrease in flow while the seepage in the left abutment appears to have increased. The right abutment flows dropped from around 500 gpm to about 380 gpm under similar reservoir levels from 1992 to 1996 and appeared to drop further by 2001. In the left abutment seepage flows in weir SM-4 had been small and sporadic prior to 1995 and have since produced flows of 30 to 40 gpm when the reservoir is above elevation 6480 feet.

Piezometers. Data from the piezometers indicates that the seepage barrier has been effective in lowering the piezometric levels in the embankment and foundation downstream of the barrier. Piezometers located downstream of the barrier in Zone 1 of the embankment dropped as much as 10 feet in the first year after barrier installation and continue to drop at decreasing rates. In the foundation, eight of the 12 piezometers located downstream of the barrier were reading at or close to the tailwater level indicating a high level of seepage barrier efficiency. These piezometers show a slight response to changes in the reservoir level that is likely due to seepage beneath the barrier. The four piezometers not reading at the tailwater level are very near the abutments and, thus, likely affected by water seeping around the ends of the barrier. Piezometers located upstream
of the barrier were reading very close to the headwater elevation, also an indication of very high barrier efficiency.

**Summary.** While the seepage barrier appears to be effective in reducing downstream piezometric heads, short-term and long-term trends in some of the piezometer data indicate mechanisms that show reduction in effectiveness of the barrier with time. These data trends are:

1. Evidence in the Zone 1 embankment of leaks through the barrier, and
2. Evidence of deterioration of the seepage resistance indicated in data from piezometers both upstream and downstream of the barrier.

These trends are discussed in the following paragraphs.

Seven of the piezometers located downstream of the barrier in Zone 1 of the embankment (PT-VW-1, PT-VW-5, PT-VW-6, PT-VW-16, PT-VW-17, PT-VW-21, and PT-VW-22) have either shown response to reservoir fluctuations or have other cyclic piezometric head variations indicative of leaks through the barrier (USBOR 2002a). Three of these piezometers (PT-VW-1, PT-VW-21, and PT-VW-22) are located near the abutments and response to the reservoir level is thought to be dominated by seepage through the abutments. Piezometers PT-VW-16 and PT-VW-17 are located in the lower portion of Zone 1 and showed responses to fluctuations in reservoir levels of about 1 and 5 percent, respectively, in the first few years following the construction of the barrier. Other nearby piezometers did not show this reservoir response. Over time, the piezometric level and reservoir response of PT-VW-16 and PT-VW-17 has decreased and the piezometers became dry in 1993 and 2001, respectively. A plot of the data from PT-VW-17 is shown on Figure 3-30.

The data from Piezometers PT-VW-5 and PT-VW-6 represent unusual behavior (URS 2002, USBOR 2002a). A plot of these piezometers along with an adjacent piezometer that is not responsive to reservoir fluctuations (PT-VW-7) are shown on Figure 3-31. As early as 1990 Piezometer PT-VW-6 began exhibiting cyclic behavior that was independent of the reservoir fluctuations. The cycles consist of sudden increases in
piezometric level of up to 6 feet followed by more gradual decreases. This cyclical pattern has repeated on fairly regular intervals with a few exceptions since 1990. Piezometer PT-VW-5 exhibited similar behavior from 1994 through 1995 and has since changed to a behavior that responds to the reservoir fluctuations.

<table>
<thead>
<tr>
<th>Year</th>
<th>Reservoir</th>
<th>PT-VW-17</th>
<th>Tailwater</th>
</tr>
</thead>
<tbody>
<tr>
<td>1988</td>
<td>6510</td>
<td>6430</td>
<td>6410</td>
</tr>
<tr>
<td>1989</td>
<td>6490</td>
<td>6470</td>
<td>6450</td>
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<tr>
<td>1990</td>
<td>6410</td>
<td>6430</td>
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<td>1991</td>
<td>6450</td>
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<td>6490</td>
</tr>
<tr>
<td>2002</td>
<td>6510</td>
<td>6430</td>
<td>6410</td>
</tr>
</tbody>
</table>

Figure 3-30 Data from Fontenelle Dam Piezometer PT-VW-17 (after USBOR 2002a)

URS (2002) attributed this behavior to leakage through small defects between panels in the wall. URS theorizes that seepage along a pre-existing concentrated leak in the upstream portion of the core builds up on the upstream side of the seepage barrier. When the pressure builds to a certain level the defect opens and releases pressure to the downstream side of the barrier. After the defect closes the pressure dissipates over time through the downstream portion of the core. The defects appear to be small and, based on the consistency of the behavior over the years, the defects do not appear to be getting larger.
In three cases piezometer data may indicate long-term changes in the seepage regime related to the seepage barrier. The first is the behavior of Piezometers PT-VW-16 and PT-VW-17. As discussed above, the piezometric level and reservoir response of these piezometers has decreased since construction of the barrier (see Figure 3-30). These decreases may be attributable to two causes (1) clogging of the leak in the seepage pathway upstream of the barrier, or (2) reducing the seepage resistance downstream of the piezometers. The former cause is beneficial to dam performance and the latter detrimental.

The second case is the behavior of Piezometer PT-VW-5 (see Figure 3-31). As discussed above, this piezometer exhibited some of the cyclic behavior as observed in PT-VW-6, independent of reservoir changes. However, after 1996 and 1999 the piezometer readings went through a transition phase and then started responding to reservoir level changes at a level a few feet above the mean previous level. This change may be the result of the
barrier defect either eroding so that an opening is always present or the barrier defect not reclosing after relieving pressure as it had during the earlier piezometer behavior.

The third case involves piezometers around the River Outlet Works (ROW). Piezometer VW-12, a non-retrievable piezometer installed in the foundation upstream of the barrier about 12 feet right of the River Outlet Works, indicated reservoir level and was quick to respond to changes in the reservoir level prior to 1996. Since 1996, there has been a progressive decrease in the response of VW-12 to reservoir level changes. From 1996 to 1999 the peak piezometer readings decreased about 10 feet. After 1999, the change in piezometer response slowed, however, this slowing may be a result of several years of low peak reservoir levels. In addition, three piezometers (VW-6, VW-7, and VW-9) located beneath the ROW to monitor the effectiveness of the grouting have shown increasing levels since 1995. These piezometers initially were reading heads approximately at the tailwater level and have since risen to about 10 feet above tailwater. It is also worth noting that a slight depression was noted in the embankment surface in the general location of Piezometer VW-12. This change in piezometer behavior may be the result of decreasing seepage resistance in the bedrock below the barrier or through the grouted section beneath the ROW.

In summary, while it appears that the seepage barrier is effective overall in reducing the seepage through the highly jointed near surface bedrock, there are several indicators that mechanisms are in effect in localized zones that are affecting the long-term seepage performance of the dam and seepage barrier. These indicators include: (1) the increased seepage flow detected from the left abutment, (2) through-barrier seepage in Zone 1 of the embankment detected in Piezometers PT-VW-5, PT-VW-6, PT-VW-16, and PT-VW-17, and (3) increased and decreased piezometric levels in the ROW area on the downstream and upstream sides of the barrier, respectively. Indicators 1 and 3 may be the result of decreasing seepage resistance in the bedrock joints below and around the seepage barrier due to one or a combination of the following: (1) removal of soil joint infill, (2) erosion of the bedrock, and (3) erosion of the soluble minerals (such as gypsum) in the joints and bedding planes. While these indicators are not felt to represent
an immediate risk to the dam, they are signs of mechanisms that are changing the seepage regime of the dam over time. Monitoring and understanding these mechanisms is important to understanding the long-term performance of the seepage barriers in this dam as well as other dams with seepage barriers.

Deformation analyses have been performed on this dam. The main purpose of these analyses is to compare the results of analyses with the well-documented deformations measured in the inclinometers installed in this barrier. A discussion and the results of these analyses are presented in Chapter 4.

**Navajo Dam**

**Background**

Navajo Dam is managed by the U.S Bureau of Reclamation and is located on the San Juan River east of Farmington, New Mexico. The dam was completed in 1963 and is a 402-foot high (crest elevation 6,108 feet) zoned earthfill embankment with a crest length of 3,648 feet (Dewey 1988, USBOR 1963). An ungated concrete spillway, the primary outlet works, and auxiliary outlet works are located in the bedrock beyond the right abutment. A plan of the dam is shown on Figure 3-32.

A cross section through the embankment is shown on Figure 3-33. The embankment has three principal zones (USBOR 1963). Zone 1 is the low permeability core and consists of selected clay, silt, sand and gravel compacted in six-inch layers. Zone 2 is the shell, which consists of sand, gravel, cobbles, and boulders compacted in 18-inch layers. Zone 3 is a miscellaneous fill zone and consisting of random material compacted in 12-inch layers. A 400-foot wide cutoff trench extending through the alluvium to the top of the bedrock was constructed beneath the core section of the dam. A single line of grout holes was constructed on 10-foot centers along a line 100 feet upstream of the crest centerline.
The foundation consists of a thin layer of alluvium overlying interbedded sandstone, siltstone, and shale bedrock (Dewey 1988, USBOR 1963). The bedrock is horizontally layered in beds ranging from thin laminae up to 20 feet thick. The sandstone is poorly to moderately cemented and has moderate to high permeability. The siltstone and shale have very low permeability. The bedding is flat lying with no folding or faulting. Stress relief cracking has formed near-vertical joints in the bedrock of both abutments. In addition, open horizontal bedding planes exist intermittently between the sandstone and shale beds. The jointing and open bedding result in open seepage paths through both abutments of the dam.

Figure 3-32 Plan of Navajo Dam (after Dewey 1988, with permission from ASCE)

Figure 3-33 Profile of Navajo Dam (modified from USBOR 1963, with permission from USBOR)
History of Seepage Problems and Mitigation

Seepage was observed in both abutments of Navajo Dam in June 1963, approximately one year after the start of initial filling (Dewey 1988). Seepage increased steadily over time and in the mid 1980s reached a total measured seepage from both abutments of about 1,800 gallons per minute. About 600 gallons per minute of the seepage was discharging from the left abutment and about 1,200 gallons per minute was discharging from the right abutment.

Seepage investigations indicated that little seepage was coming through the dam embankment (USBOR 1986, Dewey 1988). Most of the seepage was coming through a system of horizontal, open bedding planes and vertical joints in the bedrock of the abutments. The increase in flow was attributed to erosion of the bedrock and solution of gypsum layers interbedded in the bedrock. The seepage investigations concluded that there was a high probability that water seeping through bedding planes or joints along the contact between the abutments and the core material could erode the core material. It was concluded that, if left unmitigated, this mechanism could eventually lead to failure of the dam (USBOR 1986, Dewey 1988).

Design and Construction of Seepage Barrier

The seepage mitigation plan for the dam consisted of a tunnel drainage system in the right abutment and a concrete diaphragm seepage barrier in the left abutment. This study will discuss the mitigation performed in the left abutment only. The seepage barrier was designed to increase the seepage path length through the bedrock in the left abutment and prevent seepage paths from forming along the contact between the core and the abutment (Dewey 1988, USBOR 1998). The resulting barrier extends 436 feet from the left end of the dam toward the center of the dam. A profile through the left abutment showing the limits of the seepage barrier is shown on Figure 3-34. The right end of the barrier is located where the dam reaches its maximum height. To increase the seepage path length
through the abutment, the barrier was extended as deep as 180 feet into the bedrock. This resulted in a maximum barrier height of 399 feet, which, at the time of construction, was a record for seepage barriers constructed in existing dams (Davidson 1990).

The seepage barrier was constructed from May 1987 to April 1988 using the slurry trench panel method by the Soletanche company of France (Dewey 1988). Soletanche used a hydrofraise designed and built specifically for this project, taking into consideration the extreme depth of the barrier. The barrier was constructed in 40-inch wide primary and secondary elements that were 18.9 and 6.7 feet long respectively. Concrete was tremmied into place using two 10-inch pipes for primary elements and one 10-inch pipe for secondary elements. Because each pass of the hydrofraise cuts an 8-foot length, 3 to 6 inches of the primary panels are milled off the adjacent primary panels during secondary panel excavation. This was done to provide a good concrete-to-concrete bond between the panels. The minimum 28-day concrete compressive strength was specified at 3,000 psi, however, 28-day compressive strengths as high as 5,000 psi were measured (Dewey 1988).

During construction, five slurry losses occurred, with the worst resulting in the loss of 500 cubic yards of slurry (Dewey 1988). The losses occurred in the bedrock near the contact with the embankment and were not unexpected by the designers. Significant slurry losses did not occur deeper in the bedrock.

Instrumentation

The behavior and performance of the left abutment seepage barrier has been assessed and monitored since construction using piezometers, weirs, core holes in the barrier, and inclinometers (Davidson 1990, USBOR 1997, 1998, 2004). A description of each of these elements and a summary of their observed behavior is described below.

Piezometers. Thirty-eight piezometers have been used to monitor the piezometric levels around the left abutment seepage barrier (USBOR 1997, 1998). Thirteen piezometers
that were installed before the barrier remained after the barrier construction and allow for a comparison of pre- and post-barrier conditions (Davidson 1990). Immediately after the construction of the seepage barrier, there was a decrease in the piezometric levels downstream, and the piezometer responses to changes in reservoir elevation was reduced on the order of 50 percent.

The piezometers located in the bedrock downstream of the barrier indicated a significant drop in pressure shortly after the construction of the barrier. Most of the upstream piezometers located in the bedrock indicated pressures very close to the reservoir level after the barrier was constructed. This resulted in differential heads across the barrier in the bedrock of as high as 166 feet (USBOR 1998, 2002). Piezometers located in the embankment on the downstream side of the barrier showed much less drop in head soon after construction. As a result, differential heads across the barrier in the embankment were much less, with a maximum measured difference of 19.5 feet. Readings taken in
1997, 10 years after the barrier construction, indicated very little change in the pressures and differential heads in the bedrock. However, in the embankment, the 1997 readings indicated additional drops in pressure of up to 21 feet. These pressure drops resulted in the differential head across the barrier increasing to a maximum of 23 feet in the embankment (USBOR 1998).

Two piezometers located in the dam core beyond the end of the barrier stabilized at levels about 5 feet above the preconstruction water levels in the months following construction. This increase is likely due to concentration of flow in this area due to the end effects of the seepage barrier (Davidson 1990).

With a few exceptions, the piezometers indicate little change has taken place in the seepage regime since shortly after construction of the seepage barrier. One exception is 3- to 25-foot drops in the water levels in the piezometers located in the dam core downstream of the barrier. The larger of these drops occurred in piezometers that are located the furthest distance from the embankment/bedrock contact. Piezometers located in the bedrock around the barrier have not changed significantly over time since shortly after barrier construction.

Weirs. Two weirs, L-5881 and L-5730, have been used to measure the seepage flow from the left embankment since before the seepage barrier was constructed. Table 3-2 presents the data from the weirs from before seepage barrier construction, immediately after construction and 10-years after construction. All three weir measurements were taken with the reservoir at an elevation of 6050 feet.

<table>
<thead>
<tr>
<th>Weir</th>
<th>Measured Flow (gallons per minute)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L-5881</td>
<td>210</td>
</tr>
<tr>
<td>L-5370</td>
<td>597</td>
</tr>
</tbody>
</table>
It is evident from the data in Table 3-2 that the seepage barrier was effective in reducing the flows through the left abutment immediately after construction. However, readings 10 years after construction indicate the flows are increasing again. It is possible that the increase is due to other factors such as rain water or heavy snow melt. Nonetheless, this data may also indicate the effectiveness of the seepage barrier is decreasing with time.

Core Holes in Barrier. After completion of the seepage barrier, 14 core holes were drilled into the barrier to evaluate the condition of the concrete and the locations of cracks. Some of these cores were drilled at an angle in order to intercept joints between barrier panels. Sonic density logging was performed on 13 of the core holes to further evaluate crack locations and concrete density. After testing, the core holes were filled to the top with water, allowed to reach equilibrium, and the water levels recorded. The detected crack locations and equilibrium water levels are presented in Figure 3-35.

Figure 3-35 Detected crack location and water levels in seepage barrier core holes (after Davidson 1990, with permission from USSD)
The following observations were made of the cores and the core hole testing (Davidson 1990):

- Cracking was found in all of the cores. Most of the cracks were not open.
- An open crack was detected near the contact between the embankment and the bedrock near the right end of the barrier. This crack resulted in water loss during drilling.
- The one open crack detected, the largest concentration of cracks, and one of the four low velocity zones detected were all located near the contact between the embankment and the bedrock. Minor cracks were detected at this contact in 6 of the 11 cores that crossed the contact.
- Water levels in most of the core holes reached equilibrium at elevations above the water surface downstream of the wall but below the water surface level in the reservoir. However, in three of the holes the water level reached equilibrium up to 40 feet below the downstream water surface.
- The joints between panels ranged from a thin coating of bentonite to bentonite seams up to 0.25 inches wide.

**Inclinometers.** Inclinometers were installed in four of the core holes described above in October 1988, about 6 months after completion of the wall (Davidson 1990). The inclinometers were read again in October of 1989 and showed considerable movement (up to 8 inches) in the downstream direction with the maximum displacement occurring at the top of the barrier. Measurements since 1989 have shown little further movement of the barrier (USBOR 1997).

**Assessment of Seepage Barrier Performance**

Comparison of piezometer and weir readings before and after construction of the seepage barrier indicates that the barrier had a considerable influence on the seepage regime in the left abutment. The clearest evidence of the barrier effectiveness is the drop in the seepage outflow measured in the two weirs where reductions on the order of 50 percent were measured. This reduction in flow can be attributed mostly to increasing the seepage.
path length through the bedrock joints and, in doing so, reducing the hydraulic gradient. A portion of the seepage flow reduction may also be attributable to forcing the seepage deeper into the bedrock where less weathering has taken place and the joints have not been eroded. Eventual erosion along these joints should be considered as a possible mechanism for future deterioration of the seepage barrier performance.

The increase in seepage flow detected in the weirs may be attributable to changes in the seepage from the reservoir or other factors affecting the flow of water to the weirs. Increases in seepage flow may be attributable to erosion of bedrock joints and bedding planes, erosion of soil infilling in bedrock layers or solutioning of gypsum layers. Other factors that may be influencing this flow are the melting of snow in the areas above the weirs or annual differences in seasonal rainfall.

The piezometric levels in the bedrock downstream of the wall dropped considerably when the barrier was constructed as did the response to reservoir fluctuations. Piezometric levels in the embankment downstream of the barrier remain high, in part due to the slow drainage of the low permeability core material and in part due to seepage around the end of the barrier. Due to the higher permeability and lower head in the bedrock, it appears that the bedrock is acting as a drain for the water seeping in the embankment core (USBOR 1998). Because there is no filter at this interface, the possibility of erosion of the embankment into bedrock joints should also be considered a possible mechanism for future deterioration of seepage performance.

Based on the differential water pressures developed across the seepage barrier in the bedrock, it is apparent that the barrier is very effective with respect to flow through the barrier. This effectiveness has remained essentially constant since the barrier was constructed. This performance suggests the apertures of the cracks detected in the barrier soon after construction remain small.
Virginia Smith Dam

Background

Virginia Smith Dam (formerly known as Calamus Dam) is managed by the U.S. Bureau of Reclamation and is located on the Calamus River about 6 miles northwest of Burwell, Wyoming. The dam was constructed between 1980 and 1985. The 127,400 acre-feet reservoir formed by the dam is used primarily for irrigation water storage with secondary uses including recreation, wildlife habitat, and flood control.

The dam is a 7259-foot long zoned earth embankment and has a structural height of 96 feet with a crest elevation of 2259 feet (USBOR 2003a). The upstream slope is 3:1 (horizontal to vertical) and the downstream slope is 2.5:1 (horizontal to vertical) and has a bench midheight. There is a “morning-glory” drop inlet spillway that leads to a circular conduit below the central portion of the embankment (at Station 51+50). The outlet works are located in the right abutment and consist of the following listed in order from upstream to downstream: a 10-foot diameter steel lined concrete conduit, an emergency gate located below the dam crest, a 9-foot diameter steel pipe, and two high-pressure slide gates to regulate flow into either an irrigation canal or the Calamus River.

The dam is located in an area of sand dunes through which the Calamus River has cut a wide shallow valley. The foundation conditions are a mixture of eolian sand dune deposits and fluvial deposits up to 120 feet thick (USBOR 1995). The site geology has been divided into eight units that are shown on a generalized geologic cross section along the dam axis in Figure 3-36. The units are comprised of the following:

- Unit 1 – Poorly graded dune sand,
- Unit 2 – Silt and interbedded silt and fine sand,
- Unit 3 – Peat, organic silts and lean clays (slack water deposits),
- Unit 4 – Recent Calamus River deposits (river channel and marshy deposits),
- Unit 5 – Interbedded poorly graded fine sand and silty sand,
- Unit 6 – Coarse to fine sand containing varying amounts of gravel,
• Unit 7 – Lower fine sand and silt, and
• Unit 8 – Ogallala Formation – fine-grained sandstone and siltstone.

Figure 3-36 Generalized geologic section along the longitudinal axis of Virginia Smith Dam (modified from USBOR 1995, with permission from USBOR)

A cross section of the dam is presented on Figure 3-37. The embankment consists of a central core (Zone 1) that is specified as a mixture of sand, silt, and clay with a minimum of 50 percent fines, compacted in 6-inch lifts. The shells of the dam are specified as Zone 2 or 2A material which consists of a mixture of sand, silt, and clay with a maximum of 20 percent fines compacted in 12-inch lifts. Downstream of the core and along the base of the embankment the material is specified to be Zone 2 (less than 10 percent fines). The Zone 2 material acts as a transition zone between the core and the chimney and blanket drain. Depending on the type of surface soils encountered, the base of the core or the base of the entire dam was extended downward as a foundation trench. The vertical location of the base of the foundation trench is presented in Figure 3-36.

Figure 3-37 Cross section of Virginia Smith Dam (modified from USBOR 2003a, with permission from USBOR)
An 8-foot thick seepage blanket of Zone 1 material extends from the core 800 feet upstream of the dam centerline. At a location 300 feet from the dam centerline, a soil-bentonite seepage barrier extends down from the seepage blanket. Between the right abutment and Station 43+00 the seepage barrier is 5 feet thick and extends down to the top of the Ogallala Formation (see Figure 3-36). From Station 43+00 to the left abutment the seepage barrier is 3 feet thick and extends to a depth of 45 feet.

To reduce hydraulic gradients and uplift pressures downstream, 43 relief wells were installed (USBOR 2003a). Twenty relief wells were installed along the downstream toe of the dam, 11 surrounding the river outlet works stilling basin, eight surrounding the spillway stilling basin, and four around the river outlet channel. The wells consist of 18-inch holes with 8-5/8-inch stainless steel casing and a graded gravel pack in the annular space.

**Design and Construction of Seepage Barrier**

The seepage barrier was designed to reduce the amount of seepage beneath the dam as well as to intercept any high permeability channels that may exist in the near-surface alluvium that could result in concentrated seepage pathways near the top of the foundation (McAlexander and Egemoen 1985, USBOR 2003a). Along the right side of the foundation, where recent Calamus River deposits (Unit 4) and coarse sand (Unit 6) exist, the barrier was constructed through all of the soft sediments and embedded into the bedrock. Along the left side of the foundation, where the deeper sediments consisted of fine sand and silty sand (Unit 5), the trench was constructed to a depth of 45 feet, as noted above.

The width of the trench was designed to limit the hydraulic gradient across the wall (USBOR 1981a). Based on a maximum gradient of 32 and a factor of safety of 4.0 (recommended by Xanthakos (1979) to prevent blowout of the barrier fill into the adjacent soil) and the results of seepage analyses, the required width of the barrier was
designed to be 5 feet for the right (fully penetrating) portion of the barrier and 3 feet for the left (partially penetration) portion.

The seepage barrier was constructed using a drag bucket and a clamshell excavator and the slurry trench technique (USBOR 1995, 1981a, 1981b). The open trench was maintained full of water-bentonite slurry in order to provide stability. The trench backfill was designed to provide moderate seepage resistance \((1 \times 10^{-6} \text{ cm/sec})\) and to minimize the consolidation potential. In order to achieve a balance of performance the backfill was designed to contain 15 to 30 percent fines mixed with sand and up to 40 percent gravel. The in-situ sandy soils were used as the base for the backfill and soils from borrow areas were added to increase the fines and gravel contents. The materials were mixed on the ground surface using a bulldozer and then pushed into the trench to form the wall.

In order to avoid developing a gap between the top of the seepage barrier and the seepage blanket when the barrier backfill settled, the barrier was constructed after half of the seepage blanket had been constructed (USBOR 1981b). After the seepage barrier was complete, the upper half of the seepage blanket was constructed. The seepage blanket fill was placed at a water content 2 percent wet of the optimum water content to provide ductility and better maintain the connection.

**Instrumentation**

Monitoring instruments installed during and shortly after dam construction that are relevant to the performance of the seepage barrier consist of: (1) toe drain flow monitoring, (2) relief well flow monitoring, (3) piezometers, and (4) dam embankment measurement points (USBOR 2003a). Details of each of type of instrumentation are described below.

**Toe Drain Flow Monitoring.** Three V-notch weirs were set up to measure flows emitting from the toe-drain (USBOR 2003a). However, toe-drain flows to date have been non-existent, apparently due to the effectiveness of the relief well system.
Relief Well Flow Monitoring. Flows from the 43 relief wells were monitored by diverting flows through a system of V-notch weirs and rectangular flumes (USBOR 2003a). With a few exceptions, the relief well monitoring has measured modest flows that have remained reasonably constant over the life of the dam. As would be expected, the flows show correlation with fluctuations in the reservoir level. The major exception to the consistency of flow over time was experienced in measurements at SM-52R and SM-41R, which measure flows emitting from wells at the downstream toe to the left of the spillway and between the spillway and Station 41+00 (near the center of the dam). Both of these measurement points experienced a decline in flow between 1991 and 1997, and then returned to near their pre-1991 levels.

Piezometers. At the end of construction 16 vibrating-wire piezometers were installed to measure water pressures immediately upstream and downstream of the seepage barrier at four locations (USBOR 2003a). In the years immediately following construction, these piezometers indicated piezometric head drops across the barrier of 10 to 40 feet in three locations where the barrier was extended down to the Ogallala Formation, and very little head drop where the barrier did not extend to the Ogallala Formation. Unfortunately, these piezometers were not installed with adequate lightning protection, and were all rendered inoperable by lightning strikes within a few years after construction. Thus, long-term changes in the areas immediately around the seepage barrier cannot be assessed.

The remaining piezometers consist of 48 pneumatic piezometers and 38 porous-tube piezometers in the dam embankment and foundation, and 70 porous-tube piezometers and 10 observation wells located in the abutments and downstream areas. The majority of the piezometers in the embankment, foundation, abutments, and downstream area have a reasonably constant behavior pattern since the construction of the dam (USBOR 2003a). The few exceptions to the constant behavior are discussed below.
Comparison of the embankment piezometers with the foundation piezometers indicates that the piezometric heads in the embankment are higher than in the foundation (USBOR 2003a). This is attributed to the effectiveness of the seepage blanket, seepage barrier, toe drain, and relief well system.

Piezometers located in the core of the embankment indicated high water pressures much sooner after initial filling than would have been expected, and unusually high pressures developed in the downstream portions of the core (USBOR 2003a). These readings have remained high since first filling. The Bureau of Reclamation (2003a) attributes these high pressures to lower than expected seepage resistance. It is likely that the high pressures may be a result of hydraulic fracturing of the embankment similar to that described by Sherrard (1986) in Stockton Creek Dam. Analyses indicate the high pressures do not represent a risk to embankment stability and the filter is expected to prevent internal erosion of the core.

Several piezometers in the embankment and foundation have experienced some anomalous behavior over the life of the dam (USBOR 2003a). Rises and drops of up to 15 feet were recorded in piezometers before the piezometers returned to the previously recorded water levels. This behavior has been observed in both embankment and foundation piezometers. The durations of these changes have ranged from a few months to several years. In a few cases the abrupt changes have not returned to the original levels.

**Settlement Monuments.** Settlement monuments consist of 14 base plates installed at the embankment foundation interface and 66 embankment surface measurement points (USBOR 2003a). The base plates indicate that most of the foundation consolidation took place during embankment construction and little settlement has occurred since. A maximum settlement of 0.89 feet was recorded during construction and a maximum settlement of 0.16 feet has been recorded since construction. The embankment surface measurement points also indicate very small settlements since construction with a maximum recorded settlement of 0.14 feet.
Assessment of Seepage Barrier Performance

The initial readings of the vibrating-wire piezometers indicate that the fully penetrating portion of the seepage barrier was effective in reducing piezometric heads across the barrier, while the partially penetrating seepage barrier was largely ineffective in this regard. It is unfortunate that these piezometers were destroyed by lightning strikes as they would have been a highly efficient way of tracking changes within the seepage barrier itself. However, the remaining seepage flow and piezometric data indicate that the overall performance of the seepage control elements of the dam (the seepage blanket, seepage barrier, toe drain, and relief wells) has not changed significantly over the life of the dam. Isolated perturbations in a small percentage of the instruments have been recorded that may be the result of localized changes in seepage resistance either upstream or downstream of the measurement points. These changes may be due to redistribution of fines within the soil matrix (a process often referred to as suffusion) that results in a higher or lower permeability along the critical seepage path. The locations where such changes take place are impossible to ascertain and, thus, it is not possible to discern whether these perturbations are associated with the seepage barrier performance or another portion of the dam.

The dam is performing exceptionally with regard to settlement. Thus, there is no indication of removal of sediments from below the dam.

Seepage analyses were performed on the dam and seepage barrier in order to assess the effectiveness of the barrier and effective permeability of the barrier backfill. Details of these analyses are presented in Chapter 4.
New Waddell Dam

Background

New Waddell Dam is owned by the U.S Bureau of Reclamation and is operated by Central Arizona Water Conservation District and Maricopa County Water District (USBOR 2000a). The dam is located on the Aqua Fria River Northwest of Phoenix, Arizona, and retains a pump-storage power generation reservoir that uses lower-cost electricity to fill the reservoir from October to March, and releases the water during peak power periods during the rest of the year. The dam was constructed between 1987 and 1992 to replace the original Waddell Dam, a concrete arch structure located approximately one-half mile upstream from the new dam. The original dam was breached upon completion of New Waddell Dam. New Waddell Dam is a 4,800-foot long earth and rockfill embankment with a crest elevation of 1,728 feet and a maximum height of 440 feet. The dam has two spillways: an ungated service spillway with a crest elevation of 1,706.5 feet and a concrete sill and fuse plug spillway with a fuse elevation of 1,714 feet.

Two outlet works control the reservoir operation (USBOR 2000a). The River Outlet Works in the right abutment is used primarily for emergency releases and regulation of the pool level, and the Central Arizona Project (CAP) outlet works that carries water through the right abutment to the pump/generating plant. The River Outlet Works discharges into Hank Raymond Lake which is located on the downstream side of the dam embankment and comes in contact with the lower portions of the downstream toe. The CAP Outlet operates in both directions, pumping water upstream to fill the reservoir and releasing water to generate electricity.

The dam is constructed on a geologically complex foundation consisting of intrusive and extrusive volcanic rock overlain with conglomerate and varying amounts of old and recent alluvium (USBOR 2000a, 2002b). The stratigraphy of the bedrock is an assemblage of interlayered deposits of tuff, andesite, and rhyolite that is overlain by
varying thicknesses of conglomerate. Jointing in the conglomerate consists of near vertical joints oriented perpendicular to the dam axis along with pervasive bedding plane joints. Joints in the volcanic rocks are randomly oriented. The joints vary from tight to open and are filled with a variety of fine-grained soil, calcium carbonate, and carbonate cemented sandy soils.

Foundation preparation for the dam consisted stripping of surface deposits, extensive grouting of the bedrock, and construction of two seepage barriers (USBOR 2000a). The limits of the various foundation treatments are shown on the longitudinal section presented in Figure 3-38. The alluvium was stripped beneath the entire dam footprint (with the exception of where seepage barriers were constructed) and weathered conglomerate was stripped from beneath the dam core. The foundation surface was treated by blanket grouting the area beneath the dam core to a depth of about 30 feet. A single-line 150-foot to 300-foot deep grout curtain was constructed beneath the dam centerline along the entire length of the foundation except for where deep alluvial channels where cut off by seepage barriers (described below). Between Stations 10+00 and 22+00 the grout curtain was increased to a triple line grout curtain because more frequent joints were found in that area. In the areas of the tunnel portals and the pump-generating plant, where the seepage path through the foundation has been reduced due to the excavations, the grout curtain was increased to five lines. In two locations along the alignment deep, alluvial-filled channels exist that were deemed too deep to dewater for removal and replacement of the alluvial soils. In these locations, concrete seepage barriers were constructed down into the volcanic bedrock. Details of the seepage barriers are presented below.

The embankment is a zoned earth structure with a central core, a one-stage upstream filter, a two-staged downstream filter and blanket drain, and outer shells (USBOR 2000a). A cross section of the embankment is shown on Figure 3-39. The core consists of clay, clayey sand, silt, and silty sand compacted from 12-inch loose layers. The lower portion of the core consists of select erosion-resistant soil consisting of clay, clayey sand, or clayey gravel. The foundation surface beneath the core was treated with dental
concrete and slush grouting. The shells consist of sand, gravel, cobbles and boulders compacted from 3-foot loose layers.

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**Figure 3-38** Longitudinal section showing foundation treatments for New Waddell Dam (after USBOR 1999, with permission from USBOR)

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**Figure 3-39** Cross section of New Waddell Dam embankment (After USBOR 1999, with permission from USBOR)

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1 – Selected CL, SC, SM, and ML materials compacted by temping rollers from 12” loose layers
2 – Processed SP materials compacted by vibratory smooth drum rollers from 12” loose layers
2A – Processed SP and GP materials compacted by vibratory smooth drum rollers from 18” loose layers
2B – Selected CL, SC and GC materials compacted by temping rollers from 12” loose layers
3 – Processed GP materials compacted by vibratory smooth drum rollers from 18” loose layers
4 – Selected materials from required excavation compacted by vibratory smooth drum rollers from 3’ loose layers
5 – Sand, gravel, cobbles, and boulders from borrow compacted by vibratory smooth drum rollers from 3’ loose layers
6 – Selected rockfill materials from required excavation compacted by vibratory smooth drum rollers from 3’ loose layers
Design and Construction of Seepage Barriers

The two seepage barriers were constructed from 1987 to 1989 during the early stages of construction of New Waddell Dam (USBOR 2000a). The smaller barrier was constructed to a depth of about 85 feet and has a total area of about 5,500 square feet. The larger barrier has a maximum depth of about 180 feet and a total area of about 50,000 square feet. The barriers extend about 10 feet into bedrock and rise about 20 feet into the dam core. The locations of both barriers are shown on Figure 3-38.

The barriers were excavated in 20-foot long sections and supported with bentonite-water slurry until backfilled (USBOR 2000a). Excavation was accomplished using a conventional clamshell supplemented with a chisel where boulders were encountered and to embed the barriers into the bedrock. Blasting was also used to assist in removal of large boulders and rock blocks.

The panels were backfilled with plastic concrete through tremie pipes (USBOR 2000a). Sleeve pipes were installed in each panel joint to allow for water testing and grouting. The joints were tested for water tightness by pressurizing water in the pipe sleeves. Following testing grout was pumped down the pipe sleeves to provide a better seal between panels.

During excavation of the larger barrier, an alluvium-filled cave was discovered in the right sidewall of the channel (USBOR 2000a). The cave was about 110 to 125 feet high and extended back from the channel wall about 30 feet. The seepage barrier was extended to cut off the cave by constructing overlapping 24-inch diameter secant piles for the full length of the cave. The secant piles were drilled using a crane-mounted down-hole hammer and tremie backfilled with concrete.
Instrumentation

The New Waddell Dam embankment and foundation are extensively instrumented to monitor embankment movements, pore pressures, and seepage volumes (USBOR 2000a). The instrumentation pertinent to this study consists of 78 surface movement measurement points, seven inclinometers, nine V-notch weirs monitoring outflows from the toe drain, eight additional seepage locations measured by bucket and stopwatch, 69 vibrating wire piezometers, and 24 porous tube piezometers. Most of the piezometers are arranged in six cross sections designed to monitor specific areas of interest along the dam. The two instrumentation cross sections pertinent to this study are C and D, which monitor the conditions along cross sections containing the small and large seepage barriers.

The embankment movement measurement points are arranged in four lines parallel to the dam axis, one each on the upstream and downstream edges of the crest, one 200 feet downstream of the centerline, and one 400 feet downstream of the centerline (USBOR 2000a). Monitoring of these points has indicated that the dam is experiencing settlements of a magnitude expected for this size of structure. Maximum crest settlements are on the order of 1 foot or about 0.6 percent of the embankment height. No extraneous settlements have been recorded by the monitoring.

One of the inclinometers installed in the embankment is located several feet downstream of the large seepage barrier (USBOR 2000a). Deflections in this inclinometer have occurred mostly in the embankment above the seepage barrier. Foundation deformations have been negligible.

Ten vibrating wire piezometers are installed in cross section C, the cross section that incorporates the smaller seepage barrier (USBOR 2000a). Two piezometers are located in the lower dam core, one in the blanket drain, and seven located in what was thought to be the foundation alluvium downstream of the barrier. All but two of the foundation piezometers indicate tailwater elevations. The two exceptions show slight but delayed responses to reservoir level changes. The delayed response of these piezometers is
evidence that these piezometers are measuring hydraulic heads in bedrock rather than alluvium. Because the water pressure upstream of the seepage barrier is expected to be near the reservoir level due to the direct connection of the alluvium with the reservoir, the delayed response in these piezometers is believed to indicate seepage through bedrock joints rather than leakage through the barrier that would have a more immediate reservoir response.

Seventeen vibrating wire piezometers are installed in Cross Section D, the section that incorporates the larger seepage barrier (USBOR 2000a). Two piezometers are located in the lower core, one in the blanket drain, and the remaining 14 in the foundation alluvium downstream of the barrier. All of the foundation piezometers indicate tailwater elevations. The core piezometer located furthest upstream in this cross section became hydraulically connected with the reservoir faster than core piezometers on other cross sections. This is believed to have occurred due to hydraulic heads upstream of the seepage barrier that are close to the reservoir pool elevation.

The weirs measuring seepage water exiting the toe drain have only detected seepage water from the left abutment area (USBOR 2000a). Seepage from the areas near where the seepage barriers are located likely exits below the level of Hank Raymond Lake at the toe of the embankment and, thus, cannot be measured or observed. Other seeps that are monitored are in the abutments and in the switchyard excavation beyond the toe of the dam and are not relevant to the assessment of the seepage barrier performance.

Assessment of Seepage Barrier Performance

The vibrating-wire piezometer data indicate that the seepage barriers are performing very well. While no piezometers are located upstream of the barriers, the high permeability of the alluvium and the connectivity of the alluvium-filled channel with the reservoir would suggest that the hydraulic heads upstream of the barriers is near the reservoir level. Thus, with the downstream hydraulic heads close to tailwater elevations, the seepage barriers appear to have head efficiency values of close to 100 percent.
The high head efficiency of the barriers is likely more a function of the high permeability of the alluvial soils than it is the low transmissivity of the barrier. Because any water leaking through the barrier will quickly drain through the alluvium, small amounts of barrier leakage (similar to what has been detected in other dams) would be essentially impossible to detect as pore pressure increases. Thus, while the barriers appear to be performing their intended purpose well, it is not feasible to assess the amount of leakage that may be occurring through minor defects and cracks in the barriers.

**Beaver Dam (Dike 1)**

**Background**

Beaver Dam is managed by the U.S Army Corps of Engineers, Little Rock District and is located on the White River about 6 miles northwest of Eureka Springs, Arkansas. The dam was constructed between 1960 and 1966 and consists of a combination concrete gravity and earth embankment main dam and three saddle dikes. The gated spillway and hydroelectric power station are located on the main dam. The top elevation of the flood control pool is 1,130 feet and the maximum pool elevation is 1,137 feet. Dike 1, the subject of this assessment, is located adjacent to the north end of the main dam.

Dike 1 is a 740-foot long earth embankment with a structural height of 62 feet and a crest elevation of 1,142 feet (USACE 1995, Bruce and Stefani 1996). The upstream and downstream slopes have inclination of 2.5:1 (horizontal to vertical). A cross section of the dike is shown in Figure 3-40. The original design for the dike called for it to be a zoned embankment with more permeable random fill materials comprising the shells. However, due to an abundance of clay material available on the site, the dam was constructed essentially as a homogenous embankment. A key trench was excavated to a depth of at least 5 feet below the ground surface, and a grout curtain of variable depth was constructed from the base of the key trench. It can be seen from Figure 3-40 that
the base of the embankment below the centerline of the dike is only about 15 feet below
the flood control pool elevation, resulting in very little hydraulic head acting on the dike.

Figure 3-40  Cross section of Beaver Dam, Dike 1 (modified from USACE 1995, with
permission from USACE)

The foundation conditions for Dike 1 are distinguished by a graben feature that is
responsible for the seepage problems encountered at this location (USACE 1995). A
profile of the foundation geology is shown in Figure 3-41. Two northeast trending faults
traverse the site near the north end of Dike 1 (Station 63+00 to 64+00) and the north end
of the main embankment (Station 73+00 to 74+50). The interior of these faults has been
downthrown over 200 feet. This movement has resulted in large areas of disturbed
material and en echelon faulting particularly in the south fault zone. Some of the faulted
material was healed as breccia, but in some areas the material remained broken and open,
leaving conduits for water flow. The interior of the graben consists of younger
(Mississippian aged) Boone limestone that is susceptible to solutioning, while the
bedrock outside of the graben areas consists of Ordovician aged dolomitic limestone and
Devonian aged shale that are not as susceptible to solutioning. The Boone limestone is
highly solutioned along joints and faults, resulting in very low seepage resistance within
the graben. Outside of the graben area (along most of the alignment of the main dam
embankment) the bedrock is much tighter and has much greater seepage resistance.

History of Seepage Problems and Mitigation

During the initial filling of the reservoir, seepage totaling about 800 gpm was detected
from five areas downstream of Dike 1 (USACE 1995). In response to this seepage,
remedial grouting was performed between 1968 and 1971 to depths of up to 200 feet
below the dam crest. This grouting was only minimally effective, reduced the total seepage flows to about 500 gpm. However, between 1971 and 1984 seepage continued and two new seepage areas developed.

![Generalized geologic cross section along Beaver Dam, Dike 1 (modified from USACE 1995, with permission from USACE)](image)

In 1984, the pool of record (elevation 1,130.4 feet) was recorded. During and shortly after this event, three additional seepage areas developed (USACE 1995). This event prompted initiation of a seepage investigation that consisted of instrumentation and measurement of the seepage volumes and piezometric pressures. This investigation lasted several years and concluded with the recommendation to proceed with design and construction of a seepage barrier.

**Design and Construction of Seepage Barrier**

The seepage barrier was constructed 65 feet upstream from the centerline of Dike 1 between Stations 62+00 and 76+75. The barrier was constructed through a working platform built for the barrier construction. The 1,475-foot long barrier extends from about 150 feet north of the trace of the northern fault, through the graben area, and about 350 feet south of the trace of the southern fault. This resulted in the seepage barrier
extending beyond the Dike 1 embankment at both ends, and extending onto the northern end of the main dam embankment.

During the design investigation for the seepage barrier 28 borings were drilled along the seepage barrier alignment to investigate rock strength and seepage resistance (USACE 1995). Unconfined compression tests were performed on rock cores to assess excavatability of the rock. Downhole pressure tests (packer tests) were performed in the borings to assess the seepage resistance of the bedrock. The base elevation of the barrier was designed based on the results of the rock cores and pressure tests so that the barrier would intercept any zones of low seepage resistance or poor quality rock. The resulting design called for barrier depths ranging from 80 to 185 feet. A longitudinal section showing the extent of the secant pile seepage barrier is shown in Figure 3-42.

In 1990 construction of the seepage barrier was started using a hydrofraise rock mill (USACE 1995). This method proved ineffective for excavating the sound bedrock and, due to contract problems, the construction was halted and the contract terminated. In 1992 a second contract was awarded to construct a secant pile seepage barrier. The secant-pile design consisted of drilling 34-inch diameter holes on 24-inch centers as shown on Figure 3-43. If perfectly aligned, this specification would result in a contact...
length of 24 inches between adjacent piles. However, a minimum 18-inch contact length was specified to allow for a small amount of misalignment between piles. The secant-pile seepage barrier was constructed between April 1992 and December 1994.

**Instrumentation**

Instruments to measure the seepage and piezometric pressures downstream of Dike 1 were installed soon after the first seepage was observed during initial filling (USACE 1995). A temporary weir was installed in 1966 to measure the seepage from the areas that developed during initial filling. In 1967, a permanent Parshall Flume was installed to collect the seepage from all of the seepage areas that existed at that time. In 1985, a second Parshall Flume was installed to monitor the additional seepage areas that developed during the record pool event in 1984.

![Figure 3-43 Specified configuration for adjacent secant piles at Beaver Dam](image)

Between 1968 and 1972, 26 piezometers were installed to monitor piezometric heads in the overburden, weathered rock and sound rock (USACE 1995). Thirty additional piezometers were installed in 1985 to bring the total number of piezometers to 56. At the time of construction, 50 piezometers were being monitored. In 2001, a total of 30 piezometers were still being monitored. The piezometers were connected to an automated data collection system in 1987 which continues to collect data on a daily basis.
Assessment of Seepage Barrier Performance

Prior to construction of the seepage barrier all of the piezometers in Dike 1 and the downstream area were showing direct and immediate response to changes in the reservoir level, and seepage flows from 10 areas downstream from Dike 1 were emitting close to 500 gpm (USACE 1995). As the construction of the seepage barrier progressed across the Dike, the water levels and reservoir response in piezometers dropped dramatically and flows from seepage areas decreased. The effects of the seepage barrier were observed in the piezometers and seepage areas at different times as construction of the seepage barrier intercepted the flow paths. Piezometer drops as large as 40 feet were recorded during and soon after the barrier construction period. By the end of construction all of the seepage areas above elevation 1,040 feet had ceased to flow.

Between 1995 and 2001 the levels of the piezometers continued to drop slowly (USACE 1996, 2001). This continued drop is likely due to continued drainage of the area below Dike 1. About half of the piezometers continue to respond to reservoir fluctuations, although the magnitude of the response is much lower that before construction of the seepage barrier.

The data described above indicate that the Beaver Dam Dike 1 seepage barrier was highly effective in intercepting the seepage pathways responsible for the seepage problems experienced during and after the initial filling of the reservoir. The immediate effectiveness can be largely attributed to the effort made to identify the depths at which seepage pathways existed, and construction of the barrier past these depths. The favorable long-term performance of the barrier is likely attributable to extending the barrier into sound bedrock where the potential for erosion of soil infill from solution voids and faults is low. Although seepage is still occurring in the area, as evidenced by the piezometer response to the reservoir, it appears that the deeper seepage pathways do not contain erodible material, or the hydraulic gradients are low enough so that significant erosion of this infill has not occurred.
**Meeks Cabin Dam**

**Background**

Meeks Cabin Dam is managed by the U.S Bureau of Reclamation and is located on the Blacks Fork River, 22 miles southwest of Fort Bridger, Wyoming. The dam was completed in 1970 and is a 174-foot high (crest elevation 8,706 feet) zoned earthfill embankment with a crest length of 3,200 feet (USBOR 2003b). An uncontrolled concrete spillway with a crest elevation of 8686 feet is located near the center of the embankment. The outlet works runs below the dam 500 feet to the right of the spillway and consist of a 9.5 foot horseshoe-shaped concrete conduit containing a 62-inch diameter steel pipe. A plan of the dam is shown in Figure 3-44.

![Plan of Meeks Cabin Dam](after USBOR 2003b, with permission from USBOR)

Figure 3-44  Plan of Meeks Cabin Dam (after USBOR 2003b, with permission from USBOR)
The dam foundation consists of interlayered deposits of glacial till and glacial outwash overlying the Bridger Shale formation. Minor amounts of recent alluvium exist at the ground surface near the valley bottom. A longitudinal section along the left abutment of the embankment where the seepage barrier was constructed in 1992 is shown in Figure 3-45. The alluvium (Qa1), which was removed beneath the dam core, consists primarily of granular soils ranging from sand to boulders with minor amounts of silt. Three units of glacial till (Qt1, Qt2, and Qt3) exist in the foundation, interbedded with two units of glacial outwash (Qg1 and Qg2). The glacial till deposits consist of varying amounts of clay, silt, sand, gravel, cobbles, and boulders. The youngest glacial till deposit (Qt3) is medium dense and has moderate to low permeability. The glacial till deposits increase in compaction and decrease in permeability with age. The oldest glacial till (Qt1) is very compact and has low permeability. The glacial outwash deposits consist of silt, sand, gravel, cobbles and boulders, and have high to very high permeability. The Bridger Shale Formation consists of tuffaceous siltstone and clay shale and exhibits low permeability.

Figure 3-45  Longitudinal section through the portion of Meeks Cabin Dam with a seepage barrier (after USBOR 2003b, with permission from USBOR)
A cross section through the dam is shown on Figure 3-46. The embankment is made up of four zones: the core (Zone 1), the upstream shell (Zone 2), the upstream toe berm (Zone 3), and the downstream rock fill shell (Zone 4) (USBOR 2003b). Zone 1 consists of a low permeability mix of silt, sand and gravel compacted in 6-inch layers. Zone 2 consists of sand, gravel, and cobbles compacted in 12-inch layers. Zone 3 consists of miscellaneous materials obtained from necessary excavations for dam construction and is placed in portions of the upstream toe in 12-inch layers. Zone 4 consists of cobbles and boulders and was dumped in three-foot layers. The upstream slope of the dam is inclined at 3 horizontal to 1 vertical and is armored with a 3-foot thick layer of riprap. The downstream slope is inclined at 2 horizontal to 1 vertical from the crest to elevation 8,650 feet, 2-½ horizontal to 1 vertical between elevations 8,650 and 8,590 feet, 4 horizontal to 1 vertical between 8,590 and 8,555 feet and 2 horizontal to 1 vertical from 8,555 feet to the stream bed. A 10-inch sand and gravel blanket drain leading to a toe drain was constructed beneath the downstream shell.

Figure 3-46  Cross section of Meeks Cabin Dam near Station 20+00 (after USBOR 2003b, with permission from USBOR)
The original embankment design did not include filters between the various zones (USBOR 2003b). Locations of note where filters were not constructed include: (1) between the Zone 1 material and the shells (Zones 2 and 4), (2) between the Zone 1 material and course grained foundation soils, and (3) around the toe drain pipes to prevent migration of sand into the drain.

A 50-foot wide cutoff trench was constructed beneath the core of the dam (see Figure 3-46). While the cutoff trench was originally intended to provide a cutoff down to the Bridger Shale Formation in the center of the valley and tight glacial till (Unit Qt1) in the abutments, construction records and subsequent subsurface investigations indicate that considerable amounts of glacial till and outwash deposits exist in the foundation between Stations 17+50 and 25+25 (USBOR 2003b). No foundation grouting was performed.

History of Seepage Problems and Mitigation

In June 1971, soon after completion of construction and initial filling of the reservoir, increased flow from natural springs and slope instability occurred in a 600-foot long section of the left downstream abutment (see Figure 3-44 for location) (USBOR 2003b). Seepage flows were measured at 235 gallons per minute with the reservoir at elevation 8,666 feet. From September to November 1971, a series of 27 1-½-inch diameter horizontal drains ranging in length from 25 to 90 feet were installed in the abutment in the area of instability. All but one of the horizontal drains produced water. After completion of the drains, the slope dried up and the instability ceased.

By 1984, the seepage and slope instability had redeveloped and had spread to the left groin of the dam (USBOR 2003b). Seepage flows were measured at 435 gallons per minute with the reservoir at 8,675 feet. A second set of 23 horizontal drains were installed in the abutment and the area substantially dried up and the instability again ceased.
In 1985, a collection system was installed to collect the seepage water from the two sets of horizontal drains. When the pipes from the 1971 set of drains were cut, it was noted that the pipes were filled with fine to medium sand and roots.

From 1971 to 1980 a small sinkhole was observed on the upstream slope of the dam at Station 24+50. The sinkhole was observable every time the reservoir level dropped below 8,670 feet. It was noted that the sinkhole enlarged over time, and dye tests indicated direct connection with the horizontal drains in the left downstream abutment. From 1980 to 1987 the number of sinkholes in this area increased to 12.

In 1986 three Parshall Flumes were installed to monitor seepage flows from the outlets of the horizontal drain outlet system (see location of Flumes 1 through 3 on Figure 3-44). The seepage volume measured in these flumes was found to steadily increase between 1987 and 1991, with the largest increases occurring in Flume 1, which is the closest to the left groin. Additionally, sand accumulated in the stilling basin below Flume 2 each year between 1987 and 1991.

Between 1986 and 1991, investigations were conducted to evaluate the seepage in the left abutment. These investigations concluded that the two deposits of glacial outwash located between Stations 17+50 and 25+25 were the source of the seepage problems, and recommended that a seepage barrier be constructed to cut off seepage through these deposits.

**Design and Construction of Seepage Barrier**

The seepage barrier at Meeks Cabin Dam was constructed between September 1993 and September 1995. The barrier was designed to block seepage flow through the two glacial outwash deposits located in the left abutment (Gagliardi and Routh 1993, USBOR 1992, 2003b). The design called for construction of a 3-foot wide plastic-concrete barrier extending a minimum of 10 feet into the lower glacial till deposit (Qt1). The barrier was constructed 14 feet upstream from the dam crest centerline between Stations 17+25 and
25+50. The limits of the seepage barrier are shown on the plan in Figure 3-44 and the longitudinal section in Figure 3-45.

Prior to construction of the barrier, the top 6 feet of the dam was removed to provide a working platform. Guide walls were constructed on the working platform to help align the barrier excavation. The upper portion of the barrier in the dam embankment was excavated using two 3-foot wide, crane-supported hydraulic clamshell excavators while the glacial till and outwash deposits were excavated with a Bachy BC-30 cutter (hydrofraise) with carbide tipped tooth-style bits or diamond roller bits. The excavators were capable of excavating a 9.2-foot long section of wall in a single pass. The excavations were supported with bentonite slurry until tremie backfilled with plastic concrete.

Primary panels were 23 feet long. The ends were excavated first, and the remaining material from the center was excavated last. A space of 8.2 feet was left between primary panels. After sufficient curing of the plastic concrete in the primary panel, the secondary panels were excavated with the cutter excavating about 6 inches into each of the adjacent primary panels.

After excavation the panels were cleaned and checked for depth and verticality and the slurry was desanded. Plastic concrete was placed in each panel from the bottom up using a 10-inch diameter tremie pipe. The plastic concrete was specified to have a minimum 28-day compressive strength of 300 psi with a preferred range of 385 to 415 psi. Thirty percent of the plastic-concrete samples taken met the preferred criteria, 31 percent exceeded the preferred criteria, and 46 percent fell below the preferred criteria. Seven percent of the samples tested fell below the 300 psi minimum compressive strength criteria.
Instrumentation

Seepage through the left abutment of Meeks Cabin Dam has been monitored since 1971, when the first seepage problems were observed. In 1986, three Parshall Flumes were installed to enhance the ability to monitor seepage volumes through the left abutment areas. Piezometers were installed in the left abutment area in 1986 through 1991. Surface deflections have been monitored since construction by an array of surface monuments. Descriptions of each of these elements and a summary of their observed behavior are described below.

Survey Monuments. Thirty surface survey monuments have been monitored on the embankment since the end of dam construction (USBOR 2003b). The monuments are arranged in five lines located: (1) 180 feet upstream, (2) 20 feet upstream from the centerline, (3) 20 feet downstream, (4) 137 feet downstream, and (5) 285 feet downstream. The maximum settlement on the dam crest is about 0.7 feet and the maximum lateral deflection is 0.2 feet. Settlements and deflections generally decrease away from the dam crest.

Piezometers. Meeks Cabin Dam is instrumented with 29 porous-tube and slotted-pipe piezometers, 17 of which are located in the left abutment area in close proximity to the seepage barrier (USBOR 2003b). Of the 17 left-abutment piezometers, seven are located upstream of the barrier, six are located downstream of the barrier, three are located to the left of the barrier, and one is located to the right of the barrier. The piezometers located upstream of the barrier follow the reservoir elevation closely with a very quick response to reservoir fluctuations. The downstream piezometers measure a constant piezometric elevation of about 8,590 feet in the foundation below the barrier. While the downstream piezometers do not generally respond to changes in reservoir level, small dips were observed when the reservoir level was very low in 2001. The piezometer to the right of the seepage barrier has indicated a piezometric level of 8,540 since before the seepage barrier was constructed and remained constant until 2003 when it was noted as plugged. The piezometers located to the left of the seepage barrier all show a response to changes
in reservoir level with the response decreasing in the downstream direction. The behaviors of all of the piezometers have remained essentially constant since construction of the seepage barrier.

Weirs. Seepage flows are monitored at nine locations downstream of the dam. Flows from the toe drain are measured at five locations, two on each abutment and one in the center. Seepage flows from the right abutment are measured by a single weir. Seepage flows from the left abutment are measured by the three Parshall Flumes installed in 1986.

Toe drain seepage flow measurements indicate that the left abutment toe drain is nearly dry, with a maximum recorded flow of 0.5 gallons per minute since the installation of the seepage barrier. The central toe drain and right abutment toe drains have picked up maximum flows of 2 to 7 gallons per minute, respectively.

The effects of the seepage barrier construction are most graphically illustrated by the seepage flows measured in Flumes 1 through 3. Prior to seepage barrier construction the total flows through these flumes was over 500 gallons per minute. Immediately after barrier construction the seepage flows were reduced to less than 13 gallons per minute. In 2000, during a period of high reservoir level fluctuation, the seepage measured in these flumes decreased to less than 3 gallons per minute.

Assessment of Seepage Barrier Performance

It is apparent from the measurements recorded in the piezometers and flumes that the seepage barrier is performing very well. The seepage flows measured downstream of the barrier are about two percent of the pre-barrier flows and the piezometers indicate near 100 percent head efficiency. A large portion of the seepage flows that are being measured downstream of the seepage barrier are likely due to seepage around the barrier through the upper layers of glacial till that have higher permeability than the lower glacial till in which the seepage barrier is embedded. A small amount of seepage is also likely coming through and below the barrier. However, it is not possible to quantify this
seepage since this water is quickly dissipated in the high-permeability outwash deposits and cannot be isolated from the seepage flows occurring beyond the ends of the barrier.

One reason for the good performance of this barrier appears to be the combination of flexible barrier infill material and uniform stiffness of the foundation soils. The low strength and the bentonite content of the plastic-concrete seepage barrier backfill will result in a lower elastic modulus for the barrier thus making it more resistant to cracking. The foundation stiffness is expected to decrease gradually from the lower glacial till to the upper till layer. In addition, there does not appear to be a large difference in stiffness between the foundation and the embankment. The gradual stiffness change reduces the chances for stress concentrations in the barrier that were calculated in dams with abrupt changes in material stiffness at the top of bedrock between the foundation and the embankment. The result of these two factors is a greater likelihood that the barrier has remained intact with less cracking than has occurred in more rigid barriers.

**Mill Creek Dam**

**Background**

Mill Creek Dam is managed by the U.S Army Corps of Engineers, Walla Walla District and is offstream storage dam, about one mile southeast of Walla Walla, Washington. The dam was completed in 1941 and is a 120-foot high (crest elevation 1,270 feet) homogenous earthfill embankment with a crest length of 3,050 feet (USACE 2002b). A 42-inch diameter outlet conduit runs below the central portion of the dam. Control of flow through the outlet was originally at the downstream toe, however, modifications in 1948 moved control to the upstream side of the dam to depressurize the outlet pipe. A plan of the dam is shown in Figure 3-47.

The reservoir was originally intended to be used for both flood control and storage of irrigation water. However, due to seepage problems, the dam was never used for long-term storage of water above elevation 1,205 feet. The reservoir does serves as a retention
pond to retain peak flows in Mill Creek during high flow events (USACE 2002b). A diversion structure on Mill Creek diverts flow into an intake canal located about a mile north of the dam. A portion of the flow is diverted into the Mill Creek Reservoir until the flow in Mill Creek returns below the level that can be safely passed through the City of Walla Walla. The water retained by Mill Creek Dam can be released back into Mill Creek via the return canal or it can be released into Russell Creek via the outlet canal. Control boxes on the downstream side of the dam can divert the release water either into the return canal or the outfall canal. The outlet channel was added in 1943 and 1944 to allow quicker drainage of the reservoir. Because the dam has very little natural watershed, no spillway was constructed for the dam.

Figure 3-47  Plan of Mill Creek Dam (after USACE 1979a, with permission from USACE)

The dam foundation consists of wind-blown silt (loess) overlying and interbedded with a deposit of silt, clay and alluvial gravels (referred to as “conglomerate” in project documents) which overlies the Columbia River Basalt Formation (USACE 1978, 2002b). The loess, which has low plasticity and generally very low natural density, varies in thickness from 40 to 90 feet beneath the dam. While the loess has low permeability, it is also highly susceptible to erosion. The conglomerate varies from 10 to 110 feet thick beneath the dam and varies from un cemented to lightly cemented. The permeability of the conglomerate is described in Corps of Engineers documents as varying from “low
permeability” to “high permeability” due to variation in the gradation, the degree of cementation, and the fracturing of the material (USACE 1989b). The upper portion of the basalt consists of a thin layer of weathered rock, beneath which the rock is fresh. The basalt is fractured but has relatively low permeability. A longitudinal section of the foundation conditions beneath the dam is shown on Figure 3-48.

Figure 3-48 Longitudinal section through Mill Creek Dam and foundation showing limits of 1981 seepage barrier and grout curtains (after USACE 1987b, with permission from USACE)

A cross section of the dam is shown on Figure 3-49. The homogenous embankment was constructed of compacted silt borrowed from the loess deposits (USACE 1978). The upstream slope was constructed at an inclination of 4 horizontal to 1 vertical and the downstream slope was constructed at an inclination of 3 horizontal to 1 vertical. During construction it was determined that the compaction efforts were achieving low relative compaction on the order of 85 percent of the Standard Proctor density. By the time this
assessment had been made, most of the lower portion of the dam had been placed, and it was concluded that changing the method of compaction would do little to improve the dam performance. Thus, the entire embankment was placed using the original compaction methods.

The embankment is constructed with a cutoff trench on the upstream side of the dam crest (USACE 1978). The original design called for the trench to be excavated through the loess and into the conglomerate. However, during construction it was discovered that what was thought during design to be the base of the loess were actually 2-foot to 12-foot thick layers of conglomerate underlain by more loess. The decision was made to terminate the cutoff trench in the upper conglomerate layers provided the layers were at least 5 feet thick.

On the downstream side of the cutoff trench, a 10-foot wide drainage blanket was constructed. At the base of the drain there is a 10-inch diameter perforated iron pipe surrounded by 3/4-inch to 3-inch gravel. The 10-inch pipe flows toward the low point of the cutoff trench and connects to a 30-inch diameter outfall line. Although the Board of
Consultants for the dam design recommended filter tests be performed to design the gradation of the drain material, no record of such tests have been located (USACE 1978). Tests performed at a later date indicated that the D_{15} of the drain material was about 1,000 times the d_{15} of the silt (for a filter designed to modern standards this ratio should be less than 5) (USACE 2002b).

Pre-Seepage Barrier History of Seepage Problems and Mitigation

Flow from Mill Creek was first diverted into the reservoir in November 1941, one month after completion of the dam. When the pool reached an elevation of 1,219.8 feet the diversion was stopped. Over the next 21 hours, the pool level dropped 0.8 feet over an area of 75 acres, representing a water loss of 33 cfs. The water continued to leak from the reservoir and, over the next 60 days, 75 percent of the water originally retained had leaked from the reservoir.

Six days after the start of the initial filling, water started seeping from the drain at the back of the cutoff trench. Within a few days after the outfall seepage was observed, seepage was observed at the toe of the dam. This wetting of the fill caused considerable settlement of the backfill around structures at the dam toe. Seepage was also observed in farm land about a mile downstream from the dam.

The reservoir was emptied for inspection and numerous sinkholes were observed in the bottom of the reservoir. Large, deep holes were observed as far as 1,000 feet upstream of the dam and smaller features were observed up to 1,500 feet from the dam. The sinkholes were found to be underlain by a porous silt formation with numerous \( \frac{1}{4} \)-inch channels leading down to pervious gravel layers. A boring drilled at mid-height of the downstream slope encountered a very wet and soft layer of silt in the foundation directly beneath the embankment. In an effort to reduce seepage from the reservoir, a portion of the reservoir bottom was covered with a blanket of compacted silt.
In November 1942, a second filling was started and the reservoir reached a maximum elevation of 1,200 feet in January 1943. Although the rate of drop of the reservoir was slower than after the first filling, the overall behavior of the reservoir was similar to the first filling. No saturation of the toe area was observed. Seepage from the cutoff trench outfall was observed to be heavily loaded with silt. After the reservoir was emptied, a few small sinkholes were observed in the reservoir area.

Following the 1942-1943 filling, a 300-foot long toe drain and relief well system was constructed at the downstream toe of the dam. The toe drain was eight feet wide and three feet deep and located directly beneath the downstream toe. Five relief wells were drilled to depths between 50 and 100 feet and had tip elevations ranging from of about 1,060 to 1,140 feet. Apparently, the relief wells did not penetrate into the highly permeable portions of the foundation conglomerate and, as a result, were only marginally effective.

In January 1945, a test pool raise was performed that quickly filled the reservoir to an elevation of 1,200 feet. Filling continued in 5-foot increments to an elevation of 1,233 feet, with each increment being maintained for a week. Several observation wells and settlement monitoring holes were installed prior to the test. During this test, the cutoff drain outflow started flowing clear 5 days after the start of filling and seepage was observed 3,000 feet downstream of the dam 45 days after the start of filling.

Approximately 60 days after the start of filling, discharge containing up to 2.5 percent silt by weight was observed flowing from the cutoff drain outflow. This flow continued for two months with the silt content of the flow varying. The total amount of soil carried by the drain over the two months was estimated to be on the order of 750 cubic yards (USACE 2002b).

High piezometric pressures were observed at and beyond the downstream toe during the filling test. Water pressures near the downstream valve box were so high that water was observed jetting six inches from cracks in the bottom of the outlet canal. A well drilled to
the top of the basalt 50 feet downstream of the toe registered water levels 5 feet above the ground surface.

The reservoir was not filled in 1946 or 1947. In 1947, a review of the dam was performed by a Board of Consultants to assess the seepage problems. The Board made four remedial recommendations:

- Plug the interior drains with concrete or grout,
- Compact the soils on the surface of the reservoir area,
- Change the outlet control from downstream to upstream, and
- Enlarge the downstream relief well system by adding six new wells.

These recommendations were carried out in 1948.

In 1950, a reservoir pool raise test was performed that consisted of filling the reservoir to elevation 1,235 feet in 32 days, maintaining that elevation for 26 days, allowing the reservoir to drop by seepage and outlet releases to elevation 1,208 in a period of 23 days, and then drop by seepage alone to elevation 1,182 in 123 days (USACE 2002b). Piezometric levels downstream of the dam were measured in 43 wells and seepage volumes were measured in weirs. The observed results of this test filling were very similar to the 1945 test filling with the exception of the discharge from the interior drains.

Based on the results of the 1950 test, the Board of Consultants recommended that the reservoir only be used when necessary for flood control and not be used to store irrigation water as had originally been planned. Operational controls were put on the reservoir elevation that limited the amount of time the reservoir could be maintained at given elevations.

In 1964 and 1965 an emergency diversion was made into the reservoir raising the level to elevation 1,214 feet. Observations made during this filling were similar to those made in 1945 and 1950 (USACE 2002b).
In 1979, a pool raise test was performed by filling the reservoir to elevation 1,225 feet in 11 days and maintaining at that elevation for 7 days. By the end of the test, flows of approximately 450 gallons per minute were measured from the toe drain and relief wells. There was no record of seepage in the farmland downstream of the dam. Following the 1979 test, the Corps of Engineers recommended construction of a concrete cutoff wall along the upstream toe of the embankment (USACE 1978).

**Design and Construction of Seepage Barrier**

The seepage barrier at Mill Creek Dam was constructed in 1981 (USACE 1980, 2002b). The two-foot wide concrete barrier was designed to extend from the downstream toe of the dam through the silt and conglomerate layers and embed two feet into the basalt. However, because the top of the basalt in the right side of the dam is very deep, costs to extend the seepage barrier to the basalt were deemed to be unjustifiably high. The designed seepage barrier was terminated 680 feet shy of the right end of the embankment. A 300-foot long, double-row grout curtain extending 5 feet into the basalt was constructed at the right end of the seepage barrier and a similar 100-foot long grout curtain was constructed on the left end. The locations of the seepage barrier and the grout curtains are presented in Figure 3-50. A compacted clay facing was constructed on the downstream slope from the top of the seepage barrier to elevation 1,230 feet. A cross section of the seepage barrier design is presented on Figure 3-51 and a longitudinal section of the limits of the seepage barrier and grout curtains are shown on Figure 3-50.

The seepage barrier was constructed in 20-foot long panels using the slurry trench method. Grout sleeves were embedded in the barrier during construction. After the barrier had cured, drilling was continued through these grout sleeves and the top 30 feet of the basalt was grouted beneath the barrier.
Instrumentation

Instrumentation at Mill Creek Dam consists of a variety of piezometers and wells, seepage flow measuring instruments, and the ability to estimate total seepage from losses in the reservoir storage volume. These elements are discussed below.
Piezometers and Wells. A combination of 55 wells and piezometers are used to monitor piezometric pressures in the dam, foundation and areas downstream of the dam. Some of the earliest wells date back to between 1941 and 1945. Most of the piezometers were installed from 1978 to 1985, coinciding with the design, construction and initial testing of the seepage barrier. Most of the piezometers are normally read on a monthly basis and with increased frequency when the reservoir pool elevation is high.

Seepage Flow Measurement. Seepage flows emitting from the toe drain and relief wells are measured at two manholes. Seepage flows are generally measured only during the infrequent periods when the reservoir pool elevation is high.

Seepage Losses. Estimates of total seepage losses from the reservoir are made during filling tests and emergency fillings. Such estimates are possible due to the ability to accurately measure the flow entering the reservoir from the Mill Creek diversion structure and the change in reservoir elevation. Estimates of total seepage losses are made during high reservoir pool events.

Post-Seepage Barrier History of Seepage Problems and Mitigation

In 1984, a test pool raise was performed to test the effectiveness of the seepage barrier. The reservoir was raised to elevation 1,225 feet and held for 10 days and then raised to 1,235 feet and held for 28 days. Comparison of the data from the 1984 test filling with data from the 1979 test filling indicated that the overall seepage rate had decreased by 30 percent, seepage flows from the downstream relief wells had decreased by 84 percent and the area of farmland saturated downstream of the dam was decreased by 56 percent. Seepage flows from the relief wells during the 1979 and 1984 tests are plotted along with the reservoir levels on Figure 3-52. Piezometers that were in service in 1979 appeared to behave similarly during the 1979 and 1984 pool raise tests, although few piezometers were located in the toe area in during the 1979 test. The wells that were located in the toe area
area in 1979 were flowing during the test and did not provide an accurate measure of the piezometric level.

The Corps of Engineers assessed that the low level of seepage barrier effectiveness was due to continued seepage through the right abutment. A later assessment of the seepage barrier construction records also revealed that in a few reaches, the base of the seepage barrier is slightly above the contact with the bedrock, leaving gaps of conglomerate below the barrier.

In 1996, flows from Mill Creek were diverted into the reservoir as a flood control measure. A record high reservoir elevation of 1,257 feet was recorded two days after the start of the diversion. The reservoir level was then drained back to elevation 1,205 feet in about 30 days. While this event was of short duration compared to the 1979 and 1984 tests, flows of up to 17 gallons per minute were recorded from the toe drain system.
Several large depressions were found to have developed in the reservoir areas and a large depression formed near the downstream toe.

In 2002, a 590-foot long, two-row grout curtain was constructed from the right end of the seepage barrier to the right dam abutment. The grout holes were spaced on 2.5-foot centers with every other hole set 2 feet downstream from the primary alignment. The location of the grout curtain is shown on Figure 3-50 along with the locations of the seepage barrier and the grout curtains constructed in 1981.

In 2002 a test pool raise was conducted to assess the effectiveness of the grout curtain. The reservoir level was raised to elevation 1,225 feet, however, due to the construction of a new fish screen and environmental regulations on diverting water from Mill Creek, the filling took 60 days to complete. Once at 1,225 feet, the reservoir was maintained at that level for 30 days and then gradually lowered. Because this filling and lowering sequence is substantially different from the test pool raises conducted in 1979 and 1984 (see Figure 3-52), it is more difficult to assess the differences in seepage behavior caused by the new grout curtain than it was to assess the effects of the seepage barrier construction from the results of the 1979 and 1984 tests. Nonetheless, observations based on the 2002 test pool raise are presented below.

The construction of the 2002 grout curtain appears to have significantly reduced the amount of seepage from the toe drain and relief wells. Seepage flows from the relief wells and reservoir levels during the 2002 test are plotted along with the data from the 1979 and 1984 tests on Figure 3-52. It should be noted that the 2002 test flows on Figure 3-52 include flows from relief wells that were installed in 1998 in addition to the wells that comprise the flows reported for 1979 and 1984.

The difference in the reservoir raising and lowering sequence in 2002 also makes it difficult to assess the difference in the behavior of piezometers at the dam toe. A comparison of the piezometric levels in Piezometer RD44, located at the downstream toe of the central portion of the dam, is shown of Figure 3-53. In the later stages of the pool
raise test the reservoir level was higher in 2002 than in 1984 but the opposite is true of the piezometric levels in RD44 during the same time span. Also, the time it takes for the piezometer to react to the pool raise is longer in 2002 (about 12 days) than it is in 1984 (about 4 days). Similar behavior was observed in all seven piezometers in the central portion of the dam toe. While the difference in reservoir elevation makes it difficult to quantify the change in piezometric behavior, it is apparent that the 2002 grout curtain has reduced the hydraulic heads at the toe of the dam.

Figure 3-53  Reservoir levels and levels in Piezometer RD44 at the toe of Mill Creek Dam during pool raise tests conducted in 1984, and 2002 (after USACE 2002b, with permission from USACE)

Assessment of Seepage Barrier Performance

Seepage through the conglomerate layer has been a major problem at Mill Creek Dam since its construction in 1941. The seepage is characterized by flow through very high permeability joints or layers within the conglomerate that have the ability to transmit water over long distances with low head losses. This seepage behavior is evidenced by the sinkholes located as far as 1,500 feet upstream and the seepage in the farmland up to a
mile downstream. The problematic conglomerate layer is not limited to the dam foundation but extends far beyond the limits of the embankment.

The limited effectiveness of the seepage barrier at Mill Creek Dam is largely a consequence of the seepage conditions described above. The barrier and grout curtains constructed in 1981 were successful in reducing the flow volumes at the toe area downstream of the barrier by 84 percent. It can be argued that a large portion of the remaining 16 percent of the flow is due to seepage around the ends of the barrier. Small amounts of seepage may also be coming through the conglomerate gaps beneath the barrier, through the upper basalt, and through the barrier. However, the 1981 mitigation did little to reduce the overall seepage losses from the reservoir, resulting in only a 30 percent decrease in overall seepage rate. Much of the post-1981 seepage was likely occurring below the right end of the dam and in the right abutment, where the seepage paths were the shortest.

It was argued by some in the Corps of Engineers that the seepage barrier had actually increased the risk of seepage related failure in the right end of the dam by concentrating the seepage through this area. Others in the Corps argued that the barrier had little effect on the seepage-related risk in this area. It is reasonable to say that the upstream heads in the right end of the dam were controlled by the reservoir level before and after construction of the seepage barrier. Furthermore, there was little change in the heads downstream of the right end of the dam after barrier construction. Thus, with the heads upstream and downstream remaining essentially the same, the gradients in the right abutment area were not expected to be significantly changed. Therefore, it can be concluded that the seepage barrier did not decrease the risk of seepage-related failure in the right end of the dam, and any increase in risk due to seepage in this area was negligible.

The 2002, extension of the grout curtain in the right abutment significantly decreased seepage flows to the area directly downstream from the dam, and further reduced the evidence of seepage in the farmland far below the dam. The effectiveness of the grout
curtain is due to increasing the seepage resistance in the right end of the dam where the seepage paths are the shortest. It can be argued that the 2002 grout curtain decreased the risk of seepage-related failure by retarding the seepage through the shortest seepage paths, thus mitigating the highest seepage gradients. However, the 2002 grout curtain was also effective in reducing the hydraulic heads downstream from the dam. These reduced downstream heads tend to increase the hydraulic gradients in the right abutment and may increase the internal erosion potential in this area. Furthermore, while the grout curtain increases the overall seepage resistance of the right end of the dam foundation, it does not provide a positive cutoff to seepage and has a higher potential for windows than does a seepage barrier. The consequence of this is that any windows with high permeability that exist in the grouted area will now have higher gradients acting on them, and thus be more susceptible to erosion. The Corps of Engineers understands these concerns and continues to operate the reservoir as a flood control facility rather than storing water at high pool levels. Piezometer levels and flows continue to be closely monitored.

**Private Dam Number 1**

**Background**

The dam presented in this case history is privately owned and, as a condition for allowing the dam to be part of this study, the dam name, location, and owner will not be presented. The information presented in this case history was obtained from a Safety Inspection Report prepared by a private consultant to meet the requirements of the Federal Energy Regulatory Commission (FERC). References to the specific report have been omitted to prevent identification of the dam.

The dam is a combination earth embankment and concrete structure constructed in the 1950s. A cross section of the earth embankment is shown on Figure 3-54. The concrete portion of the dam is on the right abutment and consists of the outlet structure and spillway. The left portion of the dam consists of a 1,600-foot long zoned earth
embankment. The crest of the embankment is at elevation 715 feet and the embankment has maximum height of 113 feet. A 29-foot wide paved roadway runs along the dam crest and comprises the full width of the embankment. The upstream slope is inclined at 3 horizontal to 1 vertical and the downstream slope is inclined at 2.5 horizontal to 1 vertical in the upper 40 feet and 3 horizontal to 1 vertical in the lower portions of the slope.

Figure 3-54  Cross section of the earth embankment portion of Private Dam Number 1

The dam foundation consists of glacial and lacustrine soils overlying granitic gneiss bedrock. The lower and uppermost portions of the glacial and lacustrine soils contain a significant amount of fines and have relatively low permeability. However, along the northern portion of the dam (left abutment area), a layer of sandy glacial till that has relatively high permeability exists within the glacial and lacustrine soils.

The dam embankment is comprised of three principal zones as shown on Figure 3-54. The “impervious core” consists of very dense silty fine sand with traces of gravel. The “pervious fill” consists of very dense, coarse- to medium-grained sand and serves as the outer shells of the embankment. In the taller sections of the dam, rock fill zones were constructed in the lower portions of the upstream and downstream slopes. Two-staged filters separate the rock fill zones from the soil zones of the embankment.

A relatively shallow, 30-foot wide cutoff trench was constructed beneath the upstream portion of the embankment. In the taller dam sections near the center of the dam the cutoff was extended to the top of the granitic gneiss bedrock. Along the left portion of the dam the cutoff was extended into competent, low-permeability soil.
History of Seepage Problems and Mitigation

After completion of the dam and filling of the reservoir, seepage was noted beyond the downstream toe in the lower right abutment area. Over about 30 years of operation the seepage continued and in the early 1980s was observed to be moving fine-grained soil material from the dam foundation. In 1984, a filter blanket, stone drain, and pipe drainage system was installed in the seepage areas in an effort to stop the migration of fines. The filter blankets consist of filter fabric covered with gravel. The blankets lead into coarse rock drains that remove the seepage water from the area. The limits of this system are presented on Figure 3-55.

In 1988, a significant boil formed approximately 500 feet downstream from the crest of the dam (see Figure 3-55 for location). It was assessed that the seepage problems in the left abutment area were due to a layer of high permeability sandy glacial till that existed between the layers of finer-grained lacustrine and glacial soils.

In response to the 1988 boil and continued seepage through the left abutment, a plastic-concrete seepage barrier was constructed along 850 feet of the northern portion of the dam and the left abutment (see Figure 3-55 for location). Details of the seepage barrier are presented below.

In 1996, a boil formed along the bank of the river beneath the tallest portion of the dam (see Figure 3-55 for location). Two relief wells were constructed approximately 40 feet upstream of the boil. Following construction of the relief wells the seepage from the boil ceased.

Design and Construction of Seepage Barrier

The seepage barrier was constructed in 1990 and 1991 using the slurry trench method. The barrier is located in the left-most 500 feet of the embankment and extends 350 feet
into the abutment beyond the left end of the dam as shown on Figure 3-55. The barrier has a nominal width of 30 inches and is backfilled with plastic concrete consisting of a mixture of cement, bentonite, aggregates, and water.

A cross section through the embankment in the left abutment showing the location of the seepage barrier is presented in Figure 3-56. The top of the barrier is at an elevation of 710.5 feet (about 5 feet below the dam crest) and it extends to elevations ranging from 585 to 600 feet, resulting in a maximum depth of over 125 feet. The base of the barrier is
embedded in the dense glacial sands that lie below the highly permeable gravelly glacial till deposit.

Figure 3-56  Left abutment embankment of Private Dam Number 1 showing location of seepage barrier

Instrumentation

Prior to construction of the seepage barrier, the left abutment of the dam was instrumented by a weir ("main north abutment weir") and 18 piezometers. The main north abutment weir is located near the site of the 1988 boil and monitors seepage flows coming from the boil and the filter blanket and drain areas that were constructed in 1984. The existing piezometers are located on the dam embankment upstream of the barrier alignment, downstream of the embankment toe, and on the left abutment upstream of the barrier alignment.

An additional 20 piezometers were installed in 1991 following the construction of the seepage barrier to monitor the performance of the barrier. Ten more piezometers were installed at various times between 1992 and 1997 to monitor specific areas of interest downstream of the barrier.
In 1997, two relief wells were constructed to relieve pressures around the boil that had formed at the downstream toe. Flows from the relief wells are directed to weirs and the flows have been regularly recorded since the wells were installed.

**Piezometric Levels.** The construction of the seepage barrier resulted in immediate changes in water level in the piezometers that were in place prior to the barrier construction. Piezometers located below the toe of the dam measured water level drops ranging from two to nine feet with the largest drops occurring near the center of the barrier just downstream of the dam toe. In the embankment above the seepage barrier, piezometers measured rises in water level ranging from five to 10 feet. In the left abutment above the seepage barrier, piezometers measured water level rises between five and eight feet.

After the immediate response to the barrier construction, most of the piezometers indicate very little change in water level since barrier construction. Directly downstream of the middle portion of the barrier, the piezometers indicate changes of less than a foot since the initial response to the barrier. Some of the piezometers downstream of the ends of the barrier have experienced a small but steady increase of up to 2 feet since the initial response to the barrier construction. Most of the piezometers in the embankment upstream of the barrier indicate no changes since the initial response, although two of the six piezometers located in this area show water level increases of two and five feet. Most of the upstream piezometers in the left abutment indicate a one- to two-foot increase in water level since the initial response. Most of the piezometers do indicate a small annual fluctuation on the order of one to three feet that is independent of the reservoir level.

**Seepage Flow Rates.** Prior to construction of the seepage barrier, flows measured in the main north abutment weir varied from 1.0 to 1.5 cubic feet per second (cfs). Between the time of seepage barrier construction and about 2000, the flows were generally steady at about 1.0 cfs with a few readings that were slightly higher. Between 2000 and 2003, the flows gradually increased to an average of about 1.3 cfs. From December 2003 to about March 2004, the flow dropped back to about 1.0 cfs.
In addition to seepage flows, sand accumulation in the stilling box for this weir has been recorded since 1988. From 1991 to 1993 (the two years following construction of the seepage barrier), sand accumulated in the weir box on several occasions, resulting in a total of about 43 cubic feet of sand. From 1993 to 2004, only trace amounts of sand were recorded. Between February and April 2004, a total of about 10 cubic feet of sand accumulated in the weir box. This sand accumulation was accompanied by a drop in the weir flow from 1.5 to 1.0 cfs.

Flows from the two relief wells above the 1997 boil have steadily decreased since they were installed in 1997. Both relief wells flowed at rates of about eight gallons per minute (0.02 cfs) when first installed. The flow rates at both wells steadily decreased and in 2004 the rates were five and three gallons per minute (0.01 and 0.007 cfs).

**Assessment of Seepage Barrier Performance**

The seepage barrier was effective in reducing the flow beneath the dam in the left abutment. This is evident by the drop in piezometric water level just below the seepage barrier and below the toe of the dam, along with a rise in piezometric level in the embankment foundation piezometers upstream of the barrier, and a rise in the piezometers in the upstream portion of the left abutment. Contours of the hydraulic heads measured in the piezometers immediately after the construction of the seepage barrier are presented on Figure 3-57. Within a short time after the construction of the seepage barrier, a differential hydraulic head of 20 feet had developed across the barrier. With a total differential head across the dam of 100 feet (normal reservoir elevation of 705 feet and normal tailwater elevation of 605 feet) the barrier has a relatively low head efficiency of 20 percent.

It is evident from the water levels contours on Figure 3-57 that flow is occurring around the ends of the barrier. The areas around the ends of the barrier are also the locations where there have been gradual head increases in many of the piezometers since the
seepage barrier was constructed. These head increases may be the long-term result of the increased piezometric pressure behind the dam due to the seepage barrier construction. The increases may have occurred slowly due to the low permeability soils in this area. The flow around the ends of the barrier is likely in part responsible for the relatively low efficiency of the barrier.

Another reason for the poor head efficiency of the seepage barrier is that it is keyed into sandy soils that, while having permeability significantly lower than the high-permeability layer, will also allow transmission of seepage and hydraulic heads. A better indicator of the effectiveness of this seepage barrier would be the change in seepage flow through the
embankment. However, since most of the exiting flow likely flows undetected into the river, an assessment of this sort is not possible.

The importance of the sand accumulations in the main north abutment weir following the construction of the seepage barrier and in 2004 is difficult to assess. The weir is located at a distance of over 500 feet from the dam crest and there are many opportunities for sand to be eroded along the seepage paths between the dam and the weir. While the sand accumulation may be the result of increased erosion along a preferred seepage pathway, it may also be the result of surficial disturbance in the area of the blanket drains system. Thus, while continued monitoring of the sand accumulation is warranted, a connection between sand accumulation and seepage barrier performance would be highly speculative.

**Walter F. George Dam**

**Background**

Walter F. George Dam is managed by the U. S. Army Corps of Engineers, Mobile District and is located on the Chattahoochee River on the border between Georgia and Alabama approximately 84 miles southeast of Mobile, Alabama. The dam consists of a central concrete section with long earth embankments extending to the edges of the flood plain on both sides as shown on Figure 3-58. The concrete section is about 1,440 feet wide and consists of a powerhouse, the spillway, a navigational lock structure, and a non-overflow wall section. A longitudinal section through the concrete portion of the dam is presented in Figure 3-59. The powerhouse is 335 long and houses four generating bays and one erection bay. The spillway is composed of 14 tainter gates, is 708 feet wide, and has a crest elevation of 163 feet. The navigational lock is about 200 feet wide and has an 82- by 450-foot lift chamber with a maximum lift of 88 feet. The non-overflow wall section is about 200 feet wide and provides a transition from the Alabama side embankment to the powerhouse structure.
The dam location is on the Chattahoochie River located near the middle of a wide flood plain. The flood plain is covered with a layer of alluvium that averages about 38 feet thick below the dam footprint (MACTEC 2004). The lower portions of the alluvium consist of silty to clean sand with some gravel. This permeable alluvium is generally capped with five to eight feet of low-permeability clay and clayey sand. Underlying the alluvium, in descending order, are the three limestone members of the Clayton Formation: the Earthy Limestone, the Shell Limestone, and the Sandy Limestone. A geologic profile representative of the dam alignment is shown on Figure 3-60.
The Earthy Limestone is a massive earthy limestone unit with thickness varying from 30 to 72 feet along the dam alignment. The upper portion of this member is highly cavernous from solutioning, and the surface is irregular and pock marked with sinkholes. The solutioning of the unit is generally along two joint sets trending at N10°E and N80°E. The base of the Earthy Limestone is at an elevation of about 78 feet.

The Shell Limestone is a highly fossiliferous, hard, and porous unit with a thickness of about 44 feet. One joint set was identified in the unit with a trend of about N43°W (roughly parallel with the dam axis). The base of the Shell Limestone is at an elevation of about 38 feet.

The top of the Sandy Limestone consists of a four-foot thick layer of hard calcareous sandstone underlain by a layer of unconsolidated medium-grained sand. Below the sand layer are alternating layers of sandy limestone, crystalline limestone, earthy limestone, and uncemented sand. Beneath the Sandy Limestone is the Providence Sand Formation consisting of micaceous clay with varying amounts of sand.

The concrete portion of the dam is located across the main riverbed and is founded in the Shell limestone. The embankments are founded on the alluvial overburden. In order to lengthen the seepage path below the embankments, the natural clay layer on the top of the alluvium was supplemented with a five-foot thick compacted clay seepage blanket.

The earth embankments are located on the Georgia and Alabama sides of the concrete portion of the dam and are 5,810 and 6,130 feet long, respectively. A cross section representative of both embankments is shown on Figure 3-60. The crests of both embankments are 30 feet wide with a 20-foot wide roadway and have a crest elevation of 215 feet. The upstream one-third of the embankments is constructed with select low-permeability fill and the downstream two-thirds are constructed with random fill. Both embankments have a maximum height of 68 feet above the flood plain. The upper portions of upstream slopes are inclined at 2.5:1 (horizontal to vertical) and protected with riprap. The lower portions of the upstream slopes are inclined at 3:1 (horizontal to
vertical). The downstream slope is inclined at 2.5:1 (horizontal to vertical). Blanket drains are constructed beneath the lower two-thirds of the downstream slopes of the embankments.

Figure 3-60 Cross section representative of both earth embankments of Walter F. George Dam (after MACTEC 2004, with permission from USACE)

History of Seepage Problems and Mitigation

Seepage problems at the dam began to appear during construction (USACE 1972, MACTEC 2004). In 1961, several sinkholes developed at the base of the excavation for tying in the foundation of the Alabama embankment with the concrete portion of the dam. Exploration of these sinkholes with core borings and testpits revealed extensive cavities, some of which extended to the top of the permeable Shell Limestone. The sinkholes were excavated and filled with concrete and a 10-foot thick layer of low-permeability fill was placed over the base of the excavation. As a result of the sinkholes, the decision was made to grout the entire earth embankment portions of the dam down to the top of the Shell Limestone. A single line of grout holes were drilled on about five-foot centers and total grout takes were nearly one-million cubic feet.

In 1962, when the reservoir level had reached an elevation of 166 feet, numerous small boils developed at the downstream toe of the Alabama embankment. To mitigate this seepage problem, 350 relief wells were constructed at the downstream toes of both embankments. The six- and eight-inch diameter relief wells were installed at 40-foot centers and extended to the top of the Earthy Limestone. After completion of the relief wells, no additional seepage related problems were reported until 1968.
In 1968, sinkholes formed downstream of the Georgia embankment in the area adjacent to the concrete portion of the dam and a spring developed adjacent to the lock wall. In 1969, a large sinkhole was discovered below reservoir level on the upstream side of the Georgia embankment. As a result of these developments, remedial grouting was performed in the embankment foundation and a sand filter trench was installed along the lock.

A Board of Consultants was engaged in 1972 to evaluate the stability of the dam. The Board of Consultants concluded that the only permanent remedial procedure would be to construct a concrete seepage barrier extending through the Shell Limestone for the entire length of the embankments.

Design and Construction of Embankment Seepage Barriers

Between 1981 and 1984, a 24-inch thick concrete seepage barrier was constructed in both embankments. In 1981, a 1,244-foot long test section was installed in the Georgia embankment adjacent to the concrete portion of the dam. This reach was selected so that the caverns that were discovered as a result of the boils and seepage observed in 1968 and 1969 would be sealed; facilitating needed dewatering and repairs on the navigational lock. Following the successful construction of the test section, the seepage barrier was constructed along the remainder of the embankments between 1981 and 1984. Along the length of both embankments, the seepage barrier was constructed down to near the bottom of the Earthy Limestone. In select reaches, identified by pre-construction borings, the seepage barrier was extended deeper, usually to the base of the Shell Limestone.

The seepage barrier was constructed in panels using the slurry trench technique (USACE 1981b, Bryan 1987). A pair of guide walls were constructed to help align the excavation equipment. Primary panels of various widths were excavated with a hydraulic clamshell and supported until backfilled with bentonite-water slurry. After excavation of the
primary panels, 24-inch shoulder piles were installed at each end of the primary panel and concrete was placed by the tremie method. Secondary panels were excavated between the shoulder piles of adjacent primary panels. Prior to placing concrete in the secondary panels, the shoulder piles were removed. This provided a clean concrete face to place the concrete and a concaved joint between the two panels. The contacts between the seepage barriers and the concrete portion of the dam were grouted with chemical grout in order to improve the seal.

**Embankment Seepage Barrier Performance**

One indicator of performance of the embankment seepage barriers is the flow from the relief wells at the downstream toes of the embankments. Prior to installation of the seepage barriers, total flows from the relief wells were about 10 million gallons per day. After completion of the seepage barrier, flows from the relief wells ceased. Photos of the relief well collection trench before and after construction of the seepage barrier are presented in Figure 3-61.

![Figure 3-61](image)

**Figure 3-61** Relief well collection trench before (left) and after (right) seepage barrier construction (after MACTEC 2004, with permission from USACE)

Although the seepage barriers appear to have been effective in reducing the flow beneath the embankments and lowering the piezometric levels downstream of the embankments, seepage problems began occurring beneath the concrete section of the dam as the
embankment seepage barrier was being constructed. In March 1982, a large boil was observed in the tailwater just downstream of the powerhouse. It was determined that the boil was caused by water flowing from a construction dewatering well that had been left in place after the original dam construction. The entrance point of the water was found to be a construction piezometer on the upstream side of the dam. Over the next several months, the water flowing from the dewatering well and the powerhouse drainage system increased from about 9,200 gpm to over 34,000 gpm. In July of 1982, the upstream piezometer was plugged with concrete and the channel beneath the concrete section of the dam was plugged with 175 cubic yards of concrete. Additional grouting was performed upstream of the concrete portion of the dam between September 1982 and May 1983. As a result of this grouting, flow into the powerhouse drainage system and flow out of the downstream construction well ceased.

Following completion of the embankment seepage barriers, numerous episodes of high piezometric pressures beneath sections of the concrete dam were experienced. In each of these episodes, the inlet sources were found to be sinkholes or old piezometers on the upstream side of the dam. In several instances the sinkholes were determined to be associated with joints in the Earthy Limestone that extended down to the Shell Limestone unit. Grouting of the inlet sources was generally successful in lowering the piezometric pressures in each case. However, due to the continued occurrence of seepage problems below the concrete portion of the dam and confirmation that foundation materials were being piped from the powerhouse drainage system, a 1996 Board of Consultants characterized the dam as being in a “grave maintenance condition”.

The 1996 Board of Consultants recommended construction of a secant pile seepage barrier on the upstream side of the concrete portion of the dam, tying in with the embankment seepage barriers at both ends. The barrier was extended down through the three members of the Clayton Formation and embedded into the clay at the top of the Providence Sand Formation. The barrier was constructed between 2002 and 2004. The barrier appears to be effective in reducing seepage and piezometric levels beneath the concrete portion of the dam.
Assessment of Seepage Barrier Performance

Seepage problems were experienced at Walter F. George Dam since the beginning of construction in 1961 and became progressively worse with time. The seepage problems can be attributed to three foundation conditions:

1. The presence of the permeable alluvium beneath the surficial clay,
2. The presence solution cavities in the Earthy Limestone, and
3. The high permeability in the Shell Limestone.

The seepage problems beneath the embankments appear to have been the result of the first two conditions. Upon first filling of the reservoir, water began seeping through the alluvium and Earthy Limestone. The upstream entrance points for this seepage are likely (1) sinkholes upstream of the seepage blanket that allow water to flow directly to the sandy alluvium and the cavities in the Earthy Limestone, and (2) sinkholes that developed or redeveloped beneath the seepage blanket compromising the integrity of the blanket and allowing entrance of seepage water near the embankments. The seepage barriers constructed in the embankments appear to be very effective in controlling the seepage mechanisms associated with the first two conditions as evidenced by the complete cessation of flows from the relief wells. While seepage through the Shell Limestone is likely still occurring, because of the long flow path and relatively uniform seepage resistance as a result of limited solutioning in this layer, erosion problems do not appear to be developing in this layer beneath the embankments.

The seepage problem beneath the concrete section of the dam appears to be due to a mechanism involving the second two foundation conditions listed above. The clearing of soil infill from solution cavities in the Earthy Limestone has allowed connection of the reservoir water pressures with the highly permeable Shell Limestone. This has resulted in high seepage volumes through concentrated seepage pathways, high piezometric heads beneath the concrete structures, and observed erosion of the foundation materials.
The connection between the initiation of the seepage beneath the concrete structure and the construction of the embankment seepage barriers is unclear; however, the coincidence of the barrier construction and the seepage development cannot be ignored. One way that the embankment seepage barrier could have contributed to the concrete portion underseepage problems is that an upstream inlet may have existed that fed a system of solution cavities in the Earthy Limestone located both beneath the embankment and in front of the concrete section. Blocking a portion of the seepage with the barrier would cause seepage velocities in the remaining cavities to increase significantly and increase the potential for erosion of the cavity soil infill or soft portions of the rock. It is possible that the underseepage problems beneath the concrete section would have developed in time without the seepage barrier construction. However, based on the coincidental timing of the seepage barrier construction and the seepage development; it appears that the barrier construction exacerbated the seepage problems beneath the concrete section.

Mud Mountain Dam

Background

Mud Mountain Dam is managed by the U. S. Army Corps of Engineers, Seattle District and is located on the White River about 5 miles southeast of Enumclaw, Washington. The dam was originally constructed from 1939 to 1948, and is a zoned earth and rock fill embankment used solely for flood control (USACE 2003). A plan of the layout of the dam and pertinent structures is shown on Figure 3-62. The embankment has a maximum height of 395 feet and a crest length of about 700 feet. The original crest elevation of the dam was 1,250 feet and the crest was raised to 1,257 feet after the addition of the seepage barrier. The uncontrolled, concrete-lined spillway is located on the right abutment and has a crest width of 315 feet, a crest elevation of 1,215 feet, and a capacity of 245,000 cfs. The outlet works consist of a single intake tower leading to the entrances of 9-foot diameter and a 23-foot diameter tunnels. The intake tower is located on the right side of the reservoir, about 1,000 feet upstream of the right abutment, and consists of a low flow entrance and trash rack to elevation 960 feet, a main upper trash rack to elevation 1,100
feet, and an airshaft rising to elevation 1,260 feet. The outlet tunnels run through the right abutment and exit about 500 feet downstream of the dam.

The reservoir elevation is maintained at a low level and rises during high flows in the White River that are the result of storms or snow melt (Eckerlin 1992, USACE 2003). During the spring and fall the reservoir is often at the base level of around elevation 910 feet. During the winter and summer months the reservoir is usually at a higher level up to 940 feet due to winter storms and summer snow and glacier melt on Mount Rainier. Large storms push the reservoir level even higher, with peak levels in the last five years exceeding elevation 1,050 several times and reaching 1,100 feet on one occasion.

The dam is located across a narrow canyon with steep walls on both sides (Davidson et al. 1992, Eckerlin 1992, 1993). A longitudinal section along the dam crest is shown on Figure 3-63. The lower portion of the canyon has very steep (near vertical) abutment slopes nearly 200 feet high. These slopes are comprised of moderately hard to hard volcanic bedrock consisting of blocks of andesite in a matrix of welded tuff and ash. This rock contains zones of open jointing and several fault zones up to 12 feet wide. The
upper walls of the canyon are in the Mud Mountain Complex, consisting of soils deposited by mudflows that contain varying amounts of clay, silt, sand, gravel, and cobbles with varying amounts of wood fragments and pumice. Although deposited by mudflows, the soils are relatively competent.

The foundation preparation during the original construction consisted of flattening the slopes to a maximum inclination of 1:20 (horizontal to vertical), removing overhangs and fault gouge, and surficial grouting (Davidson et al. 1992, Eckerlin 1992). The grouting consisted of filling surface irregularities and pumping grout into open joints and fault traces from the surface. Grouting at the base of the canyon was performed before construction, and grouting of the canyon walls was performed from the rising embankment surface as it was constructed.

A cross section through the embankment is presented in Figure 3-64. The central core consists of a broadly graded mix of sands and gravels containing from 15 to 20 percent fines (Davidson et al. 1992, Eckerlin 1993, USACE 2003). In order to lower the moisture content of the core material to a workable level, the soil was dried in rotating heated drums and the entire core area was covered with a tent suspended from the canyon walls. The lower portions of the core contain more gravels while the portion of the core
above elevation 1050 feet is largely composed of silty sand. The transition zones separating the core from the shells consist of crushed diorite and do not have gradations meeting modern filter criteria. The dam shells consist of dumped diorite with individual rocks weighing up to 1,000 pounds.

![Figure 3-64 Cross section of Mud Mountain dam embankment (after USACE 1991, with permission from USACE)](image)

**History of Seepage Problems and Mitigation**

In the 1980’s, monitoring of a single piezometer installed in the core from the dam crest in the deepest part of the canyon indicated that the effectiveness of the dam core was deteriorating (Davidson et al. 1992, Eckerlin 1992, 1993). In 1982 and again in 1984, the reservoir level was held at elevation 920 feet for several months. The water level in the piezometer was found to be about 5 feet higher in 1984 than in 1982, and the piezometer was responding fairly rapidly to reservoir fluctuations. Subsurface investigations were performed and additional piezometers installed in 1985 and 1986. These investigations revealed the following:

1. Zones consisting of water-bearing clean gravel existed near the bottom of the core.
2. Loose zones and cracks existed in the upper, sandier portions of the core (above elevation 1050 feet). Cracks at lower elevations were filled with water.

The clean gravel zones were interpreted to be the coarse portion of the original core material with the fines washed out by fluctuating water levels in the bedrock joints (Davidson et al. 1992, Eckerlin 1992). The cracks and loose zones were apparently due
to differential settlement. Arching was thought to have occurred in the lower portions of the core due to drag on the steep canyon walls. In the upper portions of the core, arching was thought to have occurred due to drag on the stiff rock fill shells and transition zones. The arching likely led to conditions of tension or low horizontal stress which allowed development of cracks or hydraulic fractures.

Design and Construction of Seepage Barriers

The decision was made to construct a concrete seepage barrier through the core embedding it a minimum of 15 feet into the bedrock (Davidson et al. 1992, Eckerlin 1993, USACE 2003). The large embedment depth was due to the presence of numerous open joints and overhangs in the bedrock. The barrier was to have a minimum width of two feet. The original design divided the barrier into two types, as shown on Figure 3-63. The Type I (thinner) seepage barrier was to be constructed along the ends of the barrier where the height of the embankment was relatively small. The Type II (thicker) seepage barrier was constructed in the area of the deep canyon and steep canyon walls. Due to the great depth of the seepage barrier and the difficulty maintaining panel alignment while excavating into the canyon walls, there was concern about the ability to construct a continuous barrier. The original specifications required that primary elements have a steel beam embedded in the ends to help guide the excavation of the secondary elements. The requirement for this beam was later eliminated by using other means (discussed below) to ensure continuity.

Construction of the seepage barrier started in May 1991 (Davidson et al. 1992, Eckerlin 1993, USACE 2003). A working platform was constructed by lowering the crest of the dam 10 feet to an elevation of 1,240 feet. Guide walls were constructed and the seepage barrier panels were excavated using a hydrofrase. The ability to achieve a reliable minimum barrier thickness depends on the ability to maintain sufficient overlap between adjacent panels. This was achieved by constructing panel elements wider than the specified minimum, and by maintaining tight controls on verticality of the individual panels. The panel width was increased to 34 inches for the Type I panels and 40 inches...
for the Type II panels, thereby allowing for 10 and 16 inches of misalignment between adjacent panels, while maintaining a minimum thickness of two feet.

Alignment was monitored using dual inclinometers mounted on the hydrofraise (Davidson et al. 1992, Eckerlin 1993). The hydrofraise could be steered by tilting the cutter head and pushing off of the sides of the excavation with hydraulic jacks. Alignment of adjacent panels was checked using two means. First, the concrete in primary panels was dyed red and black in alternating sequence. When the secondary panels were excavated, red and black concrete chips could be observed in the cuttings as evidence that the hydrofraise was in contact with both adjacent primary panels. The second check was to core the sides of the primary panels two feet from the upstream and downstream faces of the secondary panel using a specialized downhole coring guide. If both cores encountered concrete, a minimum two feet of overlap was established.

During the excavation of the initial seepage barrier panels, significant slurry losses were experienced on numerous occasions (Davidson et al. 1992, Eckerlin 1993). Associated with several of the slurry losses were the development of cracks propagating from the ends of the panel. In one case, the cracks ran the entire length of the dam crest to the opposite river bank. It was determined that the cracking was caused by high slurry pressures that caused hydraulic fracturing of the core. The core was susceptible to hydraulic fracturing due to the low stresses in the core due to the arching mentioned above. At this point, work with the hydrofraise was temporarily halted in order to develop an alternative plan for constructing the seepage barrier.

To fill the cracks that had developed and increase the stresses in the core, a program of gravity and recompression grouting was performed (Davidson et al. 1992, Eckerlin 1993). The gravity grouting was performed by drilling a line of four-inch holes along the dam crest with a bentonite-cement slurry mix that would set after drilling. The grouting was performed until the level of the slurry in the hole began rising. The recompression grouting was performed in two lines of grout holes spaced at six-foot intervals. The grout was injected in successive stages in order to develop controlled hydraulic fractures in a
number of directions off of the grout holes. This resulted in a system of intersecting planes of grout that successively increased the stresses in the soil. On the average, grout take volumes were on the order of 9 percent of the treated volume of soil.

After completion of the compaction grouting, the remainder of the seepage barrier was constructed by hydrofaise (Davidson et al. 1992, Eckerlin 1993). The length of panels was limited to a single pass width of the hydrofaise to minimize excavation size and the length of time the panel was open. The deepest panel reached a world-record depth of 402 feet below the working platform. The top of the seepage barrier was extended upward 13 feet by constructing a one-foot thick, cast-in-place concrete wall on top of the subsurface barrier. Fill was placed around and above the cast-in-place wall to an elevation of 1,257 feet, raising the dam crest seven feet above the previous crest elevation. Construction of the seepage barrier was completed in April 1990.

Instrumentation

Instrumentation pertinent to the performance of the seepage barrier consists of piezometers, settlement monuments, and inclinometers. Descriptions of these instruments are presented below.

Piezometers. Eighty piezometers are located in the dam embankment and abutments downstream of the seepage barrier (USACE 2003). Many of these piezometers indicate hydraulic heads higher than the water levels in the reservoir. These levels are likely due to residual pressures in the core due to the recompression grouting or seepage into the dam from the abutments.

A number of piezometers located in the lower portions of the core and in the bedrock below the dam and in the abutments react to fluctuations in the reservoir level to varying degrees (USACE 2003). All of the reacting piezometers are located below an elevation of 1,000 feet. The behavior of these piezometers can be generalized as follows:
1. Most piezometers located in the rock upstream of the seepage barrier show responses to the reservoir fluctuations of nearly 100 percent.

2. Piezometers in the rock downstream of the seepage barrier show responses to reservoir fluctuations that vary from 20 to 100 percent.

3. Piezometers located upstream of the seepage barrier in the core show responses to the reservoir fluctuations of up to 100 percent.

4. Piezometers located downstream of the seepage barrier in the core below an elevation of about 950 feet show responses to the reservoir fluctuations varying from 20 to 50 percent.

5. Piezometers located downstream of the seepage barrier in the core above an elevation of about 950 feet show small responses to peaks in reservoir level. These responses usually take several weeks to develop.

**Settlement Monuments.** The settlement monuments indicated that up to a half a foot of settlement occurred in the central portion of the dam during and soon after the seepage barrier construction. Since construction, settlements have continued at a slower rate. The largest settlements since 1991 were detected in a line of three settlement points located about 200 feet upstream of the dam crest and ranged from 0.2 to 0.3 feet. Settlement monuments located along the dam crest have detected settlements of up to 0.18 feet since 1991, with the largest settlements occurring toward the center of the dam.

**Inclinometers.** Six inclinometers were installed within the seepage barrier during construction. Since completion of construction, lateral movements of two to three inches in the upstream direction have been recorded in the upper 150 feet of the seepage barrier.

**Assessment of Seepage Barrier Performance**

The seepage barrier was constructed to mitigate deterioration of the dam core by placing a continuous, non-erodible barrier in the core. The effectiveness of this barrier in blocking flows through the core is difficult to assess due to the apparent flow around the barrier through open bedrock joints.
The following is an interpretation of the seepage behavior of the dam, core, seepage barrier and bedrock based on the history of the observations made in the piezometers summarized above.

1. As the reservoir level fluctuates, water in the upstream dam shell and bedrock joints upstream of the seepage barrier respond quickly to the reservoir level. The response is essentially 100 percent.

2. Downstream of the seepage barrier, the response to reservoir fluctuation in the bedrock varies from 20 to 100 percent. The decrease in reservoir response can be attributed to (1) the seepage barrier blocking some of the most open and developed joints in the top 15 feet of bedrock, and (2) longer seepage paths through the deeper portions of the bedrock that contain constrictions that result in head loss. However, based on several downstream piezometers that show 100 percent or near 10 percent response to reservoir fluctuations, it appears that some seepage pathways through the bedrock provide nearly direct connections with the reservoir.

3. Piezometers in the lower portion of the core indicate up to 100 percent responses to reservoir fluctuations upstream of the seepage barrier and 20 to 50 percent responses below. These responses are likely due to seepage into the portions of the core that had deteriorated prior to the construction of the barrier. Borings into the lower portions of the core encountered zones of clean gravels that, based on the assumed mechanism of their formation, align with open bedrock joints. The upstream seepage paths into the core have not been affected by the seepage barrier. Seepage paths into the portions of the lower core downstream of the seepage barrier appear to have been restricted by the construction of the seepage barrier.

4. Piezometers located higher in the core (above elevation 950) indicate a small response to reservoir fluctuations that lag the reservoir levels by several weeks. This behavior is likely due to a less compromised core than was observed in the lower levels, combined with lower gradients driving the seepage. Seepage in this
portion of the core is likely occurring through cracks in the core as discussed above.

Although long term changes in the piezometer behavior are difficult to assess due to the erratic changes in reservoir water level, there does not appear to be substantial evidence of long-term changes in piezometer behavior since construction of the seepage barrier.

The settlement and lateral movements detected in the dam since seepage barrier construction are relatively small for a dam the size of Mud Mountain Dam. Furthermore, the settlements along the dam crest may be associated with the placing of 17 feet of fill on the dam after completion of the seepage barrier. The pattern of movement (more settlement upstream, and upstream deflection in the inclinometers) could be indicative of continued erosion of the fine portion of the core upstream of the seepage barrier. However, because the magnitude of settlement observed to date is small, linking the observed settlements to an erosion mechanism would be speculative.

While long-term monitoring of the dam does not indicate significant changes in the seepage regime since seepage barrier construction, the piezometers do indicate significant groundwater fluctuations in the core upstream and downstream of the seepage barrier. These groundwater fluctuations were responsible for the core deterioration prior to seepage barrier construction and should be considered a mechanism for possible continued removal of fines from the core. This process could lead to deformations within the dam that could eventually affect the dam performance. Therefore, continued monitoring of the dam is essential.

**Jackson Lake Dam**

**Background**

Jackson Lake Dam shown in Figure 3-65 is managed by the U.S Bureau of Reclamation and is located on the South Fork of the Snake River within Grand Teton National Park in
northwestern Wyoming. The dam was originally constructed as a rock crib dam in 1907, partially failed in 1908, and completely failed in 1910. The dam was reconstructed in 1911 and enlarged in 1916. The 1911 dam consisted of a 202-foot long concrete spillway and a 2,600 foot long earth embankment with a 300-foot long concrete core wall and a 2,300-foot sheetpile cutoff. In 1916 the spillway was replaced with a 222-foot long spillway and the embankment was lengthened, raised and realigned using hydraulic fill construction (Miedema and Farrar 1988, USBOR 2005).

Investigations conducted in from 1975 through 1984 concluded that the portions of the 1916 hydraulic-fill embankment and the upper parts of the foundation of the dam were susceptible to liquefaction during seismic shaking (Miedema and Farrar 1988, USBOR 2005). The uppermost soils beneath the embankment consist of a 20 to 30 foot thick layer composed of sands, gravels, and very low blow count silts. Underlying this layer is a layer consisting of medium-dense sand and silty sand that is approximately 20 to 40 feet thick and a silty sand layer about 10 to 20 feet thick. Portions of these layers where deemed potentially liquefiable during the design seismic event. Below the potentially liquefiable layers the remainder of the deposits consist of low plasticity fine grained soils that were assessed to have low liquefaction potential. The boundary between the
potentially liquefiable and non-liquefiable soils was referred to as the “silt boundary” in the design documents.

Modifications to the dam were made between 1986 and 1989 to correct these deficiencies (Miedema and Farrar 1988, Ryan and Jasperse 1989, Pujol-Rius et al. 1989, Farrar et al. 1990, USBOR 2005). These modifications included a complete reconstruction of the embankment north of the concrete spillway section. After stripping away the embankment the central portion of the foundation beneath the embankment was densified using deep dynamic compaction and deep-mixed soil-cement columns arranged in a honeycomb pattern to create 50-foot wide buttresses under the upstream and downstream toes of the embankment. Finally, in order to reduce the water pressures in the embankment and further reduce the liquefaction susceptibility, a 4,000-foot long soil-cement seepage barrier was constructed by the deep mixing method along the upstream toe of the embankment. The buttresses and the seepage barrier were constructed to depths 10 and 20 feet below the “silt barrier”, respectively.

The embankment has a maximum height of 45 feet and a crest elevation of 6780 feet. The embankment is 4580 feet long and decreases in height with distance away from the concrete spillway. A photograph of the reconstructed dam is shown in Figure 3-65.

A cross section of the new embankment, constructed from 1986 to 1989, is shown in Figure 3-66. One of the primary design considerations for the new embankment was minimizing the pore pressures in the foundation beneath the dam. To impede the seepage flow the embankment design includes a low-permeability core connected to an upstream low-permeability blanket. The low-permeability blanket ties in with the seepage barrier located 20 feet upstream of the toe of the embankment. Underlying the low-permeability Zone 1 blanket is a blanket drain (Zone 2) consisting of filter/drain material that runs beneath the entire embankment. The blanket drain is connected to the toe drain beneath the downstream toe of the dam. Shell material supports the slopes of the embankment and the upstream face of the dam is protected with a soil-cement revetment.
Figure 3-66 Cross section of Jackson Lake Dam embankment and foundation modifications (modified from USBOR 2005, with permission from USBOR)

Design and Construction of Seepage Barrier

The seepage barrier was constructed using the deep mixing method, DMM, also referred to as deep soil mixing, DSM, and soil mixed wall, SMW (Ryan and Jasperse 1989, Pujol-Rius et al. 1989, Farrar et al. 1990, USBOR 2005). This method uses hollow-stem augers to inject a slurry of cement and water into the in-situ soil and blend the soil and slurry into a soil-cement material. The equipment used on the Jackson Lake Dam project consisted of a rig equipped with three overlapping 3-foot diameter augers. The overlap of the augers results in a nominal wall thickness of about 2 feet. The seepage barrier was constructed in primary and secondary elements. The primary elements consisted of one pass with the three augers, and were installed leaving the width of one auger (minus the overlap between augers) between primary elements. The secondary elements were installed shortly after the primary elements (before the soil cement had hardened) so that the outside augers of the element could follow the previously treated paths of the adjacent primary elements. This procedure helped maintain continuity of the seepage barrier.

The mix design for the seepage barrier consisted of injecting 200 pounds of cement per vertical foot of each column by way of a slurry having a water to cement ratio of 1.35 by
weight (Ryan and Jasperse 1989, Farrar et al. 1990). This procedure resulted in a 573 pounds of cement injected for every cubic yard of treated material. Tests performed on specimens prepared from wet samples of the in-situ soil-cement mixture resulted in 28-day unconfined compressive strengths ranging from 400 to 800 psi.

Instrumentation

The instrumentation installed during and shortly after the dam modifications made from 1986 to 1989 consist of: (1) seepage measurement points (weirs), (2) observation wells, (3) vibrating-wire piezometers, and (4) dam embankment settlement measurement points (USBOR 2005). Details of each of type of instrumentation are presented below.

Seepage Measurement Points. Four V-notch weirs measure flows emitting from the toe-drain. Data from these weirs has been collected since 1989. While flow measurements are often a very good indicator of the overall seepage performance of a dam, evidence suggests that factors other than seepage are affecting the seepage flows measured in these devices (USBOR 2005). The most compelling evidence to this effect is that the measured flows peak each year as the reservoir is still rising and decline as the reservoir continues to rise. It is thought that groundwater from melting snow and spring rain infiltrating the toe drain is responsible for most of the seasonal fluctuation detected by the weirs. Nonetheless, the seepage volumes measured have remained relatively constant since 1989, and the measured flows are not considered problematic.

Observation Wells. Three observation wells were drilled adjacent to the toe drain to monitor groundwater levels near the downstream toe of the dam. Similar to the seepage measurement weirs, seasonal fluctuations in the observation wells do not appear to be controlled by reservoir fluctuations and sources other than seepage from the reservoir are thought to be affecting the readings in these wells. Nonetheless the fluctuations in the observation wells appear to be consistent since 1989.
Piezometers. In all, 58 vibrating-wire piezometers that have been monitoring pore pressures in the dam and foundation since 1989 (USBOR 2005). In a majority of the cases, the current piezometer levels and response to reservoir fluctuations are essentially the same as those observed immediately following the construction of the new embankment and seepage barrier. Six piezometers, two in the foundation and four in the dam core or low-permeability blanket, have experienced either an increase or decrease in response to reservoir fluctuations since 1990. These fluctuations are minor and likely due to small changes in the seepage path or the piezometer performance and are not thought to be significant with respect to the performance of the dam.

Three piezometers located on the downstream side of the seepage barrier. Two of these piezometers have experienced head drops on the order of 5 feet between 1990 and 2000. It is theorized that the cause of the observed head drops could be due to one or a combination of the following: (1) the dissipation of excess pore pressures developed during construction, (2) increased efficiency of the barrier due to the clogging of cracks or defects, or (3) decreased seepage resistance downstream of the piezometers.

Settlement Monuments. Four lines of settlement and deflection monitoring points are located on the embankment located 60 and 15 feet upstream of the centerline and 18 and 60 feet downstream of the centerline. These monuments have been surveyed every three years since embankment reconstruction. These measurements indicate settlements of up to 0.6 feet in the embankment since reconstruction. The settlements are greatest at the crest where the embankment height is greatest and decrease toward the toes of the embankment. The observed settlement pattern and magnitudes are considered within the anticipated range for consolidation-related settlement of the embankment and foundation.

Assessment of Seepage Barrier Performance

The piezometer data collected since the construction of the embankment and seepage barrier indicates that the behavior of the piezometers has not changed significantly since the installation of the barrier. Annual fluctuations observed in some of the measured
piezometric levels measured are generally tied to the pool level in the reservoir and were observed to be following the same trends as they did shortly after the seepage barrier construction. Thus, based on the piezometer data, it can be assessed that the performance of the seepage barrier has not changed significantly since construction.

**Twin Buttes Dam**

### Background

Twin Buttes Dam is managed by the City of San Angelo and the U.S Bureau of Reclamation and is located directly outside of the City of San Angelo, Texas. The dam is 8.2 miles long and spans three drainages: the South Concho River, Spring Creek, and the Middle Concho River. Two pools are retained behind the dam - the South Concho Pool and the Middle Concho Pool. The pools are connected by an equalization channel that keeps the water levels in the two pools the same when the reservoir levels are above elevation 1,926 feet. Twin Buttes Dam is operated in conjunction with Lake Nasworthy, which is located just below the left end of Twin Buttes Dam. Lake Nasworthy is generally kept full and maintained at an elevation of 1872 feet.

Construction of the dam was completed in 1963. The dam is a homogeneous rolled earthfill embankment with a maximum height of 134 feet (crest elevation 1,991 feet) (USBOR 2000b). An uncontrolled concrete spillway with a crest elevation of 1,969 feet is located on the left abutment. The outlet works are located in the left abutment and consists of three 15-½-foot diameter pipes on the upstream side of the outlet that transition to three horseshoe shaped outlets on the downstream side. A map showing the layout of the dam is shown in Figure 3-67.

The dam is constructed in a wide, alluvium-filled valley underlain by shale bedrock of the San Angelo Formation. The upper portion of the alluvium consists of clayey wind-blown and alluvial deposits that vary in depth from 10 to 50 feet. Beneath the clayey soils are deposits of partially-cemented, gravelly alluvium that range in thickness from
zero to 60 feet. The gradation, cementation, strength and permeability of the gravel layers vary greatly, making characterization of the seepage behavior and strength of the soils difficult.

A cross section of the dam embankment is shown in Figure 3-68. Most of the dam embankment is constructed of Zone 1 material consisting of select clay, silt and sand excavated from borrow areas in the upper layers of the alluvium (USBOR 2000b). While the embankment is predominantly homogenous, selected gravelly material from required foundation excavations was allowed in the downstream portion of the embankment (Zone 2). The inclination of the downstream slope varies from 2:1 (horizontal to vertical) in the upper portion of the dam to 4:1 in the lower portions. The downstream face is protected by a 10-foot thick layer of compacted caliche. The inclination of the upstream slope...
varies from 2:1 (horizontal to vertical) above the spillway elevation and flattens to 3:1 in the lower portions of the dam. The downstream face is protected by riprap.

![Figure 3-68 Cross section through Twin Buttes Dam (modified from USBOR 2000b, with permission from USBOR)](image)

A cutoff trench was constructed beneath the embankment where the alignment of the embankment crossed Spring Creek, the Middle Concho River, and the South Concho River. The reaches where the cutoff trench was constructed are indicated on Figure 3-69. The criteria for deciding where the cutoff was to be constructed were: (1) the clay layer was not thick enough to prevent vertical percolation of water into the gravel layers, and (2) the construction did not require “excessive excavation” (USBOR 2000b). The cutoff is centered 100 feet upstream of the crest centerline and the base of the cutoff varies from 20 to 100 feet wide with side slopes inclined at 1-½:1 (horizontal to vertical). The depth of the cutoff varied up to about 50 feet. Zone 1 material was used to backfill the cutoffs. Approximately 70 percent of the dam was constructed without a positive cutoff.

Similarly, a five-foot thick drainage blanket was constructed in selected reaches under the lower half of the downstream slope. The reaches where the seepage blanket was constructed are shown on Figure 3-69. Approximately 65 percent of the embankment was constructed without a blanket drain.

**History of Seepage Problems and Mitigation**

Since reservoir filling, the central portion of the dam has been prone to foundation seepage. During construction, borrow areas for the embankment fill were located as
close as 150 feet from the upstream toe of the dam (USBOR 2000b). These borrow areas removed the upper clayey soils and exposed the gravel areas to direct contact with the reservoir. Additionally, a portion of the equalization channel was excavated into gravel layers.

Seepage was first noted in 1964, one year after completion of construction (Shadix 1981, USBOR 2000b). The seepage occurred in areas beneath the reaches of dam without cutoff trenches and increased with rising reservoir levels. In 1974, when the reservoir level rose to elevation 1940 feet, the seepage areas expanded and the groundwater level as far as 4,000 feet downstream from the embankment rose to within 2 feet of the ground surface. In response, the City of San Angelo constructed a series of drainage ditches to remove the surface water. In 1975, seepage from these drains was measured at 21 cubic feet per second.
Between 1976 and 1980, grouting was performed in the gravels beneath the embankment where no seepage cutoff had been constructed (USBOR 2000b). The reaches of the dam where the grouting was performed are indicated on Figure 3-69. A total of 889 grout holes were drilled and 175,385 sacks of cement were pumped into the foundation. While the grouting appeared to decrease the seepage volume, it did little to relieve the uplift pressures at the downstream toe of the dam.

From 1982 to 1984, a series of relief wells and a subdrainage system were constructed in two critical areas along the downstream toe of the dam (USBOR 2000b). The reaches where the relief wells were constructed are indicated on Figure 3-69. Forty-four wells ranging in depth from 30 to 95 feet were constructed at 100- to 150-foot spacings between Stations 89+00 and 180+00. An additional 15 wells ranging in depth from 40 to 65 feet were constructed at 150-foot spacings between Stations 245+00 and 280+00. Subdrains to collect near-surface water were installed along both reaches.

The amount of flow into the relief wells was highly dependant on the water surface level in the reservoir. When the reservoir level was below elevation 1920 feet, the relief wells did not flow. In 1986 and 1990, when the reservoir rose to a level of 1936 feet, piezometric levels downstream of the embankment indicated that the relief well system was ineffective. High piezometric pressures developed at the downstream toe of the dam, and the piping potential in this area was deemed unacceptably high.

Following assessment of the relief wells, it was decided to construct a seepage barrier on the upstream side of the embankment in locations where a positive cutoff was not provided in the original construction. Details of the design and construction of the seepage barrier are presented below.

**Design and Construction of Seepage Barrier**

Construction of the barrier started in August of 1996 and was completed in February of 1999. The location of the seepage barrier with respect to the embankment is shown on
Figure 3-69. The barrier is about four miles long and spans the portion of the embankment between the reaches where cutoff trenches were constructed during the original embankment construction. Construction of the barrier was performed on a working platform constructed on the upstream toe of the embankment. Details of the barrier construction are shown on Figure 3-70. The barrier has a minimum width of 2.5 feet, was constructed through the alluvial deposits, and is embedded a minimum of 2.5 feet into the underlying shale bedrock. The depth of the barrier varies between 44 and 104 feet with an average depth of about 65 feet.

The trench excavation was held open with water-bentonite slurry until backfill could be placed. The backfill consisted of a mixture of the excavated soils, cement, and bentonite (SCB backfill). The SCB backfill was selected for use over a soil-bentonite backfill due to concern with hydraulic fracturing under high reservoir water pressures and blowout due to high hydraulic gradients acting across the barrier. After the SCB backfill was set, a geomembrane was placed on top of the barrier and connected to the low permeability portion of the original embankment. A six-foot thick layer of clay was placed on top of the barrier and geomembrane.
Instrumentation

The performance of the seepage barrier is monitored by 18 vibrating wire piezometers installed during the construction of the barrier (USBOR 2000b). All of the piezometers are located in the alluvium beneath the embankment. Monitoring of the piezometers began in 1997 for two of the piezometers, in 1998 for eight more, and in 1999 for the final eight. Sets of three piezometers are located along cross sections at each end of the seepage barrier. Ten piezometers are installed in sets of two along the barrier alignment with one piezometer upstream of the barrier and the other on the downstream side. The final two piezometers are located downstream of the barrier near the bend in embankment alignment.

In addition to the 18 piezometers installed around the seepage barrier, there are 160 piezometers and observation wells near the downstream toe of the dam. Forty of these wells were selected to provide a representative piezometric section with which to monitor and evaluate the seepage barrier performance (USBOR 2000b). In addition, flows in drainage ditches and relief wells below the toe of the embankment can be monitored to evaluate the seepage barrier effectiveness.

Since construction of the seepage barrier, the reservoir has been at its lowest levels since the original construction in 1964 and has not risen past elevation 1910 feet in the Middle Concho Pool. Water levels in the South Concho Pool have remained higher, and since barrier construction have been close to the invert level of the equalizing channel (elevation 126 feet) as no other outlet exists for this pool. The downstream groundwater elevation is generally controlled by Lake Nasworthy which is maintained at a level of 1,874 feet. Because of the low reservoir levels, it has not been possible to monitor the performance of the seepage barrier under the hydraulic loading conditions for which it was designed. However, there are still differential water levels across the dam and the performance of the seepage barrier under these conditions is discussed below.
The three piezometers at the right end of the seepage barrier indicate a steady gradient in the alluvium below the embankment from the South Concho Pool elevation of about 1,925 feet to the downstream water elevation of about 1,884 feet. The sets of piezometers installed across the seepage barrier measure differential hydraulic heads across the barrier that range from about 35 feet near the right end of the barrier to about 20 feet near the left end of the barrier. The decrease in differential head across the barrier is largely due to a change in the upstream piezometric surface due to the difference in water level between the South Concho and Middle Concho Pools.

Most of the piezometers located just downstream of the seepage barrier indicate steady water levels that do not fluctuate with water level. The exception is Piezometer VW-18, which fluctuates with reservoir level, but at a piezometric level about 20 feet lower than measured by other piezometers upstream of the barrier. VW-18 is located in an area where vertical joints were identified in the foundation bedrock. Thus, it can be assumed that the reservoir response of VW-18 is likely due to seepage beneath the barrier through the bedrock joints.

The downstream piezometers and observation wells indicate the water levels downstream of the dam are constant and largely controlled by Nasworthy Reservoir. Since construction of the seepage barrier, the relief wells and drainage ditches have not flowed. This is a consequence of the low reservoir levels, and the effect of the seepage barrier on the relief wells and drainage ditches cannot be assessed until reservoir levels rise significantly.

Assessment of Seepage Barrier Performance

Because the reservoir levels have not been near the elevations for which the seepage barrier was designed or levels where significant seepage problems have been observed in the past, the ability to assess the performance of the seepage barrier is significantly limited. Under the present conditions, the barrier appears to be effective in lowering the piezometric levels beneath the embankment and beyond the downstream toe. However,
with increased upstream piezometric levels will come increased hydraulic gradients across the wall and around its boundaries as well as higher differential water pressures across the barrier. These conditions will have a higher likelihood of initiating mechanisms of internal erosion and barrier deformation that can lead to changes in the performance of the barrier.

**Chambers Lake Dam**

**Background**

Chambers Lake Dam is managed by the Water Supply and Storage Company of Fort Collins, Colorado, and is located in the on Joe Wright Creek a few miles north of Rocky Mountain National Park in Larimer County, Colorado. (Woodward-Clyde Consultants 1988). The dam was first constructed in 1910 and raised to its present height between 1923 and 1926. A plan of the dam is shown on Figure 3-71. The dam consists of a 1,400-foot long homogenous earth embankment and has a structural height of about 55 feet with a crest elevation of 9,167 feet. The embankment consists of two sections: the main embankment section comprising the left-most 900 feet and the Right Dike section comprising the right-most 400 feet. The embankment in the Right Dike section is less than 10 feet high. The upstream slope varies from 2.5:1 (horizontal to vertical) in the upper 11 feet and 3:1 (horizontal to vertical) in the lower portions of the embankment. The downstream slope is inclined at 3:1 (horizontal to vertical). The spillway is located beyond the left abutment of the dam and consists of a four-foot wide concrete sill with a crest elevation of 9,158 feet. The outlet works are located near the center of the dam and consists of a four-barrel box conduit controlled by cast iron gates located just downstream of the dam crest. A cross section of the dam is shown of Figure 3-72 indicating configurations before and after modifications added in 1992.

Most of the terrain in the area of the dam is covered with Quaternary age glacial moraine deposits consisting of poorly sorted silt, sand, gravel, cobbles, and boulders (Woodward-Clyde Consultants 1988). The dam foundation consists of loose to very dense clayey and
silty sands and gravels interbedded with firm to stiff silt and clay layers overlying siltstone and claystone bedrock. Underlying the main dam embankment, the foundation soils consist predominantly of silty and clayey sand and gravel. Piezometers in this portion of the foundation react to changes in the reservoir elevation with a time lag of about one day, indicating these soils have moderate permeability. Under the Right Dike area, the upper foundation soils consist of a layer of slightly silty sand about 25 to 35 feet thick underlain with silt and silty gravel. Prior to the installation of the seepage barrier (described below) piezometers in this area and a spring downstream of the dike responded rapidly to reservoir level changes, indicating these soils are highly permeable.

Figure 3-71  Plan view of Chambers Lake Dam (after Woodward-Clyde Consultants 1992, with permission from URS Corporation)

Figure 3-72  Cross section of Chambers Lake Dam (after Woodward-Clyde Consultants 1992, with permission from URS Corporation)
The dam embankment consists of variable mixtures of sands, silts, clays, and gravels ranging in consistency from firm to stiff. Piezometers located in the dam embankment react to changes in reservoir level with a time lag varying between two and eight weeks, indicating a greater resistance to seepage than in the foundation soils.

History of Seepage Problems and Mitigation

Seepage problems at Chambers Lake Dam were first documented during an inspection by the Colorado Division of Water Resources in 1974, when leakage at the right side of the dam appeared to increase when the reservoir level rose above elevation 9,154 feet (Woodward-Clyde Consultants 1988). As a result of this observation, two V-notch weirs were installed in 1975 to measure seepage volume downstream of the Right Dike area and a subsurface investigation was performed in 1977.

The dam was inspected by the U.S. Army Corps of Engineers (USACE) in 1978 and was found to have an “unsatisfactory steady seepage condition” in the lower half of the embankment and excessive leakage in the area of the Right Dike (Woodward-Clyde Consultants 1988). The USACE recommended (1) restricting the reservoir level to below elevation 9,154 feet, and (2) engaging a consultant to investigate the source of the seepage and evaluate the stability of the embankment.

In 1983, a slump occurred on the downstream slope of the main embankment adjacent to the headwall for the outlet works (Woodward-Clyde Consultants 1988). The slump involved about 15 cubic yards of material and the downstream face of the embankment was observed to be saturated below an elevation 9,136 feet. The slump was attributed to the melting of an unusually large snow pack on the embankment. The slumped zone was removed and replaced with clean granular fill.

In 1987 and 1988, Woodward-Clyde Consultants conducted a study to investigate the observed seepage and stability problems (Woodward-Clyde Consultants 1988). Based on
that study, it was recommended to construct a seepage berm on the downstream side of the main embankment and to construct a seepage barrier through the Right Dike.

**Design and Construction of Seepage Mitigation**

To mitigate the seepage and stability problems along the dam, a seepage berm was designed and constructed along the downstream slope of the main dam embankment as shown in Figure 3-74. To mitigate the underseepage problems occurring in the Right Dike area, a slurry wall seepage barrier was constructed through the permeable foundation soils. The locations of the elements of the mitigation are presented in Figure 3-73. A cross section through the area treated with the seepage berm is shown in Figure 3-72 and a cross section through the area treated with the slurry trench seepage barrier is shown on Figure 3-74.

![Figure 3-73 Plan of seepage mitigation elements at Chambers Lake Dam (after Woodward-Clyde Consultants 1992, with permission from URS Corporation)](image)

The seepage barrier was constructed using the slurry trench technique (Woodward-Clyde Consultants 1992). The embankment was removed to the ground surface and a five-foot deep key trench was excavated and backfilled with low permeability material along the barrier alignment. The barrier was then constructed through the key trench, providing five feet of overlap between the top of the barrier and the bottom of the low permeability fill.
The seepage barrier was originally planned to be a two-foot wide cement-bentonite barrier with the trench supporting slurry hardening in place as the excavation advanced. However, boulders encountered in the trench slowed the progress of the excavation to the point where the cement-bentonite slurry was setting before the section of the excavation could be complete. After about 50 feet of the cement-bentonite barrier was constructed in about five days, the design was modified to a three-foot wide soil-bentonite barrier and the remainder of the construction (about 450 feet) was completed in about 18 days.

Along most of the alignment, the barrier was extended down into the siltstone and claystone bedrock. At the far right end of the seepage barrier, excavation difficulties prevented excavation to bedrock, and the base of the barrier was terminated in silty to clayey gravel layers beneath the permeable sand layer.

Instrumentation

Instrumentation of the Chambers Lake Dam seepage barrier consists of four piezometers and three V-notch weirs. These instruments are discussed below.
Piezometers. The piezometers are located on the right end of the embankment downstream of the seepage barrier. Prior to seepage barrier construction the piezometers in this area fluctuated with reservoir levels with little or no time lag. With the seepage barrier in place the piezometer behavior has changed significantly. The piezometers generally read their highest levels in late June or early July when the reservoir level is rising. The piezometer levels then drop through the remainder of the summer while reservoir level remains within a few feet of the peak reached in the early summer. This behavior indicates that the piezometers are influenced by factors other than the reservoir. In this case, a significant portion of the piezometer rise is likely due to the melting snow pack downstream of the Right Dike and on the hillside above. Although the effects of the snow pack make assessing the effects that reservoir level has on the piezometers difficult, it is clear that the piezometers respond significantly less than before the barrier construction, and with a significant time lag.

Weirs. The three weirs located below the right end of the dam have a similar response to reservoir fluctuations as do the piezometers. Peak flows are measured in late June and early July as a result of the melting of the snow pack, and then drop to lesser values while the reservoir remains at a high level. It is apparent by the late summer flows that some of the seepage measured in the weirs is from the reservoir. However, the flows measured after the seepage barrier construction are less than half of the flows measured before barrier construction.

Assessment of Seepage Barrier Performance

Although the effects of the snow melt make it difficult to quantify the efficiency of the seepage barrier, the piezometer and weir measurements indicate that the seepage barrier has significantly reduced the flow through the clean sand layers underlying the right end of the dam. Although the water levels in the piezometers and flow volumes through the weirs drop over the summer, they remain at levels higher than in the fall, winter, and spring when the reservoir is at a low level. This indicates that seepage from the reservoir is still occurring, albeit at a reduced rate.
The post-barrier seepage can be attributed to three potential sources: (1) seepage around the left end of the barrier, (2) seepage below the barrier, and (3) seepage around the right end of the barrier. The volume of seepage around the left end of the barrier (beneath the main embankment) is likely small due to the moderate permeability of the foundation soils and the tendency of water seeping in this area to flow toward the drains in the seepage berm. Although the barrier was not excavated to bedrock in a few locations, the seepage volume beneath the barrier is also likely small due to the moderate permeability of the soils between the base of the barrier and the bedrock. The largest volume of flow around the barrier is likely occurring around the right end of the barrier where the sand layer that the barrier was designed to intercept is still intact. Flow through this sand layer around the right end of the barrier likely accounts for most of the seepage flows occurring in this part of the dam.

The performance of the seepage barrier appears to be stable since construction. While there are likely high hydraulic gradients beneath and around the ends of the barrier, the soils upon which these gradients act appear to be internally stable and protected from erosion by the adjacent down-gradient soils. Therefore, the seepage resistance around the barrier has not changed over time.

Manasquan Dam

Background

Manasquan Dam is managed by the New Jersey Water Supply Authority and is located on Timber Swamp Brook in the Howell Township in Monmouth County, New Jersey. The dam was constructed between 1987 and 1990 and consists of a 4900-foot long homogeneous earth embankment with a soil-bentonite seepage barrier as the primary means of seepage resistance (Woodward-Clyde Consultants 1990). The embankment has a structural height of 53 feet with a crest elevation of 113 feet. The upstream slope is 3:1 (horizontal to vertical) and the downstream slope is 2.5:1 (horizontal to vertical) and has
a bench midheight. The emergency spillway is an earth lined channel located on the left abutment. The service spillway is incorporated in the inlet/outlet works which are located near the center of the dam, near the maximum dam section.

The dam is located in an area underlain by coastal plain sediments. Three geologic formations comprise the dam foundation: the Kirkwood, Manasquan, and Vincentown Formations (Khoury et al. 1992). The Kirkwood Formation consists of a predominantly sandy upper stratum and a lower stratum that consists of interbedded layers of silt, sand, and clay. Underlying the Kirkwood Formation is the Manasquan Formation. The upper unit of the Manasquan forms the base clay layer beneath the dam and reservoir. The lower portion of the Manasquan Formation grades into a sandy soil. The Vincentown Formation underlies the Manasquan and consists predominantly of sand. The Vincentown Formation forms the main groundwater aquifer in the area.

A longitudinal section along the axis of the dam is shown on Figure 3-75. The base of the embankment is in contact with the Upper and Lower Kirkwood Formation soils along most of the alignment (Khoury et al. 1992). Original plans called for foundation of the maximum embankment section to be constructed on the Upper Manasquan clay layer. However, during construction it was discovered that the Manasquan clays had been mined under the central portion of the embankment footprint and the clay blanket that the seepage barrier was to keyed into was missing in this area. The dam foundation was modified where the clay had been excavated by constructing a 10-foot thick imported clay layer beneath the dam crest and a five-foot thick clay blanket extending upstream for the extent of the mining as shown on Figure 3-76-A.

Two cross sections through the embankment are shown on Figure 3-76. One cross section (Figure 3-76-A) depicts the area where the clay had been mined and the other (Figure 3-76-B) depicts the areas where clay has not been mined. The embankment was constructed with fine sands borrowed from the Kirkwood Formation. Due to the high permeability of the embankment and foundation soils, a soil-bentonite seepage barrier was included in the original design. The seepage barrier varies from three to five feet
wide, has a top elevation of 108 feet (five feet above the normal pool elevation) and keys a minimum of five feet into the Manasquan clay layer or the compacted clay repair in the mined area. Chimney and blanket drains were constructed in the downstream portion of the embankment. The upstream face of the dam is armored with riprap and the downstream face has grass erosion protection.

Figure 3-75 Longitudinal section of Manasquan Dam embankment and foundation (modified from Khoury et al. 1992, with permission from ASTM)

A 10-foot diameter prestressed reinforced concrete outlet pipe penetrates the base of the embankment near the maximum section (see Figure 3-75). The pipe is connected to the outlet tower upstream of the dam. An envelope of compacted clay was constructed around the pipe as shown on Figure 3-75.

Design and Construction of Seepage Barrier

The soil-bentonite seepage barrier design was selected over a cement-bentonite barrier based on cost and the ability to achieve a very low permeability. The main concern in the soil-bentonite barrier design was the impacts of differential settlement between the barrier and the embankment (Khoury et al. 1992). The settlement potential of the barrier backfill was expected to be much higher than the compacted embankment sands. As the barrier backfill settles, drag on the embankment soils tend to reduce the total stresses in the barrier, making the barrier susceptible to hydraulic fracturing when the reservoir water pressures come in contact with the barrier.
Figure 3-76  Cross sections of Manasquan Dam at (A) Station 16+00, the maximum section, and (B) Station 13+00 (modified from Woodward-Clyde Consultants 1990, with permission from New Jersey Water Supply Authority)
The width of the seepage barrier was designed to take into account the potential for hydraulic fracturing. Based on the design recommendations of Xanthakos (1979) and the U. S. Army Corps of Engineers (USACE 1986b), the barrier width was designed to be at least 0.1 times the anticipated differential hydraulic head acting across the barrier (Khoury et al. 1992). This resulted in a barrier width of five feet in the central portion of the dam where the embankment is at its maximum height and three feet where the lower portions of the seepage barrier are in native materials.

The seepage barrier was constructed in two stages (Khoury et al. 1992). The lower stage was constructed when the top of the embankment was at an elevation of about 98 feet and extended a maximum depth of 64 feet. The bottom of the first stage was embedded a minimum of 5 feet into the Manasquan clay layer. The upper stage was constructed from an elevation of 108 feet and had an average depth of 18 feet. The upper stage was constructed two months after the completion of the lower stage.

Both stages were excavated using a backhoe and the trenches were supported with bentonite slurry until backfilled (Khoury et al. 1992). Backfill material consists of the excavated soils mixed with bentonite slurry. The mixing was performed on the top of the embankment using a bulldozer. Backfilling was started in the south ends of the trenches by pumping the backfill through a 21-inch diameter tremie pipe. Once a slope of backfill had been established for the full height of the trench, the remainder of the trench was backfilled by pushing the backfill into the trench at the top of the slope and allowing the backfill to slump into the open portion of the trench. Backfill was also tremied into the lower trench on the north side of the 10-foot pipe penetration so that there would not be a vertical drop as the backfilling progressed over the penetration.

The sand content of the slurry was closely monitored to prevent the accumulation of sand at the base of the trenches. The bases of the trenches were also monitored for the accumulation of sand. This was especially important in the upper trench where any sand filled windows in the barrier would be in contact with permeable embankment sands.
Instrumentation

Monitoring instruments installed during and shortly after dam construction that are relevant to the performance of the seepage barrier consist of: (1) 41 vibrating wire piezometers, (2) two flow weirs, (3) four inclinometers, (4) five vibrating wire stress cells, (6) 33 surface movement monuments, and (7) five settlement plates (Woodward-Clyde Consultants 1990). The locations of the instruments are shown on Figure 3-77. The piezometers, weirs, and stress cells are connected to an automated data collection system that records readings from the instruments on a daily basis. Details of each of type of instrumentation are described below.

Vibrating Wire Piezometers. The 41 piezometers are arranged to monitor five different cross sections of the dam (Woodward-Clyde Consultants 1990). The locations of the piezometer cross sections are presented on Figure 3-77. One section is at the maximum section of the dam and two sections are located north and south of the maximum section. In the maximum section, most of the piezometers are located in the embankment fill with three piezometers located in the underlying Vincentown Formation, as shown in Figure 3-76-A. In the north and south sections, the piezometers are located predominantly at the interface between the embankment and foundation or in the underlying foundation soils of the Kirkwood and Manasquan Formations (see Figure 3-76-B for typical piezometer arrangement in these sections).

The hydraulic head readings in the five piezometer sections indicated similar conditions along the length of the embankment soon after the filling of the reservoir. The piezometer readings indicate the following (Woodward-Clyde Consultants 1990, Khoury et al. 1992):

- Hydraulic heads upstream of the seepage barrier were at or slightly below the reservoir level.
- Hydraulic heads downstream of the seepage barrier were controlled by the blanket drain or were slightly higher than the blanket drain in the area between the drain and the seepage barrier.
Figure 3-77  Plan view of Manasquan Dam showing locations of instrumentation (from Woodward-Clyde Consultants 1990, with permission from New Jersey Water Supply Authority)
Hydraulic heads in the seepage barrier were measured at intermediate levels between the upstream and downstream levels. Lines representing the piezometric surface are shown on the two sections in Figure 3-76.

Since the first filling, the position of the piezometric surface has remained essentially the same (NJWSA 1992, 2000, 2005). Readings of many of the piezometers indicate a small gradual change in water elevation that, in most cases, is less than 2 feet. While these small changes may reflect actual changes in water pressure at the piezometers, they are also in the range that may be attributable to zero drift of the piezometers. Zero drift is a phenomenon common in vibrating-wire piezometers where the base reading (zero pressure reading) drifts over time resulting in a gradual change in the piezometer reading. In any respect, the piezometer readings indicate that the hydraulic pressure regime in the dam has not changed significantly since the initial filling.

Flow Meters. Flows from the chimney and blanket drains are measured in two weirs located at the downstream toe of the embankment. Prior to reservoir filling, the north and south weirs indicated average flow rates of 6.3 and 1.5 gallons per minute (gpm), respectively (Woodward-Clyde Consultants 1990). Flows from the south weir were relatively constant while flows from the north weir showed fluctuations of up to 2 gpm. These fluctuations appeared to be due to changes in the groundwater level due to seasonal variation and rainfall events.

The filling of the reservoir did not appear to affect the flow rates measured at the weirs (Woodward-Clyde Consultants 1990). Flow volumes and fluctuations observed after reservoir filling are similar to those observed prior to reservoir filling. Since the initial reservoir filling, the measured flows have remained essentially the same with fluctuations attributable to seasonal variation, rainfall events, and instrument calibration (NJWSA 1992, 2000, 2005).

Inclinometers. Four inclinometers were installed in the embankment to measure horizontal deformation (Woodward-Clyde Consultants 1990). Three inclinometers are
installed in the maximum section; one each on the upstream and downstream sides of the
dam crest and one on the downstream bench (see Figure 3-76-A for locations). The
fourth inclinometer is located at the dam crest on a section to the north of the maximum
section.

Upon initial filling of the reservoir, the inclinometers located near the dam crest all
experienced small deflections in the downstream direction and small deflections toward
the maximum section of the embankment (Woodward-Clyde Consultants 1990).
Maximum deflections were on the order of 0.25 inches with most of the deflection
occurring in the upper 60 feet of the embankment. The inclinometer on the downstream
bench indicated lateral deflections on the order of 0.1 inches. Since reservoir filling, the
pattern of deflection with depth has remained the same while the maximum magnitude of
the deflection measurements has varied from about 0.25 inches to about 0.6 inches
(NJWSA 1992, 2000, 2005). This small amount of variation may be due to instrument
calibration.

Stress Cells. Five vibrating-wire stress cells were installed in the seepage barrier backfill
to measure total horizontal stresses parallel and perpendicular to the alignment of the
seepage barrier. The stress cells are located at the same locations along the dam as the
piezometer cross sections at varying depths. The initial measurements of these stresses
in the seepage barrier indicated the following (Woodward-Clyde Consultants 1990,
Khoury et al. 1992):

- The total stress increase in cells facing the reservoir was approximately equal to
  the rise in reservoir water pressure.
- Cells facing the reservoir measured higher total horizontal stresses than those
  facing in the direction of the dam axis.
- Total horizontal stresses in directions parallel and perpendicular to the dam axis
  were greater than the pore water pressures in the wall.
- Horizontal stresses increased with depth.
The stress cells measured relatively constant stresses after the initial filling until August of 1991, when all of the cells indicated a sudden jump in horizontal stress on the order of 4 to 7 psi (NJWSA 1992). Since August 1991 the stress cells have measured relatively constant horizontal stress at the elevated levels (NJWSA 2005). Because the August 1991 increase occurred simultaneously in all of the stress cells, the sudden change is likely due to a change in the recording equipment rather than a real change in the barrier.

**Settlement Monuments.** Vertical deformations are monitored at the dam with 33 monuments on the surface of the embankment and 4 settlement plates installed in the seepage barrier (Khoury et al. 1992). The surface monuments indicate that there has been very little vertical movement of the embankment since the monuments were installed just after construction in 1990. The maximum recorded settlements since 1990 are on the order of about 0.1 foot.

The settlement plates in the seepage barrier were used to monitor the rate and magnitude of settlement of the barrier backfill (Khoury et al. 1992). In the lower stage barrier, settlements of up to five feet were recorded in the five-foot thick barrier and settlements of up to three feet were measured in the three-foot thick barrier. In the upper stage barrier, settlements of up to 1.5 feet were recorded in the five-foot thick barrier and settlements of up to 0.75 feet were measured in the three-foot thick barrier. Most of the settlement was complete in the lower barrier by about two months after construction and in the upper barrier most of the settlement was complete in about three weeks. Since the initial settlements described above, measurements indicate that additional settlements of the barrier backfill have been on the order of 0.1 feet (NJWSA 2000, 2005).

**Assessment of Seepage Barrier Performance**

The seepage barrier in Manasquan Dam has performed well from the first filling of the reservoir and no significant changes in the water pressure regime or seepage rates have occurred over the life of the structure. The elevations of the piezometric surface upstream of the dam (headwater), just downstream of the seepage barrier, and at the
blanket drain outlet pipe for the five piezometer sections in the dam are presented in Table 3-3. The calculated head efficiencies (head drop over the barrier divided by the total head drop across the dam) for the five sections are presented on Table 3-3 and range from 81 to 89 percent. The head efficiencies tend to be lower where the barrier is narrower and where the height of the embankment and the hydraulic gradient across the barrier are greatest.

Table 3-3  Head efficiency calculations for Manasquan Dam seepage barrier.

<table>
<thead>
<tr>
<th>Station</th>
<th>Seepage Barrier Thickness (ft.)</th>
<th>Headwater Elevation (ft.)</th>
<th>Piezometric Surface Elevation Downstream of Barrier (ft.)</th>
<th>Elevation of Blanket Drain Outlet Pipe (ft.)</th>
<th>Seepage Barrier Head Efficiency (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10+00</td>
<td>3</td>
<td>103</td>
<td>78</td>
<td>74</td>
<td>86</td>
</tr>
<tr>
<td>13+00</td>
<td>5</td>
<td>103</td>
<td>70</td>
<td>66</td>
<td>89</td>
</tr>
<tr>
<td>16+00</td>
<td>5</td>
<td>103</td>
<td>67</td>
<td>60</td>
<td>84</td>
</tr>
<tr>
<td>19+50</td>
<td>5</td>
<td>103</td>
<td>72</td>
<td>68</td>
<td>89</td>
</tr>
<tr>
<td>22+00</td>
<td>3</td>
<td>103</td>
<td>86</td>
<td>82</td>
<td>81</td>
</tr>
</tbody>
</table>

The good immediate and long-term performance of the seepage barrier can be attributed to: (1) keying the seepage barrier into a continuous low-permeability layer, (2) a design that provides several layers of defense against the development of leaks in the barrier, and (3) quality control during construction. By keying continuously into the Manasquan clay layer a complete cutoff is achieved by the barrier and the efficiency of the barrier is not diminished by the effects of seepage around the ends of the barrier. The ends of the barrier are embedded into the flat-lying abutments so that seepage around the barrier is either driven by low hydraulic pressure (in the upper portions of the reservoir) or has a long seepage path (in the lower portions of the reservoir).

The design of the seepage barrier took into account several mechanisms that had the potential to degrade the performance of the seepage barrier. First, the designers recognized the potential for stress redistribution within the barrier that could lead to hydraulic fracturing and designed the width of the barrier accordingly. The design also provided two other lines of defense should a leak develop in the barrier. The $D_{15}$ of the fine sand embankment fill is less than the 0.7 mm size recommended by Sherard and
Dunnigan (1989) for designing downstream filters against sandy silts and clays and silty and clayey sands (Khoury et al. 1992). Thus, any leak that may develop in the barrier would be filtered by the embankment, allowing the barrier material to heal. In the event that a crack developed in the embankment downstream of the barrier, the designers provided the chimney drain as a filter and crack stopper against propagation of the crack. In this case, while high hydraulic gradients may force barrier backfill material into the crack, the filter would stop the migration of material and the crack would fill. With these multiple lines of defense against leaks in the barrier, the potential for degradation of the performance of the barrier is low.

Good quality control during construction is also key in the performance of the barrier. The designers recognized the potential for developing windows in the barrier due to accumulation of sand at the base of the trench. This was especially important where the upper trench keyed into the lower trench in the middle of the embankment. Strict monitoring of the sand content of the slurry and the condition of the bottom of the trench were maintained throughout the construction. Other quality control testing included tests of slurry and backfill properties.

Seepage analyses have been performed on Manasquan Dam to assess the effective permeability of the barrier and the gradients developed below the base of the barrier. A description of these analyses and the results are presented in Chapter 4.

**West Lewiston Levee**

**Background**

The West Lewiston Levee, in the city of Lewiston, Idaho, is managed by the U.S Army Corps of Engineers, Walla Walla District. The Lewiston levee system consists of over 8 miles of levees protecting the city from waters in the Snake and Clearwater Rivers. In this area the water levels in the Snake and Clearwater Rivers are controlled by Lower Granite Dam, located about 30 miles northwest of Lewiston. The portion of the levee
that is the subject of this case history, the area around Station 47+70 on the West Lewiston Levee, was constructed in 1973. This levee section was constructed over a low portion of the river bank that was below the current level of the river. A low area remains on the landside of the levee forming a pond adjacent to the landside toe. A plan of the levee reach and the surrounding area is shown on Figure 3-78.

![Figure 3-78 Plan of Lewiston Levee Station 47+70 and surrounding area (after Shannon & Wilson 2000a, with permission from USACE)](image)

A cross section of the levee is presented on Figure 3-79. The base of the levee consists of a two- to six-foot thick layer of well-graded sand and gravel with traces of silt. This layer was placed as the first stage of construction to provide a working platform above the water level. The second stage of construction consisted of a seepage barrier constructed through the highly permeable, sandy gravel river alluvium and tying in with the claystone and basalt bedrock. The six-foot wide seepage barrier is backfilled with a soil-bentonite mixture and is described in detail below. The top of the seepage barrier was capped with a 5-foot thick blanket of low-permeability, silty, sandy gravel. Above the cap blanket is
an 12-foot thick sandy silt to silty sand core and an eight-foot thick filter. The shells of the embankment consist of cobbles and boulders with trace amounts of sand.

Figure 3-79  Cross section of Lewiston Levee Station 47+70 (modified from Shannon & Wilson 2000b, with permission from USACE)

History of Seepage Problems and Mitigation

From 1973 to 1991, water levels in the Snake River varied from about elevation 736 feet to 741 feet. The water level in the landside pond varies from elevation 719 feet to 722 feet. Prior to 1991 seepage was noted along several sections of the levee; however, none was noted in the area around Station 47+70.

From February to April 1992, a drawdown test was performed in the Snake River where the water level was lowered to an elevation of about 710 feet. When the Snake River was returned to normal levels, personnel from the Corps of Engineers noticed seepage and sand boils on the landward toe of the levee near Station 47+70. In October 1993, the Corps of Engineers installed a discharge pipe perpendicular to the levee toe (see location on Figures 3-78 and 3-79). Seepage and sand boil activity in the area continued and by the end of 1994 a sand bar had built up at the outlet of the drain pipe. Seepage was also observed for a distance 15 feet downstream and 20 feet upstream of the outlet pipe.
In July 1994, Piezometer PN1711 (see Figure 3-78 for location) was being installed using air-rotary methods at the toe of the levee at Station 47+50. When the drill penetrated what was noted to be an “impermeable layer” on the top of the working platform, air bubbles were observed in the pond until completion of drilling. After the piezometer was installed, muddy water flowed from the discharge pipe at a rate of about 75 gallons per minute. The flows cleared and decreased to about 1 gallon per minute over a period of about 3 months.

In April 1995, drain pipes were installed parallel to and downstream of the levee crest and connected to the discharge pipe (see Figure 3-78). Over the next several years flow from the pipe and seepage from the embankment continued at rates varying from “no seepage to medium flows” (Shannon & Wilson 2000a).

In July 1998, a seepage berm was constructed along about 80 feet of the levee toe, extending about 26 feet into the pond (see Figures 3-#a and 3-#b for location). The berm consists of a geotextile filter fabric placed against the levee fill and the native soils, covered with crushed rock. The drain pipe was extended to the end of the berm. After the berm was constructed, piezometers directly above the berm indicated that the piezometer levels in the levee were rising. By December 1998, the two piezometers (PN1711 and PN1371) had flooded, water was flowing from the outlet pipe at a rate of 15 gallons per minute, and seepage was exiting from the toe of the levee upstream and downstream of the berm. In 1999, seepage from the outlet pipe increased to rates ranging from 50 to 75 gallons per minute. It was hypothesized that the filter fabric on the berm had clogged, forcing the seeping water above and around the berm.

In April 2000, a 5-foot deep trench drain was installed above the berm (see Figure 3-78 for location) and connected to the outlet pipe. Flow from the outlet pipe was estimated to be about 90 gallons per minute after construction of the trench drain.

Due to the ineffectiveness of the previous seepage control measures, and the risk the observed seepage posed to levee stability, a Deep Soil Mixed (DSM) seepage barrier was
constructed along the levee crest in 2001. As the DSM barrier was constructed, seepage through the levee steadily decreased. No seepage was observable at the end of construction.

**Design and Construction of Seepage Barrier**

The original seepage barrier was constructed using the slurry-trench method from the top of the working platform that was placed to raise the working surface above the level of the river (approximate elevation of 715 feet). The six-foot wide seepage barrier was excavated using backhoe excavators for the upper portion of the trench and a dragline for the lower portion. The trench was filled with a bentonite and water slurry to support the sides of the trench until the backfill was placed. In the area related to this study the bottom of the seepage barrier is at an approximate elevation of 695 feet, resulting in a barrier depth of about 20 feet.

The backfill consists of a mixture of the sandy gravel to gravelly sand alluvium from the trench excavation blended with bentonite to form a low permeability material. The backfill was mixed on the working platform next to the trench by spreading bentonite on the excavated soil and mixing with bulldozers. The resulting backfill consists of a well-graded gravel with 10 to 18 percent fines and a 6-inch maximum particle size.

Backfill placement was performed by pushing material into the end of the trench and forming a slope of backfill material below the slurry. Once the slope had formed, backfill was added at the top of the slope, causing the slope to slump and fill the lower portions of the trench. This procedure is used in lieu of dumping the material though the slurry because it results in a smaller chance of creating high permeability windows due to segregation of the backfill.

While the trench is open and filled with slurry, the slurry will tend to seep through the sides of the trench and the suspended bentonite will be filtered out of the slurry, forming a “filter cake” on the trench walls (Xanthakos 1979). The resulting filter cake, while
being very thin, has very low permeability. In this barrier, with a predominantly granular backfill, it is likely that the filter cake accounts for a large portion of the barrier’s seepage resistance.

Once backfill was complete, a low-permeability blanket consisting of well-graded gravel was compacted on top of the trench. As compaction of the blanket was being performed, several depressions occurred due to sinkholes in the seepage barrier. These sinkholes were repaired by removing the trench backfill for the full depth while supporting the trench with bentonite slurry and replacing it with similar backfill.

**Instrumentation**

Between 1986 and 1992 when the seepage problems developed, the portion of the Lewiston Levee seepage barrier that is the subject of this case study was instrumented by three piezometers installed along the levee crest (PN1702 through PN1703 on Figure 3-78) and one on the landside slope (PN1371). After the seepage problems became evident, an attempt to control and measure the seepage was made by inserting the discharge pipe in 1993. Estimates of discharge volume from this pipe were made visually from 1993 to the time of levee repair in 2001. Two additional piezometers (PN1710 and PN1711) were installed in 1994 to further monitor the developing seepage. The behavior of these piezometers, as it applies to the assessment of the seepage problems, is discussed previously under the “History of Seepage Problems and Mitigation” heading.

**Assessment of Seepage Barrier Performance**

After 19 years of satisfactory performance, significant seepage developed through the reach of levee around Station 47+70 in 1992. The increased seepage followed a 30-foot test lowering of the Snake River to elevation 710 feet. As a result of the test lowering, the water level in the landside pond was higher than the river and the seepage flow direction in the levee and foundation was reversed for about 3 months. Based on the
coincidence of the river lowering and the development of seepage, it can be assessed with a high level of confidence that the initiation of seepage development was caused by lowering the river level.

As described previously, during the drilling of Piezometer PN1711, a low permeability zone was drilled through, after which a surge of seepage flowed from the discharge pipe. It has been hypothesized (Shannon & Wilson 2000a) that the low permeability layer is the result of the mixing of the seepage barrier backfill on top of the working platform and the infiltration of the backfill material into coarse gravel of the working platform. It was hypothesized that water seeping through the seepage barrier was confined below this low permeability layer. Once the layer was penetrated during the piezometer drilling, the confined water flowed through the bore hole to the upper levels of the levee and drained through the discharge pipe. Therefore, it is apparent that the seepage is occurring through the seepage barrier rather than the core of the levee.

The cause of the deterioration of the seepage barrier performance cannot be definitively determined. However, there are two likely scenarios to describe how the test lowering of the Snake River caused the deterioration of the seepage barrier. These scenarios deal with the increased in effective stress due to the water lowering and the reversal of seepage gradient across the levee.

The first scenario is that the increased effective stress due to lowering the piezometric level within the levee caused consolidation of the levee and foundation materials. In the foundation, the seepage barrier backfill is much more compressible than the alluvium and, thus underwent a greater amount of settlement. This resulted in two situations that may have caused stress redistribution and low effective stresses in the barrier material. First, the top of the barrier tended to settle away from the overlying blanket, causing the blanket to bridge over the top of the barrier, and reducing the effective stress at the interface. Second, as the barrier backfill settles, friction along the sides of the trench results in an upward drag on the backfill and causes a reduction in effective stress. After the Snake River water levels returned to normal, the water pressure against the barrier
may have been greater than the total stresses in the barrier material, resulting in hydrofracturing of the barrier. Once fractured, the finer portion of the soft barrier backfill eroded, leaving behind a permeable, open-graded soil.

The second scenario is that there was a high permeability window in the barrier backfill due to segregation during placement, or a collapse of the trench wall. Segregation of the backfill may have occurred during backfill of the repairs of the sinkholes that developed following completion of the barrier, where there was less room for development of a backfill slope. If this is the case, during the first 19 years of the barrier performance the riverside filter cake may have eroded into the window. However, the landside filter cake would be more likely to remain intact during this time period, as it was supported and filtered by the well graded alluvial soils. When the seepage gradient was reversed during the test lowering, the landside filter cake was then susceptible to erosion into the open graded window. With both filter cakes compromised, the seepage would be concentrated through the barrier defect and the potential for further erosion of the barrier increased.

Both of these scenarios are plausible explanations for the observed seepage development. It is not possible to ascertain which of the two is responsible for the seepage without excavation of the levee and careful observation of the seepage barrier.

**Camanche Dike-2**

**Background**

Camanche Dike-2 is one of six dikes and a dam that retain Camanche Reservoir. The dam, dikes, and reservoir are managed by the East Bay Municipal Utilities District and are located on the Mokelumne River about 30 miles northeast of Stockton, California. Construction of the dike was completed in 1965 (Anton and Dayton 1972). The dike consists of a 2550-foot long zoned earth embankment and has a structural height of about 80 feet with a crest elevation of 263 feet. The upstream and downstream slopes vary from 1.75:1 (horizontal to vertical) in the upper portion of the embankment to 2.5:1
(horizontal to vertical) below the maximum controlled water surface elevation of 235.5 feet. The reservoir spillways and outlet works are located on the main dam and are not associated with the Dike-2 structure. A plan of the dike is shown on Figure 3-80.

![Plan view of Camanche Dike-2](image)

**Figure 3-80** Plan view of Camanche Dike-2 (after Anton and Dayton 1972, with permission from ASCE)

The Camanche Dike-2 foundation consists of alluvial deposits overlying bedrock of the Mehrten Formation (Cedergren 1969, Anton and Dayton 1972). The alluvium beneath the dike is up to 95 feet deep. A longitudinal section through the upstream toe of the dike is shown on Figure 3-18. The upper 20 feet of the alluvium consists of relatively low permeability clayey sand and the lower portion consists of fine sand to silty sand. Portions of the lower alluvium have high permeability. The Mehrten Formation consists of welded tuff and has relatively low permeability.

A cross section through the dike is shown in Figure 3-82. The zoned earth embankment consists of seven zones (Anton and Dayton 1972, Bechtel 1967). The central core of the dike consists of compacted clayey sand borrowed from the upper portion of the alluvium.
A blanket consisting of core material extends from the upstream portion of the core to a point 150 feet upstream of the toe of the main embankment. Beyond the toe of the upstream blanket, the upper portion of the alluvium was disked and recompacted for a distance of 1,000 feet above the embankment toe to reduce the permeability (see “surface compacted area in Figure 3-80). On the downstream side of the core, a chimney drain was constructed that connects to a blanket drain and toe drain. The upstream shell consists of two zones: an inner zone consisting of a mixture of sand and gravel adjacent to the core, and an outer zone consisting of gravel and cobbles. A transition zone of processed sand and gravel was placed between the inner and outer upstream shells. The downstream shell consist of material similar to the inner upstream shell. The upstream slope and low-permeability blanket are armored with riprap and the downstream slope is armored with gravel and cobbles.

Figure 3-81  Longitudinal section through upstream toe of Camanche Dike-2 (after Bechtel 1967)

History of Seepage Problems and Mitigation

The reservoir was partially filled to elevation 218 in April 1965 (Anton and Dayton 1972). Significant seepage developed soon after filling and seepage was observed flowing from an extensive area of privately owned property downstream of the Dike-2.
The reservoir was lowered below the downstream toe of the Dike-2 embankment and the amount of seepage rapidly decreased. Inspection of the upstream blanket revealed holes in the compacted surface.

![Cross section of Camanche Dike-2 embankment and foundation](image)

**Figure 3-82** Cross section of Camanche Dike-2 embankment and foundation (modified from Anton and Dayton 1972, with permission from ASCE)

**Design and Construction of Seepage Barrier**

To mitigate the excessive seepage, a soil-bentonite seepage barrier was constructed in 1966 about 50 feet upstream of the toe of the compacted core material seepage blanket. The location and limits of the seepage barrier are presented on Figures 3-#a, 3-#b, and 3-#c, and a detail of connection between the seepage barrier and the seepage blanket is shown on Figure 3-83. The seepage barrier has a width of 8 feet and extends to the top of the Mehrton Formation. The maximum depth of the barrier is about 95 feet and the length is about 1,660 feet. An 8- to 11-foot thick low-permeability blanket was constructed above the top of the trench and connected to the toe of the existing low-permeability blanket.
The trench was excavated using a dragline bucket attached to a crane. The open trench was supported with bentonite-water slurry until the backfill could be placed. Backfill material consisted of the excavated soils mixed with clayey borrow soils and bentonite slurry. The specifications called for a well-graded backfill material consisting of 80 to 100 percent passing the 3/4-inch sieve and 10 to 30 percent passing the number 200 sieve. The mixing was performed on the ground surface using a bulldozer.

After placement of the backfill the top of the barrier was surcharged with soil to consolidate the backfill. The minimum depth of surcharge was five percent of the depth of the barrier. The surcharge was left in place for a minimum of seven days. Measured settlements due to the surcharge were reported to be very small, indicating the surcharge was ineffective in consolidating the backfill. Difficulty was experienced in placing the blanket above the barrier due to the barrier backfill not being capable of supporting the weight of the compaction equipment. The compaction procedure was modified during construction to make compaction easier and still provide a good contact between the barrier and the blanket.
Instrumentation

Piezometers were installed below Camanche Dike-2 during construction of the embankment and downstream of the seepage barrier at the time of seepage barrier construction. Because many of these original piezometers failed over time, additional piezometers were installed in 2001. The locations and performance of the original and 2001 piezometers are discussed below.

Original Piezometers. At the time of seepage barrier construction, 24 pneumatic piezometers existed in the upper portions of the alluvium below the Dike-2 embankment. These piezometers were installed in pairs, with one piezometer approximately 15 feet above the other. During construction of the seepage barrier an additional 9 pneumatic piezometers were installed in the alluvium immediately downstream of the seepage barrier. These piezometers were installed in sets of three with the piezometers located in the top, middle, and lower portions of the alluvial layer. The piezometers have been monitored from the time of seepage barrier construction. However, most of these piezometers have failed over time and have since been abandoned.

The piezometers that existed at the time of seepage barrier construction indicated that the barrier construction resulted in a drop in the piezometric surface of about 15 to 20 feet beneath the embankment, as shown in Figure 3-84. Since barrier construction, the piezometers indicate that the lower piezometric level is remaining relatively constant beneath the embankment at a hydraulic head of about 175 to 180 feet.

The three groups of three piezometers installed immediately downstream of the seepage barrier at the time of construction exhibited varying behavior. The group located at Station 247+50 indicated hydraulic heads slightly above the levels measured beneath the embankment before the piezometer failed in 1970. The three piezometers of the group located at Station 253+60 all measured the same head elevation of about 175 feet, also similar to the piezometers under the embankment. These piezometers failed in 1982.
The piezometers located downstream of the barrier at Station 250+00 indicated a very different local seepage regime. A plot showing the measured hydraulic heads in these piezometers from 1967 to 1971 is presented in Figure 3-85. The lowermost piezometer (P38) measured hydraulic heads similar to those measured in the alluvium under the embankment. However, the piezometers located higher in the alluvium (P36 and P37) measured increasing heads with increasing elevation. This behavior indicates there is a downward hydraulic gradient at this location and may be indicative of a leak in the barrier or blanket at or near the interface between the barrier and seepage blanket. These piezometers ceased to operate in 1977.

**Piezometers installed in 2001.** In 2001, 12 vibrating wire piezometers were installed beneath the dam crest and immediately downstream of the seepage barrier. Four sets of two piezometers were installed from dam crest. In two of these sets, one piezometer was located in the alluvium and the other was located in the embankment core fill. In the other two sets, one piezometer was in the alluvium and one in the Mehrten Formation bedrock. Two sets of two piezometers were installed downstream of the seepage barrier. In both of these installations one piezometer was installed in the alluvium and one in the upper portion of the Mehrten Formation.
The piezometer sets located beneath the dam crest measured hydraulic heads in the alluvium varying from 170 to 178 feet. Hydraulic heads in the embankment core were measured between 200 and 210 feet. This indicates that there is a hydraulic gradient from the core into the underlying alluvium. Of the two sets of piezometers that had a piezometer in the Mehrten Formation bedrock, one measured piezometric pressures about 10 feet higher in the bedrock than the alluvium, and the other measured pressures about 10 feet lower in the bedrock. This indicates that in some locations there is a hydraulic gradient from the bedrock to the alluvium and in some location the opposite is true.

The two piezometer sets located downstream of the seepage barrier both indicate slight hydraulic gradients from the bedrock to the alluvium. The head difference between the two piezometers in each pair varies from about 2 to 5 feet. All four piezometers indicate hydraulic heads between 170 and 180 feet. These heads are similar to those measured in the alluvium below the embankment.
Assessment of Seepage Barrier Performance

The overall behaviors of the piezometers indicate that the seepage barrier has a high level of efficiency in controlling seepage through the alluvial layer beneath the dike. Because of fluctuating reservoir level and uncertainties with the groundwater level downstream of the barrier, it is not possible to calculate precisely a head efficiency of the barrier. However, at times of high reservoir level, the barrier has a head efficiency of about 90 percent.

One exception to this high efficiency was detected in Piezometers P36, P37, and P38 where a downward gradient indicates a barrier or seepage blanket leak near the top of the barrier. The efficiency of the barrier in this location, calculated based on the uppermost piezometer, would be on the order of 30 percent. However, the increased heads are not detected in other piezometers, indicating the leak is small and localized. The piezometers did not indicate the leak was increasing in size between the time when it was detected and the time the piezometers failed in 1977. It is likely the leak is not getting larger due to the well-graded nature of the upper alluvium preventing the migration of fines away from the barrier or seepage blanket.

Hydraulic gradients were detected from the embankment core and bedrock into the alluvium. These gradient patterns are expected since, once the barrier has been constructed, the alluvial layer tends to act as a drain for the less permeable core and bedrock. The gradients detected immediately downstream of the barrier indicate there is less seepage resistance through the bedrock than through the barrier. This is qualitative evidence of low effective permeability of the seepage barrier.

The failure and replacement of the original piezometers precludes a precise assessment of any long-term changes in the seepage regime. Differences between the original and new piezometers are more likely attributable to local variation than to changes occurring over time. However, the new piezometers do indicate hydraulic heads in the alluvium similar
to those measured by the older piezometers. This indicates that there has been little overall change in the seepage barrier efficiency since construction.

Old Dams with Seepage Barriers

This section provides short summaries of 11 dams built in the early 1900s that contain seepage barriers constructed using methods and materials available at the time of construction. Such barriers include concrete core walls and driven sheet pile walls. In general, the instrumentation on these dams is minimal or was added fairly recently, making assessment of changes of the seepage barrier performance difficult. However, the observed performance of these dams does provide insight into the mechanisms that may be affecting the performance of the more modern seepage barriers discussed previously in this chapter.

Warranga Basin Dam. Warranga Basin Dam is located about 8 miles southwest of Tatura, Australia, and is owned and operated by Goulburn-Murray Water. The dam is a 4.3-mile long embankment that was originally constructed between 1905 and 1908 (URS 2001). The embankment is a puddle-clay core structure with select compacted clayey fill shoulders. The original embankment had a crest elevation of about 396 feet and a maximum height of 26 feet. From 1915 to 1919 the embankment was raised 14 feet to a crest elevation of 410 feet. The puddle clay core was extended during this raise by constructing a four-foot thick horizontal section connected to a new vertical section as shown in Figure 3-86.

![Cross section of Waranga Basin Dam](after URS 2001, with permission from URS Corporation)
Soon after the raising of the embankment, several leaks developed through the dam (URS 2001). It was determined that these leaks followed a path along the upper surface of the horizontal section of the puddle clay core. While these leaks were originally attributed to burrowing of crayfish, a recent assessment of the dam performance suggests the leaks were associated with erosion of dispersive clays (URS 2001). In response to these leaks, a six-inch thick reinforced concrete core wall was constructed near the downstream toe of the dam in 1927. The core wall was constructed by (1) removing the toe of the embankment, (2) constructing the reinforced-concrete wall, (3) placing a nine-inch thick layer of crushed rock behind the wall to act as a drain, and (4) reconstructing the original embankment and a 15-foot wide berm around the core wall. The core wall configuration is shown on Figure 3-86.

Since completion of the core wall there have been no additional reports of piping (URS 2001). In 2001, several seepage exit points were noted on the downstream toe of the dam after a significant rainfall. An investigation of these exit points indicated that they were associated with buried outlet pipes leading out from the crushed rock drain behind the core wall. The remainder of the pipes were located and are being monitored for flows and signs of internal erosion. It is likely that the cause of the seepage was rainfall collecting in the core wall drain. Thus, based on the past performance of the dam, the core wall appears to have performed well since its construction.

**Hume Dam.** Hume Dam is located about 10 miles north of Albury-Wodonga, Australia, and is owned and operated by the Murray-Darling Basin Commission. The dam was originally constructed in the late 1920s, and consists of a 1,043-foot long gravity concrete section with earth embankments abutted to both ends (McDonald and Wan 1999). The north embankment is 328 feet long and has a maximum height of 59 feet, and the south embankment is 3,884 feet long with a maximum height of about 132 feet. Both embankments are poorly compacted earth fill with two-foot wide concrete core walls beneath the crests. The core walls are slotted into the concrete section as shown on Figure 3-87.
Between 1950 and 1961, modifications were made to the dam and spillway crest to raise the reservoir level. By raising the spillway crest 5 feet and adding spillway gates the maximum storage elevation of the dam was increased by nearly 30 feet (Germanis 1975, McDonald and Wan 1999). With the reservoir being operated at a higher level, the southern embankment began to deflect in the downstream direction. In the early 1990s seepage and piping problems became apparent at the interface between the northern embankment and the concrete section. It was determined that the deflection of the embankment caused cracking of the concrete along the slot where the core wall ties into concrete section of the wall. In 1996, at the advice of the Hume Dam Technical Review Committee, the reservoir was drawn down to prevent further piping.

Repairs to the core wall and concrete structure interface were made in 1997. These consisted of the following (NSWDPWS 2001):
• Grouting of the gap between the core wall and the upstream portion of the tie-in slot in the concrete section.
• Stitching of the cracked portion of the concrete section with stainless steel bars.
• Construction of a 150-foot long, 5-foot thick seepage barrier upstream of the core wall. The barrier backfill consisted of controlled low strength material (CLSM) to provide ductility.
• Construction of a waterstop, consisting of a 16-inch pipe filled with cement-bentonite, at the upstream corner of the core wall and the concrete structure.
• Construction of an L-shaped soil-bentonite seepage barrier on the upstream side of the CLSM barrier to seal the end of the CLSM barrier.
• Construction of a downstream filter curtain consisting of 5-foot diameter augered holes filled with filter material.

These repairs are shown on Figure 3-87.

The repairs have been successful in mitigating the concentrated seepage at the interface between the core wall and the concrete section. The cracking that developed as a consequence of increased water pressure acting on the seepage barrier is an example of the susceptibility of rigid seepage barriers (or structural tie-ins) to cracking at locations of high stress.

Lake Wolford Dam. Lake Wolford Dam is located northeast of Escondido, California, and is owned and operated by the Escondido Mutual Water Company. The dam was originally constructed in 1895 and 1896 as an 80-foot high rock fill dam with upstream facing of redwood planking supported by dry walling (rock rubble hand placed in a Portland cement mortar). From 1923 to 1924, the dam was raised to a height of just over 100 feet (crest elevation of 1,485 feet) by placing hydraulic fill on the upstream side of the existing dam. The upper 20 feet of the dam was placed in layers compacted with horse-drawn “rollers” (McFarland 1930). In 1932, a sheet pile seepage barrier was constructed through the compacted fill, embedding in the fine portion of the hydraulic fill. The sheet pile wall was constructed in response to seepage through the upper portions of the dam when the reservoir level was near the spillway elevation of 1,480
feet. A cross section of the dam with the various stages of construction indicated is shown in Figure 3-88.

Although available records are sparse, problems were not reported until 1978 when settlement occurred on the upstream side of the dam crest (Dames & Moore 1979). The settlement was coincident with an increase in seepage at the dam toe, from the normal flow of 30 gpm to 60 gpm. The seeping water was also noted as being cloudy. An investigation was conducted and it was determined that the most likely cause of the seepage and settlement was seepage though gaps in the sheetpile wall and between the sheetpiles and the concrete spillway structure. The problem was mitigated by excavating the affected area down to the top of the fine hydraulic fill and replacing the material with well-compacted fill with sand filters against the sheet pile wall.

This case study illustrates how the observed performance of a seepage barrier can change rapidly due to processes that have been occurring over many years. It is likely that material had been piped from the leaks in the seepage barrier at a slow rate for many years. Once the seepage path had been developed to a point where velocity of the seeping water increased, the effects of the piping began to manifest as settlement and cloudy seepage water.

Lake Valley Dam. Lake Valley Dam is located near the Town of Cisco Grove in Placer County, in northern California, and is owned and operated by the Pacific Gas and Electric
Company. The dam was originally constructed in 1911, and has a maximum height of 74 feet with a crest elevation of 5,855 feet and a crest length of 940 feet. The dam was originally constructed as a homogenous earth embankment consisting of silty sand and gravel, with a six-inch thick wooden core wall constructed through the center of the dam. Details of the construction of the dam are scarce, however, based on photographs of the core wall during modifications of the dam (Vanberg 1980), it appears that the core wall was constructed by stacking and nailing two-inch by six-inch wooden boards.

In 1980, the dam was upgraded by adding a rock fill berm on the downstream side of the dam and placing low-permeability fill around the upper portion of the wooden core wall. While excavating adjacent to the core wall, a 10-feet deep by 14-feet long (parallel to the dam) by 4-feet wide void was discovered on the upstream side of the seepage barrier (Vanberg 1980). The base of the void was below the elevation where most of the work was being performed and the condition of the core wall at the base of the void was not noted. However, the remaining thickness of the portion of the core wall that was exposed was noted to be from three to five inches, as compared to the original six inches. The void was excavated by hand and backfilled with compacted soil.

Due to the limited investigation of the void, its cause can not be determined with certainty. However, based on the location of the void and the observed deterioration of the wooden core wall, it is likely that the void was associated with a leak in the core wall. It can be theorized that concentrated flows through a leak in the barrier eroded the embankment fill. No investigation was performed on the downstream side of the barrier, and, therefore, no assessment of the mechanism for removing the eroded soil from the embankment could be made.

Crane Valley Dam. Crane Valley Dam is located in Madera County in northern California, and is owned and operated by the Pacific Gas and Electric Company. The dam was originally constructed in 1910, and has a maximum height of 146 feet, with a crest elevation of 3,381 feet and a crest length of 1,880 feet. The dam was built by constructing a vertical concrete core wall and placing hydraulic fill and rock fill against
the core wall (Perkins 1932, Garber 1972). The left third of the dam, which includes the highest sections, was constructed by placing hydraulic fill on the upstream side of the core wall and rock fill on the downstream side. The right two-thirds of the dam was constructed with hydraulic fill on both sides of the core wall. Embankment slopes constructed by the hydraulic fill method were inclined at 2:1 (horizontal to vertical) and rock fill slopes were constructed at inclinations of 1.25:1 (horizontal to vertical). From 1916 to 1929 additional rock fill was placed on the downstream slope of the highest portion of the dam, widening the crest from 40 to 80 feet. Some of this fill was placed to accommodate a lumber railroad that ran across the crest of the dam in the earlier part of the 1900s. In 1970, rock fill berms were placed at the toe downstream of the hydraulic fill portions of the dam to increase the stability of the embankment.

Since completion of the dam, the top of the seepage barrier in the rock fill portion of the dam has deflected in the downstream direction. In 1932, measurements indicated the deflection was on the order of 11 feet (Perkins 1932). By 1972, the maximum deflection was reported as 14.5 feet (Garber 1972). From 1946 to present, no appreciable movement of the core wall has occurred. The movement of the core wall was attributed to the densification of the loosely placed rock fill (Garber 1972). This densification may have been exacerbated by the vibrations from the lumber railroad crossing the dam.

In 1932, shafts were excavated and inspected on the upstream and downstream sides of the core wall to investigate the condition of the wall. The upstream shaft was excavated to a depth of 80 feet and the downstream shaft was excavated to near the base of the core wall. Several horizontal and vertical cracks up to “several inches wide” were observed on the upstream side of the core wall and tight cracks were observed on the downstream side of the wall (Perkins 1932). The cracks were filled with mud and did not appear to be transmitting a significant amount of water.

The dam has been monitored with 20 piezometers and 4 weirs since 1979 (PG&E 2005). None of the piezometers or weirs have indicated an appreciable change in the seepage regime since their installation. Leakage through the dam that is measured by the weirs
indicate average flows of about 15 gpm, with peaks of about 40 gpm during high reservoir levels.

This case study illustrates how the effectiveness of the core wall may not be significantly affected by extensive cracking. In this case, the material above the core wall is soft, low permeability material that was able to fill the cracks, preventing the development of seepage pathways, high seepage velocities, and internal erosion.

Cherry Flat Dam. Cherry Flat Dam is located about four miles northeast of Alum Rock, California, and is owned and operated by the City of San Jose. Construction of the dam was completed in 1936 (DSOD 2004). The dam is a zoned earth embankment with a maximum height of 67 feet, a crest elevation of 78 feet, and a crest length of 230 feet. A cross section of the embankment is shown on Figure 3-89. The core consists of compacted clayey sand with gravel, and the shells consist of compacted clayey gravel. The upstream and downstream slopes are 2.5:1 (horizontal to vertical) at the base of the dam and steepen to 2:1 (horizontal to vertical) near the crest. The foundation consists of mélange of the Franciscan Complex in the right abutment and landslide debris in the left abutment. The Franciscan mélange consists of sheared sandstone, chert, and shale.

![Cross section of Cherry Flat Dam](image)

Figure 3-89 Cross section of Cherry Flat Dam

A six-foot wide concrete seepage barrier was constructed at the base of the dam to depths as great as 50 feet below the ground surface. The barrier was designed to cut off seepage through the highly fractured Franciscan mélange. The barrier was excavated by hand
and supported with wooden shoring until backfilled. The backfill, consisting of concrete mixed on site with four small cement mixers, was placed by dumping from the top of the excavation. The wooden shoring was removed as the concrete filled the trench. A photograph of the filling operation is shown as Figure 3-90.

![Figure 3-90 Photo of construction of Cherry Flat Dam core wall (from Engle 1933)](image)

Seepage through the dam is measured in a weir near the downstream toe. In 2004, flows through the weir varied from one to six gallons per minute and were reported to be consistent with flows measured during the earlier life of the dam (SJDRPNS 2005). Additional seepage is likely flowing through the foundation and remaining in the fractured bedrock located downstream of the dam. The consistent low amount of seepage emerging at the toe is indicative that the seepage barrier is performing well, and the performance remains steady over the life of the structure.

**Lower Franklin Dam.** Lower Franklin Dam is located just outside the city limits of Beverly Hills, California, and is owned and operated by the City of Los Angeles. Construction of the dam was completed in 1916 (LADWP 2001). The dam is an earth embankment with a maximum height of 100 feet, a crest elevation of 589 feet, and a crest length of about 500 feet. A cross section of the embankment is shown on Figure 3-91.
The dam was constructed in stages, with the original dam constructed in 1916 using a combination of wagon fill and hydraulic fill techniques. A 50-foot deep concrete core wall was constructed through the alluvium and into the bedrock. In 1952 the dam was raised by adding rolled fill to the crest of the dam and adding a downstream berm of rolled fill. The foundation consists of about 30 feet of streambed alluvium underlain with shale bedrock (Leventon 1930a). The dam was taken out of service in 1976 due to seismic stability concerns associated with liquefaction of the hydraulic fill.

The reinforced concrete core wall was constructed a year prior to the embankment by hand excavating a trench through the alluvium and shoring the sides with timbers. The core wall is four feet wide in the bottom two-thirds and three feet wide in the upper third of the wall, and was formed and cast in place. The spaces between the core wall and the sides of the excavation were backfilled with puddle clay. The core wall extends about 10 feet into the hydraulic fill portion of the embankment, as shown in Figure 3-91.

The effectiveness of the core wall was tested before construction of the embankment when water backed up behind the wall to the point of overtopping, while a downstream well measured groundwater levels 45 feet below the ground surface (CDWR 1916). The only instrumentation data available on Lower Franklin Dam are measurements through the main seepage weir from 1930 to 1977. The flows through the weir remained constant at about two to four gallons per minute from 1930 to 1972, when the dam was taken out.
of water storage service. Much of this seepage is likely through the embankment as seepage through the core wall would be expected to remain in the alluvium.

The satisfactory performance of the core wall in this dam is likely due to following factors:

- The thickness and good quality of the wall preventing large cracks from forming,
- The puddle clay surrounding the wall tending to fill and seal cracks that may have developed in the wall, and
- The 10-foot embedment into the hydraulic fill, which reduces the hydraulic gradients above the wall that could lead to erosion of the hydraulic fill.

Fairmont Dam. Fairmont Dam is located in Antelope Valley, California, and is owned and operated by the City of Los Angeles. Construction of the dam was completed in 1928 (Leventon 1930b, Coluzzi 1972). The dam is an earth embankment with a maximum height of 121 feet, a crest elevation of 3043 feet, and a crest length of about 875 feet. A cross section of the embankment is shown on Figure 3-92. The dam was constructed in stages between 1913 and 1928 using a combination of wagon fill and hydraulic fill techniques. The foundation consists of about 10 feet of alluvium underlain by weathered granitic bedrock (Coluzzi 1972). A concrete core wall was constructed from the top of the bedrock, through the alluvium, and to the top of the hydraulic fill. The core wall is about six feet wide at the base and tapers to four feet at the top. The dam was taken out of service in 1977 due to seismic stability concerns associated with liquefaction of the hydraulic fill.

Seepage problems developed within a few years after construction. Seepage was first observed in 1931 at a location 300 feet from the embankment, and increased from 0.06 to 0.28 cfs in a short period of time (Leventon 1930b). The seepage appeared to be coming from the bedrock in the abutments and not the embankment fill. The reservoir was drained and the abutments were grouted. The grouting reduced the amount of seepage until 1964 when excessive downstream seepage again developed. As a result of
the 1964 seepage the water surface elevation was restricted to below elevation 3,020 feet. This reduced the seepage to a tolerable level.

![Diagram](https://via.placeholder.com/150)

Figure 3-92  Cross section of Fairmont Dam (after Coluzzi 1972)

The observed seepage problems at Fairmont Dam appear to have been through the bedrock in the abutments and do not appear to be related to the core wall. The seepage volume through the core wall appears to be small because after the foundation seepage was mitigated in 1931 the total seepage volumes dropped until 1964. The satisfactory performance is likely due to the good quality of the wall through the alluvium and the low permeability of the surrounding hydraulic fill material in the embankment.

Private Dam 2. The dam presented in this case history is privately owned and, as a condition for allowing the dam to be part of this study, the dam name, location, and owner will not be presented. The information presented in this case history was obtained from a Safety Inspection Report prepared by a private consultant to meet the requirements of the Federal Energy Regulatory Commission (FERC). References to the specific report have been omitted to prevent identification of the dam.

The dam was constructed in the early 1920s and is comprised of a 487-foot long concrete section on the right side and a 410-foot long earth embankment on the left side of the dam. The concrete section houses the overflow spillway, the gated spillway, and the powerhouse. The embankment is a zoned earth embankment and has a maximum height of 32 feet and a 10-foot wide crest. A cross section of the embankment and core wall is
shown on Figure 3-93. The downstream slope is 2:1 (horizontal to vertical) and the upstream slope is 3:1 (horizontal to vertical). No information on the soil types were available, however, it is reasonable to assume that the core material (indicated as soil in Figure 3-93) was intended to have a significantly lower permeability than the upstream shell. The embankment includes concrete core wall extending from the top of the limestone bedrock, through about 10 to 20 feet of glacial till, and up through the entire embankment.

![Cross section of Private Dam 2](image)

**Figure 3-93** Cross section of Private Dam 2

The core wall is about 5 feet thick at the base and tapers to 2 feet thick at the top of the embankment. No indication of reinforcement is noted in the plans or report. The plans do indicate that contraction joints were constructed in the core wall at spacings of no more than 30 feet.

The dam is not instrumented with piezometer or weirs. Inspections of the dam have not identified any signs of seepage though the dam or in the foundation or any abnormal movement of the embankment. Thus, it can be assessed that the seepage barrier and dam have performed well over the past 80-plus years. This performance is likely due to the stoutness of the seepage barrier and the relatively low head across the dam.

**Private Dam 3.** The dam presented in this case history is privately owned and, as a condition for allowing the dam to be part of this study, the dam name, location, and

225
owner will not be presented. The information presented in this case history was obtained from a Safety Inspection Report prepared by a private consultant to meet the requirements of the Federal Energy Regulatory Commission (FERC). References to the specific report have been omitted to prevent identification of the dam.

The dam was constructed in 1925 and is comprised of a 432-foot long concrete section on the left side and a 1,260-foot long earth embankment on the right side of the dam. The concrete section consists of a buttress dam, an overflow spillway, a gated spillway, and a powerhouse. The embankment is a zoned earth embankment and has a maximum height of 58 feet and a 30-foot wide crest. A cross section of the embankment and core wall is shown on Figure 3-94. Vague descriptions of the soils comprising the embankment are provided on the cross section. These descriptions indicate the soil placed upstream of the core wall is of relatively low permeability and the upstream and downstream shells are largely granular. The downstream slope is 2.5:1 (horizontal to vertical) and the lower portion of the upstream slope is 3:1 (horizontal to vertical) steepening to 2:1 above the normal water level.

The foundation consists of a surficial silty clay layer 10 to 15 feet thick overlying a 30-foot thick layer of dense silty sand with gravel. Beneath the silty sand clay hardpan exists at a depth of about 45 feet.

Figure 3-94  Cross section of Private Dam 3

A concrete core wall extends through the full height of the embankment. The wall was formed and poured before the construction of the embankment. The wall is about two
feet thick at the top of the dam and tapers to about four feet thick at the base of the wall. The available plans show the core wall to be founded on a pile cap and sheet piles. The plans indicate the pile cap is founded on top of the silty sand layer. The plans have no indication of the depth of the sheet piles.

In 1995, five piezometers were installed for a subsurface investigation: three in the gated spillway area and two in the embankment. The spillway piezometers and one of the embankment piezometers were located downstream of the core wall and sheetpile wall and the measured heads in these piezometers are very close to the tailwater elevation. The piezometer located in the embankment upstream of the core wall measured a head 13 feet higher than those below. These piezometers have not been read since 1995.

Based on the piezometer readings, it appears that the core wall and sheet pile seepage barrier are performing quite well 80-plus years after construction. This performance is likely attributable to a stout seepage barrier and soils of low to moderate permeability upstream of the barrier.

**Summary of Observations and Findings from Case Histories**

The following is a summary of observations and insights from review of the 30 case histories presented above. Additional insights have been developed through the analyses presented in Chapter 4, and these are presented at the end of Chapter 4.

**Seepage Barrier Effectiveness**

In an attempt to compare the effectiveness of multiple seepage barriers, several methods of quantifying barrier effectiveness were considered. The effectiveness has been quantified by others using measures such as head efficiency, flow efficiency, and reduction in exit gradient (Telling et al. 1978a, 1978b, 1978c). While these quantifying measures are often useful in assessing and understanding a seepage barrier’s performance, the large number of variables associated with a barrier’s effect on the
seepage regime often complicate the comparison of barriers in multiple dams. Discussions of each of the methods for quantifying seepage barrier effectiveness are presented below.

**Head Efficiency.** Head efficiency is defined as the ratio of the head drop across the seepage barrier to the head drop across the entire dam. Head efficiency can be easily calculated if there are piezometers located near the seepage barrier on the upstream and downstream sides. It is also possible to make good estimates of head values on the upstream and downstream sides of the barrier by interpolating and extrapolating from other piezometer locations. While head efficiency may be useful for comparing the behavior of seepage barriers in dams with similar seepage conditions, differences in the seepage regime can significantly affect the head efficiency achieved by a barrier. To illustrate this point, consider two identical dams, one on a foundation with moderate permeability (say $10^{-4}$ cm/s) and one on a foundation of high permeability (say $10^{-1}$ cm/s). If identical seepage barriers are installed through both dams and foundations, the head efficiency for the barrier in the dam with the moderately permeable foundation will have a lower efficiency because of the head losses occurring in the foundation soils. In the dam with the high permeability foundation, the seepage barrier provides nearly 100 percent of the seepage resistance, and therefore the head efficiency would be near 100 percent. Even in a case where the seepage barrier was compromised by poor construction or post-construction deterioration, the effective permeability of the barrier would still be orders of magnitude lower than the foundation, and the head efficiency would still be very high.

**Flow Efficiency.** Flow efficiency is defined as the ratio of the reduction in seepage flow as a result of seepage barrier construction to the seepage flow prior to seepage barrier construction. In the limited number of cases where the seepage through a barrier can be accurately measured, the flow efficiency is a good indicator of seepage barrier performance. However, in a majority of cases, a significant portion of the seepage remains in the subsurface and cannot be observed or measured. Thus, flow efficiency has
limited application for quantification of seepage barrier performance. Seepage volume can often be assessed from the change in size of observed seepage areas.

**Exit Gradient Efficiency.** In many cases the purpose of the seepage barrier is to reduce exit gradients or seepage volume downstream of the dam. The reduction in exit gradient can be assessed from piezometric data and observed performance.

**Hydraulic Conductivity of Barrier.** The effective hydraulic conductivity of a barrier can be assessed by back-calculating the apparent hydraulic conductivity of the seepage barrier material based on the seepage regime that develops as a result of the barrier construction. Calculations of effective hydraulic conductivity are presented for several case studies in Chapter 4. The effective hydraulic conductivity is a good way to quantify the effects of defects in barriers where the hydraulic conductivity of the layer being cut off is within a few orders of magnitude of the seepage barrier backfill. In cases where a highly permeable layer is being cutoff, order of magnitude changes in the modeled seepage barrier backfill have little effect on the seepage regime, and thus the ability to precisely back calculate the effective hydraulic conductivity is lost.

It is apparent from the above discussion that quantitative assessment of seepage barrier performance is somewhat subjective and quantitative comparisons between dams is difficult. Thus, performance assessments of seepage barriers for individual dams need to be performed on an individual basis based on the desired results and the specific site conditions.

**Dams on Solutioned Limestone**

Three dams from the study (Wolf Creek, Beaver, and Walter F. George) were constructed on foundations containing limestone with significant solutioning. The seepage barrier at Beaver Dam has performed well over time while seepage at Wolf Creek Dam has redeveloped to levels similar to the pre-barrier conditions, and seepage problems increased at Walter F. George Dam in locations around the ends of the barrier while the

229
barriers were being constructed. The good performance of the Beaver Dam seepage barrier can be attributed to extending the depth and the width of the barrier beyond the limits of heavy solutioning. Therefore, the high gradients developed beneath and around the ends of the barrier act on bedrock that is not susceptible to erosion. In contrast, the increased hydraulic gradients developed beneath and around the seepage barriers in Wolf Creek and Walter F. George Dams are in areas where soil-filled solution voids exist, and erosion of the infill can take place, thus increasing seepage flows over time.

The mechanism of increasing the hydraulic gradients below and around the ends of the seepage barrier is enhanced in the case of solutioned limestone by the interconnected solution voids. This mechanism is shown schematically in Figure 3-95. In most cases, solution voids form along joint sets in the limestone bedrock and are interconnected where the joints intersect. Seepage paths beneath dams will first flow through the soil overburden and enter the solution voids through “entrance points”. Once through the entrance points, the flows coalesce in the interconnected system of solution voids and then exit through “exit points”. The exiting flows then flow through the downstream soil overburden. The seepage resistance is often much higher in the overburden than in the system of solution voids, and therefore, the seepage volume is controlled by overburden and the entrance and exit points. When a barrier is constructed partially across the solutioned limestone, it blocks a portion of the pathways in the limestone but may have little effect on the seepage volume capacity of the entrance and exit points. As a result, the same amount of seepage enters the system and must now flow around the seepage barrier in a reduced area. This results in increased seepage flow volumes and velocities in the solution voids not blocked by the barrier, and increases the erosion potential in these areas. For this reason, it is essential that seepage barriers designed to cut off seepage in solutioned limestone be extended beyond the extent of the solutioning.

**Dams on Jointed Bedrock Foundations**

The bases of seepage barriers in several of the dams included in this study are embedded in jointed bedrock. In three of these dams (Fontenelle, Navajo, and Mud Mountain) the
bedrock jointing was a crucial part of the seepage mechanism that the seepage barriers were designed to mitigate. In other dams (Upper and Lower Clemson, New Waddel, Twin Buttes, Mill Creek, Private Number 1, and Camanche) the jointed bedrock underlies the highly permeable layer that the seepage barrier was designed to cut off. These case histories show that construction of seepage barriers can result in increased flow through bedrock joints for two reasons: (1) erosion of soil infill or erodible rock from within the joints under increased gradients, and (2) seepage over long distances through open joints with little head loss.

In several of the case histories presented above (Wister, Navajo, Fontenelle, and Mill Creek), there is evidence in the form of changing piezometric heads and increasing seepage flows that suggest the seepage resistance along bedrock joints has changed over time. In none of these cases has a decrease in seepage resistance developed into a significant stability concern, and it is quite possible that the observed changes represent adjustments in the seepage regime toward a stable and safe long-term equilibrium condition. Nonetheless, the observed changes do indicate that erosion along bedrock joints is a viable mechanism for changing the long-term performance of a seepage barrier, and should be considered in the long-term design and monitoring of the barrier.

Open joints in rock foundations provide opportunities for seepage over long distances with small head losses. This phenomenon has been observed in several of the case histories presented above (Navajo, Mill Creek, Mud Mountain) and is significant in that it decreases the effectiveness of the seepage barrier by allowing high pressures and high flows to develop downstream of the seepage barrier, due to flow through joints around the boundaries of the barrier. In addition to the erosion potential along the joint, there are two consequences that may develop as a result of this flow. First, the exiting seepage may develop high hydraulic gradients at the outlet point downstream from the barrier that have the potential to initiate piping erosion. Secondly, the high velocity flow in joints may come in contact with erodible soils in the dam and foundation, and cause internal erosion of these soils. No cases are known where this mechanism has developed into an immediate concern. However, the erosion of soils adjacent to open joints is or has been a
Figure 3-95  Schematic illustration of flow paths through solutioned limestone and overburden A) before seepage barrier, and B) after seepage barrier
concern in Navajo, Mill Creek, and Mud Mountain Dams, and should be considered in future assessments of these dams and design of similar barriers.

Dams on Soil Foundations.

A large number of the dams in this study are founded fully or partially on native soil deposits. These deposits are derived from a number of depositional environments including alluvial, fluvial, glacial, and aeolian. In a few of these dams, long-term changes in the performance since seepage barrier construction have been observed but judged most likely to be related to seepage flows through solutioned or jointed bedrock. The only signs of internal erosion related directly to soil deposits are pulses of eroded soil that have been collected in the sediment basins of weirs of Manasquan, Private Number 1, and Chambers Lake Dams. These pulses represent single or limited events and are not believed to be related to long-term continuing mechanisms.

The lack of evidence of long-term internal erosion occurring in soil deposits may be attributable to the soils in the cases studied being resistant to internal erosion in locations where high gradients have developed. High resistance to internal erosion is likely the result of soils being internally stable under the imposed hydraulic gradients (not susceptible to suffusion) or effectively filtered by adjacent soils or bedrock. However, the lack of evidence of erosion of soil foundations should not be considered as reason to discount the possibility of such erosion occurring. It seems possible that, under the proper conditions, erosion of soil deposits may be a viable mechanism for deterioration of seepage barrier performance. Such conditions would include erodible soils exposed to high gradients, and the existence of an unfiltered pathway for the removal of the eroding soils.

Post-Construction Development of Leaks in Seepage Barriers.

Several of the case histories in this study showed direct evidence of the development of cracks or leaks in seepage barriers after construction. Evidence of leaks in rigid barriers
was detected in four dams (Wolf Creek, Fontenelle, Navajo, Twin Buttes, and Crane Valley Dams). Furthermore, it appears likely, that post-construction cracking has occurred in most of the barriers having rigid barrier backfill. Most of these cracks remain undetected due to the positioning of instrumentation and the minor impact of the cracks on the overall seepage regime. In addition to the development of cracks in rigid barriers, post-construction leaks were detected in two of the five soil-bentonite seepage barriers included in this study (Camanche Dike and Lewiston Levee). The mechanisms for development of post-construction leaks in barriers are quite different for rigid backfill and soil-bentonite backfill. These are discussed separately below.

The mechanisms causing cracking in rigid barriers include: (1) deformation due to seepage forces developed due to the change in seepage regime, (2) deformation from post-construction settlement, (3) shrinkage cracking, and (4) thermal cracking. Deformation analyses have been performed to investigate the mechanisms causing barrier deflection and the potential for deformation induced barrier cracking. The results of these analyses are presented in Chapter 4. The best documentation of cracking in a rigid barrier is in Navajo Dam where core holes and sonic testing were performed to map the locations of cracks in the barrier. While many of the detected cracks were in locations where high bending moments would be expected from deformation, other cracks were detected in areas where bending moments would not be expected. These other cracks may be the result of shrinkage or thermal cracking. The largest deformations and cracking observed in the study occurred in Crane Valley Dam where placement of additional rock fill, and possibly vibrations from train traffic, resulted in horizontal deformations in excess of 14 feet at the top of the barrier, and extensive cracking of the barrier.

The mechanisms for development of post-construction leaks in soil-bentonite barriers include: (1) separation from the overlying soil or barrier cap due to settlement of the infill, and (2) cracking or hydraulic fracture of the seepage barrier due to stress reduction in the backfill caused by upward drag from the trench walls as the backfill settles. In the Lewiston Levee barrier it appears that one or both of these mechanisms was responsible
for the leaks that developed. In the Camanche Dike seepage barrier, the location of the detected leak suggests that separation from the barrier cap is the likely cause. It is worth noting with regard to the first mechanism, that the in the three case studies where leaks were not detected in soil-bentonite seepage barriers, the barriers had either extended above the water level or been constructed through a low-permeability cap that was constructed prior to barrier construction. In the two cases where leaks have developed, low-permeability caps were constructed on top of the previously constructed barriers, increasing the potential for gap development when the barrier backfill settled.

Consequences of Leaks in Barriers.

As stated previously, leaks and cracks can develop in seepage barriers after construction due to a variety of mechanisms. In addition, leaks in seepage barriers may result from construction difficulties. In either case, the consequences of having a leak in a barrier will vary depending on the type of barrier and the soil and rock conditions through which the barrier is constructed. Consequences of barrier leaks can be grouped into the following categories: (1) decreased barrier efficiency, (2) erosion of the barrier, and (3) soil or rock erosion due to concentrated flow through the barrier.

The effect that cracking and defects have on barrier efficiency is as much or more a function of the surrounding soils as it is the aperture and number of defects. Small cracks in a barrier can reduce the effective permeability of the barrier by one or two orders of magnitude (see discussion in Chapter 4). However, the effect this reduction in permeability has on the overall efficiency of the barrier is largely a function of the permeability of the surrounding soil. In cases where the surrounding soil has permeability within a few orders of magnitude of the barrier backfill material, or where the permeable layer is thin, the reduction in effective barrier permeability can have a significant effect on the overall water pressure regime. Conversely, if the permeability of the surrounding soil is significantly higher than the seepage barrier backfill, the reduction in permeability will not significantly affect the water pressure regime because the extra seepage leaking through the barrier is easily dissipated in the high permeability soil.
However, the seepage volume may significantly increase in this situation as a result of the increasing effective hydraulic conductivity of the barrier.

If the velocity of water flowing through a defect in a seepage barrier is high enough, the water may erode the barrier infill and enlarge the defect. Similar to the piezometric pressure regime, the seepage velocity within the defect is a function of the size of the defect and the permeability of the surrounding soil or rock. Velocities are highest when the surrounding soils are highly permeable.

The erodibility of the barrier depends on the barrier backfill material and the filtering ability of the surrounding soil. Concrete backfill has the highest resistance to erosion and soil-bentonite backfill has the lowest. The effect the filtering ability of the surrounding soil has on the erosion of the backfill is illustrated by the case studies on Camanche Dike and Lewiston Levee, both barriers being composed of erodible soil-bentonite backfill. When a leak developed in Camanche Dike, the leak did not appear to enlarge over time, presumably due to the relatively low seepage velocity in the defect and the filtering ability of the silty sand through which the barrier was constructed. In contrast, when leaks developed in Lewiston Levee, the leak got worse over time because the open graded gravels through which the barrier was constructed allowed high velocities in the defect, and did not provide a filter for the eroding particles.

In two dams (Lake Wolford and Lake Valley) there was direct evidence of soil piping through defects in a barrier, resulting in voids or settlement upstream of the barrier. In both cases, the barriers had been in place for more than 50 years without previous signs of distress. In another case (Crane Valley Dam) large cracks were detected in the barrier and, while the cracks had filled with soil, there was no evidence of significant amounts of soil piping through the cracks. The difference between these cases is likely the hydraulic gradient acting in the area of the defect and the presence of a pathway for removing the eroding soil from the dam. Thus, based on available data, it appears that the potential for piping soil through the barrier is a function of the backfill materials around the barrier.
Instrumentation and Monitoring

Various means of monitoring seepage performance were utilized in the cases discussed above. Instruments used in these cases consisted of piezometers, flow measurement devices (weirs or flumes), settlement monuments, inclinometers, and stress cells. Because each dam and foundation has different characteristics, there is no “best way” to instrument a dam to monitor the seepage barrier performance. However, the review of the 30 cases provides some insight into effective monitoring of seepage barriers.

Flow Measurement. Where it is possible to measure a portion of the seepage flow through a dam, the data collected provide the best indicators of changes that occur due to seepage barrier construction and long-term changes after barrier construction. Defects in the barrier or concentrated seepage paths may result in piezometric head changes that are highly localized and easy to miss even with relatively close piezometer spacing. Conversely, flow measurements tend to collect flows from a wide area, increasing the chances of detecting changes in behavior.

The downsides of flow measurement are (1) the inability to measure subsurface water flowing through permeable soils or bedrock downstream of the dam, and (2) the inability to identify the location or cause of the change in flow due to a large tributary area leading to a flow measurement point. The first problem has been addressed in a few cases by installing relief wells and measuring the outflow of the wells. However, this solution is only applicable in a limited number of cases where the flow is under pressure beneath a low permeability surface layer. The second problem may be addressed somewhat by increasing the number of flow measurement points. While this solution may provide some additional insight into the sources of the flow, it is also has limited application.

Piezometers. Arrays of piezometers can provide data points that help define the piezometric pressure regime through a dam and its foundation and provide insights into the performance of the seepage barrier. Piezometers should be located to measure pressure at selected points of interest, and are often arranged along several cross sections.
in a dam to provide piezometric profiles through the sections. The downside of piezometers is that they can only measure pressure at a single point, as mentioned above. One consequence of this problem is that, in many cases, a piezometer would have to be located very near a leak in the barrier in order to detect the resulting change of head before it dissipates into the surrounding soil.

In several of the cases discussed, sets of piezometers were installed immediately upstream and immediately downstream of the seepage barrier. Generally, these sets of piezometers were installed within the most permeable layer being cut off, although some dams have piezometers in multiple layers. These piezometers have provided valuable insight into the performance of the seepage barrier by making it easy to calculate the differential water pressure across the barrier. Knowing the differential water pressure, it is easy to calculate the forces acting on the barrier as well as the hydraulic gradients acting across the barrier and in the soil or rock below the barrier. The information on the hydraulic gradients can help assess the potential for various mechanisms of internal erosion.

**Settlement Monuments.** Settlement monuments have been installed and are monitored on most dams to detect anomalous surface settlements or settlements that are larger than anticipated. In a few of the cases discussed (Wolf Creek, Mud Mountain), anomalous settlement was detected that may be indicative of internal erosion in the dam. When anomalous settlement has been detected, other means of instrumentation, such as piezometers, can be installed to further investigate the cause of settlement.

**Inclinometers.** Inclinometers were installed in or adjacent to seepage barriers in several of the dams discussed. These instruments provide insight into the post-construction deformation of the barrier and provide a valuable check for the deformation analyses that were performed and presented in Chapter 4. The deformation of the barrier can lead to cracking at locations where high moments are developed. Knowing where cracking is likely to occur or has occurred, it may be possible to install piezometers to measure the
effects of this cracking on the pore pressure regime upstream and downstream of the barrier.

**Pressure Cells.** Pressure cells were installed in one case (Manasquan) to measure the total pressures within a soil-bentonite seepage barrier. These measurements provide insight into the behavior of soil-bentonite seepage barriers and the susceptibility of these types of barriers to cracking and hydraulic fracture. However, it is unlikely that arrays of pressure cells will become a standard for instrumentation of soil–bentonite seepage barriers due to the cost of providing enough cells for monitoring purposes.