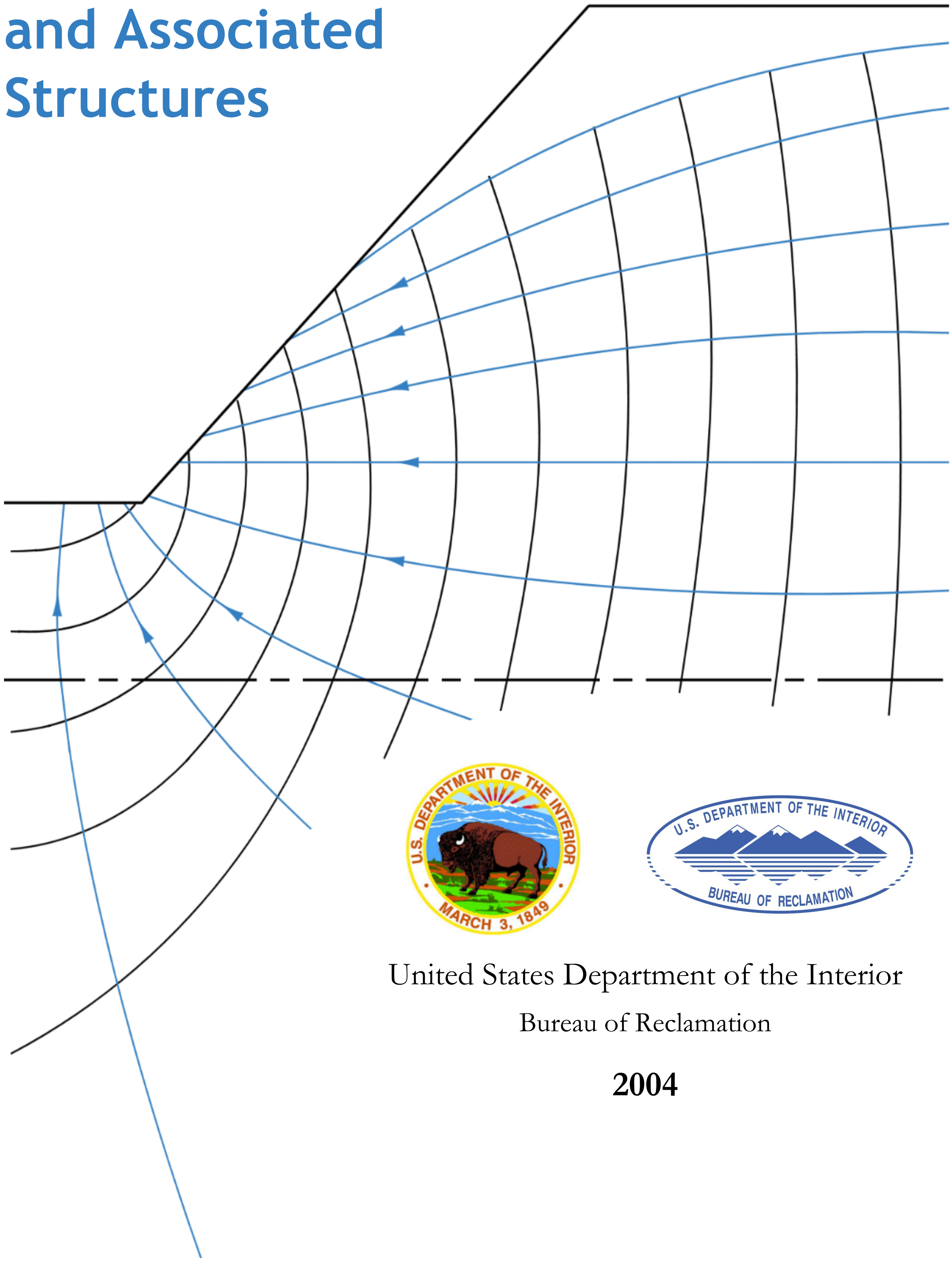
Drainage for Dams and Associated Structures



United States Department of the Interior

Bureau of Reclamation

2004

Drainage for Dams and Associated Structures

Technical Service Center
Denver, Colorado
2004

United States Department of the Interior
Bureau of Reclamation

MISSION STATEMENTS

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to tribes.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Preface

Drains are important design features for dams and appurtenant structures. It is standard practice to include drains in the design of these structures. They reduce seepage uplift pressures, control seepage, and reduce foundation driving forces on dams and other structures, improving their stability.

A primary purpose of this manual is to provide information that can be used to establish an effective drain maintenance program. Drains at Bureau of Reclamation (Reclamation) dams have received varying amounts of attention. In some cases, drains have not been maintained since they were constructed. Drains that are obviously plugged have generally been detected during an examination of the dam, and a cleaning program initiated. In other cases, instrumentation data have provided strong indications that drains are plugged, and appropriate action was taken. In still other cases, drains have become plugged without the plugging being detected for a period of time, because the drains could not be easily accessed and visual evidence did not strongly suggest that the drains were plugging. Cleaning programs have resulted in varying degrees of success.

This manual hopefully will provide a heightened awareness of the importance and benefits of drainage systems and the need for periodic monitoring and maintenance of these systems. Rather than providing strict requirements for a regular drain cleaning program, this manual is intended to provide guidelines for evaluating drainage systems and determining the need for initiating a drain cleaning program. The current Comprehensive Facility Review and Periodic Facility Review process results in a systematic evaluation of each Reclamation dam. This provides an ideal opportunity to evaluate drainage systems, considering the instrumentation data and the physical condition of the exposed portions of drainage systems.

The need for a more comprehensive inspection of the drainage system, possibly through the use of a remote video camera can also be assessed. The Performance Parameter documents now available for all major Reclamation dams also are a valuable tool for assessing the performance of drainage systems. Performance Parameter documents should indicate acceptable levels for drain flows and water pressures at a specific dam, and levels at which a further evaluation of a drainage system or a drain cleaning program should be initiated.

This manual provides background information on the purpose of drainage systems and on the design and analysis of drainage systems. This information is provided to illustrate the importance of drainage systems to the overall stability of dams and associated structures, and also as a starting point for the design and analysis of drainage systems. The design and analysis information is basic and not intended to be a comprehensive design guide.

The manual is organized as follows: general descriptions and the purposes of various types of drainage systems for dams and appurtenant structures (ch. 1); design guidelines and analysis methodology on various drainage systems (ch. 2); installation methods for drains (ch. 3); case histories that illustrate the performance of a variety of drainage systems (ch. 4); and, guidance on maintaining drains, including a discussion of drain plugging mechanisms, methods for evaluating drain effectiveness, guidelines for drain maintenance, site-specific considerations, and a summary of drain cleaning methods (ch. 5). Appendices are also included that contain design examples (app. A), detailed case histories (app. B) and descriptions of drain cleaning and inspection equipment (app. C).

The authors of this manual are Bill Fiedler, Rick Kelsic, John LaBoon, Becky Morfitt, Rich Munoz, Gary Turlington, Chuck Cooper, Doug Hurcomb, and Steve Young. The peer reviewers are Bill Engemoen and Gregg Scott. The technical editor for the manual is Lelon A. Lewis. The authors would like to thank Bill and Gregg for their thorough reviews and substantial feedback, which greatly improved the manual. They would like to thank Mr. Lewis for his tireless work in assembling numerous drafts and shaping the writings of multiple authors into a coherent document. The authors would also like to thank the following people who either reviewed or provided input to the manual: Chuck Cooper, Doug Craft, Dave Harris, Dave Hinchliff, Larry Nuss, Mike Romansky, and Rod TeKrony of the Technical Service Center; Jim Kinney of the Office of Policy; Lloyd Crutchfield, Lovell Parrish, and Jan Seckel of the Great Plains Regional Office; Brad Augustine of the Lower Colorado Regional Office; Marvin Lockhart of the Pacific Northwest Regional Office; Mark Neeley of the Upper Colorado Regional Office; Ken Lally of the Mid-Pacific Regional Office; Kit Kwiatkowski of the South-Central California Office; Bob Major of the Albuquerque Area Office; Keith Brooks of the Snake River Area Office; Randy Harris of the Grand Coulee Power Office; Kit Kwiatkowski of the South-Central California Area Office; and Terry Seitz of the McCook Field Office.

Funding for this manual was provided by the Dam Safety Office, the Technical Service Center, and the Office of Policy. The authors would like to thank these offices for their joint efforts in support of the manual.

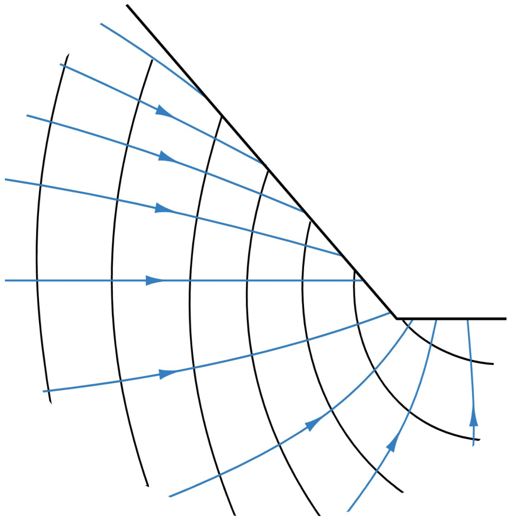
Contents

Tables

No. *page*

Figures

No *page*



Chapter 1

BACKGROUND

Drains are a common feature for dams and their foundations as well as for associated structures and their foundations. In general, all drains fulfill the same purpose—they reduce seepage pressures within a structure or foundation and improve the stability of the structure or foundation. Drains also provide the benefit of collecting and transporting seepage to a desired outfall location, while minimizing aesthetic impacts, or impacts due to erosion. There are a variety of drain types and configurations. Factors that influence the type and configuration of drains are the type of structure or foundation, the expected seepage or groundwater locations and volume of drain flows, ease of construction, and accessibility. This section describes the types and purposes of various drainage systems for concrete dams and their foundations, embankment dams and their foundations, spillway and outlet works structures and their foundations, and for other features.

1-1. Purpose of Drains.—Drains are provided in dams, in dam foundations, in certain appurtenant structures, and at structure/foundation interfaces to safely intercept, collect, and transport seepage that may compromise the stability of any of these features. An important component of dam stability, drains reduce uplift/pore water pressures associated with reservoir loading that can develop below and/or within a structure. Uplift/pore water pressures develop when water enters discontinuities, such as joints and faults, in rock foundations; when water seeps through soil foundations and embankments; and when water accesses structural interfaces within a dam or foundation. Uplift/pore water pressures can reduce the ability of a structure to resist sliding and overturning. The magnitude of the uplift/pore water pressure depends on a number of factors, including relative material permeabilities and the extent to which water is limited from exiting discontinuities or materials.

Collected seepage can be measured to detect changes in seepage flows that may indicate changes in the condition of the dam or foundation, or possible clogging of drains. Furthermore, collected seepage can be inspected for the presence of sediments that may indicate a possible loss of soil materials. The effectiveness of drainage can be evaluated from instrumentation measurements through the use of piezometers, observation wells and/or uplift pressure pipes installed upstream and downstream of a drainage system.

a. *Concrete Dams*.—Major concrete storage dams, regardless of dam type, are typically constructed on rock foundations which may contain joints, faults, and other discontinuities. Within the concrete structure itself, formed and unformed joints exist, associated with concrete placements, cantilever interfaces, and appurtenant structure contacts. Furthermore, cracks may develop within concrete structures due to tensile stresses that exceed the concrete tensile capacity caused by shrinkage, loading, and/or differential settlements.

When reservoir water enters the material, joints, cracks, and/or discontinuities within a dam or dam foundation, uplift pressures develop proportionally to the hydrostatic reservoir head. Uplift pressure within these discontinuities will increase the overturning moment and reduce the sliding stability within concrete dams and foundation blocks. Generally speaking, except for foundation discontinuities, the consequences of uplift are greater for gravity dams than for arch dams due to the greater surface area over which uplift can act and the assumed lack of three-dimensional load transfer. In arch dams, the effect of uplift is often critical within the foundation and abutments, since it reduces the stability of foundation blocks that carry loads. Historical data show foundation failure to be the major cause of concrete dam failures [2].

b. *Embankment Dams*.—Embankment dams and foundations are not totally impervious. Control of the leakage is necessary to prevent uneconomical loss of water and prevent water forces and pressures related to seepage from contributing to static instability and piping or blowout. Cutoff trenches, cutoff walls, and upstream impervious blankets are methods used to reduce the flow of water and control water pressures. Chimney drains, downstream drainage blankets, toe drains, drainage trenches, drainage tunnels, and relief wells are used to collect seepage and reduce pressures to prevent static instability, blowout, uplift failure, and piping failure in the downstream zones and foundations of the embankments.

A number of failure mechanisms can develop for embankment dams that are related to seepage/drainage:

- *Stability of Embankment (High Pore Pressures)*.—The water table in sloping ground (an embankment) can affect the stability of the mass and cause a slope failure. The factor of safety for slope stability is evaluated as the ratio of the resisting force to the driving force. The resistance is a function of the effective stress which is affected by saturation (effective stress = total stress - pore pressure). A saturated material has a lower effective stress, which can also be looked at as water generating pore pressure that decreases the effective stress and resisting force. To a lesser extent, the seepage force imparted by seeping water on the saturated medium through friction can contribute to the driving force (fig. 1). Therefore, lowering the line of saturation increases the resisting force (fig. 2). Clogged drains prevent the release of water and can allow the pore pressures (line of saturation) to increase, which could cause a stable slope to become unstable.

- *Piping or Internal Erosion*.—Piping or internal erosion involves the progressive removal of soil particles from a mass by flowing water. Several conditions must exist in order for there to be piping and a piping-related failure. First, there must be a source of flowing water. Second, there must be erodible soils, which are usually fine grained soils such as silts and clays. Clays tend to be nonerodible under low pressures due to attractive forces between particles. Then there must be a free surface where the water is released and the soil particles can be removed. This can be open ground, concrete cracks, rock cracks, or soils with voids such as coarse sands, gravels, rockfill, or riprap. The material must also be capable of sustaining a roof or "pipe." As the water exits, the small soil particles are carried with the water, and a hole starts forming. As a piping hole forms it enlarges and the velocity of the water flow increases, which allows larger and larger particles to be carried away and the erosion "pipe" to progress toward the reservoir. Eventually, the hole becomes large or progresses backward to the point where it is in contact with the reservoir, and the embankment erodes rapidly and releases the reservoir uncontrollably as shown on figure 3. Sinkholes and depressions can be formed as material from above falls into the piping hole. If the piping is occurring through cracks in concrete or rock, the rate and extent of the piping are restricted by the size of the opening, and as larger particles are moved, the crack can actually become plugged.

  To prevent piping, either the water level can be lowered or the pressure reduced, so that the force required to move the particles does not develop. This can be done with relief wells and downstream drainage trenches, or the seepage path can be increased with impervious upstream blankets and cutoff walls, which in turn reduce the pore pressure and exit-gradient. Another means of preventing piping is to place a material (filter) at the exit location that has void spaces large enough to allow the water to flow out but small enough to prevent the soil particles from moving. This can be accomplished with a filter and toe drain system.

Piping is progressive and can result in a rapid failure (often occurring within the first filling of the reservoir) or a failure that starts slowly and then progresses rapidly. Careful monitoring of seepage exit areas for evidence of material being moved is important if early intervention is to occur to prevent continuation of piping and an uncontrolled release of the reservoir. Clogged drains and improperly functioning relief wells can cause the water pressure and exit-gradient to increase and cause initiation of piping.

- *Blowout (High Pore Pressures).*—Foundation pore pressure can exert significant uplift force on a confining layer of soil downstream of the dam. This occurs when a more permeable layer underlies a less pervious layer, also known as a confining layer (e.g., a clay layer overlying a gravel layer). Failure begins to occur when the pore pressure on the bottom of the confining layer exceeds the overburden pressure created by the weight of the confining layer. The resulting uplift eventually breaches the confining layer, providing a flow path that can lead to a piping or fluidization failure. The factor of safety can be calculated as shown on figure 4. A minimum factor of safety of 2 is recommended, depending upon the confidence level of the variables.

c. *Spillways and Outlet Works.*—Drains beneath and/or adjacent to appurtenant structures should be provided to control (mitigate) excessive water pressures which might lead to instability, including failure of appurtenant structures and/or their foundation (see fig. 5 for examples). Appurtenant structures include spillways, outlet works, canals, pipelines, retaining walls, drainage adits, access adits, powerplants, and pumping plants. Historically, it has been the Bureau of Reclamation's (Reclamation) practice to design appurtenant structures to withstand part or all of an anticipated water pressure (i.e., assuming drains do not function or are only partially effective). However, it has been and is considered prudent engineering practice to provide drainage for critical appurtenant structures (An appurtenant structure is considered critical if increased risk to the dam and/or downstream consequences could result from misoperation, inability to operate, and/or failure). It should be noted that the science of drainage and groundwater has progressed to the point where drain designs can incorporate considerations/details that will result in a high level of confidence that drainage effectiveness can be maintained over the service life of the appurtenant structure. Other guidelines/criteria recommend that drainage provisions should be considered for any appurtenant structure where the permanent water table is above the foundation or the temporary water table, due to local ponding or seasonal variation, could exceed the foundation level [3].

d. *Cut Slopes.*—The purpose of drains associated with rock or soil cut slopes is to depressurize the slope. This reduces driving forces and uplift pressures on rock wedges or soil masses formed along a circular failure surface and improves their stability.

e. *Tunnels*.—The purpose of drains in tunnels is to relieve groundwater pressures surrounding the perimeter of a tunnel. This is important for concrete-lined tunnels that might be associated with a spillway or outlet works. The presence of a reservoir upstream of the tunnel can significantly increase groundwater levels and water pressures in the abutments. A concrete lining seals off joints and fractures along the perimeter of the tunnel and prevents drainage and relief of groundwater pressures, unless the lining is drained.

1-2. Types of Drains.—Various types of drains are used to control seepage and limit uplift in concrete dams, embankment dams, dam foundations, and appurtenant structures associated with either concrete or embankment dams as well as in natural slopes. Each drain type is generally suited to a particular application such as within foundations, within massive concrete structures, within embankment dams or their foundations, at interfaces between concrete and rock, and at interfaces between existing concrete and new concrete placements. Drain types include drilled drains, formed drains in concrete, underdrains, and prefabricated drains. The various types of drains are described below:

a. *Concrete Dams* .—

1. *Foundation Drains*.—Foundation drains, typically drilled in a fixed pattern of hole spacing and depth referred to as a drainage curtain, are used to collect seepage that passes through rock foundations and abutments of concrete dams. Such drains are drilled from galleries within the dam, from the downstream toe, or from adits excavated into the abutment rock. As a rule, a foundation drainage curtain is located downstream of the grout curtain and consists of one or more lines of holes parallel to the dam axis. Most Reclamation drainage curtains consist of 3-inch diameter drain holes at a 10-foot spacing, drilled to a depth of up to 40 percent of the dam height. Drain hole diameters may range from 2 to 3 inches, spacings from 5 to 15 feet, and depths from 20 to 40 percent of the dam height. Because drain efficiency can decrease significantly as the depth of hole is reduced, depths of 40 percent of the dam height are recommended. The diameter, spacing, and depth of drain holes may be adjusted based on a prior knowledge of foundation discontinuities and expected seepage paths and quantities. A typical foundation drain layout, figure 6, shows foundation drains drilled from the drainage gallery and from foundation adits.

It is common practice in arch dam design to provide drainage in the abutments by drilling drains from foundation adits. Foundation adits extend from galleries in the dam into the abutments where drains can be drilled into the abutment at any desired angle and depth. Adits can also be excavated from the downstream face of the abutment and provide access for drilling drains into the abutment. This is often the case in dam rehabilitations where the only access is from the downstream face. Figure 7 is a layout drawing of an abutment adit and drain system. Section views of the adit and drains are shown in figure 8.

2. *Formed Drains*.—Formed drains are placed within the mass of a concrete structure to intercept seepage that may enter through joints, unbonded lifts, or cracks in the dam. These drains, which are generally vertical in orientation and circular in cross section, extend the height of the dam and are connected to galleries where seepage is collected for transport to a downstream location. Typically, formed drains have a diameter of 5 inches and are spaced at 10-foot centers. Holes may have diameters that range from 3 to 8 inches and have spacings as close as 5 feet. Figure 9 shows a layout for formed drains. Figure 10 shows the layout of foundation drains (foundation drains shown in this figure extended to a depth less than the recommended 40 percent of the dam height) and formed drains with each of the drainage systems tied into the foundation gallery. Also shown in this figure are adits from which foundation drains are drilled in the upper abutments.

3. *Prefabricated Drains*.—Prefabricated drains are generally used at structural interfaces, such as between an existing structure and a concrete overlay, where seepage control is needed to ensure stability and where formed drains are not feasible. Prefabricated drains are manufactured in a variety of solid and flexible shapes for different applications.

Prefabricated drain applications have been used in dam modifications, as shown in figure 11, with details shown in figure 12. In this application, 2-inch diameter perforated polyvinyl chloride (PVC) pipe was placed at the interface between the downstream slope of the concrete arch dam and a roller-compacted concrete (RCC) overlay. Welded wire fabric and burlap were used to attach the drains to the existing concrete surface and prevent infiltration of the new concrete into the drains.

b. *Embankment Dams*.—

1. *Blanket Drains*.—A blanket drain is a horizontal or inclined layer of sand and gravel material that carries the seepage water to a pipe or natural exit. These drains are also known as filter/drain layers, chimney drains, inclined filters, and horizontal filters. Drainage blankets are used beneath slope protection, beneath membrane liners, beneath embankments, and on abutments, and can be used as a part of an internal drainage system in embankment dams and dikes. Figure 13 shows the general location of blanket drains in an embankment dam.

2. *Toe Drains*.—Toe drains are designed as a downstream extension of the embankment internal drainage system to colllect seepage passing through the dam or foundation and should also be designed with filter protection from seepage flow entering from the foundation. Toe drains typically consist of perforated or slotted pipe sized to carry this flow and all of the flow from any attached blanket drains. Pipe sizes generally range from 8-to 24-inch diameter, depending on seepage values expected. Typically the pipe is HDPE, although concrete and PVC pipe is also used. An outfall pipe is normally placed as

low in the system as the discharge point will allow, to provide maximum drainage. Manhole access should be provided at intervals along the toe drain system for the purposes of inspection, measurement, and maintenance. The toe drain is sometimes the only drainage feature needed to mitigate or eliminate seepage exit gradients at the toe of the dam. Figure 13 shows the general location of a toe drain in an embankment dam. Section 2-4 gives more details on toe drain design. Old toe drains were oftern constructed of clay tile pipe with open joints. These drain types are prone to inflow of material and plugging. Collapse of HDPE pipe has also recently been observed.

3. *Downstream Drainage Trenches.*—Downstream drainage trenches running parallel to the toe of the dam can be used when downstream drainage of the foundation is needed and a low enough discharge point is available. An open trench should be lined with graded sand and gravel material for filter protection and riprap or other suitable material for erosion protection. A closed trench (typically 2 to 3 feet wide) should be backfilled in the drainage zone with pervious soil meeting filter criteria, have a properly sized collector pipe (typically 8-to 12-inch diameter HDPE pipe) and be provided with access manholes at intervals for inspection, measurement, and maintenance. Figure 13 shows the general location of a drainage trench in an embankment dam.

4. *Relief Wells.*—Relief wells are used to reduce artesian pressure in a confined aquifer to a tolerable level with respect to factor of safety against uplift and/or exit gradients through the confining layer downstream of the dam. Spacing of relief wells depends on a number of factors, including the required reduction in pore pressure, the permeability of the foundation materials, and the thickness of the confining layer. Drilled hole diameters are typically 12 to 18 inches, which allows for installation of a filler pack and well casing. Gravity flow relief wells can be installed at smaller diameters. Filter criteria should be used in designing the "gravel pack" around the well screen. Figure 13 shows the general location of a relief well at an embankment dam. Section 2-4 gives more details on the layout and design of a pressure relief well system.

5. *Drainage Tunnels.*—Drainage tunnels from which a series of drain holes can be drilled are sometimes necessary to relieve seepage pressure from rock abutments and remove seepage flows from near the embankment/foundation contact. Considerations should always be given to possible adverse effects of increasing the gradient through the embankment core and the possible need for filtering when such drainage of the contact occurs. Figure 14 shows the design of the right abutment drainage tunnel and drains for a dam modification.

6. *Semihorizontal Drain Borings.*—Semihorizontal drain borings can be made into the abutments of a dam to relieve excessive pore pressures or intercept seepage before it reaches the dam or an erodible exit surface. The spacing and depth of the drain borings will

vary depending on the pressure that needs to be relieved, the location of the foundation areas that need to be drained, and the permeability of the foundation materials. Drilled hole diameters are typically 4 to 6 inches, with the drain pipe diameter used to case the hole usually 2 to 4 inches. Proper drilling techniques, screening and filtering must be used when installing this type of drain to prevent erosion of materials due to the increased gradient and velocity created.

7. *Surface Drains.*—Surface drains in groins, along abutments, at the toe of a dam and in other areas are used in conjunction with slope protection to protect the structure from erosion by surface waters. An open trench should be lined with graded material for filter protection, and riprap or other suitable material for erosion protection.

c. *Spillways and Outlet Works.*—A number of different types of drains are used with spillway and outlet works structures. Drains used in Reclamation appurtenant structures include:

- *Structure underdrains.*—Structure underdrains collect seepage at the interface between foundation rock or soil and a concrete structure, typically a concrete slab where uplift is a concern. Common applications of underdrains are in chutes, conduits, and stilling basins for dam spillways and outlet works. Underdrains are typically laid out in an interconnected grid pattern, so that seepage can be collected and carried to a point downstream of the overlying structure. In some cases, seepage may be collected in an access gallery located along the axis of the overlying structure. Drains consist of split tile pipe drains, and perforated and slotted PVC drains with a sand/gravel envelope. Geotextiles are not recommended as an envelope for underdrains, because they are prone to plugging. This type is primarily used beneath slabs or adjacent to walls (see fig. 15 for examples). Also, high density polyethylene (HDPE) pipes have been used in lieu of PVC drain pipes. Split tile underdrains were common in older installations.

- *Weep holes.*—Drilled and formed drains (sometimes referred to as "weep holes") may be a stand alone drainage provision or tied into collector drains (manifold) beneath and adjacent to slabs or walls (see fig. 16 for examples).

- *Prefabricated drains.*—Prefabricated drains (such as "flat drains") are self-contained drainage features (generally, a geotextile exterior to filter seepage and an interior PVC collector). This type has been effectively used between foundation and concrete and between existing and new concrete (see fig. 17 for examples).

- *Graded Drains.*—Graded drain systems serve as a foundation drain and can envelop features of an appurtenant structure, such as a conduit through an embankment dam. These drain systems are used to control and filter seepage that flows through or beneath an embankment or rockfill impoundment structure, such as dams, dikes, levees, or similar structures (see figs. 18 and 19 for examples). These features are often placed beneath embankment dam overtopping protection, channel protection/armorment, and around conduits/pipes.

- *Free draining backfill.*—Free draining (pervious) backfill adjacent to walls is a common drainage provision of many appurtenant structures. This is a "well graded" material ranging in size from a maximum of 3 inches to less than 5 percent passing a No. 200 sieve.

As previously noted, Reclamation's historical approach to drainage features for spillways and outlet works has been to design for loss of effectiveness over time, due to the difficulty in accessing, monitoring, and maintaining drains. However, there are now design and operation and maintenance (O&M) tools that would result in a high level of confidence that drainage effectiveness can be maintained over the service life of the appurtenant structure. To date, not all of these tools have been fully implemented on Reclamation jobs, but the effort continues. As an example, figures 20 and 21 provide a recent design example of cleanouts being provided in a spillway chute for future access and maintenance. As another example, Reclamation has implemented remote controlled video inspection (RCVI) for accessing/inspecting drains as small as 3 inches in diameter. Although there are still access difficulties (such as accessing drains at interconnects, where the turning radius is less than 8 inches), this tool had potential value in visually evaluating the present condition (effectiveness) of existing drainage features, which were previously inaccessible. For additional information about RCVI, refer to appendix C.

Although many drainage provisions for appurtenant structures are designed based on historical practice, consideration should always be given to evaluating whether the drainage requirements or expectations fall within the examples and assumptions being used as guides. To evaluate the level of analysis/design required for the appurtenant structure drainage system, geology/geohydrology information (such as extent, thickness, stratification, and permeability of foundation), along with stream and groundwater fluctuations may be needed. Additional information concerning the level and method of design for appurtenant structure drainage systems can be found in Chapter 2, *Drain Design and Analysis*.

d. *Slopes.*—Water pressure can have two adverse effects on soil and rock slopes. It can increase the driving force within the slope, and it can decrease the frictional resistance along sliding surfaces. The presence of water within slopes is not critical by itself. The water pressure within a slope is the important parameter.

In order to improve the stability of slopes, drainage can be provided. The drains will depressurize the slopes and improve stability. There are several different ways of providing drainage:

1. *Horizontal or Angled Drains*.—This is the most common method of depressurizing slopes. This type of drainage system is quick to install, can be effective up to 300 feet into the slope, can penetrate steeply dipping features in a rock mass, and does not require power to operate. Disadvantages of horizontal drains are that they must be installed after excavation; they drain water on the face of the cut slope, which could lead to erosion and aesthetic concerns; they are prone to freezing unless angled; and they may require maintenance. Typically, drilled horizontal drain diameters would be 2 to 4 inches. In soil slopes, the drains are lined with a perforated PVC pipe, and the annular space between the PVC pipe and the outside of the drilled hole is plugged at the outlet end of the hole. In some cases, the PVC pipe may be wrapped with geotextile. The spacing of horizontal drains depends on the geometry of the slope and on the soil type and stratigraphy for soil cut slopes, and on the location and orientation of discontinuities for rock cut slopes.

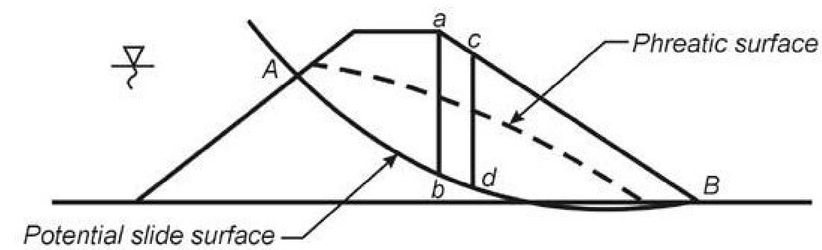
2. *Dewatering Wells*.—Wells can also be used to depressurize slopes, although they are not as common as horizontal drains. Advantages of dewatering wells are that they are effective over considerable distances, they can be located remotely from the active excavation and provide benefits during the excavation of a slope, and they direct water away from the excavated face and excavation in progress. Disadvantages of dewatering wells are that they require power and high maintenance, they must be located remotely from the area being excavated, where depressurization is most needed, and they are expensive. Dewatering wells typically have the pump located at the bottom of the well for maximum efficiency and a well screen over the full depth of the well. Wells are typically spaced on the order of 50 to 200 feet.

3. *Drainage Adits*.—Drainage adits are typically constructed parallel to the face of a rock cut slope. Drain holes are then drilled from the inside of the adit into the rock mass. The drains discharge water into the adit, where it collects and is discharged via gravity or a sump pump. The advantages of a drainage adit are that it is a highly effective method of draining a slope, it can be installed at the optimum location for depressurization, it can be installed in advance of the excavation or as a remediation effort after excavation, water is collected in one location, and it is not subject to freezing. The disadvantages of a drainage adit are the relatively high cost, and that it takes longer to construct that other depressurization methods. The drains installed from within the drainage adit are typically very similar to horizontal drains, consisting of drilled holes, occasionally lined with slotted PVC pipe.

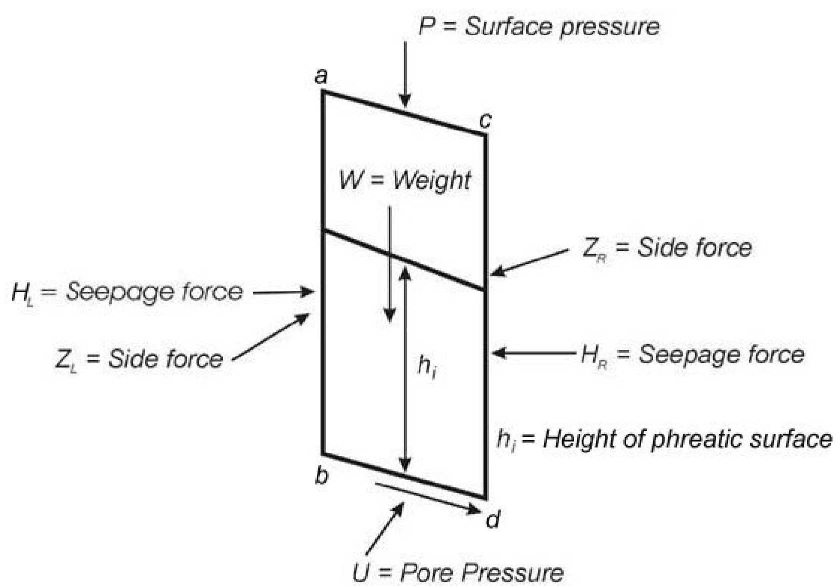
4. *Surface Drains*.—Surface drains typically consist of ditches or trenches at the top of a cut slope or at a bench. They are designed to collect surface drainage before it can infiltrate and saturate the slope.

Figure 22 provides a sketch showing each of the four drainage systems that can be provided for rock and soil slopes.

e. *Tunnels*.—In some cases, drains are required to reduce foundation water pressures on the tunnel lining and to prevent collapse of tunnel linings. Care should be used, as drains can also pressurize the surrounding material when water flows through the tunnel. Typically, drainage holes for tunnels consist of drilled drains that are 1½ to 3 inches in diameter. Drains are typically located just below springline and are drilled slightly upslope to allow for drainage into the tunnel. Drains are typically spaced at 20-foot centers and drilled 20 feet deep, with two drains (one on each side of the tunnel) provided at each location. Figure 23 provides an example of drains that were provided for a downstream river outlet works tunnel. Tunnel drains can also serve as an exit point for piped material from an embankment dam or from the dam tunnel foundation. The seepage should be carefully monitored for piped material.



a) Typical Failure Surface



H_L and H_R are seepage forces and with small slices are considered equal and opposite and cancel each other

U = Pore Pressure = (h_i) (γ_W = 62.4 lb/FT^3)

b) Typical Forces Acting on a Slice

Figure 1.—Forces used in stability analysis.

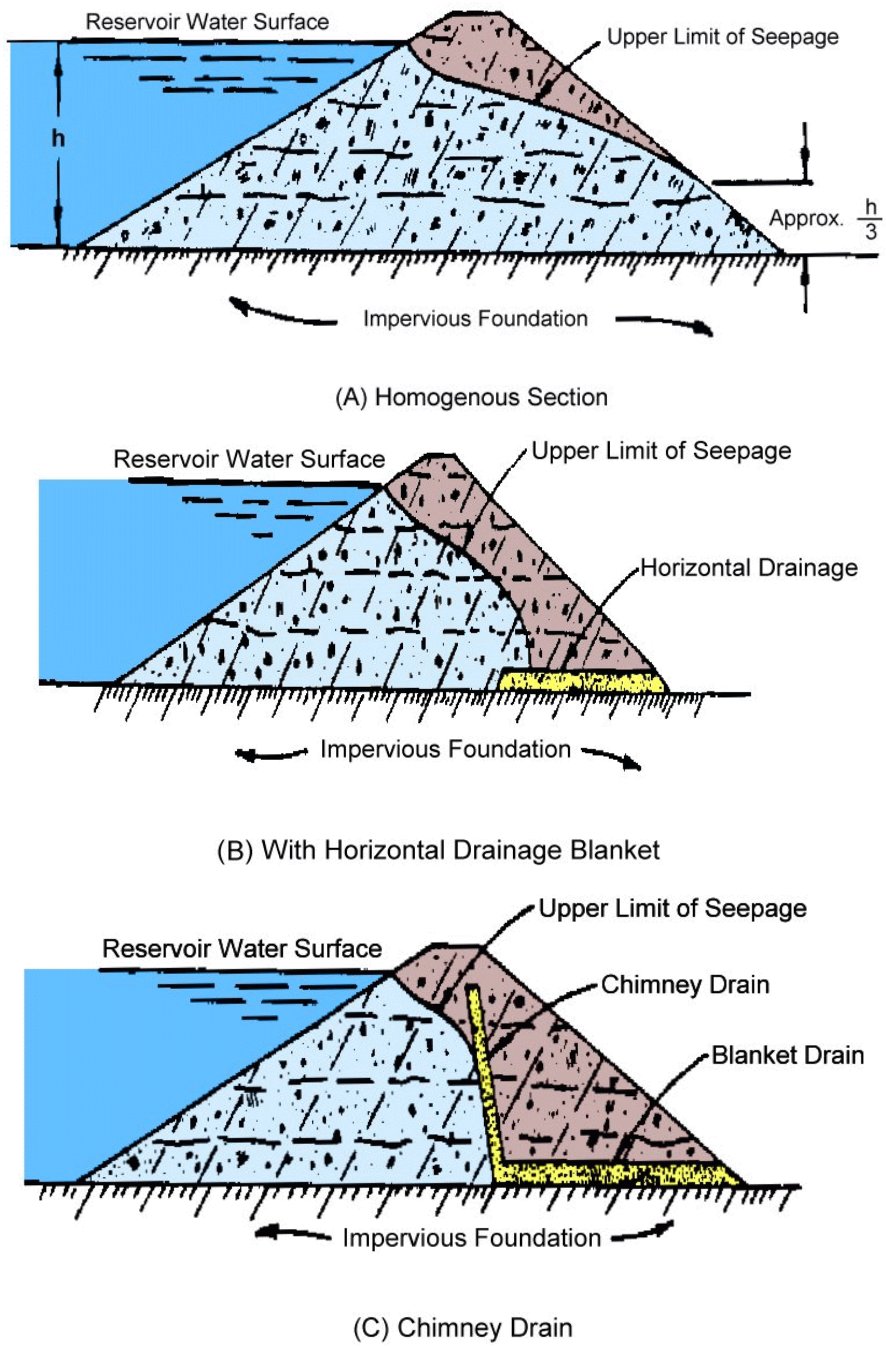
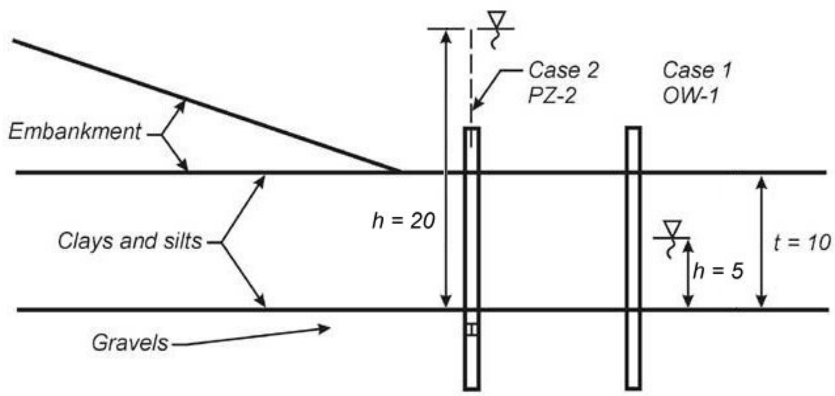


Figure 2.—The effect of internal drains on the zone of saturation in embankment dams.

Figure 3.—Piping through the embankment leads to the failure of a dam.



$$\text{Safety Factor} = \frac{(\gamma o)t}{(\gamma w)h} > 2 \text{ (Design criteria)}$$

γo = Unit weight of confining layer material
t = Thickness of confining layer
γw = Unit weight of water
h = Pore Pressure at bottom of confining layer

$$\textit{Case 1} \quad SF = \frac{(103\ lb/FT^3)\ (10\ ft)}{(62.4\ lb/FT^3)\ (5)} = 3.3 > 2 \textit{ Acceptable}$$

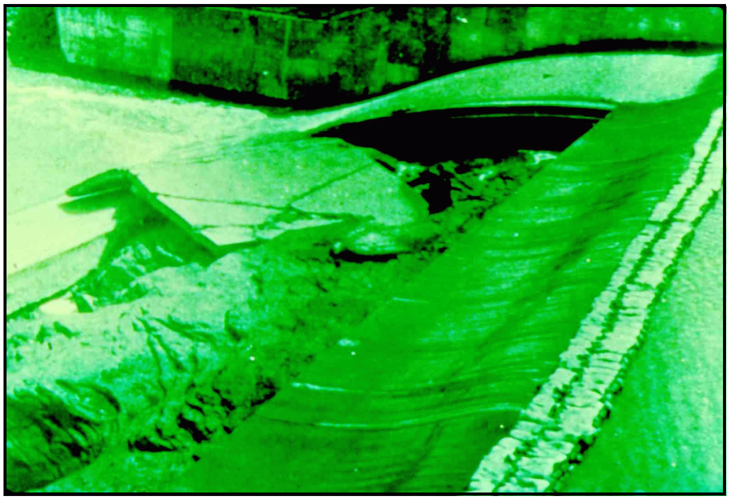
$$\textit{Case 2} \quad SF = \frac{(103\ lb/FT^3)\ (10\ ft)}{(62.4\ lb/FT^3)\ (20)} = 0.8 < 2 \textit{ Unacceptable}$$

Figure 4.—Example blowout calculation.

a. Vallecito Dam spillway stilling basin, Colorado.—Failure of the left retaining wall was the result of inadequate drainage provisions, which led to accumulation of water behind the wall, and subsequent large lateral loads, due to ice formation.

b. Vallecito Dam spillway stilling basin, Colorado.—After backfill was excavated, only portions of the counterforts remained.

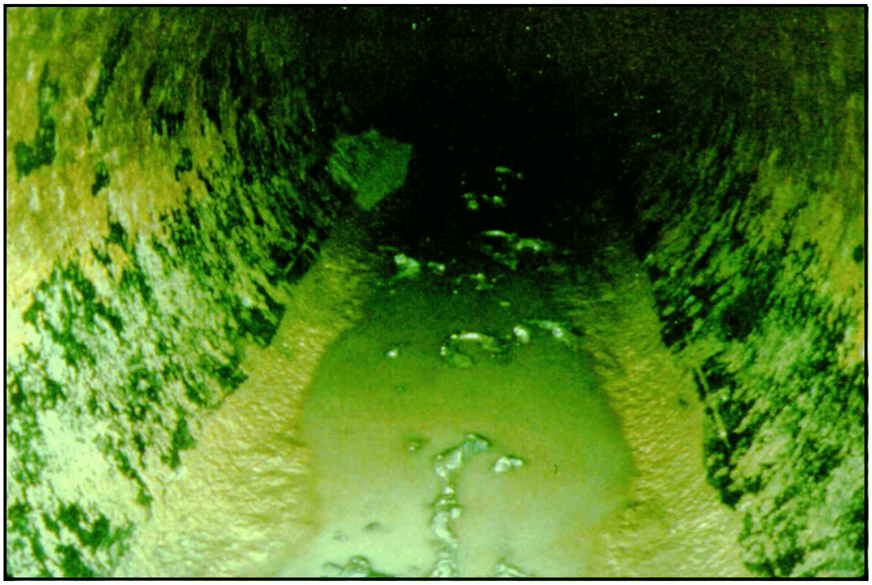
Figure 5.—Appurtenant structures—Examples of damage/failure due to inadequate drainage provisions.



c. Dickinson Dam spillway chute, North Dakota.—Due to improperly graded drain filter material, foundation was piped out and floor slab was collapsed.

d. Grassy Lake Dam spillway, Wyoming.—Due to excessive frost heave, walls have failed.

Figure 5 (cont'd)



e. Unknown dam.—Sediment accumulation in an outlet pipe could be a warning sign that piping of adjacent embankment material is occurring. This could be the result of inadequate drainage of the embankment and/or the outlet pipe.

f. Unknown dam.—The worst-case scenario for piping of adjacent embankment material can lead to failure of not only the outlet works, but also the embankment.

Figure 5 (cont'd)

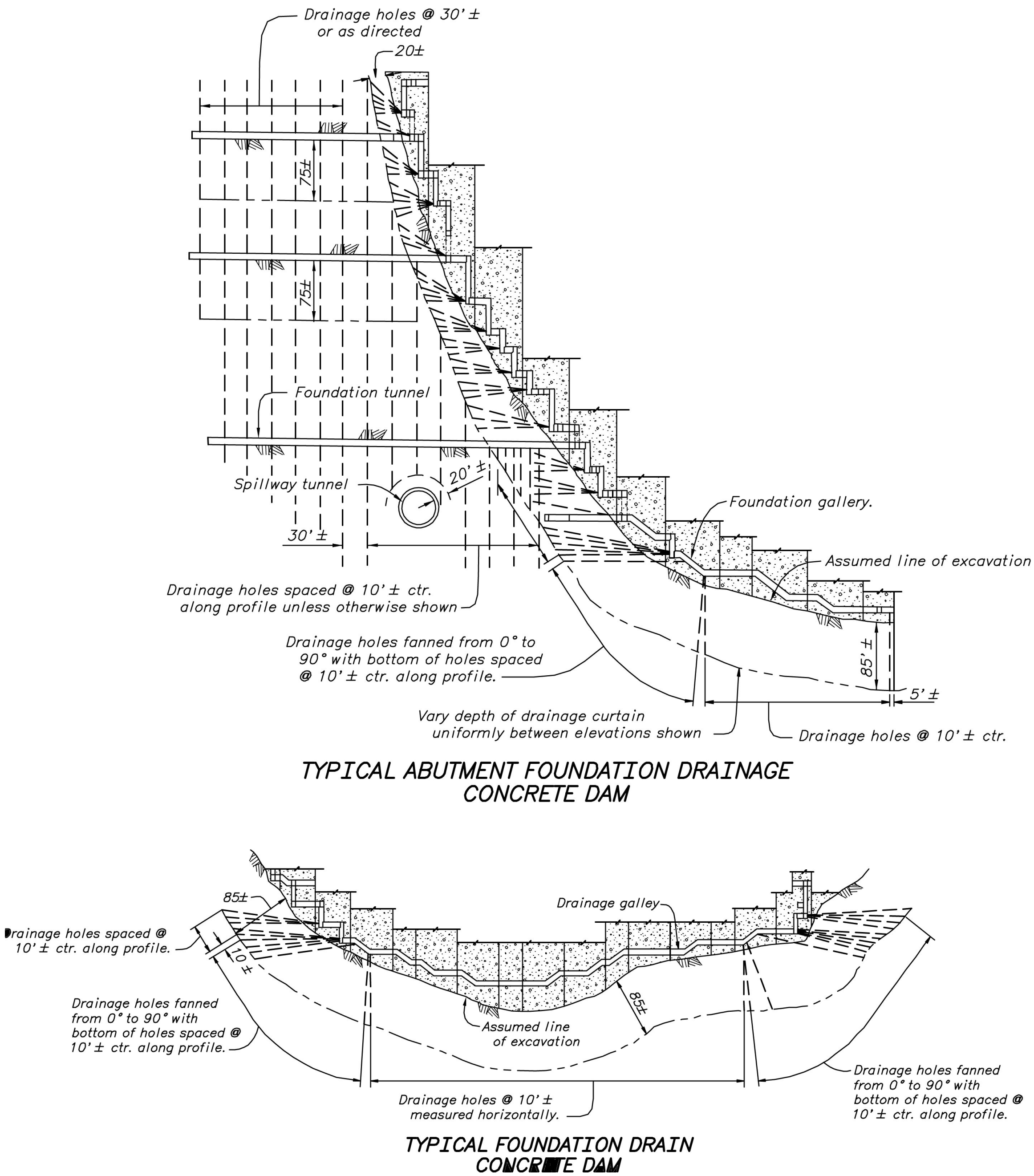


Figure 6.—Typical foundation drain layout.

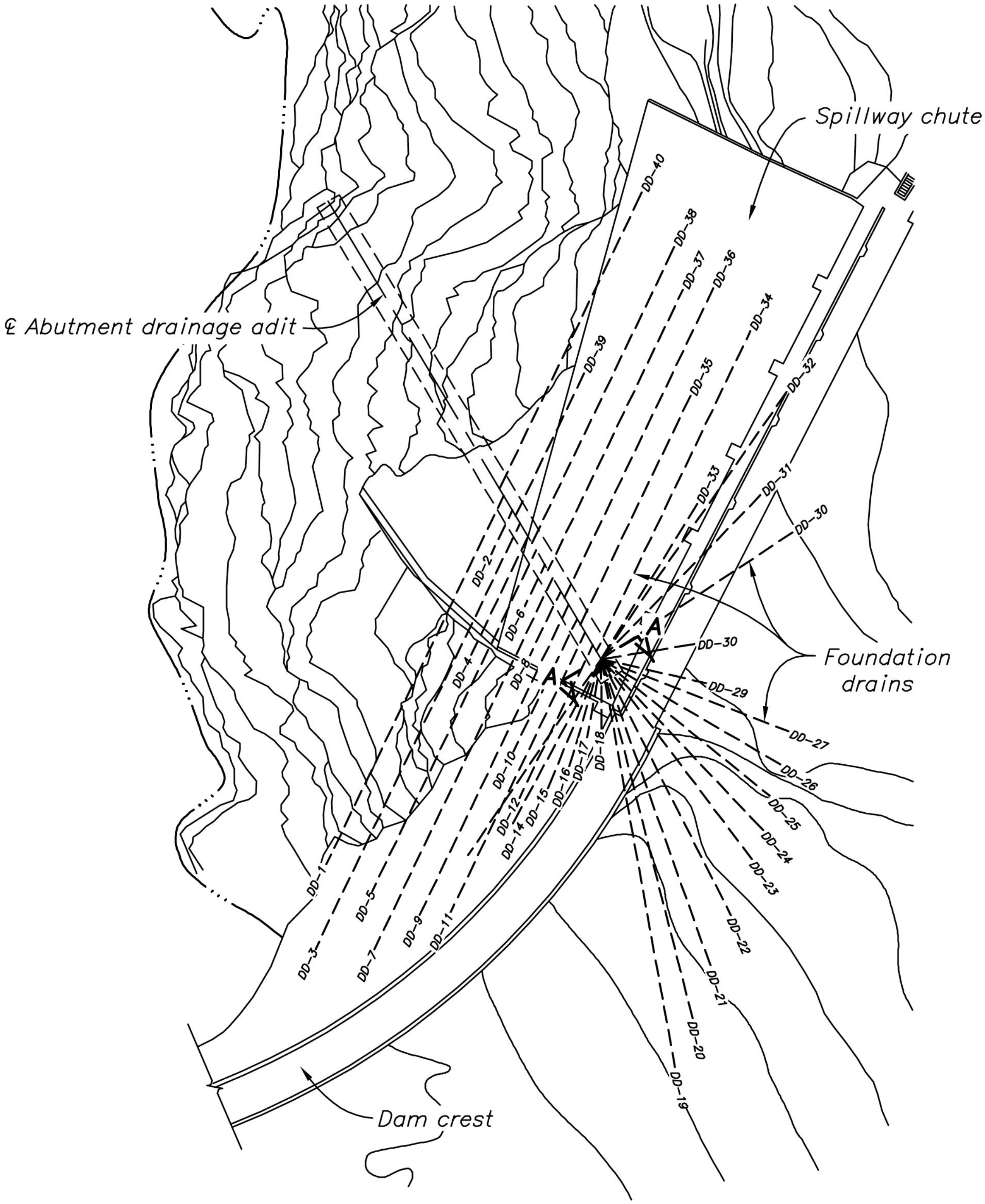


Figure 7.—Abutment drainage adit and foundation drains.

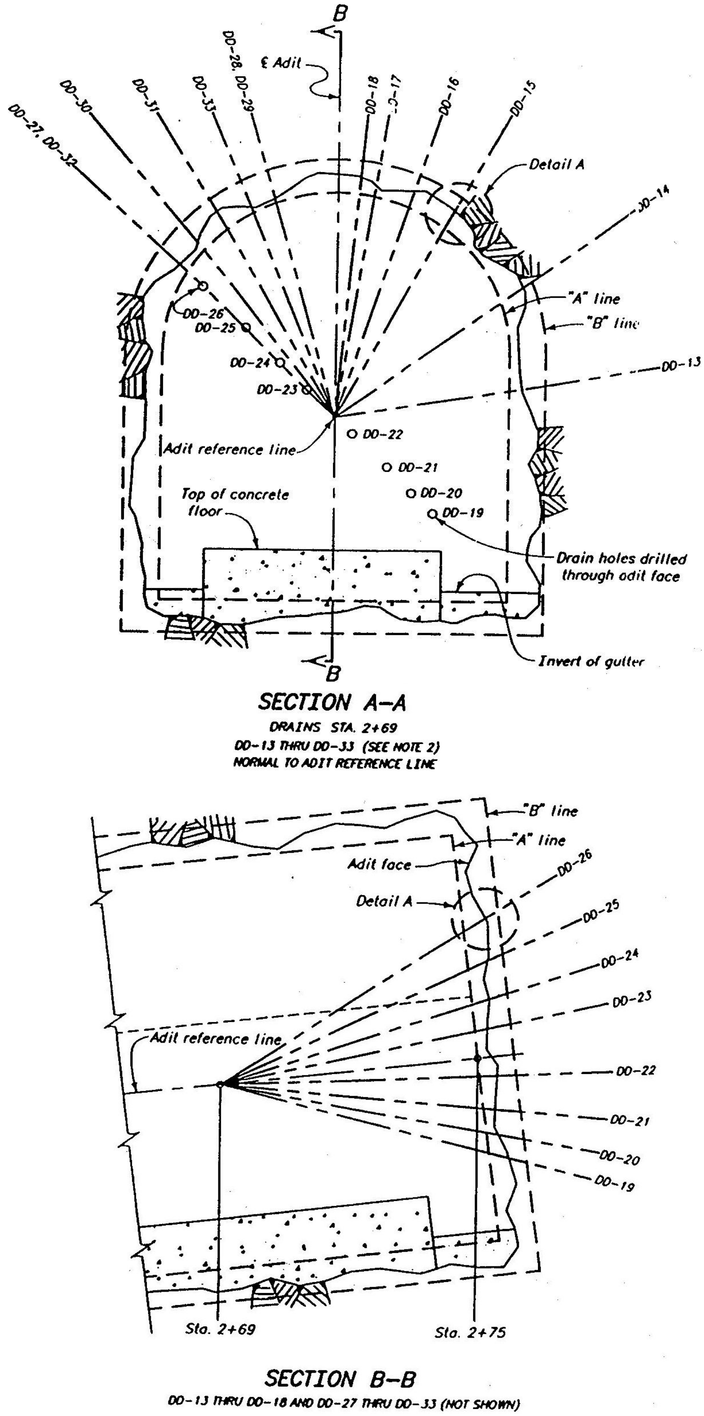


Figure 8.—Details of abutment drainage adit and foundation drains.

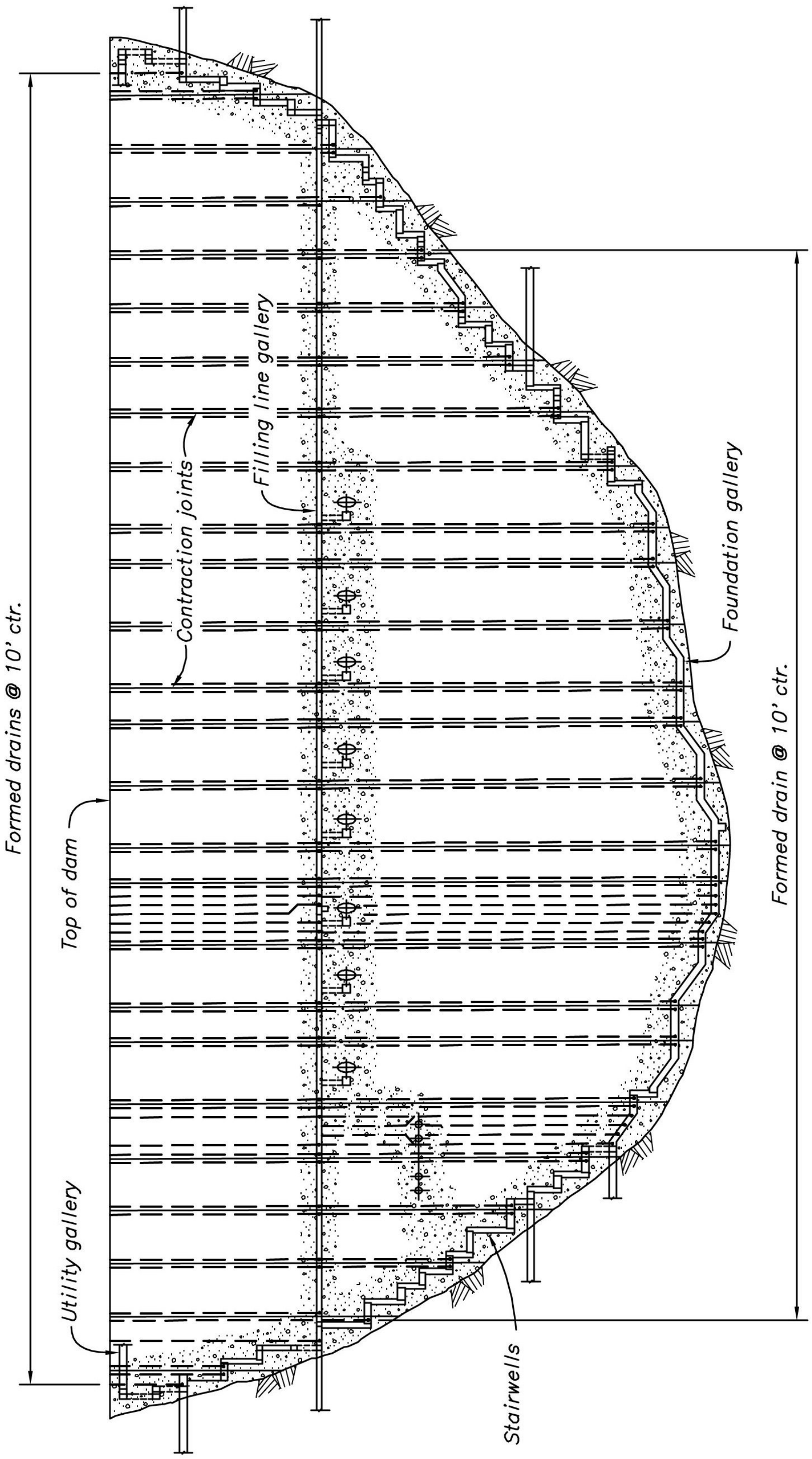


Figure 9.—Formed drain layout.

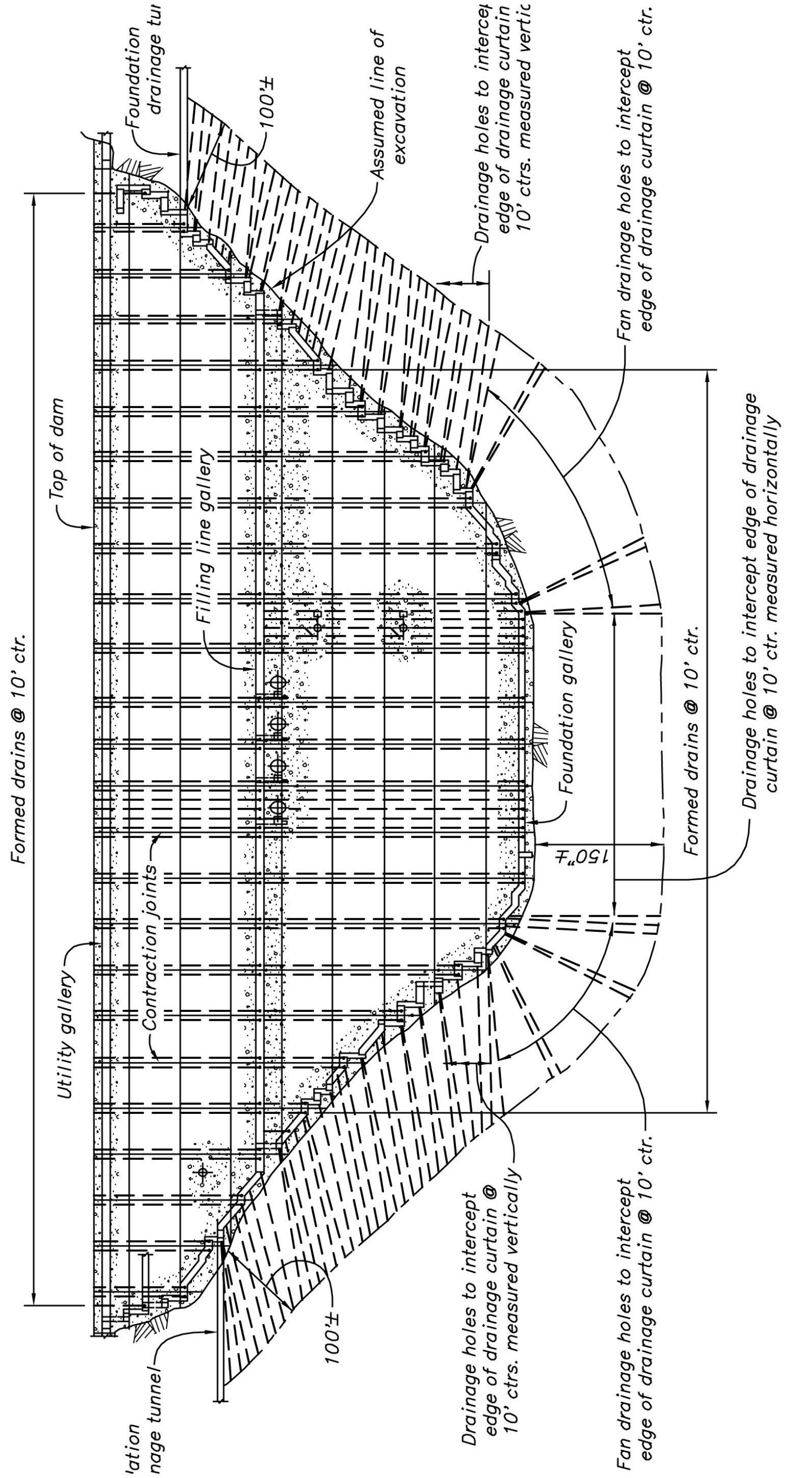


Figure 10.—Formed and foundation drain layout.

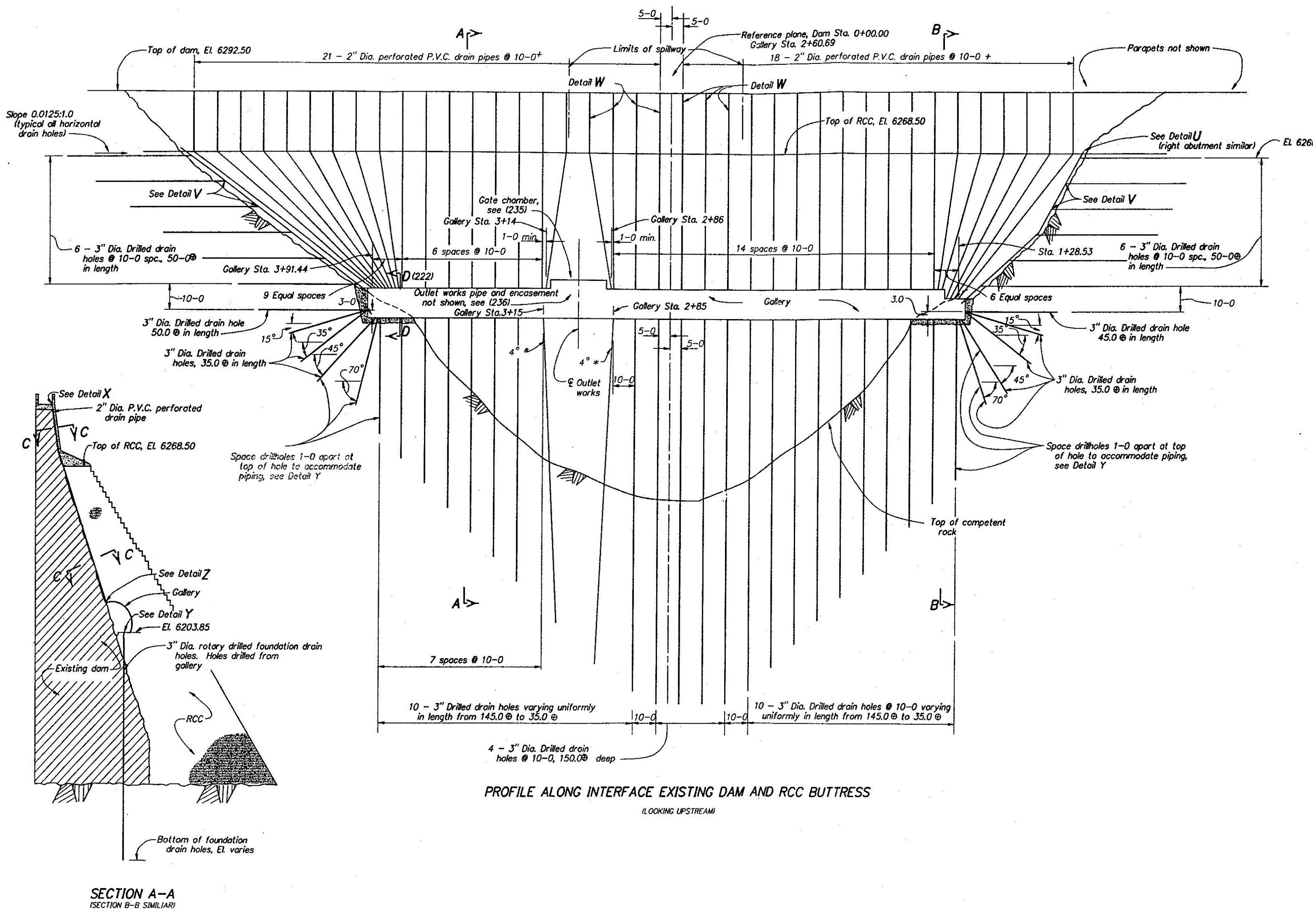


Figure 11.—Formed and foundation drain layout for RCC buttress.

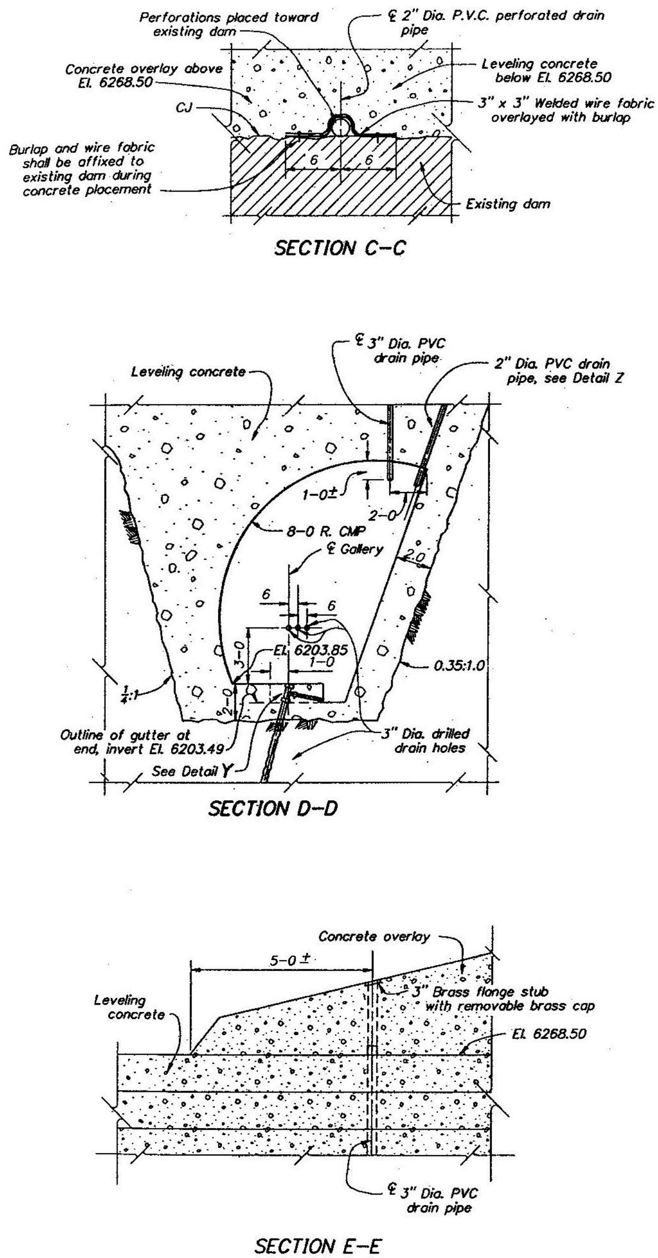


Figure 12.—Formed drain details for RCC buttress.

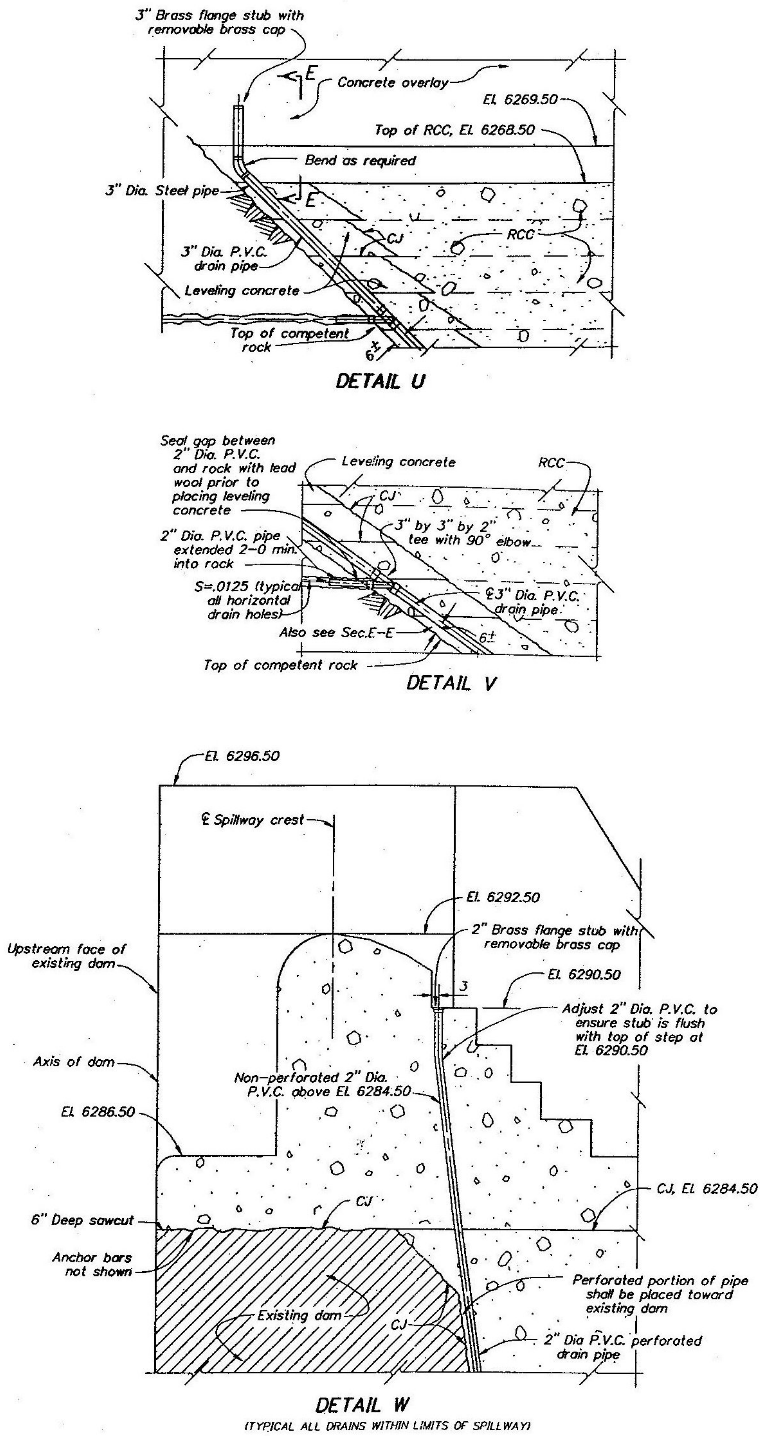


Figure 12 (cont'd)

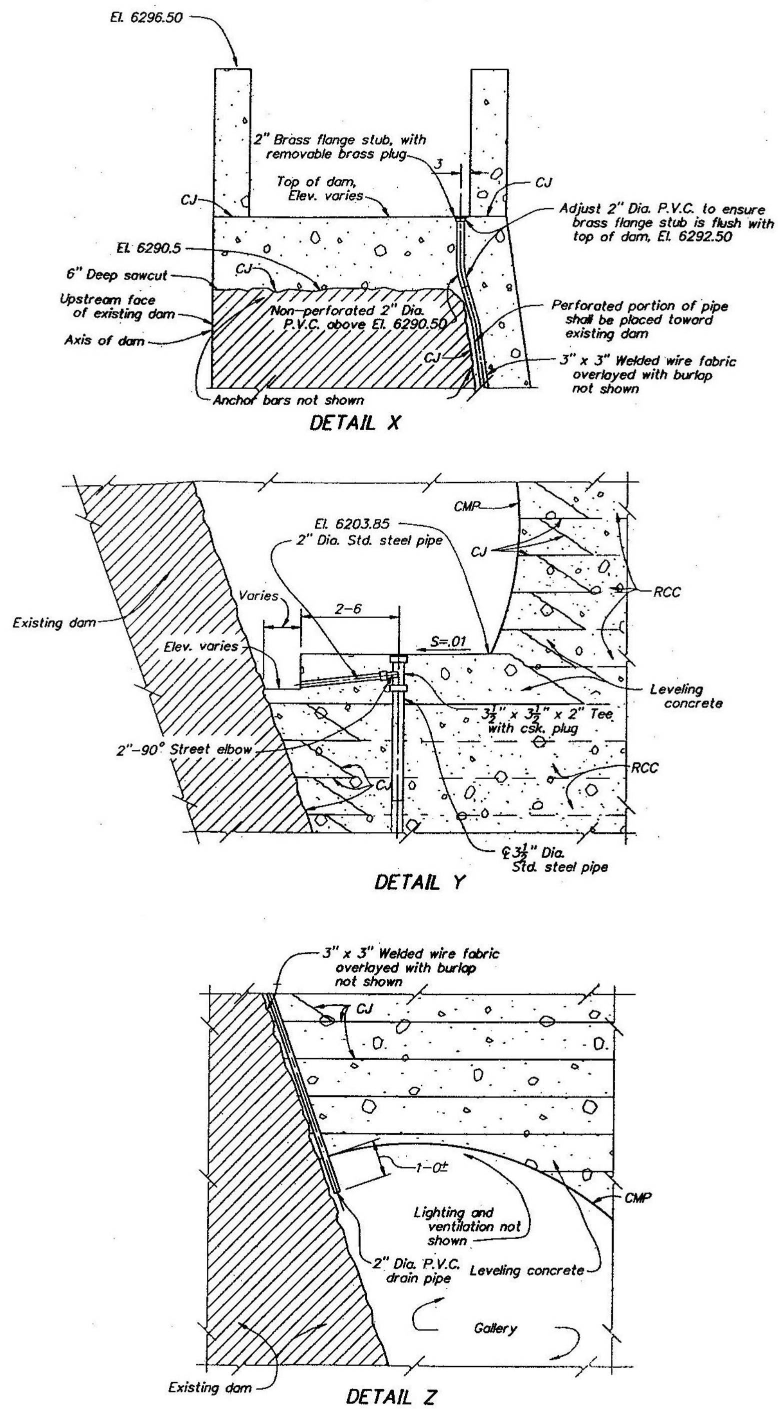


Figure 12 (cont'd)

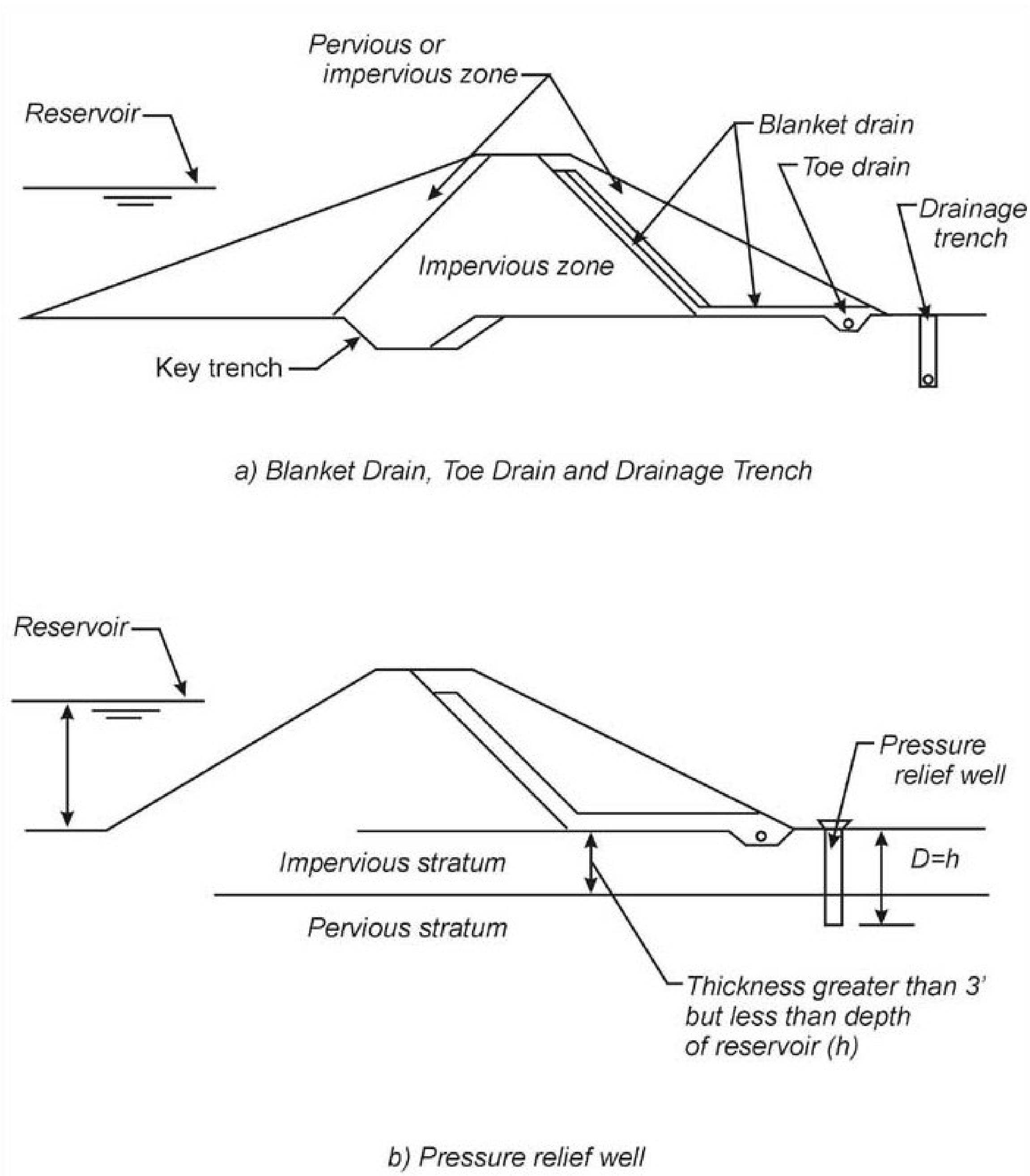


Figure 13.—Embankment dam drains.

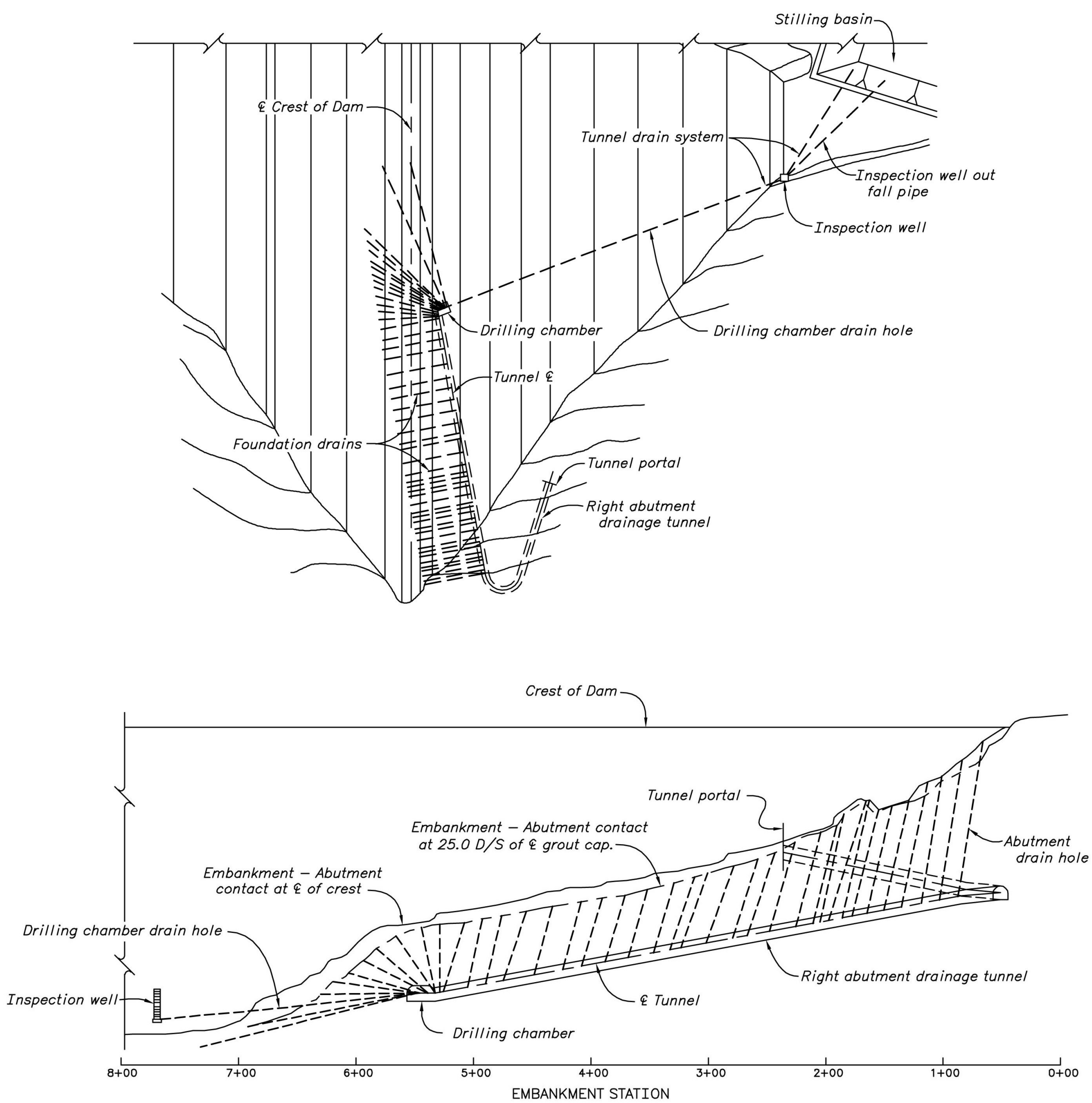
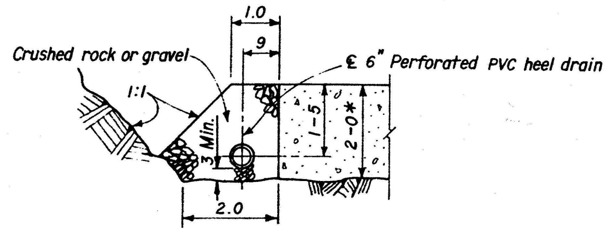
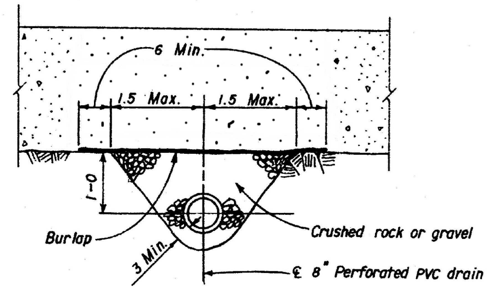


Figure 14.—Plan and profile of the drainage tunnels for the right abutment.

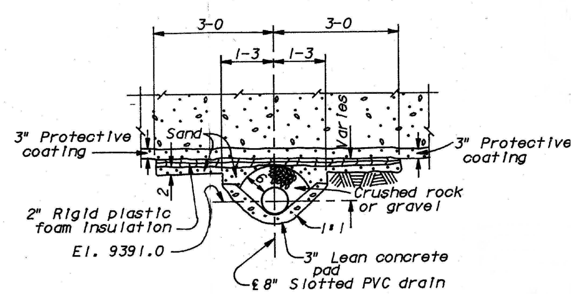
a. Stewart Mountain Dam auxiliary spillway, Arizona.—Typical drainage feature (longitudinal) which extends along the heel of a retaining wall (limited filter and no insulation requirements). Depending on fill material adjacent to drain, consideration of additional filtering (such as sand, or geotextile) may be needed.



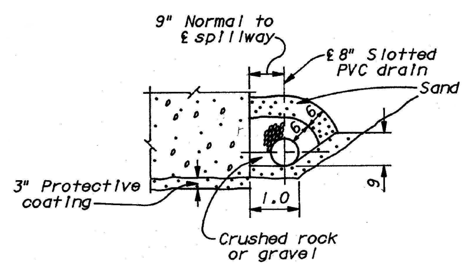
b. Stewart Mountain Dam auxiliary spillway, Arizona.—Typical drainage feature (lateral and longitudinal underdrains) for slabs on firm formation (limited filter and no insulation requirements).



c. Costilla Dam spillway, New Mexico.—Typical drainage feature (lateral and longitudinal underdrain) for slabs on soil (filter and insulation requirements).



d. Costilla Dam spillway, New Mexico.—Typical drainage feature (longitudinal), which extends along the heel of a retaining wall (filter and insulation requirements).



e. Costilla Dam spillway, New Mexico.—Typical drainage feature (lateral) adjacent to cutoffs for slabs on soil (filter and insulation requirements).

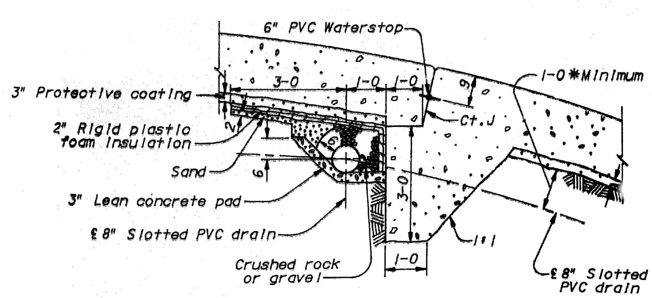
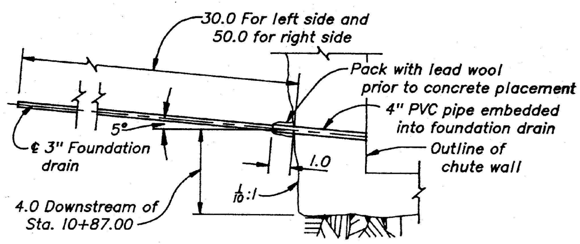
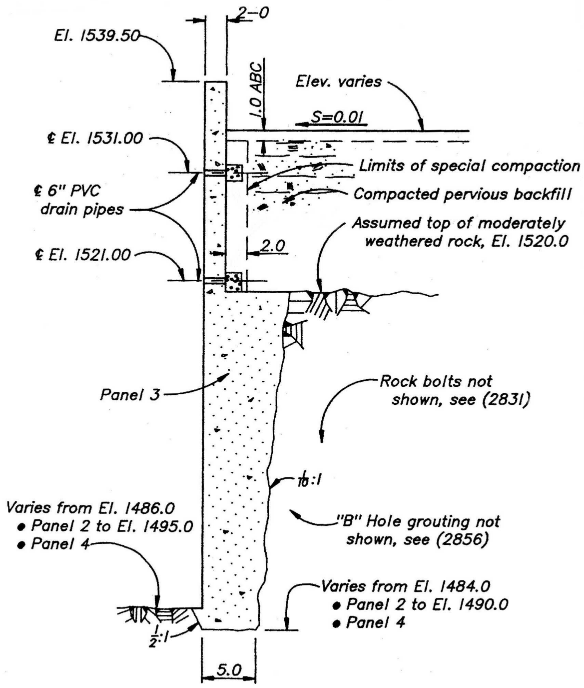
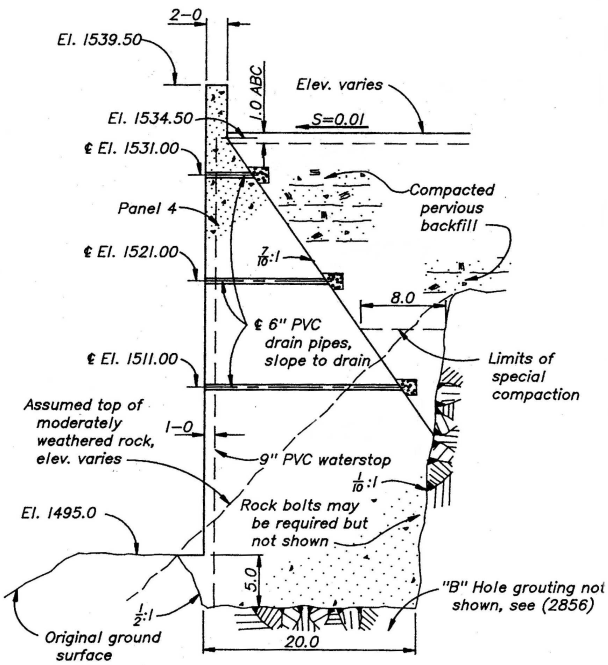


Figure 15.—Appurtenant structures—Perforated and slotted PVC drains with sand, gravel and/or geotextile envelope.



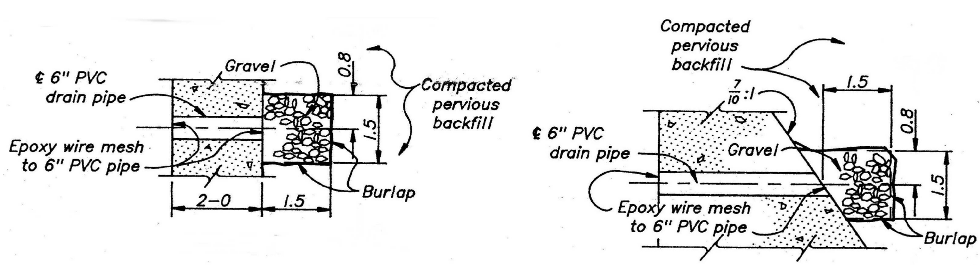
a. Stewart Mountain Dam auxiliary spillway, Arizona.—Drilled and formed “weep hole” (no filter and insulation requirements) to address hydrostatic pressure in rock adjacent to wall. Note slightly inclined hole to facilitate drainage. Also, orientation of drain holes should be based on geologic features (type of rock, thickness of layers, joint orientation, etc.). Finally, as a rule of thumb, this type of drain should extend at least to the same depth as anchor bars and/or rockbolts.





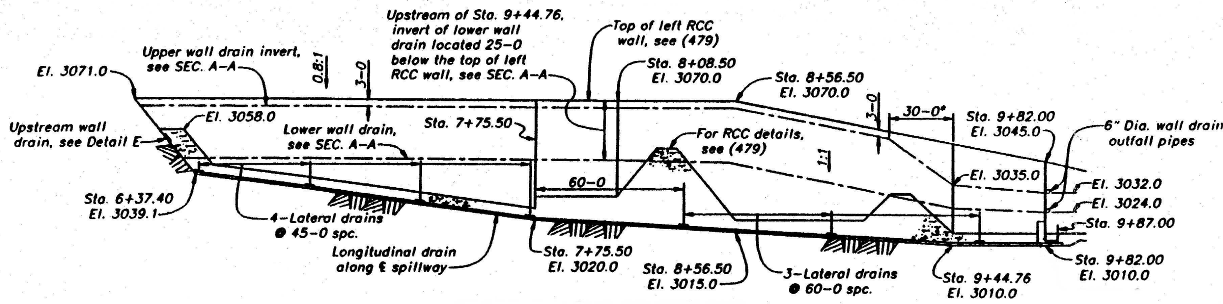
b. Stewart Mountain Dam auxiliary spillway, Arizona.—Formed “weep holes” with gravel filter (limited filter and no insulation requirements) to address hydrostatic pressure in backfill adjacent to a cantilever wall.

c. Stewart Mountain Dam auxiliary spillway, Arizona.—Formed “weep holes” with gravel filter (limited filter and insulation requirements) to address hydrostatic pressure in backfill adjacent to a gravity wall.

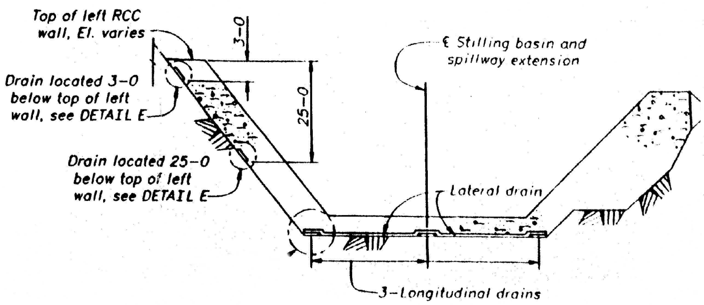


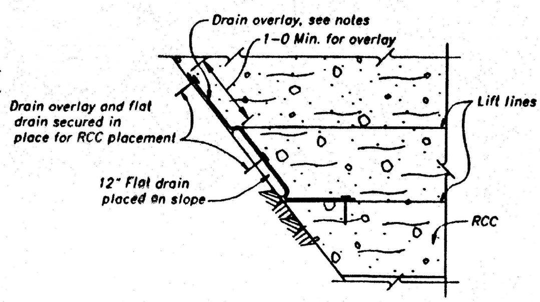
d. Stewart Mountain Dam Auxiliary spillway, Arizona.—Gravel detail used with weep holes shown in the cantilever (b) and gravity walls (c).

Figure 16.—Appurtenant structures—Drilled and formed drains (sometimes referred to as “weep holes”).



a. Ochoco Dam spillway stilling basin, Oregon.—RCC extension (drainage provisions employ flat drains), which is founded on firm formation.

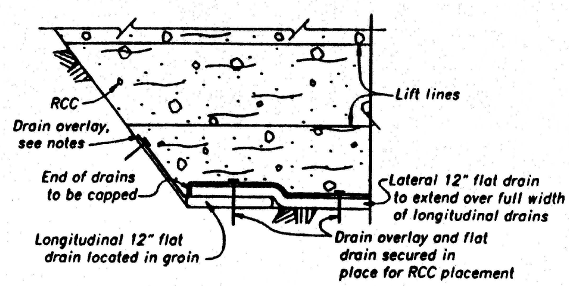




b. Typical layout of flat drains (longitudinal) adjacent to the walls and beneath floor of the RCC extension.

c. Typical drainage feature (longitudinal) that illustrates flat drain details adjacent to the walls of the RCC extension (limited filter requirements; insulation requirements met by RCC thickness).

d. Typical drainage feature (lateral and longitudinal) that illustrates details of flat drains beneath the floor of the RCC extension (limited filter requirements; insulation requirements met by RCC thickness).



e. Typical drainage feature (longitudinal) that illustrates exit details of flat drains beneath the floor of the RCC extension (limited filter requirements; insulation requirements met by RCC thickness).

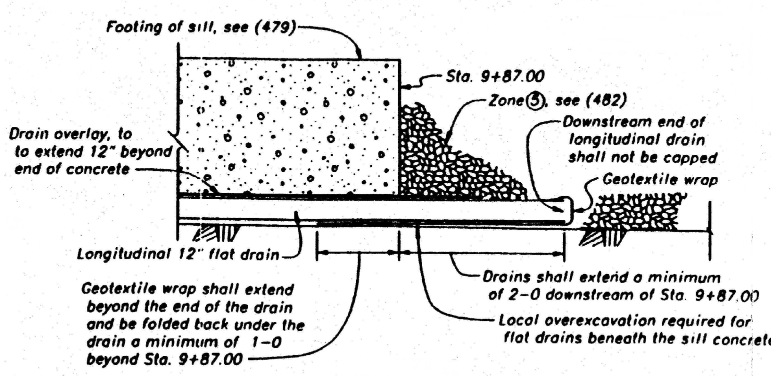
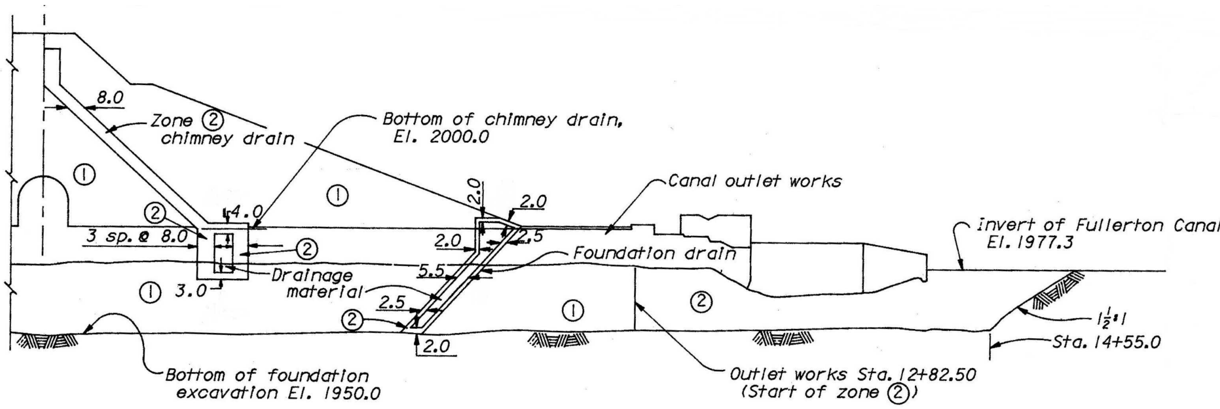
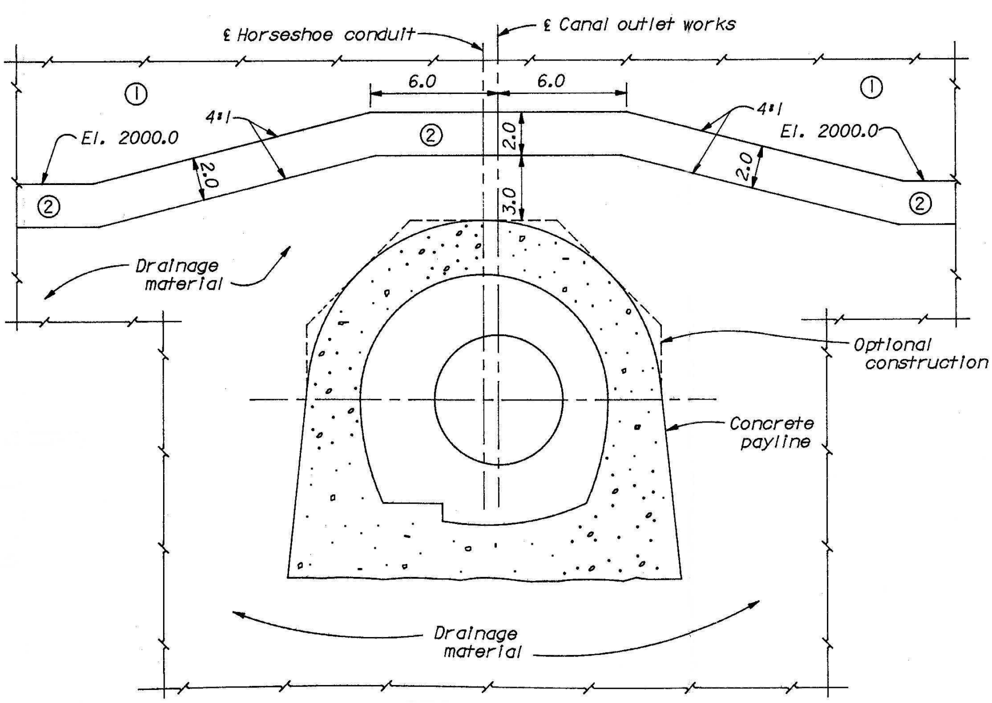


Figure 17.—Appurtenant structures—Prefabricated drains (such as "flat drains").

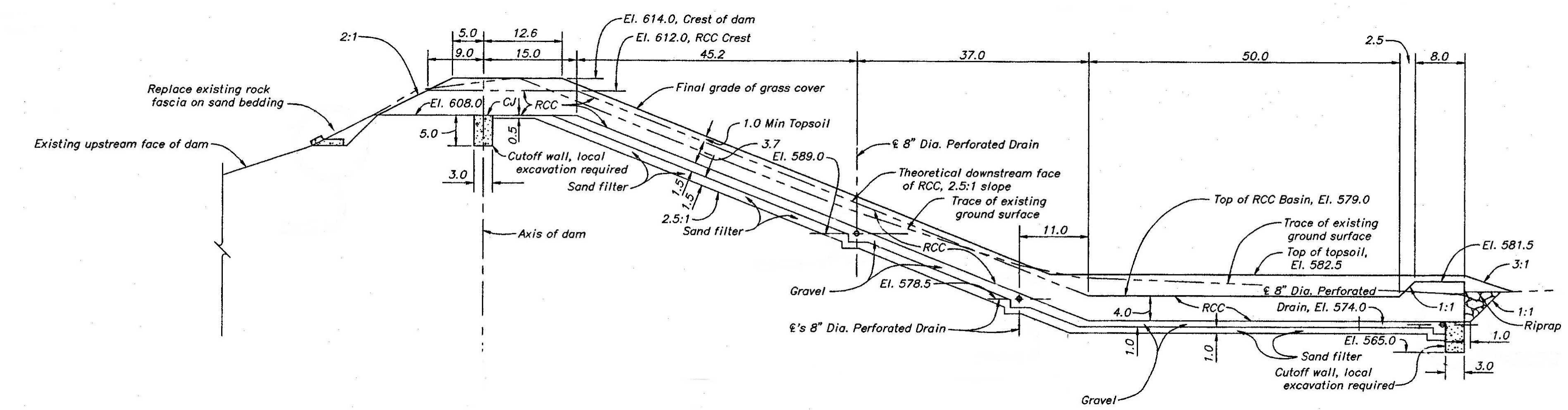


a. Outlet works.—Internal drainage for embankment dam ties into (provides foundation and wraps around) canal outlet works, which is founded on soil.

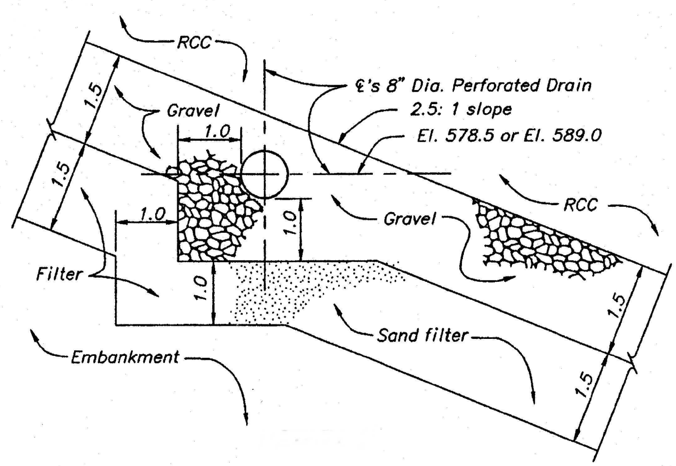


b. Outlet works.—Typical drainage feature (longitudinal) that wraps embankment filter material around/beneath outlet works conduit, which is founded on soil (filter requirements).

Figure 18.—Appurtenant structures—Graded filter drains—Drainage provisions for conduit/pipe that passes through or beneath an embankment dam.



a. Vesuvius Dam overtopping protection, Ohio—Filter blanket beneath RCC provides drainage.



» b. Typical drainage feature beneath the RCC on the downstream dam slope, which ties filter blanket (longitudinal) with perforated collector drains (filter requirements, insulation requirements met by RCC and topsoil thickness).

c. Typical drainage feature beneath and at the downstream end of the RCC, which ties filter blanket (longitudinal) with perforated collector drain (filter requirements, insulation requirements met by RCC and topsoil thickness). o

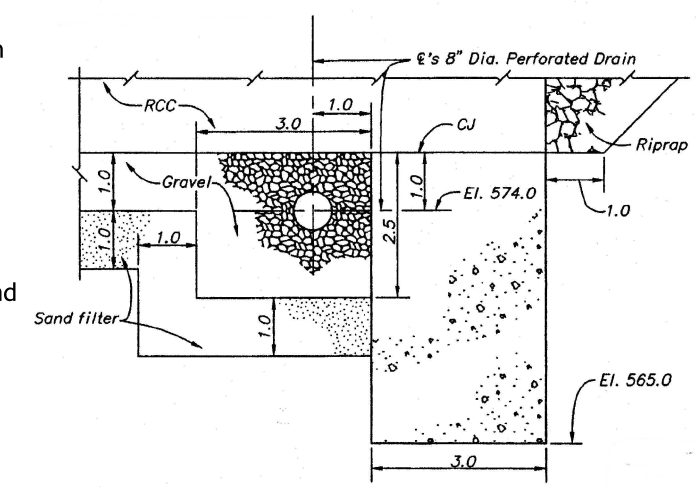


Figure 19.—Appurtenant structures—Graded filter drains—Drainage provisions for embankment dam overtopping protection.

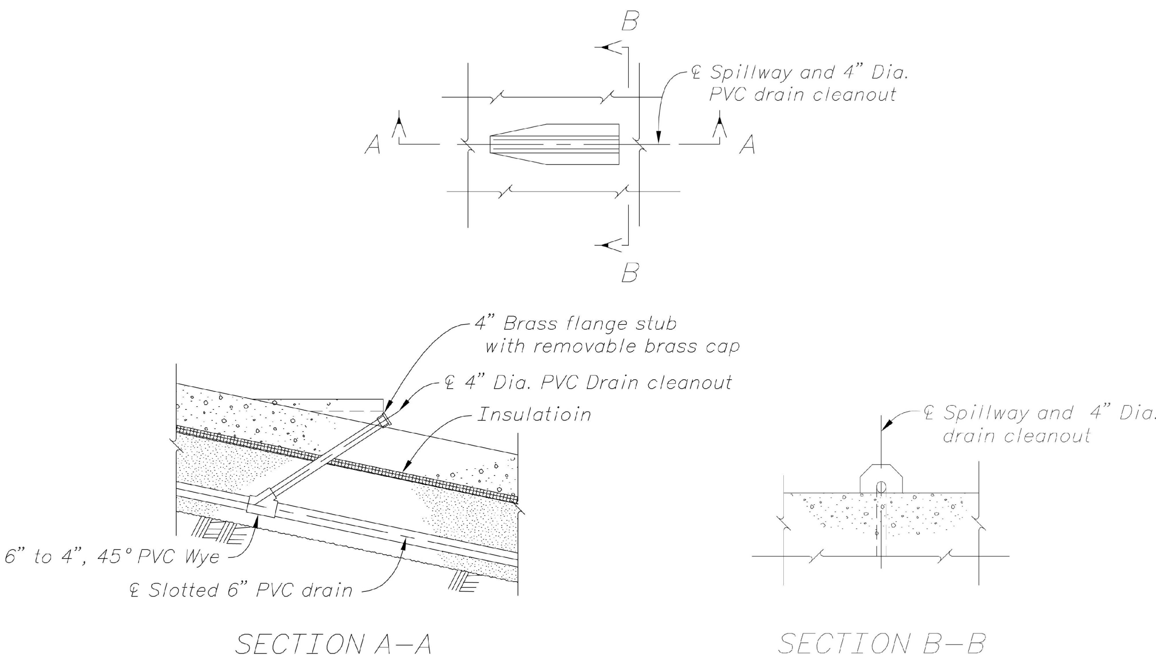


Figure 20.—Plan view and sections of a drain cleanout within a spillway chute.

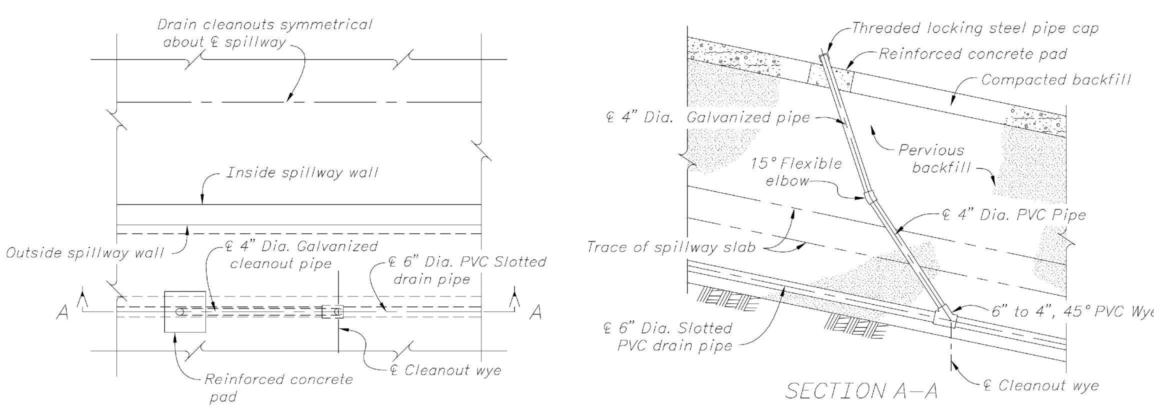


Figure 21.—Plan view and section of a drain cleanout for spillway chute, outside drains.

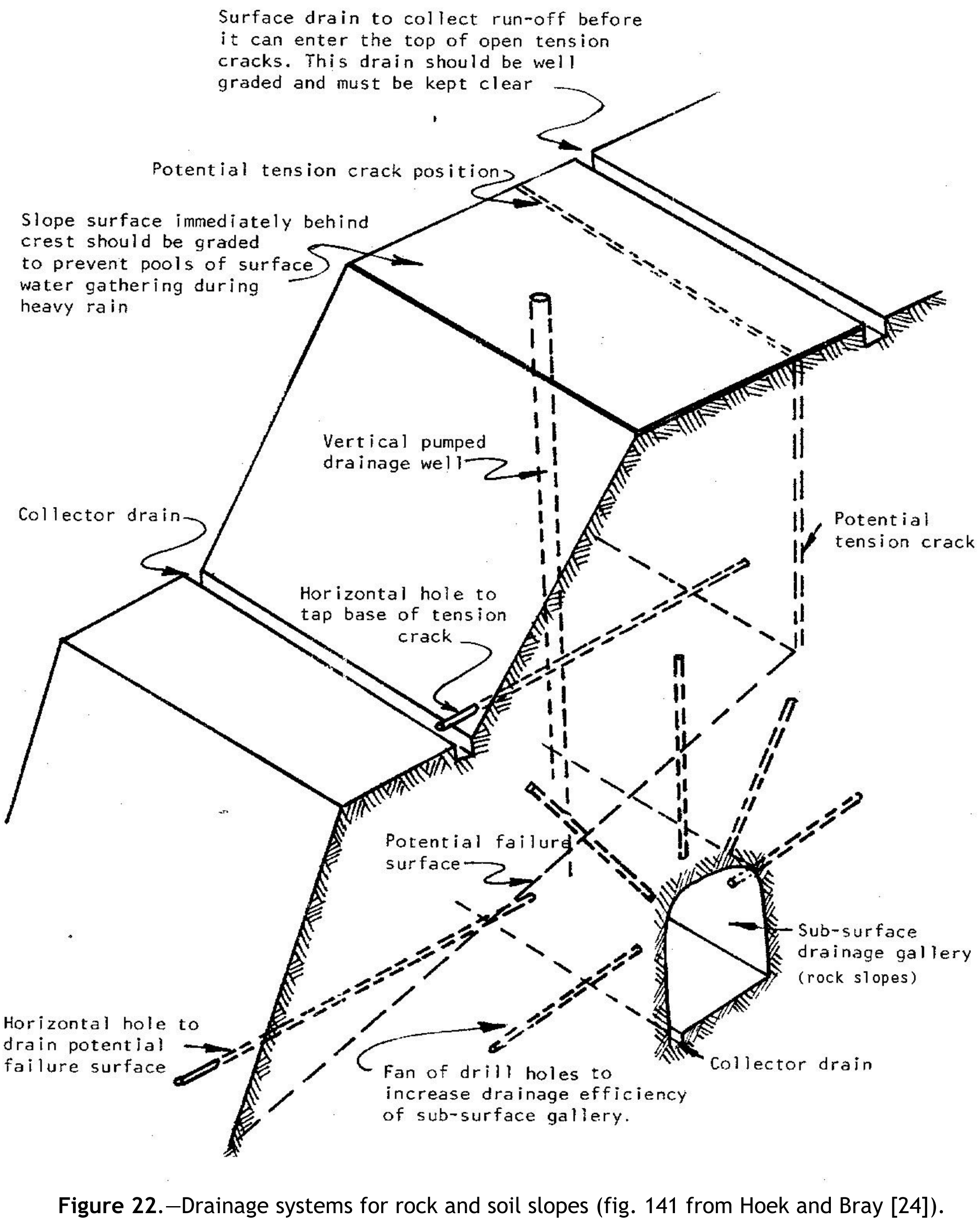


Figure 22.—Drainage systems for rock and soil slopes (fig. 141 from Hoek and Bray [24]).

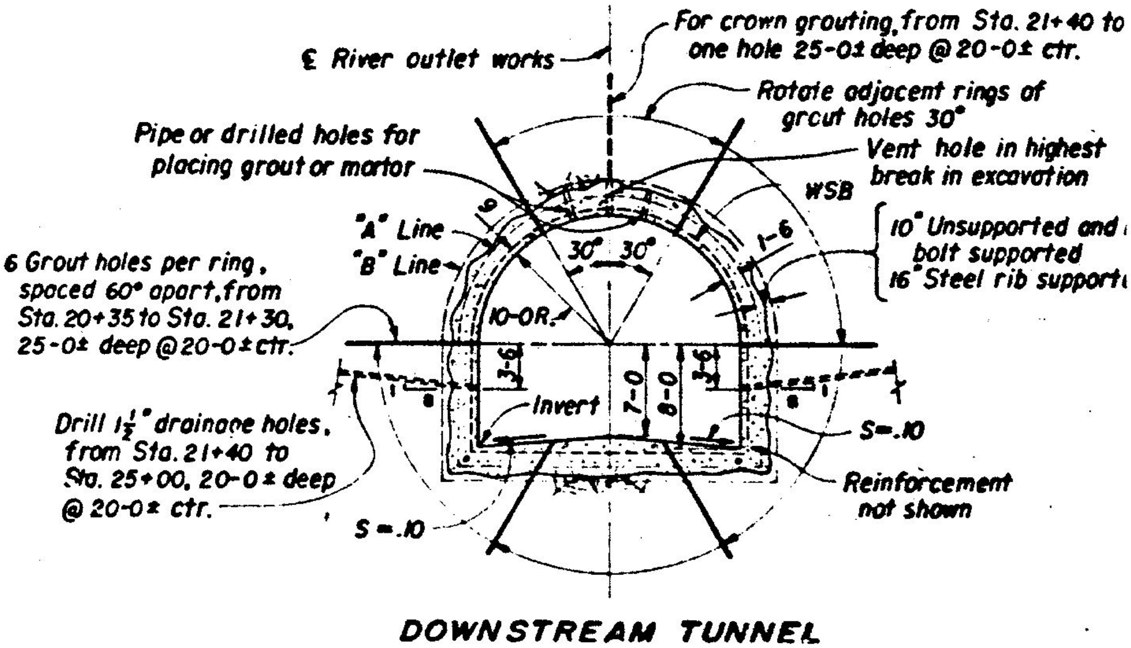
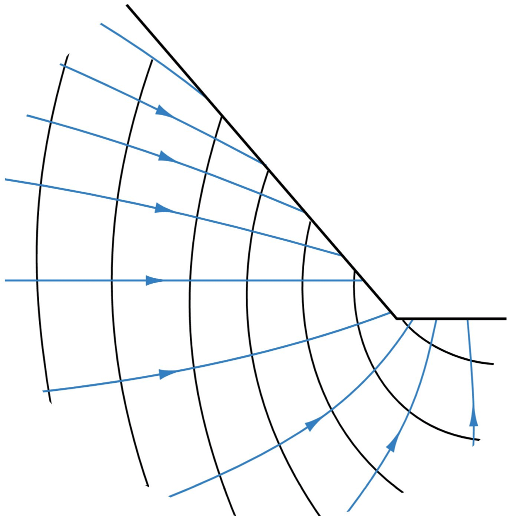


Figure 23.—Example of tunnel drains—river outlet works tunnel.



Chapter 2

DRAIN DESIGN AND ANALYSIS

2-1. Design Philosophy.—The design philosophy of a drainage system may vary, depending on the type of drainage system, the structure/foundation for which the drains are provided, or the loading condition for which the structure/foundation is being analyzed/designed. In some cases, the drainage system is a critical portion of the design, with functioning drains being critical to the assurance of a stable structure. In other cases, the drains are provided to achieve a desired factor of safety (the structure may be stable without the drains but at a reduced factor of safety). And finally, a structure may be designed to meet required factors of safety without drains, but the drains are provided as an additional line of defense or an increased margin of safety against instability (particularly under unusual loading conditions).

Procedures for designing the various types of drainage systems are provided in this chapter. In some cases, the spacing and sizing of drains is based on "rules of thumb," which provide a starting point for designing a system based on what has worked effectively for similar drainage systems. In other cases, a detailed seepage analysis may be warranted to design a given drainage system. Analysis methods will also be presented in this chapter, and further described in appendix A, for evaluating the stability of a structure/foundation with and without a fully functioning drainage system. This type of analysis may demonstrate the need for initiating a drain cleaning program.

2-2. Site-Specific Considerations.—Material types, cementation, grain size, permeability, and fabric or structure (orientation and spacing of discontinuities) are major lithological factors which influence drain effectiveness. In addition, groundwater pH and mineral content can adversely affect slots or perforations in drain pipe; well screens and the drain itself can become plugged by precipitation of minerals from solution in the groundwater, or by bacterial/organic growth.

Most foundation drains for concrete dams are installed in rock. While some embankment toe drains and horizontal drains are installed in rock, many are installed in weathered rock or soil. Movement of groundwater within the rock is largely confined to discontinuities such as joints, fractures, bedding planes, foliation planes, and fault planes. Exceptions are where the fabric of the rock matrix is permeable (volcanic agglomerate, poorly cemented dune sand, etc.). Movement of groundwater within soil is in the interconnected pore spaces between individual soil particles. Drains must intersect discontinuities or permeable materials so that influent seepage and groundwater can be removed. Most drains are normally wet or flowing year-round (fluctuating with reservoir elevation); however some drains are often dry, yielding large amounts of water only during times of high precipitation or high reservoir levels.

The most effective foundation drain installations are those which have been designed and installed based on the geology of the site. The design of drains should incorporate the location, orientation, and spacing of discontinuities and confining layers or aquatards. Groundwater barriers such as clayey zones, or confining beds such as clay layers must first be identified and then penetrated by drains to relieve the impounded water behind or below them. Spacing of drains should ideally be based on the location of productive zones where the water occurs rather than an even spacing over a large area. Spacing of rock joints may vary widely over a short distance. This will affect the effectiveness of individual drains but may be difficult to fully consider in the layout of a drainage system. It may be necessary to monitor pressures and drain flows and install supplemental drainage after reservoir impoundment.

Foundation drain layout often makes use of uniform lengths and spacings. Horizontal drain length is governed by the water-bearing strata intercepted rather than a predetermined length. In most cases, the initial drains are drilled longer than is considered adequate. Volumes and points where the water is intercepted are recorded during drilling, and subsequent drain lengths are determined from the drill data.

Slot or perforation size and well screen opening is particularly important to consider in the design of some projects. For example, 2-inch diameter pipe with d-inch perforations used in horizontal drain installations at one site may not work at another site. The drain hole casings may fill with sand, causing the drains to lose their effectiveness almost immediately. Generally, 0.020-inch slotted PVC pipe is used in the majority of current horizontal drain installations. Slotted PVC pipe sections allow roots easy access near the ground surface, and a protective sleeve of galvanized pipe should be installed over the 10 or 20 feet of drain near the ground surface to discourage root penetration. When poorly cemented, granular material is encountered, a soil gradation analysis is performed to determine the proper slot size and backfill materials.

2-3. Concrete Dam Drains.—It is possible through rigorous analysis procedures to develop a design layout for foundation and formed drains in a concrete dam based on known or assumed seepage flow characteristics. Such a design would include hole diameter, depth, spacing, and location. An approach to the design and analysis of foundation drains has been presented in a paper by Casagrande [4], which expands upon the work of Brahtz and Muskat [5, 6]. His analysis of foundation drain efficiency, which is purely theoretical, indicates that spacings of between 5 and 10 feet for a hole diameter as small as 3 inches would be expected to provide full reduction of uplift pressure (reducing the uplift pressure downstream of the drains to the uplift created by the tailwater elevation) if the water surface in the holes remains below the tailwater elevation. The analyses suggest that a typical foundation drain layout used by Reclamation for concrete dams should be adequate to reduce uplift to a level below that assumed in a stability analysis. While this procedure was applied to drilled foundation drains, a similar approach would be valid for other drain types, such as formed drains within structures and prefabricated drains.

The above approach assumes that the drainage curtain extends to a depth sufficient to intercept the seepage that could affect the stability of the structure. Studies based on flow patterns and empirical data from existing gravity dams suggest that drain depths of 40 percent of the dam height provide the necessary reduction in uplift pressure at the dam/foundation contact.

A more common approach to drain design is to base the layout on the historical precedence of similar dams and foundations for which actual empirical data are available. Readings from uplift pressure gauges, as shown in figure 24 for a gravity dam, are an indication of the effectiveness of a foundation drain layout. Uplift gauges represent discrete points at the base of the dam only. Evaluating drain flows and possibly foundation pressures at deeper depths in addition to uplift pressures at the base of the dam will provide a more comprehensive picture of drain effectiveness. This approach is valid, because flow characteristics prior to reservoir filling cannot always be determined, and design stability analyses are based on a theoretical, rather than actual, uplift profile. Modifications can be made to the foundation treatment after construction by performing additional foundation grouting or providing additional drains.

An aspect of drain design, not always addressed in the initial layout, is the question of access for the purpose of future maintenance of the drain system. The two areas of concern are access to the drain outlet (often through galleries) and access to the full length of the drain hole (both formed and drilled). Galleries should be sized and orientated so that equipment and personnel can reach drain outlets with special attention to possible obstructions such as

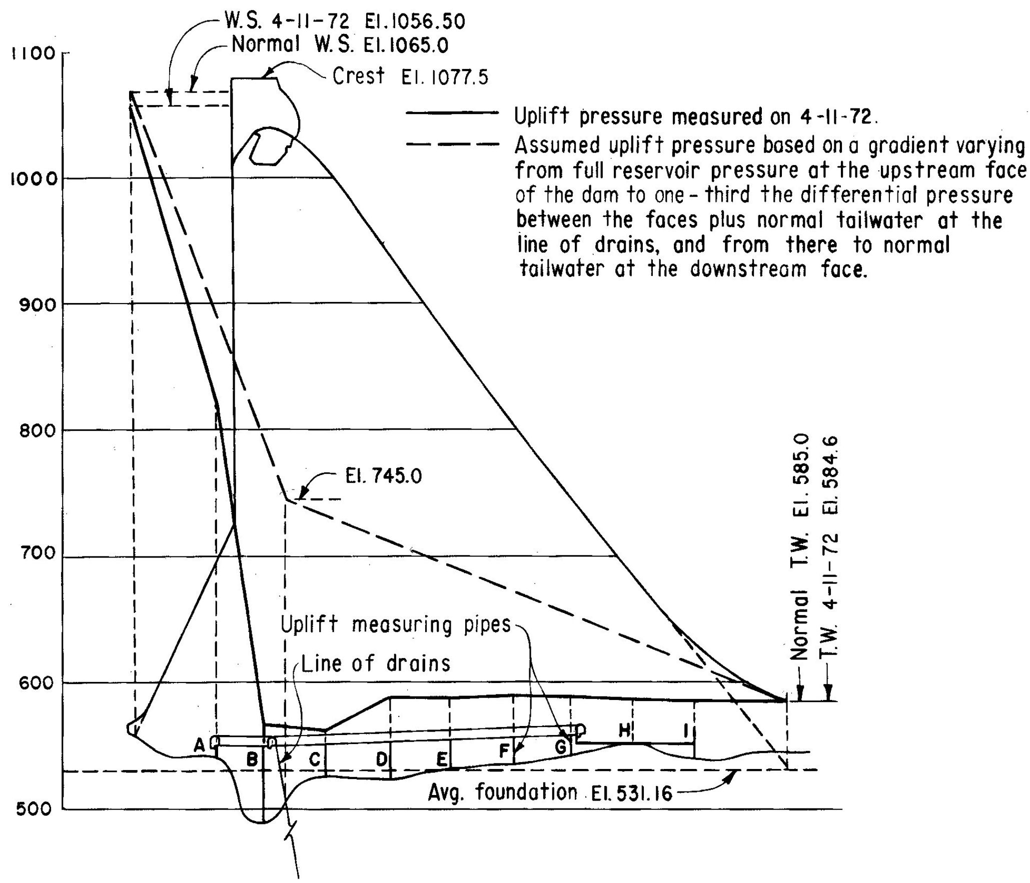


Figure 24.—Comparison of assumed and measured uplift for Shasta Dam.

stairwells. Drain holes should be made accessible at their outlet by eliminating or bypassing any bends that may be present in formed sections or in pipe attached at the drain outlet.

In considering the layout of foundation drains for concrete dams, it is important to remember that the goal is to ensure that uplift pressures do not exceed those assumed in the design and analysis of the concrete dam or foundation feature. Uplift loading is a significant factor in the foundation stability analysis of both gravity and arch dams. Within each structure, however, uplift loading and its reduction by formed drains are more critical factors in the stability of a gravity dam structure than an arch dam structure, due to the thinner arch section and the ability to transfer loads to the abutments through arch action. However, drainage may be critical to the stability of arch dam abutments, and foundation drainage should be maintained for either type of dam. The extent to which uplift loading is a significant part of the stability analysis depends upon factors such as the weight of the dam, the thickness and associated shear strength of the dam, the shear and tensile strength at the concrete lifts, the degree of load redistribution in arch dams due to arch action, and the presence of potential failure planes or wedges in the foundation. An example of a stability analysis of a gravity dam showing the effects of uplift loading with variations in cohesion and friction angle is provided in appendix A (example A7).

An understanding of effect of uplift on structures has evolved over time as a greater base of observed performance has been obtained and more research has been conducted to confirm the effectiveness of drains. Nonetheless, uplift loading assumptions used in the design and analysis of dams may differ among groups devoted to dam engineering. Reclamation guidelines generally assume full uplift at the heel of the dam, tailwater or zero uplift at the toe, and one-third of the difference of these two values at the drain line. When cracking occurs, previous Recalmation criteria assumed that full uplift extended the full length of the crack with no reduction at the drains. U.S. Army Corps of Engineers criteria differ from Reclamation criteria when cracking occurs. While full uplift is assumed over the length of the crack upstream of the drains, the drains continue to reduce uplift, although at a lower efficiency, until the crack passes the drain, at which point the drains become ineffective. Research conducted at the University of Colorado [7] indicated that drains continue to operate at full efficiency after cracking is initiated, and drain efficiency is reduced only after the crack extends beyond the drain. Reclamation currently uses this criterion. Graphical representations of the uplift criteria are shown in figure 25.

The design and analysis of foundation drains for concrete dams to a large extent are based on two factors—historical precedence and an understanding of the foundation rock conditions. Typically, dams have been designed with a single curtain of foundation drains using 3-inch diameter holes spaced at 10 feet. The depths of holes generally vary from abutment to abutment, so as to maintain a depth of about 40 percent of the hydraulic height (the height from the original streambed elevation at the dam axis to the normal water surface). Where excessive seepage flows are expected based on geologic conditions, additional drainage features should be considered. These features may include additional drains to reduce the drain hole spacing or a second drainage curtain.

In the case of existing structures where stability concerns are present which pose an unacceptable failure risk, measures to reduce uplift, especially in the abutments, may be considered. A reduction in uplift can be accomplished through the installation of a new or supplemental drainage system or the rehabilitation of an existing drainage system. In the case of a new or supplemental drainage system, the drains can be designed to optimize the potential for intercepting seepage based on an understanding of the geology, and measurements of uplift pressures can be taken before and after installation to determine the effectiveness of the system. An example of a rehabilitation project is Horse Mesa Dam [8], where a drainage adit and deep drains were installed to reduce uplift in the right abutment.

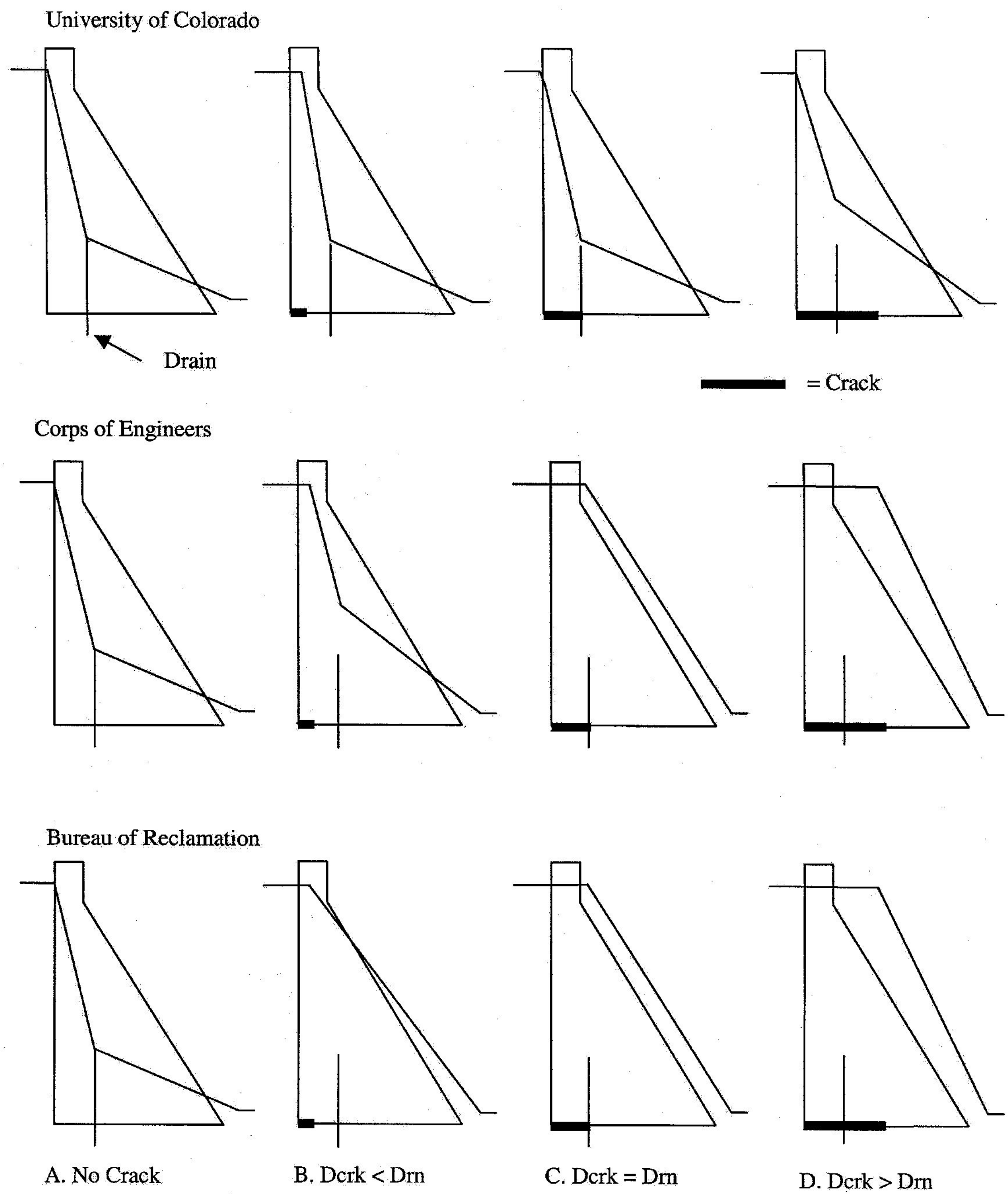


Figure 25.—Graphical representations of uplift criteria used for stability analysis.

The design, which was based on geologic conditions of the abutment and visual observation of seepage on the abutment face, is summarized in appendix A (example A8). Similarly, formed drains within a concrete structure are generally orientated vertically, spaced at 10-foot intervals, and connected to galleries within the dam. A typical diameter of 5 inches is used, and holes may be lined with pervious or slotted pipe, or unlined.

2-4. Embankment Dam Drains.—A number of different drain systems may be involved with embankment dams. The two main types of drains using pipes are toe drains and relief wells.

a. *Toe Drain Design.*—

1. *Type of Pipe.*—Toe drain pipes may be made of any material that has adequate durability and strength. The current preferred type of pipe is high-density polyethylene pipe with slots or perforations. Wood stave, vitrified clay, concrete, or asbestos-bonded corrugated metal pipes laid with open joints or perforated (laid with closed joints) have been used in the past. Open-jointed pipe laying is no longer recommended due to the potential for plugging from material flowing into the open joints and the possibility of the joints pulling apart. The outfall pipe, which carries the water collected in the perforated pipe to an appropriate discharge location, is generally the same type as the collection pipe but without perforations.

2. *Capacity.*—The capacity of the toe drain should be sufficient to handle the maximum amount of seepage expected through the embankment and the foundation (usually based on a seepage analysis), as well as precipitation drainage that might occur through the overlying materials. In order to provide a margin of safety for the drain capacity, pipes should be sized so that the depth of water in the drain pipe is less than 75 percent of the inside diameter of the drain pipe at the time of maximum expected flow. The pipe diameter can vary from smaller diameter pipes laid along the abutment sections to increasing larger pipes, with the largest pipes placed along the valley floor. The largest pipe may be used throughout the entire length for ease of construction. The minimum pipe diameter recommended for small dams is 6 inches; however, diameters up to 24 inches may be required for long reaches at flat gradients.

3. *Alignment.*—Toe drains are generally located at the downstream toe of the dam, where they can be accessed for repairs. Occasionally, they are located farther upstream to drain cutoff trenches or when intercepting springs and known seepage locations beneath the dam. The alignment should be set so that the minimum gradient of the pipe is greater than 0.01. A maximum gradient of about 0.09 (5 degrees) is recommended based on limitations of video camera and cleaning equipment (unless uphill access is provided). Pipe bends greater than 22.5° should be avoided to facilitate inspection with a remote camera and

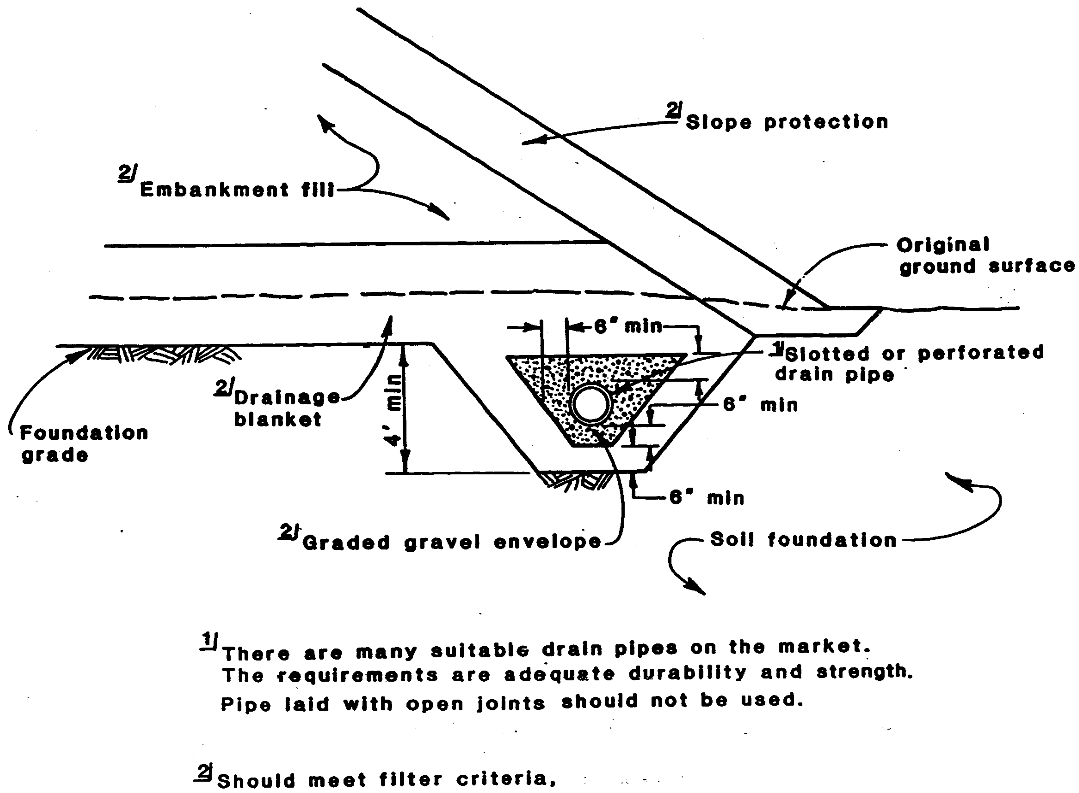


Figure 26.—Typical toe drain configuration.

cleaning. For additional recommendations on the alignment and layout of toe drains, see section C-2 in appendix C.

4. *Installation*.—The drain pipes are generally placed in trenches at a sufficient depth to ensure effective interception of the seepage flow. The minimum depth of the trench is normally 4 feet. The maximum depth is that required to maintain a reasonably uniform gradient, even if the ground surface is uneven. A gravel envelope or filter layer meeting the requirement that D_{85} size of the filter be equal to or greater than twice the size of the maximum opening in the pipe is constructed around the pipe. It is recommended, whenever practical, that the envelope consist entirely of gravels. Figure 26 shows a typical toe drain installation.

5. *Outfall*.—An outfall pipe is used to convey the water collected in the toe drain to an appropriate discharge point. The flow is usually discharged into the spillway or outlet works stilling basin, into the stream channel downstream of the dam, or into some other

drainage channel. Multiple outfalls may be required depending upon the natural ground topography. The outfall pipe is usually the same material as the toe drain but without perforations. Occasionally a corrugated metal pipe (CMP) is placed over the outlet end to prevent deteriorations of the pipe due to sunlight and weather. A screen of some type, or rodent guards, should be placed over the exit to prevent animals from entering the pipe and plugging it. The outfall pipe should daylight above flowing water to allow monitoring of flow. A sediment box, weir and/or flumes should be placed at the outfall to monitor quantity of flow and trap and monitor sediment that may be carried with the flow.

b. *Relief Well Design.*—Pressure relief wells are used to reduce and control excessive artesian pressures, thereby reducing the potential for a blowout failure or reducing uplift pressures on structures. The design and installation of a relief well system should be done by specialized staff with the appropriate knowledge and skills. This type of system requires considerable post-construction supervision, long term operation costs, and maintenance. A relief well system is generally not used if a simpler, lower maintenance system like an upstream blanket will satisfy the design requirement. The U.S. Army Corps of Engineers has done extensive research on design and installation of relief wells, and there are a number of excellent papers [9, 10] that should be referenced for additional information.

The following is a brief summary of a relief well system.

The primary components of a pressure relief well system consist of:

- *Wells.*—The wells differ little from conventional water supply wells except that the main purpose is for lowering the water table, instead of supplying water. Figure 27 shows a typical pressure relief well.

- *Collector Pipe.*—A collector pipe is connected to the wells and sometimes placed between wells to collect the flow from the wells and transport the flow to the outfall pipe.

- *Outfall Pipe.*—An outfall pipe is used to convey the water from the collector pipes to an appropriate discharge point. The flow is usually discharged into the spillway or outlet works stilling basin, into the stream channel downstream of the dam, or into some other drainage channel. Multiple outfalls may be required, depending upon the natural ground topography. A screen of some type should be placed over the exit to prevent animals from entering the pipe and plugging it. The outfall pipe should daylight above flowing water to allow monitoring of flow.

- *Measurement Devices.*—Piezometers are normally installed between the wells to monitor the drawdown of the water table. Rising piezometric levels are an indication that the well system may not be functioning properly. A sediment box, weir, and/or flumes are

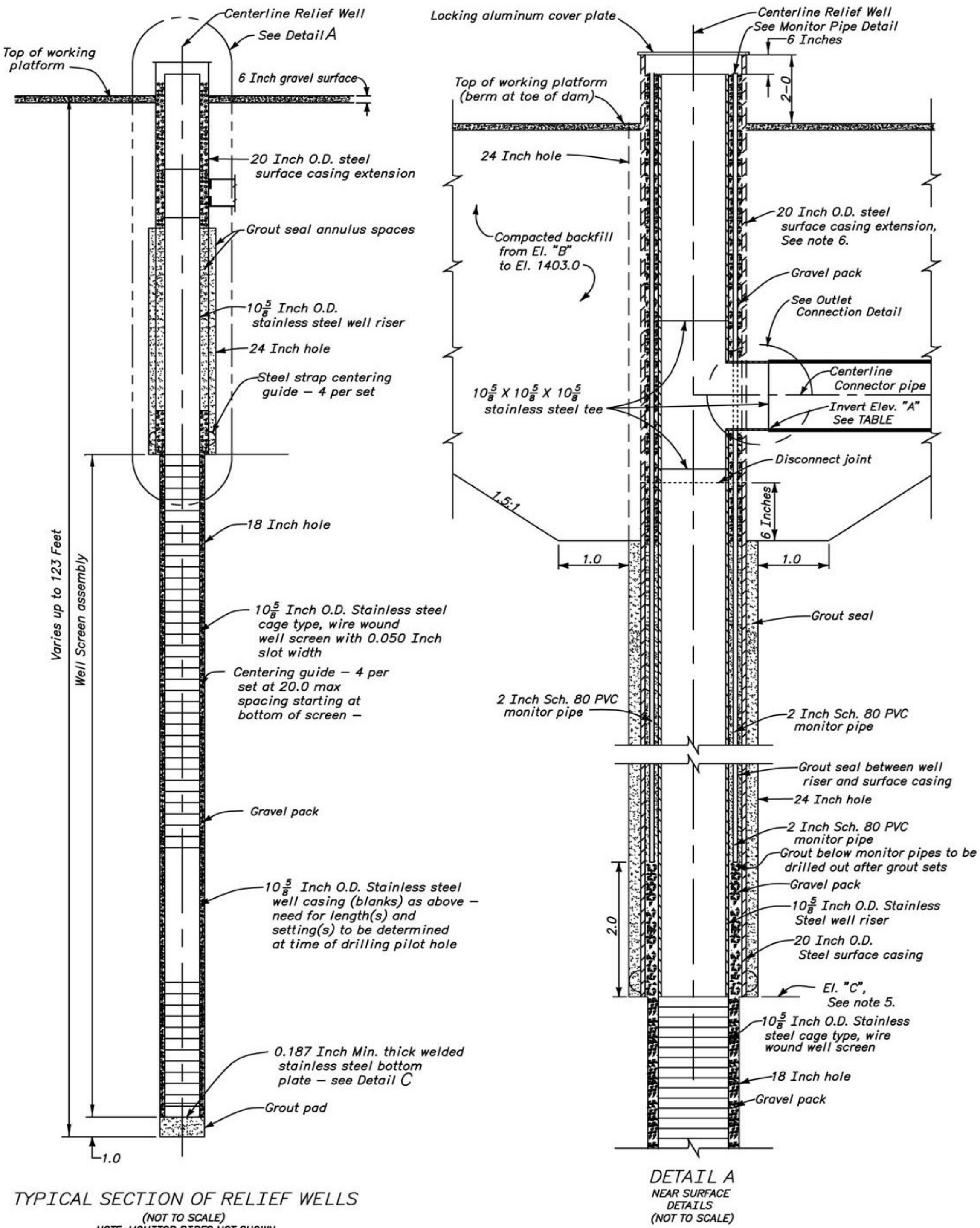


Figure 27.—Typical relief well.

placed at the outfall to monitor quantity of flow and trap and monitor sediment that may be carried with the flow. Weirs or flumes are sometimes placed at the wells or along the collector pipe to monitor flows from specific wells.

The primary requirements for a relief well system are:

- The wells should extend deeply enough into the pervious foundation underlying the confining layer to provide stability against underlying unrelieved pressures. Depths of wells up to the height of the dam are usually satisfactory.

- The wells should be spaced to intercept the seepage and reduce the uplift pressure between wells. Wells spacings of 25 to 100 feet are generally used.

- The wells should be designed to offer little resistance to the infiltration and discharge of seepage. In general, the well diameter should not be less than 6 inches with a minimum 6-inch thick filter between the well screen and the foundation. In the past, the filter thickness used for some relief wells has been less than 6 inches, but the 6-inch minimum is now strongly recommended.

- The wells should be designed so that they will not become ineffective as a result of clogging or corrosion. This is controlled by the type of well screen used and the gravel pack surrounding the well screen. Some maintenance of the wells may be unavoidable, however, due to the presence of iron bacteria or other plugging mechanism.

- The capacity of the collector and outfall pipes should be sufficient to handle the maximum expected flow from the wells (normally based on a seepage analysis). In order to provide a margin of safety for the system capacity, pipes should be sized so that the depth of water in the pipes is less than 75 percent of the inside diameter of the pipe at the time of maximum expected flow.

c. *Analysis Techniques*.—Various methods of analysis of the effect of seepage on embankments are available. The methods used depend upon available information and the particular situation that is of concern. The following types of methods, in increasing complexity, are generally used:

1. *Equations, Figures, and Charts*.—Darcy's law is the basic premise, upon which almost all seepage and water flow analyses are based: $q = k\ i\ A$, where q is flow rate, k is the hydraulic conductivity, i is the hydraulic gradient, and A is the cross-sectional area. Numerous other equations, and simplifying charts and figures can also be used. These types of analysis are generally used for preliminary studies and layouts.

2. *Graphical Methods, and Flow Nets*.—Semigraphical and graphical methods such as flow nets (fig. 28) are used to predict the location of the steady-state phreatic surface for use in stability analysis. The flow net also provides information on hydraulic potentials, flow direction, pore pressure, and quantity. These analysis types are two dimensional and limited to the more homogenous and simple conditions. A good source of additional information on flow nets is *Seepage, Drainage, and Flow Nets*, by H.R. Cedegren [11].

3. *Numerical Methods*.—Numerical methods using finite elements and boundary element techniques are used to evaluate more complex problems. The analysis can be performed based on two or three dimensions. Numerous computer programs (SEEP2D, SEEP3D, UNSAT, BIE2DCP, SEEPW) are available to perform these analyses. These types of analyses are best left to be performed by people with the appropriate technical knowledge.

4. *Other Methods*.—Numerous other methods have been used, such as the method of fragments and electrical analogy methods. Most of these methods are no longer used due to the introduction of high speed computers and the ease of running numerical methods.

2-5. Appurtenant Structures.—This section will focus on design considerations and methodology for drainage provisions associated with appurtenant structures, which include spillways, embankment dam overtopping protection, outlet works, channel protection, retaining walls, drainage adits and access adits. Design considerations and methodology will be presented in two sections: (1) general design considerations that apply to drainage provisions for most appurtenant structures, and (2) specific design considerations and methodology for specific appurtenant structures.

a. *General Design Considerations and Methodology*.—A number of "tried and true" guidelines apply to most appurtenant structures, whether they are founded on firm formation (rock) or soil. These include:

1. *Structural Foundation*.—Minimize disturbance of the structural foundation by employing drainage provisions that would limit the amount of excavation needed to install those drainage provisions. This is particularly true for structures founded on firm formations (i.e., rock). As an example, drainage provisions for the Stewart Mountain Auxiliary Spillway and Spring Creek Debris Dam Spillway chute floors (see fig. 31) incorporated an underdrain grid system of near vertical formed/drilled weep holes connected to PVC pipes in lieu of a more traditional underdrain grid system, using lateral and longitudinal perforated PVC pipe placed in excavated trenches (which, if used, would have resulted in the removal of about 50 percent of the structural foundation).

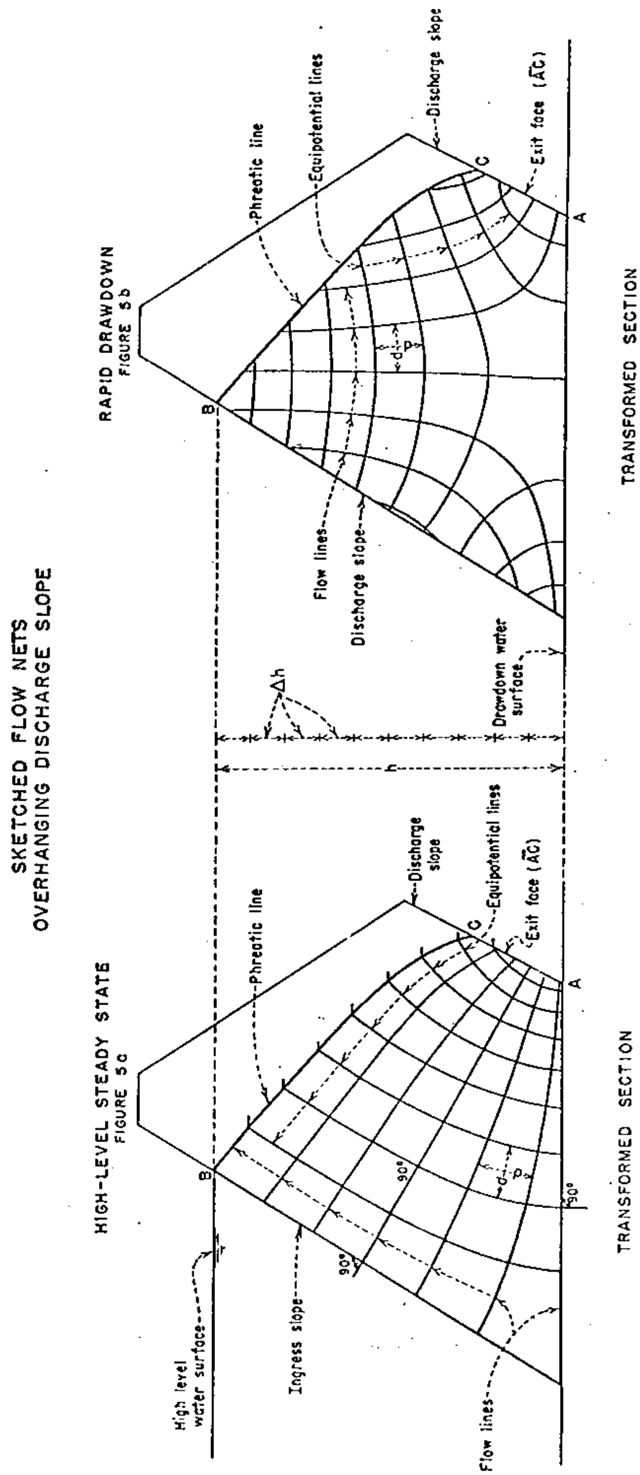


Figure 28.—Example flow nets for high level steady state and for rapid drawdown.

2. *Configuration and Size.*—As a general rule, most appurtenant structure drainage provisions are laid out in a grid pattern, whether dealing with walls, conduits, tunnels, or slabs. Spacing of the grid in both the longitudinal (along the structure) and lateral (across the structure) directions are influenced by the amount of flow expected, anticipated efficiency over the (economic) life of the structure, etc. In some cases, design by precedent (and rules-of-thumb) may be employed.

As another reminder, Reclamation's historical approach to drainage features for spillways and outlet works has been to design for loss of effectiveness over time, due to the difficulty in accessing, monitoring, and maintaining drains. However, there are now design and O&M tools that would result in a high level of confidence that drainage effectiveness can be maintained over the service life of the appurtenant structure.

The number and size of collector drain pipes (generally longitudinal) are based on the guideline that open channel flow will be maintained. To achieve this guideline, the pipes should be sized so that the maximum depth of flow does not exceed 75 percent of the pipe diameter (or if using collector drains with noncircular cross sections, such as rectangular, wetted area should be 80 percent or less of the total cross sectional area). Unstable flow conditions, such as "slug flow" can result with flow depths greater than this and lead to damage and/or failure of the drain pipe. To estimate the number and diameter (or area) of the collector drain pipes, the maximum seepage must be estimated. Design examples of estimating seepage for lined channel drainage systems can be found in the U.S. Army Corps of Engineers publication, EM 1110-2-2007, "Structural Design of Concrete Lined Flood Control Channels" [3]. A flow net analysis, assuming the collector drain pipes have an infinite permeability, can be used to estimate total seepage per foot of drain adjacent to and beneath the appurtenant structure. It should be noted that since gradients may become very steep adjacent to drains, often greater refinement is needed in this area. For more details on developing a flow net analysis, refer to section 2-4 in this chapter on drainage provision designs for embankment and rockfill dams. Once the total seepage is estimated, the number and size of drain pipes can be made by assuming normal (uniform) flow conditions and employing Manning's equation:

$$v = \frac{1.49}{n} r^{2/3} s^{1/2}, \text{ where } r = \frac{a}{p}, \text{ and } s = \frac{\Delta_{\text{elev}}}{\text{length}}$$

v: Average velocity in collector pipe, ft/s

n: Manning's roughness factor (smooth-walled pipe, such as PVC, n = 0.009; ribbed walled pipe, such as CMP, n = 0.0225; concrete/mortar pipes, n = 0.013) *r*: Hydraulic radius, ft, which is defined as the wetted area, a, divided by the wetted perimeter, p [Note: d (depth of flow) #0.75 diameter of pipe, if circular, or a #0.80 area of pipe, for any cross-sectional shape]

s: Slope of energy grade line, which can be approximated by the difference in elevation,)$_{elev}$, divided by length (the length of the longitudinal drain pipe along the appurtenant structure).

To determine the capacity, q, of the collector pipes, the average velocity, v (determined from Manning's equation above), and the wetted area, a, use the continuity equation:

$$q = va$$

Also, the U.S. Army Corps of Engineers has developed a nomograph for estimating the size of circular drains, flowing full (refer to sec. A-6 in app. A). This could be used for an initial pipe size, then applying the 75 percent of the pipe diameter (flowing full) guideline, a final pipe diameter can be estimated for the longitudinal collector drain pipes. Additionally, it is very likely that the drainage system may lose efficiency over time. This is particularly true for drainage systems associated with appurtenant structures, where inspection and cleaning may not be possible. Therefore, it is prudent to design the drainage systems for a reduced level of efficiency. The drain pipe should be designed for a reduced area (due to material deposition or calcium carbonate deposits) in the range of 75 percet of the original pipe diameter, while still allowing for free flow conditions within the pipe [3].

For design by precedent (applicable for preliminary or lower-level designs), refer to section A-1 in appendix A for guidelines (rules of thumb). In other cases, configuration and size requirements must be determined from design data and subsequent analysis. As a general guideline, drainage systems should be defined by analysis rather than by precedent whenever there is no precedent for the drainage feature being considered and/or the consequences of failure of the drainage system could lead to failure of the appurtenant structure and/or impoundment structure, resulting in uncontrolled release of part or all of the reservoir.

3. *Filter Requirements*.—Determination of filter requirements can have a significant bearing on the type, size, and location of drainage provisions, along with the stability of the appurtenant structure. In some cases, design by precedent will suffice (i.e., using drainage provisions from similar previously constructed appurtenant structures and/or rules-of-thumb). In other cases, filter requirements must be determined from design data and subsequent analysis. Refer to section A-6 in appendix A for typical design data needed and analysis to determine filter requirements.

Figure 29.—Grassy Lake Dam Spillway, Wyoming—Chute floor has succumbed to inadequate drainage provisions and frost heave.

4. *Pervious Backfill*.— As a general guideline, consideration should be given to using pervious backfill or other freely draining material adjacent to retaining walls and conduits that are not part of a water impoundment structure (i.e., dam or dike), where economically feasible. Of particular importance is providing sufficient freely draining material (such a pervious backfill) adjacent to walls and conduits in cold weather climates (i.e., in locations were freezing ground can occur). Unheated appurtenant structures surfaces in contact with frost-susceptible backfill and with access to water are subject to frost penetration, ice lensing, and subsequent frost heave that can be significant. Placement of a freely draining granular material adjacent to the appurtenant structure will limit, if not prevent "frost heave." The required thickness of non-frost-susceptible backfill material may be determined from the surface-freezing index (cumulative degree-days below freezing, based on average daily temperatures), using design curves presented in Reclamation's Frost Action Team Report [12, 13] and in section A-2 in appendix A.

5. *Insulation Requirements*.—Considerable damage and even failure can result from freezing foundations and adjacent materials, particularly if the materials are soil (refer to figure 29 for example of frost-heave damage to an appurtenant structure). As described in *Pervious Backfill* above, unheated appurtenant structures' surfaces in contact with frost-susceptible backfill or foundation and with access to water are subject to frost penetration, ice lensing, and subsequent loading that can be significant. To address this concern for

foundations and backfill, insulation requirements are employed to protect drainage provisions from freezing during cold weather. Historically, typical insulation materials for appurtenant structures' drainage provisions are rigid polystyrene insulating materials, such as Styrofoam HI brand plastic foam, available in standard thicknesses of ¾, 1, 1½, 2, and 3 inches; widths of 24 inches; and lengths of 4 and 8 feet. Also, to improve performance, and if needed, insulation should be placed in at least two overlapping layers [14]. Methods that have been used to estimate insulation requirements include:

- *Modified Berggren Equation for Multilayer Systems.*—This is a widely used method for estimating seasonal frost depths in soils. It is assumed that each layer of material is homogeneous and isotropic, and the average thermal properties of the materials (frozen and unfrozen) are applicable. The entire mass is assumed to be at the mean annual temperature for the site prior to the start of the freezing season. When the freezing season starts, the surface temperature is assumed to drop to a temperature below freezing, determined by the length of the freezing season and by the surface-freezing index. The effect of latent heat of fusion is considered as a heat sink at the moving frost line, with complete freezing assumed to occur at 32.0 °F [15, 16]. The degree-days required to penetrate each layer are accumulated until the summation equals the surface-freezing index. The sum of the thicknesses of all the frozen layers is the frost depth of the system [17]. This method cannot be used for the design of insulating materials alone, since these materials have negligible moisture contents and therefore no latent heat. However, the modified Berggren equation does give reliable results for frost depths greater than 8 to 12 inches beneath the insulating layer. More details and an example problem can be found in section A-3 in appendix A.

- *Lachenbruch 3-Layer Method.*—This method does not consider the effects of latent heat, and can therefore be used to design a system allowing no frost penetration beneath the insulating layer. A 3-layer system of a gravel base, an insulating layer, and gravel subbase is assumed. A sinusoidal temperature variation of amplitude A (based on the mean annual temperature and the surface-freezing index) is applied at the surface. The amplitude F at the interface of the insulating and subbase layers is determined, from which the ratio F/A is calculated. This ratio is then used to determine the required thicknesses of the gravel base and insulating layers [15, 16]. Assuming that no frost penetration beneath the insulation is permitted, the magnitude of F is the difference between the mean annual temperature and the freezing point of the soil moisture, or 32.0 °F. For colder climates, the effects of the gravel base over the insulating material may be neglected. A review of Lachenbruch's data indicates that increased thicknesses of insulation required to eliminate frost penetration beneath the insulating layer have a diminishing effect in colder climates (where the F/A ratio is small). In unheated structures, the insulation required is significantly reduced if clean, non-frost-susceptible fill material can be provided beneath the floor slab.

- *Finite Element Method (FEM)*.—This method was developed by C. T. Hwang and applied to the design of insulated foundations by Eli Robinsky and Keith Bespflug. The method may be used to analyze two-dimensional heat flow in multilayer systems, and includes the effects of latent heat. Thermal properties of the various materials for both the frozen and unfrozen states may be used. For open spaces and unheated structures, a sinusoidal temperature variation during the freezing season may be assumed to closely approximate actual winter conditions. The temperature at depths of 16 to 20 feet is assumed to remain constant at the mean annual temperature throughout the year. Robinsky and Bespflug developed design curves for determining insulation and granular fill thicknesses beneath unheated structures, based on the surface freezing index and this FEM . These curves were included in Reclamation's Frost Action Team Report [13].

6. *Contamination*.—Primarily concerned with contamination of drainage provisions during construction, considerations should be given to providing design features that will isolate the drainage provisions from adjacent concrete placements. Typically, this has been accomplished by using insulation material, burlap, geotextiles, geomembranes, and/or steel wool (weep holes) as a barrier between drains and fresh concrete. Some considerations related to selecting barrier materials include:

- Prior to the advent of geotextiles and geomembranes, burlap was typically used as a barrier material. However, burlap may not be readily available today, so geotextiles and/or geomembranes are another option.

- Insulation (if needed) may suffice as a barrier material, but may not satisfy filter requirements.

- Geotextiles could serve the dual purpose of a barrier material and provide filter requirements (if needed).

- Geomembranes would provide an impermeable barrier material.

- Consideration should be given to the slope of the foundation when selecting a barrier material. As a general rule, geotextiles and geomembranes should not be used on slopes greater than 3:1, unless they are anchored, and the overlying material can be shown to be stable against sliding.

7. *Hydraulic Considerations*.—

(a) *Back Pressure*.—Particularly for hydraulic appurtenant structures (such as spillways and outlet works) associated with high velocity, high volume releases, care must be taken to ensure that drainage provisions do not create adverse conditions that could lead to damage or failure of the appurtenant structure. Such situations have and can occur in chutes and hydraulic jump stilling basins (i.e., terminal structures) where tailwater is above the floor of the stilling basin and portions of the chute. During operation, particularly at releases considerably less than the maximum designed release (when a hydraulic jump might begin in the chute rather than the stilling basin), the depth of the jet just upstream of the hydraulic jump (i.e., d_1) will be considerably less than the tailwater depth (i.e, $\$\ d_2$). The weight of the floor slab and water in the jet may be less than the hydrostatic pressure under the slab (corresponding to full tailwater head), which can be introduced through the drainage provisions beneath and adjacent to the stilling basin.

It was common on many Reclamation chute and stilling basin structures to terminate the drainage provisions at the downstream face of the chute blocks (usually at the interface between the chute and stilling basin floor). For maximum design releases, subatmospheric pressures generally resulted at this location (i.e., the beginning of the hydraulic jump) which lowered hydrostatic pressures beneath the chute slabs. However, for smaller releases, the tailwater could exceed the conjugate depth needed for the jump, which causes the beginning of the hydraulic jump to move upstream of the termination point for the drainage provisions. This could lead to the introduction of increased hydrostatic (uplift) pressure beneath the chute floor, which in turn could result in damage or failure of the chute floor. Although Reclamation has not experienced this type of failure, such a failure occurred at Karnafuli Spillway in Bangladesh. Perhaps one factor that has helped Reclamation avoid this type of failure is that the majority of Reclamation hydraulic structures, particularly spillways and outlet works, have been constructed on firm formation, with anchorage (i.e., rockbolts, or anchor bars). The bond strengths between the concrete and foundation and anchorage are not usually considered as stabilizing features, but as redundancies that are considered prudent, given the potential consequences resulting from damage or failure of an appurtenant structure.

To address this situation, "eductors" (i.e., aspirators) are incorporated into the drainage provisions (refer to fig. 30 for graphical representation of eductors). These are drains which exit through the chute slab into high velocity flow. During operation, a negative pressure occurs at the exit of the eductors, which lowers the hydrostatic pressure beneath the chute and/or stilling basin slab (even when the eductors are located below tailwater). When the appurtenant structure is not operating, the eductors below the tailwater will admit water into the drainage provisions. However, since this is a balanced pressure condition, there is no concern. For more details of this concept, refer to the Spring Creek Debris Dam enlargement study [19], figure 31, and the design procedure found in appendix A4.

(b) *Stagnation Pressure.*—High velocity flow within an open channel on steep slopes is a significant source of energy, which can generate damaging uplift pressures.

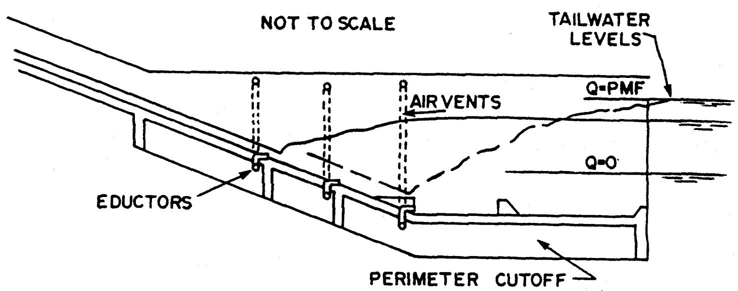
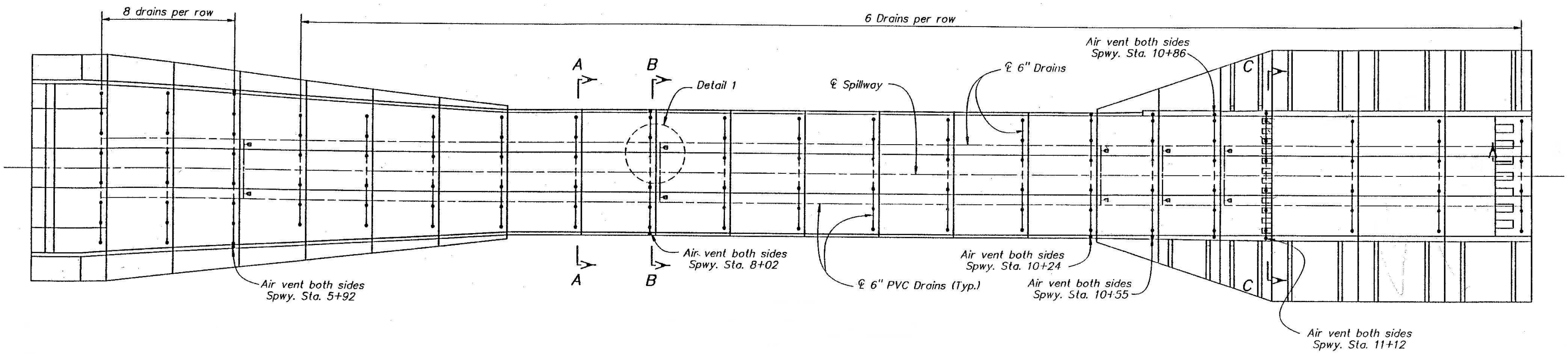


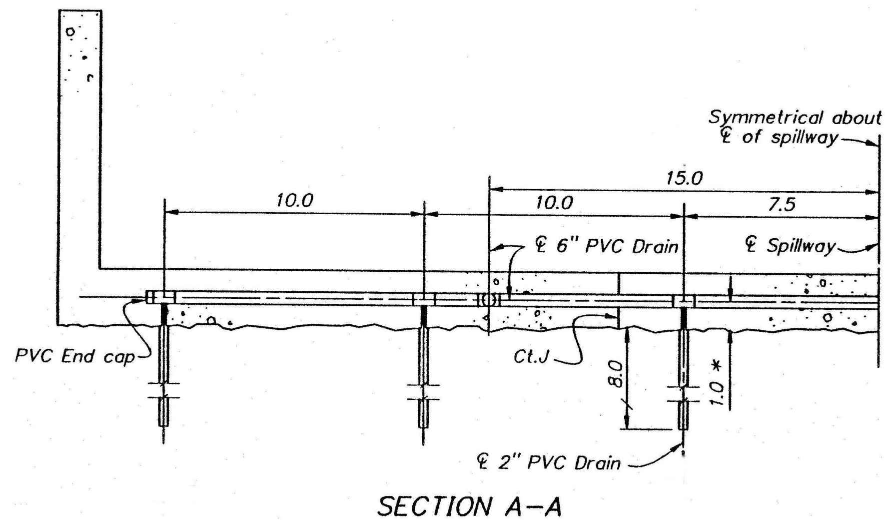
Figure 30.—Illustration of chute and hydraulic jump stilling basin with "eductors" connected to chute drainage provisions (from Smith and Gui [18])

Offsets may develop within the concrete lining at joints or cracks as a result of shrinkage, differential settlement/heaving, ice pressures, etc. In some cases, these offsets serve to direct a portion of the flow downward into openings (such as contraction and control joints) and beneath the concrete slab. The result may be stagnation pressures (refer to fig. 32 for a graphical representation of stagnation pressure conditions). If these pressures are large enough to overcome the weight of the concrete lining (slab), the weight of the water on the slab, and any mechanical or chemical bonds (anchor bars, interface bonding of concrete and foundation, etc.), the slab will be displaced, and structural failure may result [20]. Reclamation has experienced several stagnation-pressure-induced incidents. These failures are associated with older structures (built prior to 1965) that did not employ present-day design and construction considerations such as pressure grouting (where applicable); embedded waterstops in floor joints; longitudinal reinforcement and transverse cutoffs; foundation anchors; and as this discussion emphasizes, an adequately sized, filtered, and insulated underdrain system. An adequately sized, filtered, and insulated underdrain system is a key element in reducing the potential for offsets and/or cracks in the overlaying concrete slab by effectively removing seepage and mitigating potential frost heave. For related details on size, filter and insulation requirements, refer to section 2-5.a., *General Design Considerations and Methodology*, under *Configuration and Size*, *Filter Requirements*, *Pervious Backfill*, and *Insulation Requirements*.

8. *Air Demand.*—For many underdrain systems, air demand must be accounted for in the design. This demand could range from just requiring a "vacuum break" to providing sufficient air to offset lowered pressures induced by high velocity flow across drain outlets (such as "eductors" or aspirators). The vacuum break air demand concept is analogous to needing two openings in a can of fluid. Without the second opening, pressure can drop below atmospheric pressure, causing reduced flow rates under unstable conditions, such as slug flow (refer to figs. 33 and 34 for vacuum break vent). For further discussion on



a. Layout of spillway drainage provisions, including lateral and longitudinal collector drains, near vertical drilled and formed weephole drains, eductors (i.e., aspirators), and standpipes.



» b. Layout of lateral collector drain and drilled/formed weephole drains. Note that lateral collector drain is embedded in structural concrete to minimize disturbance to foundation.

c. Typical detail of drilled/formed weephole drain. o

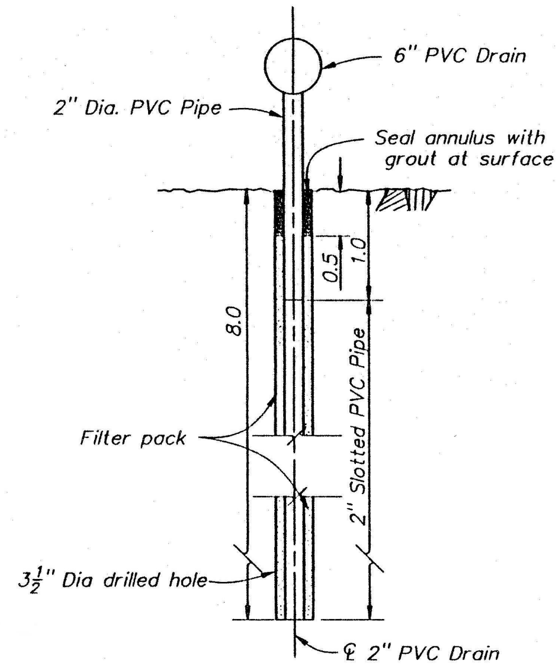
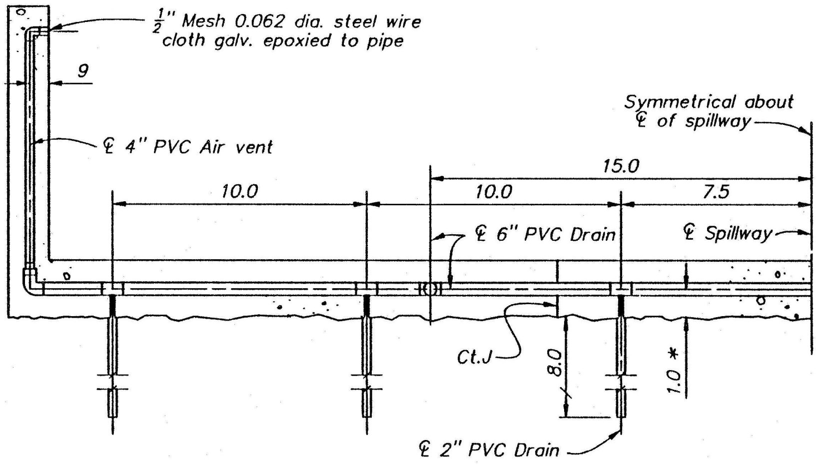
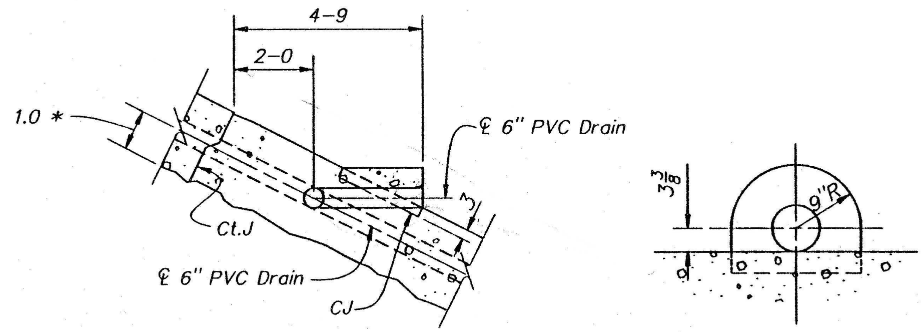


Figure 31.—Spring Creek Debris Dam, California—Using drilled/formed drains (weepholes) and "eductors" (i.e., aspirators) to mitigate excessive hydrostatic (uplift) pressure beneath chute and stilling basin floors.

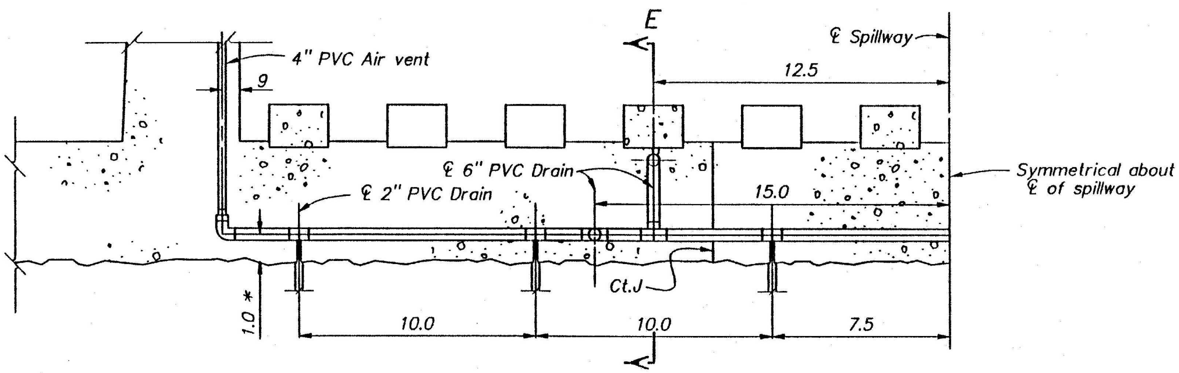


SECTION B–B

d. Section through chute illustrating lateral collector pipe tie-in with drilled/formed weephole drains and standpipe, used to provide air to eductors.

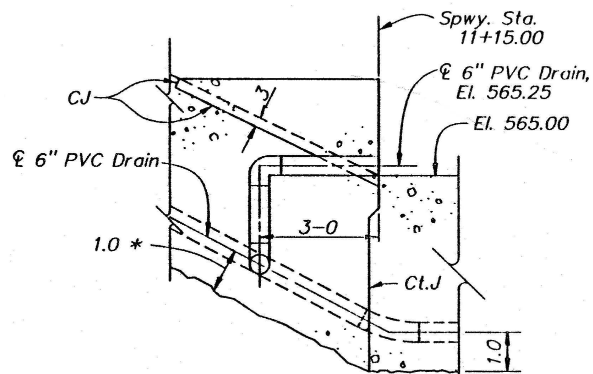


e. and f. Details of eductor in chute.



SECTION C–C

g. Section through chute-stilling basin interface illustrating lateral collector drain pipe tie-in with drilled/formed weephole drains and standpipe, used to provide air to eductors.



SECTION E–E

h. Details of eductor in chute-stilling basin interface.

Figure 31 (cont'd)

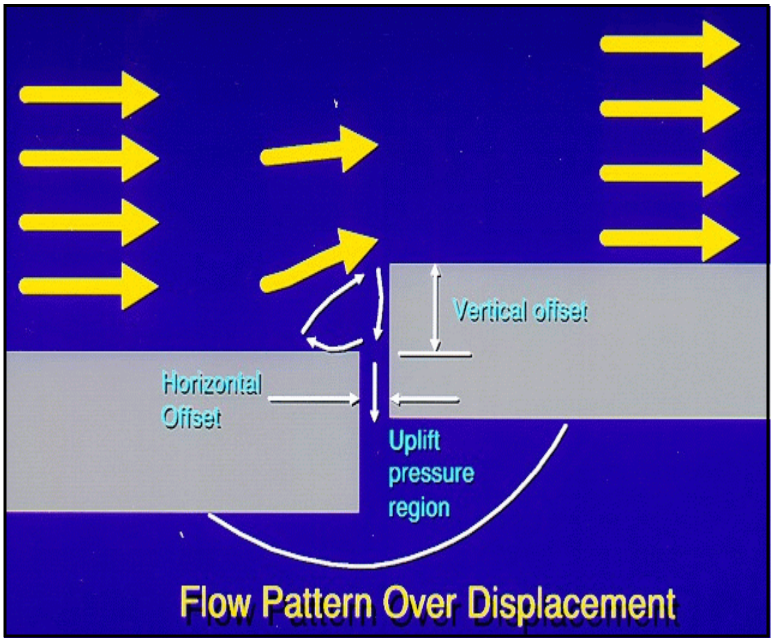


Figure 32.—Illustration of conditions that could result in a stagnation pressure failure. Laboratory modeling has indicated that uplift pressures could be as high a 0.9 velocity head.

air demand to mitigate lowered pressures induced by high velocity flow across a drain outlet, refer to section 2-5.a., *General Design Considerations and Methodology* under *Back Pressure*, section A-4 in appendix A, and Reclamation's Engineering Monograph No. 41, *Air-Water Flow in Hydraulic Structures* [21].

b. *Specific Design Considerations and Methodology*.—In addition to the general design considerations and methodology, specific or unique consideration should be given to certain types of appurtenant structures. Considerations for two high velocity, high volume, high pressure hydraulic structures follow.

1. *Spillways*.—Reclamation's definition of a spillway is a structure that passes normal and/or flood flows for the purpose of protecting the structural integrity of the dam(s) and/or dike(s). Other definitions include: an overflow channel of a dam or impoundment structure; a structure over or through which flow is discharged from a reservoir; and/or any passageway, channel, or structure designed to discharge surplus water from a reservoir. If the rate of flow is controlled by mechanical means such as gates, it is considered a controlled spillway. If the geometry of the spillway is the only control, it is considered an uncontrolled spillway.

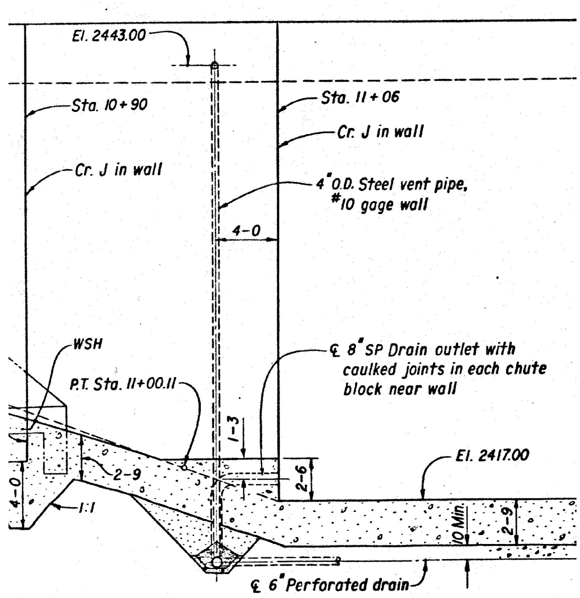


Figure 33.—Keechelus Dam Outlet Works, Washington—Section along stilling basin showing steel vent pipe (vacuum break).

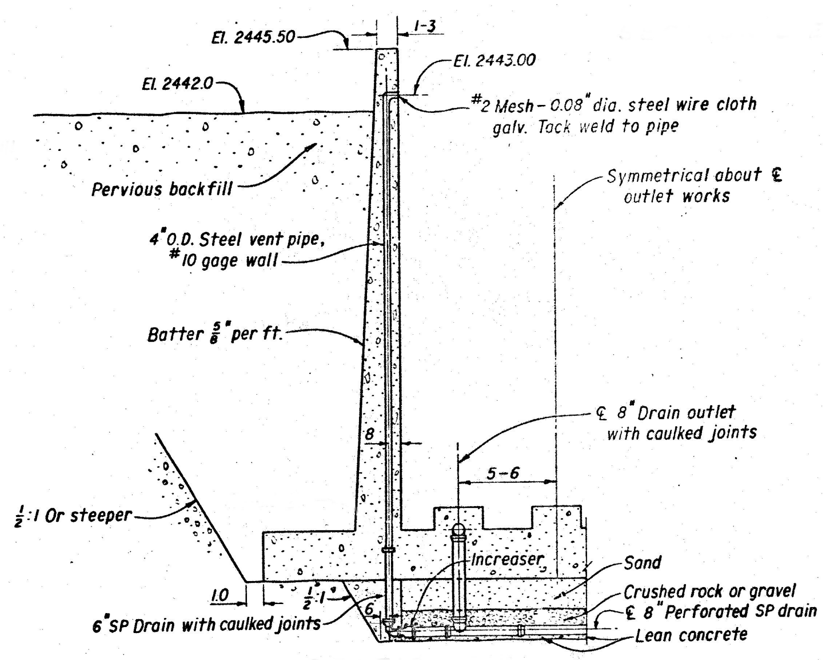


Figure 34.—Keechelus Dam Outlet Works, Washington—Section through stilling basin showing steel vent pipe (vacuum break).

2. *Outlet Works.*—Reclamation's definition of an outlet works is a combination of structures and equipment required for the safe operation and control of water released from a reservoir to serve various purposes—regulating downstream flow and quality; releasing floodwater; and providing, municipal, and/or industrial water. Other definitions include: a series of components located in a dam through which normal releases from the reservoir are made; a device to provide controlled releases from a reservoir; and/or a pipe that lets water out of a reservoir, mainly to supply downstream demands [22].

Drainage provisions are a key component of most spillways and outlet works designs. Drainage provisions should be considered for the following features of most spillways and outlet works:

1. *Inlet (or Approach) Channel or Structure.*—This feature is located upstream of the spillway crest structure or the outlet works inlet structure and generally in the reservoir. It conveys water from the reservoir to the crest or inlet structure. Drainage provisions are usually only considered for this feature if it is lined (with materials such as reinforced concrete, roller-compacted concrete, riprap, or grouted riprap). Needs for drainage provisions are based on historical or planned operation of the reservoir (e.g., the submergence of the channel, or a potential for rapid reservoir fluctuations). Generally, drainage provisions are limited to relieving hydrostatic pressure in the excavated channel subject to potential rapid fluctuations in the reservoir.

2. *Spillway Crest Structure.*—This feature is generally located upstream of the conveyance structure, in the reservoir and/or upstream of the dam/dike axis (or centerline). Its intended purpose is to serve as a control point for reservoir releases, whether it is free flow or gated flow. For crest structures located adjacent to or through dams/dikes, drainage provisions will be similar to the dam's/dike's drainage provisions, including grout curtains located at or upstream of the spillway crest, and drain holes, and/or graded filter material located downstream of the grout curtains. Drainage provisions may be needed to relieve hydrostatic pressure in the excavation adjacent to the spillway crest structure subject to potential rapid fluctuations in the reservoir (in areas at or upstream of the dam/dike axis). Also, drainage provisions may begin within the spillway crest structure (in areas downstream of the dam/dike axis) and tie into the other downstream features. For this situation, refer to section 2-5.a., *General Design Considerations and Methodology.*

3. *Outlet Works Inlet Structure.*—Similarly to the spillway crest structure, this feature is located upstream of the conveyance structure, in the reservoir and/or upstream of the dam/dike axis (or centerline). The inlet structure may or may not have control features such as gates. Its intended purpose is to provide an unobstructed opening to the reservoir. The need for drainage provisions is based on historical and/or planned operations of the reservoir (e.g., submergence of the intake, or the potential for rapid reservoir fluctuations).

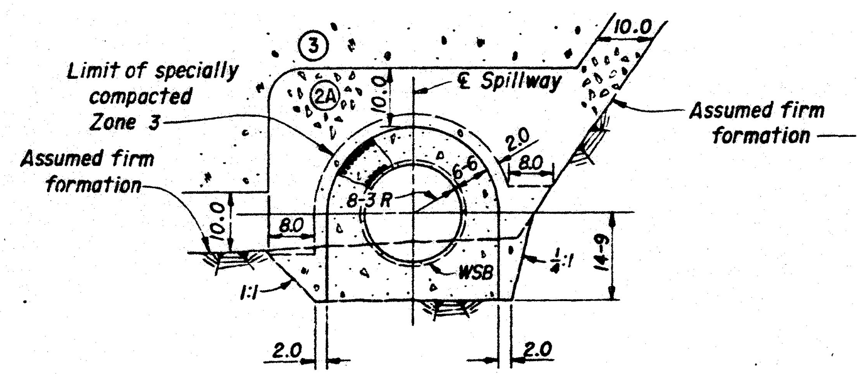


Figure 35.—Rigdway Dam spillway, Colorado—Cross section showing conduit with embankment filter wrap-around.

4. *Conveyance Structure (Chutes, Conduits, Pipes, Tunnels).*—This feature conveys discharge from the crest or inlet structure to the terminal structure. As with the crest or inlet structure, if the conveyance structure is located adjacent to or through dams/dikes, drainage provisions compliment the dam's/dike's drainage provisions, which could include drain holes, lateral and longitudinal drains, and graded filter material (which could be part of or a continuation of the dam/dike drainage provisions, such as those shown in fig. 35).

- *Chutes.*—This feature is an open structure, such as a cast-in-place concrete rectangular or trapezoidal section, riprap (or grouted riprap) trapezoidal section, gabion rectangular or trapezoidal section, grass-lined trapezoidal section, rock cut unlined rectangular or trapezoidal section, or geomembrane trapezoidal section, that discharges water under free flow conditions. Drainage provisions will be needed for most chutes. For details and specific considerations, refer to section 2-5.a.

- *Conduits or Pipes.*—This feature is a closed structure, such as precast concrete pipe, cast-in-place concrete conduit, HDPE pipe, or CMP, that could discharge water under free flow or pressure conditions. Drainage provisions are needed for most conduits and pipes.

  Of special concern are cut and cover conduits or pipes through or beneath embankment or rockfill water impoundment structures (i.e., dams, dikes, or levees), which may create an opportunity (increase the risk) for seepage and subsequent internal erosion, perhaps leading to failure, along the conduit or pipe surface. Therefore, other alternatives (such as tunnels) will be considered in lieu of conduits wherever they are technically and economically feasible. When conduits are placed through or beneath embankments, filters enveloping the conduit are a particularly important defense against internal erosion caused by seepage along the conduit. For additional details, see the following text.

- *Through Embankment/Rockfill Dams or Dikes*—When a conduit or pipe is identified as the preferred alternative for conveying water through or beneath an embankment or rockfill water impoundment structure, Reclamation's policy is that "cutoff collars" *will not* be used as a seepage control measure (refer to figs. 36 and 37 for graphical representation of cutoff collars). The use of cutoff collars can result in lower density material, nonuniform earth pressure, and seepage gradients along a conduit, which can lead to zones where a combination of higher seepage gradients and lower earth pressures will allow internal movement of less dense soil particles or piping. Seepage gradients are more uniform along a smooth conduit without cutoff collars. Seepage will be controlled by careful selection and compaction of earth backfill against the conduit, which should have battered walls, and installation of a properly graded filter around the conduit at the downstream limit of the embankment impervious zone. Designs should provide for horizontal drainage contiguous with the inclined and horizontal drainage zones of the embankment (or rockfill) dam. If it is impractical to use the dam filtering and drainage provisions along the conduit, separate filtering and drainage provisions around and along the conduit, respectively, should be added [23]. Specific guidelines that should be considered include:

    - Combine waterways for different purposes into one structure. A major concern that needs to be addressed when considering this option is that if a combined structure is inoperative, there may be no way to safely pass floods or operational releases through or around the impoundment structure.

    - Locate the conduit(s) in a cut-and-cover section into firm formation when the firm formation is at or near the ground surface. For this alternative, the specifications should include provisions for firm formation excavation to be performed to eliminate or minimize open fractures or other damage to the firm formation beyond the limits of the excavation. In this case, backfill around the conduit with nonstructural concrete through at least the dam impervious zone (or a significant portion of the impervious zone). The concrete plug should extend to an upper limit of the top of the conduit or to the original firm formation surface if lower than the top of the conduit. If the dam is to be placed against the upstream and/or downstream ends of the plug, then sloping the ends may be desirable. Depending on the nature of the foundation, and deformation characteristics, the conduit and nonstructural concrete backfill should be not be bonded together.

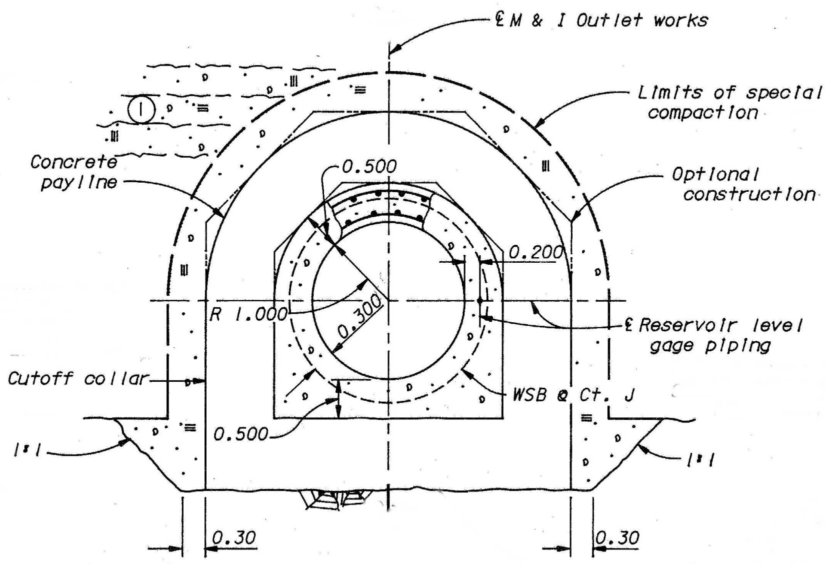


Figure 36.—McGee Creek Dam M&I Outlet Works, Oklahoma—Cross section showing conduit and cutoff collar. **NOTE**: Cutoff collars **are not** acceptable drainage provisions for future Reclamation appurtenant structures.

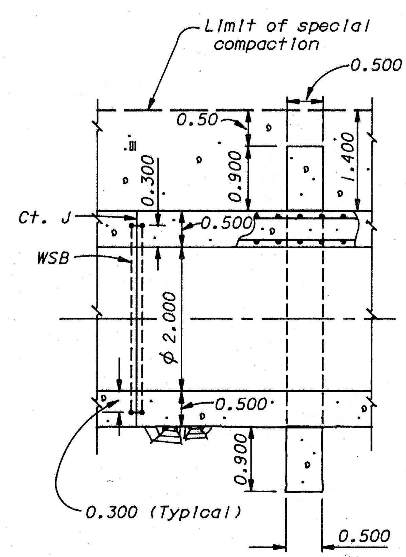


Figure 37.—McGee Creek Dam M&I Outlet Works, Oklahoma—Section along conduit centerline and through cutoff collars. **NOTE**: Cutoff collars **are not** acceptable drainage provisions for future Reclamation appurtenant structures.

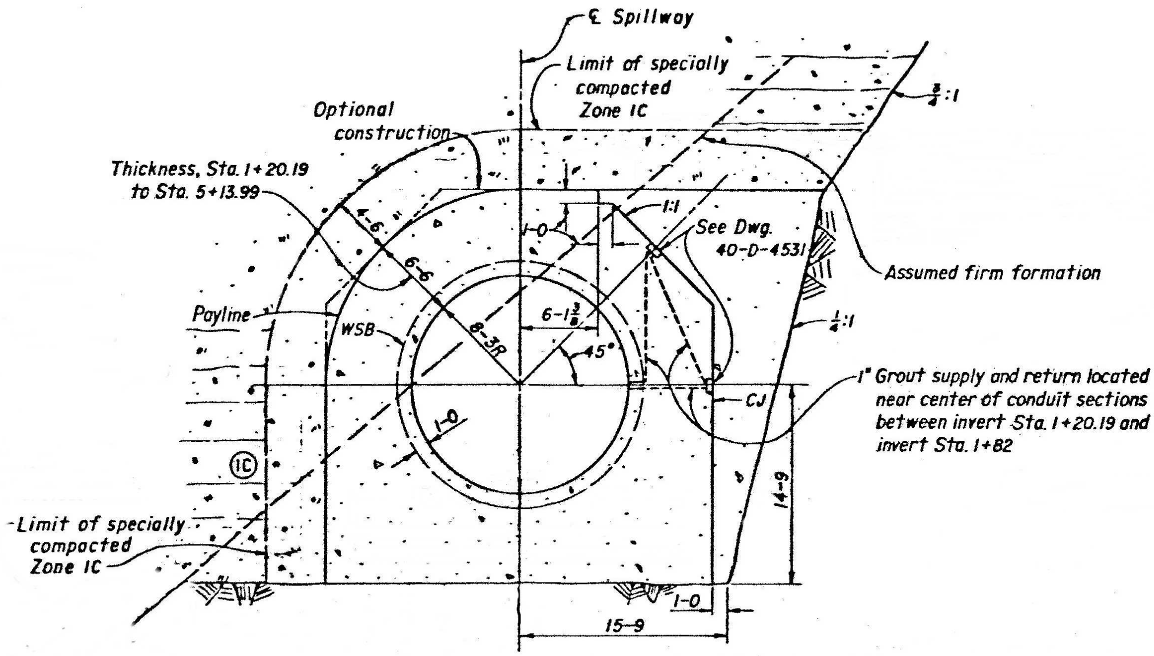


Figure 38.—Ridgway Dam spillway, Colorado—Cross section showing conduit founded on and tied into excavated bench with nonstructural concrete.

- Locate the conduit in a trench or on a bench excavated along the toe of an abutment when geological conditions and topography are favorable. Placing a lean concrete plug on the abutment side, or casting the conduit against the excavated firm formation reduces or eliminates requirements for earthfill compaction against one side of the conduit (refer to fig. 38 for a graphical representation of placing conduit against excavated firm formation).

- The conduit-foundation contact must not be overlooked as a path for potential seepage and piping, particularly when the foundation is soil. Prevention of excessive seepage (piping) along the conduit consists of providing a smooth, firm contact surface free from loose or disintegrated materials. If the foundation surface is subject to deterioration when exposed to the atmosphere, it may be necessary to protect the foundation surface with suitable earthfill, a concrete pad ("mud slab"), or an acceptable sealing compound until conduit construction commences.

R Filters placed around conduits to prevent piping should envelop conduits on soil foundations. Filters around conduits on firm formation should extend only to the foundation surface. The filters should meet the same guidelines/criteria for dry unit weight and filtering as for other filters within the dam or dike (refer to fig. 35 for a graphical representation of filter wrap-around).

N *Through Canal or Levee Embankments.*—Pipe culverts through this type of embankment should have soil-cement slurry placed to a level to ensure a firm pipe foundation under the pipe haunches. Special consideration should be given to compacting earthfill around the remainder of the pipe. Cutoff collars should not be used for seepage control. A filter should be provided around the downstream reach of the pipe.

For additional discussion, concerning design and construction considerations for controlling seepage along conduits, refer to Assistant Commissioner - Engineering and Research (ACER) Technical Memorandum No. 9 [23].

- *Tunnels.*—Section 2-7 covers *Rock Tunnel Drains.*

5. *Terminal Structure (Stilling Basin, Plunge Pool).*—This feature can be either an open or closed structure. Open structures can be rectangular or trapezoidal, and lined with cast-in-place concrete, roller-compacted concrete (RCC), and/or soil cement; constructed with gabions; or consist of an unlined rock cut. Open structures can also be trapezoidal, and lined with riprap or grouted riprap. Closed structures include rectangular, covered, cast-in-place structures. These features dissipate the kinetic energy of the discharge. Usually, the drainage provisions terminate prior to or at the terminal structure. Because of the significant pressure fluctuations (due to the energy dissipation) associated with most terminal structures, care must be given to designing and installing drainage provisions so that they do not jeopardize the structural integrity of this feature or adjacent features by, for example, introducing excessive back pressures into the drainage provisions. Details of drainage provisions can be found in section 2-5.a.

Another issue for terminal structures is unwatering these structures for inspection. Unwatering often requires sealing the end of the structure against tailwater through the use of stoplogs, sandbags or other provisions. Once this is accomplished, the inside of the terminal structure can be unwatered by pumping out water from inside of the structure. Before unwatering is attempted, it should be verified that the structure is adequate for this loading condition. Provisions may also be required for preventing tailwater from reentering the terminal structure through the drainage system.

6. *Exit Channel*.—This feature is located downstream of the terminal structure. It conveys water from the terminal structure to the stream or a canal. Similarly to considerations given for the inlet (or approach) channel or structure, drainage provisions are usually only considered for this feature if it is lined (with materials such as reinforced concrete, roller compacted concrete, or grouted riprap). Needs for drainage provisions are based on historical or planned reservoir releases and their affects on the downstream river/stream (e.g., the submergence of the channel, or the potential for rapid increasing or decreasing releases). Similarly to the inlet (approach) channel considerations, drainage provisions are limited to relieving hydrostatic pressure in the excavated channel subject to (potential) rapid tailwater fluctuations.

2-6. Slope Drains.—Water pressure (whether acting as an uplift pressure on a base plane or acting as a driving force within a tension crack or a similar nearly vertical fissure) can be a significant factor in the stability of rock slopes. Water pressure is also significant in soil slopes, because it increases the driving force on a potential slide and reduces the sliding resistance of a soil mass, through pore pressure acting along the failure surface. Examples are provided to indicate how water pressures are accounted for in the stability of slopes. Three cases are considered—the plane failure of a rock slope, a wedge failure within a rock slope, and a soil slope failure.

a. *Rock Slope Plane Failure*.—One of the assumptions for a plane failure is that the plane on which sliding occurs must strike parallel or nearly parallel to the slope face. For this type of failure, a tension crack may be assumed to form either in the slope face or on the slope bench, which provides a back-release plane for the failure. The tension crack may be assumed to be filled with water to a certain depth. This water provides a lateral driving force on the block and also provides uplift pressure on the sliding plane. Figure 39 provides two scenarios for a plane failure—a tension crack forming in the upper slope surface and a tension crack forming in the slope face.

The factor of safety (FS) for a plane failure is a function of the total force resisting sliding relative to the total forces which encourage sliding. FS can be expressed as:

$$FS = \frac{cA + \left(W \cdot \cos\psi_p - U - V \cdot \sin\psi_p\right)}{W \cdot \sin\psi_p + V \cdot \cos\psi_p} \tan\phi$$

where, from figure 39,

$A = (H - z)$ @cosec R_p (area of base plane)
$U = ½$ $(_w$@ z_w $(H\text{-}z)$ @cosecR_p (uplift force)
$V = ½$ $(_w$@z^2_w (lateral water force)

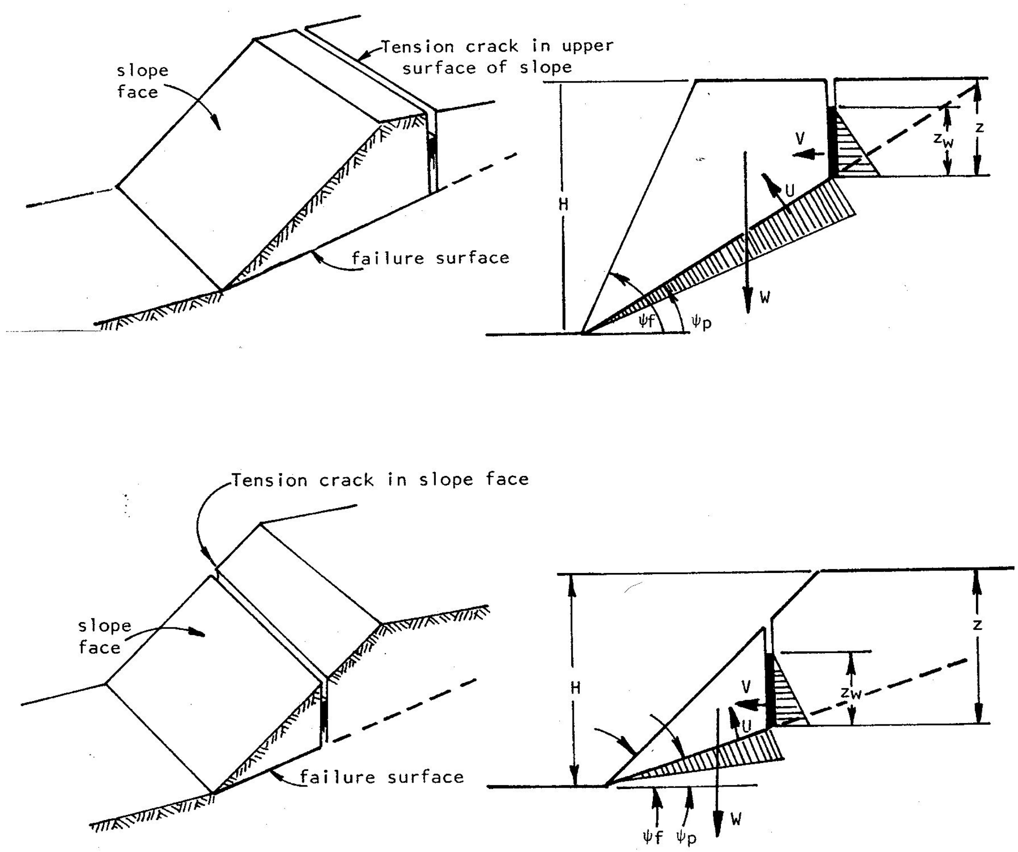


Figure 39.—Tension cracks contributing to slope failure (from Hoek and Bray, figs. 62a and 62b [24]).

For the tension crack in the upper bench surface (fig. 39),

$$W = \tfrac{1}{2} \, (\, H^2 \left((1- (z/H)^2\right) \cot R_p - \cot R_f)$$

For the tension crack in the slope face (fig. 39),

$$W = \tfrac{1}{2} \, (\, H^2 \left((1- z/H\right)^2 \cot R_p \,(\cot R_p \cdot \tan R_f - 1))$$

b. *Rock Slope Wedge Failure.*—A wedge failure is more complicated to analyze than a plane failure. Figure 40 represents a wedge failure and an assumed water pressure distribution. For this case, it was assumed that the wedge itself is impermeable, and water enters the top of the wedge along lines of intersection 3 and 4 and exits the slope face

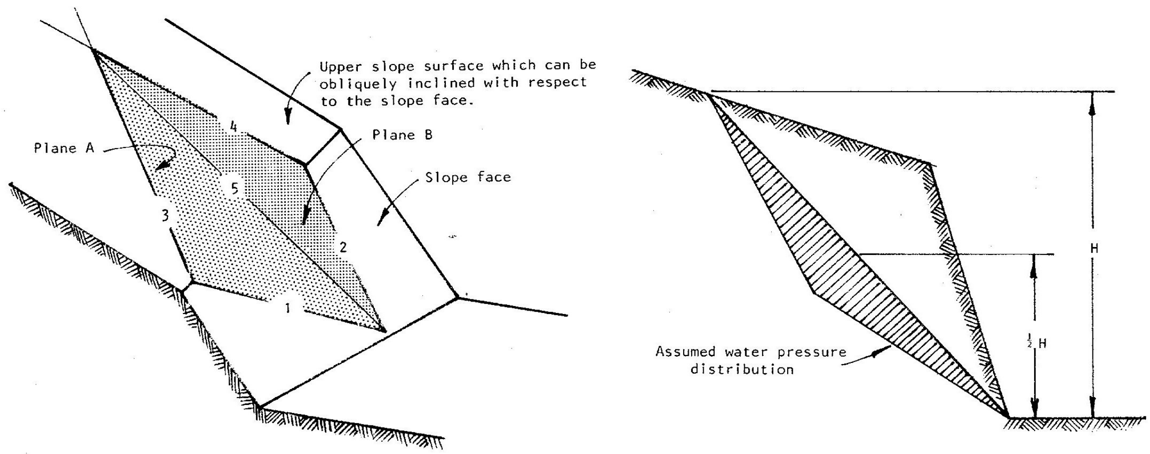


Figure 40.—Wedge failure with water pressure (from Hoek and Bray, fig. 97 [24]).

along lines of intersection 1 and 2. The maximum water pressure would occur along line 5, and zero pressure would occur along lines 1, 2, 3 and 4. The water pressure distribution shown in figure 40 and described above is an extreme case that could occur during heavy rain. The equations for the factor of safety for this type of analysis and a detailed discussion of wedge failures in rock slopes can be found in Hoek and Bray [24].

c. *Soil Slope Failure.*—A soil slope failure is similar to a rock slope plane failure. Instead of a continuous linear failure surface, however, a circular slip surface, or sometimes a noncircular slip surface, described by linear segments, is assumed in a slope stability analysis. A circular surface is applicable to homogeneous slopes, whereas a noncircular slip surface is more suited to a slope containing materials in variable layers. More detail on analysis methods for soil slopes can be found in Chapter 4 of the Embankment Dams Design Standard No. 13 [40].

d. *Rock/Soil Permeability.*—The permeability of intact rock is generally very low, as shown in table 1. However, discontinuities in the rock mass in the form of joints, fractures, or other geologic features will result in the permeability becoming significantly greater. The discontinuities form pathways in which water can travel. The permeability of soil materials is a function of the soil type, gradation, and the geologic history of the soil deposit.

The lack of surface flow on a rock or soil mass may not be an indication that groundwater is not present in the mass. If the evaporation rate is higher than the seepage rate, the surface may look dry, but there may be water at significant pressure within the mass. Water pressure, not rate of flow, is the important parameter that influences slope stability. Drainage is a very effective method of improving slope stability, and a good understanding of water flow patterns in a rock or soil mass is necessary to design an efficient drainage

Table 1.—Permeability coefficients for typical rocks and soils (from Hoek and Bray, p. 132 [24]).

General Condition	k (cm/s*)	Intact Rock		Fractured Rock	Soil
Practically impermeable	10^{-10}	Slate			Homogeneous clay below zone of weathering
	10^{-9}	Dolomite			
	10^{-8}	Granite			
		Limestone			
	10^{-7}		Sandstone		
Low discharge Poor drainage	10^{-6}				Very fine sands, organic and inorganic silts, mixtures of sand and clay, glacial till, stratified clay deposits
	10^{-5}			Clay-filled joints	
	10^{-4}				
	10^{-3}				
High discharge Low drainage	10^{-2}			Jointed rock	
	10^{-1}				Clean sand, clean sand and gravel mixtures
	1.0			Open jointed rock	
	10^{1}				
	10^{2}			Heavily fractured rock	Clean gravel

* cm/s x 0.0328 = ft/s; cm/s x (1.035 x 10^{6}) = ft/yr

system. Measurement of permeabilities and pressures are keys to understanding the groundwater patterns.

e. *Permeability Measurements.*—Permeability measurements may be helpful in designing a drainage system for a slope. While the stability of a slope depends on the water pressure within the rock or soil mass, the pressure at a point in the mass depends on the path the water took to arrive at the point. Permeability measurements may help determine where water is likely to be produced and areas where drain spacings need to be adjusted.

The following field permeability tests are typically performed in a borehole:

- *Falling Head Tests*.—A known volume of water is introduced into a borehole and the time it takes for the water level to lower to its original level is recorded.

- *Constant Head Tests*.—Water is introduced into a borehole at a rate to maintain a specific water level, and the volume of water added is measured over a time interval.

- *Pump-In Tests*.—Water is pumped into a borehole section between two packers or between a packer and the bottom of the borehole at various pressures, and the flow rates are recorded.

- *Lugeon Test*.—A Lugeon test is a specific pump-in test for rock foundations in which intervals of a test hole are tested over a range of pressures. A Lugeon is defined as 1 liter/meter/minute at 150 lb/in^2. If pressures vary from 150 lb/in^2, the calculation adjusts for the actual pressure used.

- *Pump-Out Tests*.—Water is pumped out of a borehole (packers usually not installed), and observation wells surrounding the borehole are monitored to record the drawdown.

Of the five tests, falling head tests, constant head tests, and pump-out tests are best suited for uniform soils or rock. Pump-in tests do allow for measuring permeabilities in distinct zones, where geologic conditions indicate permeabilities will likely vary. A more thorough discussion of field permeability tests and how to calculate permeability values from these tests can be found in Hoek and Bray [24, p. 136]. A detailed discussion of Lugeon tests and the supporting calculation can be found in Houlsby [25].

f. *Monitoring of Slopes*.—If an accurate assessment of the stability of slopes is needed, or if drainage is provided to ensure the stability of slopes, it will be important to monitor the water pressure in slopes. This is typically done by installing observation wells or piezometers in boreholes. A number of different types of instruments can be installed. An important factor in selecting an observation well/piezometer is the time lag. The time lag is the time it takes for the pressure in the system to reach equilibrium after a pressure change. The time lag is a function of the permeability of the ground and the changes related to pressure and volume.

The following is a summary of the more frequently used types of piezometers.

- *Observation Wells*.—This type of instrument consists of an open borehole. In rock or soil with low permeability, lag time may be significant for these instruments. This is because of the relatively large volume of water required. A device for measuring the water level in the borehole is required for these instruments. Typically, this measurement can be made using a probe, a marked cable, and a small resistance-measuring device. The probe is lowered into the borehole on a cable. When contacts in the probe encounter water in the borehole, the resistance of the electrical circuit drops, and this change in resistance can be measured. An observation well averages out water pressures over the entire hole.

- *Standpipe Piezometers*.—A standpipe piezometer (fig. 41) consists of a perforated tip sealed into a section of the borehole. A small diameter tube, or standpipe, extends through the seal, and the water level within the tube can be read with a device similar to that used for observation wells. Since the standpipes are of small diameter, several standpipes can be installed in the same borehole, allowing pressures to be read in different zones within the soil or rock. Leakage can prevent this type of installation from functioning properly. The ability to read pressures in different zones would be important if water flow were confined in certain zones within the rock mass or in areas of high gradients. A standpipe piezometer allows for measurement of water pressures in a specific interval of the borehole. This is usually more useful in understanding flow paths.

- *Closed Hydraulic Piezometer*.—A closed hydraulic piezometer is filled with de-aired water, and is capable of measuring small water pressure changes. This type of instrument will allow for pressure measurements in a specific zone within the borehole.

- *Electrically Indicating Piezometer*.—This type of device will provide an almost instantaneous response time. The deflection of a diaphragm as a result of water pressure changes is measured electrically by means of a strain gauge attached to the diaphragm. This type of instrument will allow for pressure measurements in a specific zone within the borehole.

g. *Modeling of Groundwater Flow*.—Piezometers only provide the groundwater pressure at discrete locations in the rock or soil mass. In order to get an overall picture of groundwater pressures, analyses are required to predict groundwater pressures throughout the rock mass. The flow of groundwater through a soil or rock mass can be represented graphically with a flow net. See figure 42 for an example of a flow net. Flow nets consist of flow lines and equipotential lines. Flow lines are paths followed by the groundwater when flowing through the rock mass. Equipotential lines are lines that join points where the total head is the same . A thorough discussion of the construction and calculation of flow nets is provided by Cedegren [11].

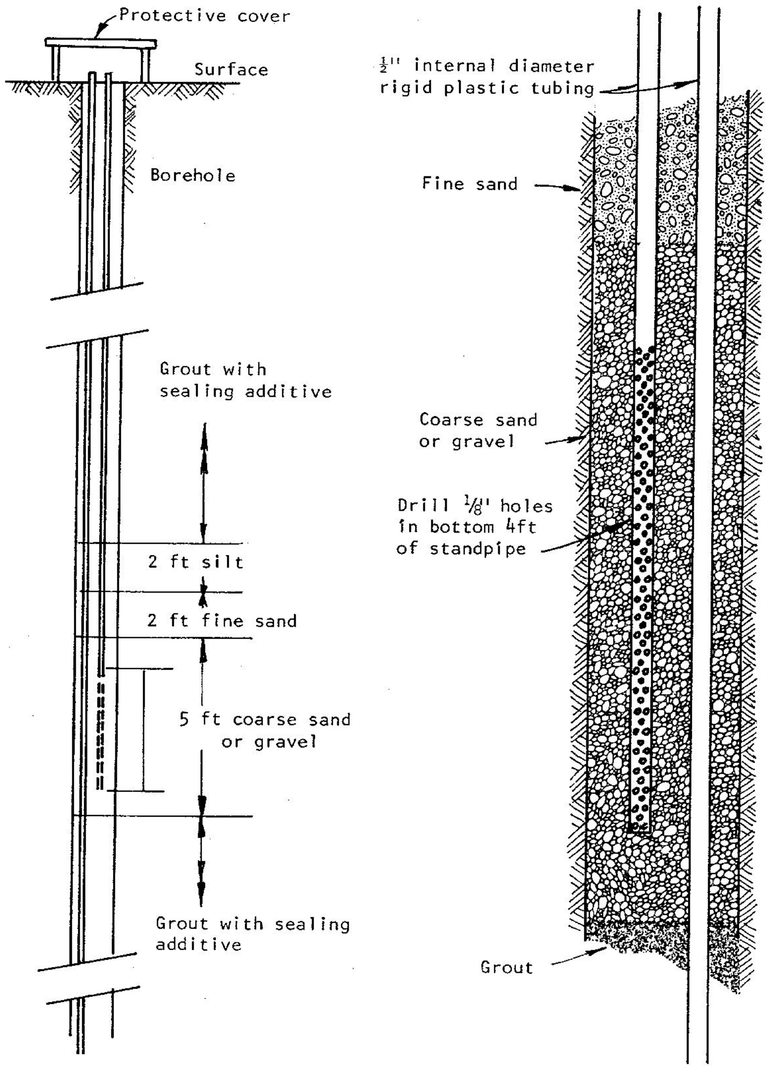


Figure 41.—Standpipe piezometer (from Hoek and Bray, fig. 59 [24]).

Computer programs are also available, which can be used to construct flow nets and evaluate groundwater flows. Calibration of the model against known responses of the actual system is necessary to develop confidence in the model. There will always be some uncertainty in the model results, due to the difficulty in comprehensively verifying the results, and in modelling geologic discontinuities.

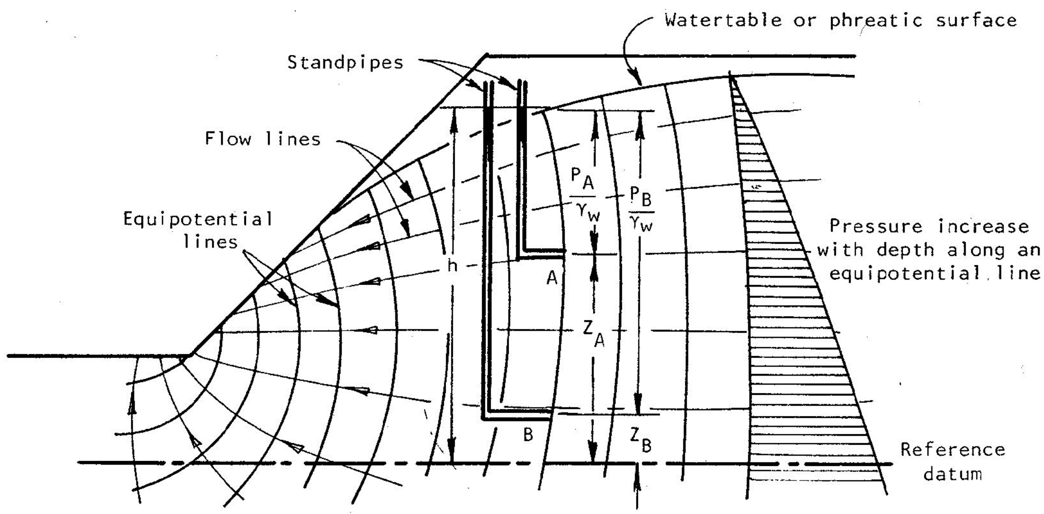


Figure 42.—Typical flow net for a rock slope (from Hoek and Bray, fig. 53 [24].

Modeling flow through fractured bedrock is difficult to do with any accuracy. Assuming equivalent porous media to represent a rock mass is only good for gross trends, and predicting pressures with this method will likely be inaccurate.

While groundwater models may have inaccuracies, they can provide a useful tool for predicting the effectiveness of drains on slope stability by comparing drained and undrained slope stability. Models are also effective in extrapolating limited information on permeabilities and water pressures to a bigger picture of groundwater flow within the slope.

h. *Depressurization.*—If the stability of a rock or soil slope needs to be improved and water pressures are significant, there are three options for improving slope stability:

1. reduce the water pressure in the slope
2. modify the geometry of the slope, or
3. reinforce the slope.

Reducing the water pressure in the slope (depressurization) is often a cost-effective method of improving slope stablility. Depressurization is effective, because it reduces the driving force on rock blocks or soil masses, and it reduces uplift pressures on sliding surfaces, increasing the shear strength and sliding resistance along these surfaces.

Depressurization is accomplished by removing water from slope materials. The amount of water that can be removed in the short term depends on the storage of water in the slope. The rate at which water can be removed from the slope depends on the hydraulic conductivity of the slope materials, k. The coefficient of consolidation, C_V, incorporates both the storage and hydraulic conductivity parameters and reflects the rate of pressure change that can be achieved.

Table 2 provides some typical field values for C_V. Table 3 provides an indication of the effectiveness of various depressurization methods as related to the coefficient of consolidation. Tables 2 and 3 are provided to give a general indication of the range over which various drainage strategies would be effective. The actual selection and design of a drainage system requires a good understanding of the site geology, groundwater conditions, and the slope geometry in relation to the above factors.

Table 2.—Coefficient of consolidation for rocks and soils

<table>
<tr><td colspan="2"></td><td colspan="4">Coefficient of consolidation (m^2/s)</td></tr>
<tr><td colspan="2"></td><td>10^{-8} to 10^{-6}</td><td>10^{-6} to 10^{-2}</td><td colspan="2">10^{-2} to 10^{2}</td></tr>
<tr><td rowspan="3">Foundation material</td><td>Soil</td><td>Clay</td><td>Silt</td><td colspan="2">Sand</td></tr>
<tr><td>Sedimentary rock</td><td>Massive clay shales</td><td>Siltstones and layered claystones</td><td colspan="2">Sandstones and layered siltstones</td></tr>
<tr><td>Igneous and metamorphic rock</td><td>Totally decomposed and gouge filled</td><td>Altered and decomposed</td><td>Weathered</td><td>Clean</td></tr>
</table>

Table 3.—Range of effectiveness of depressurization stategies

	Range of coefficient of consolidation (m^2/s) over which depressurization strategies are effective	
Depressurization strategy	Marginal	Effective
Unaided drainage	10^{-2} to 1	1 to 10^{2}
Horizontal drains	10^{-4} to 10^{-2}	10^{-2} to 10^{2}
Wells	10^{-2} to 1	1 to 10^{2}
Drainage adits	10^{-4} to 10^{-3}	10^{-3} to 10^{2}
Unloading	10^{-8} to 10^{-6}	10^{-6} to 10^{-4}

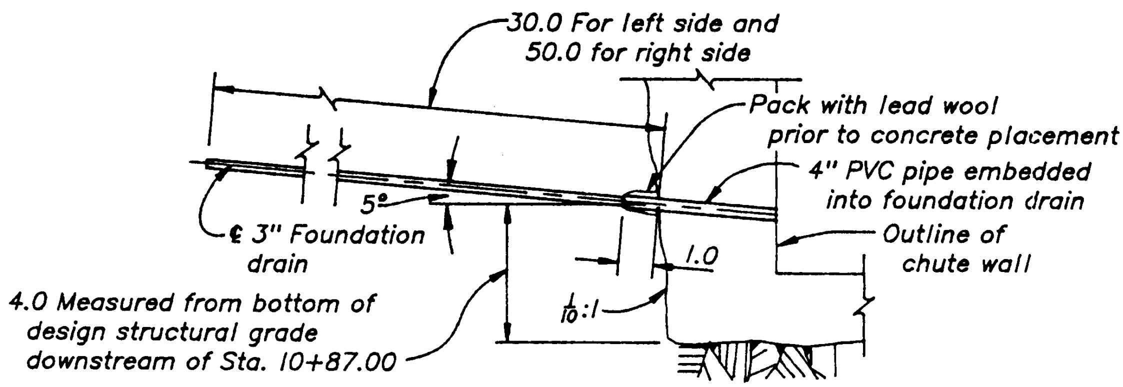


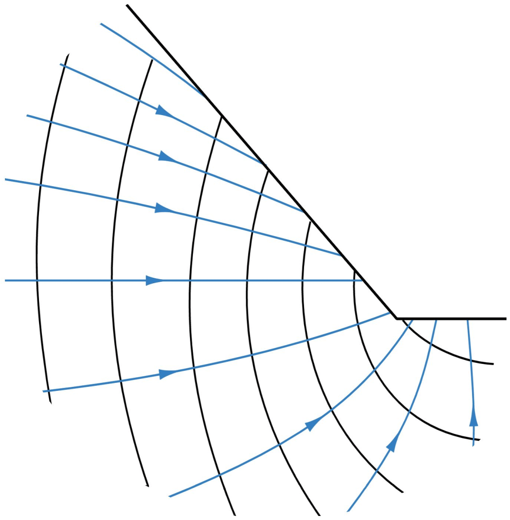
Figure 43.—Detail of the horizontal drains installed into the rock cut slopes for the spillway chute at Stewart Mountain Dam.

Figure 43 shows a detail of the horizontal drains that were installed into the rock cut slope for the auxiliary spillway chute at Stewart Mountain Dam.

2-7. Rock Tunnel Drains.—Water pressure in the rock surrounding a tunnel is not a major stability concern for unlined tunnels. In the case of underground structures, the stresses in the surrounding rock mass are typically much greater than any pressures that can be generated by groundwater, and the dangers of instability due to a reduction in effective stresses are not very significant. This is in contrast to the influence of groundwater pressures on rock slopes and foundations, where the water pressures in these features may be of the same magnitude as the stresses acting across discontinuities, which can have a significant effect on stability.

Tunnels associated with appurtenant structures are subjected to greater groundwater pressures than other types of tunnels (such as highway tunnels). Appurtenant structure tunnels are typically constructed through dam abutments, and these tunnels are near the surface, where in situ stresses may be low. The presence of a reservoir just upstream of the abutments provides a constant and high head water source. External water pressure on the outside of a tunnel is often the controlling loading condition for the design of the permanent tunnel support (often a reinforced concrete tunnel lining). The external water pressure is often assumed to be full reservoir head on the outside of the tunnel, and reinforced concrete tunnel linings are typically designed for this load. This is a conservative assumption, since the rock surrounding the tunnel and any temporary reinforcement (rock bolts, steel sets, etc.) have the capacity to carry some of this load. External water pressures during the maximum anticipated water surface should also be considered as a loading condition.

Drainage is usually provided for tunnels as a redundant feature that provides extra assurance that the permanent support system will be adequate. A typical spillway or tunnel arrangement would be an upstream pressurized tunnel, a gate chamber located at mid-length of the tunnel with guard gates and regulating gates, and a free flow downstream tunnel. Drains are not provided in the pressurized portion of the tunnel, as this would allow leakage from the tunnel into the abutment and could possibly increase the water pressure in the abutment. Drains are typically provided in the nonpressurized or free flow section of the tunnel to relieve any water pressures that could develop on the outside of the concrete lining. An example of tunnel drains is provided in chapter 1. Care must be taken when installing drains to ensure that the installation does not have adverse effects on other structures.



Chapter 3

DRAIN INSTALLATION METHODS

This chapter discusses various methods for installing drainage systems. The most common method of installing drains is drilled drainage holes (vertical, horizontal, or angled). In other cases, prefabricated drains (toe drains for dams, structure underdrains, and drains between the interface of existing and new concrete) are installed. Drilling methods and installation methods for prefabricated drains are discussed in this chapter. An important aspect of all drain installations is accurate documentation of drain locations in the form of as-built drawings. This will allow for future inspections of the drains.

3-1. Drilling Methods.—Drains are a proven, effective method for removing water from embankments, foundations, and slopes in an effort to reduce or eliminate pore pressures, uplift pressures, and ultimately, slope or structure failure. Most foundation drains are installed in rock, while many toe drains and horizontal drains are installed in weathered rock or soil. Drains must intersect discontinuities or permeable materials so that seepage and groundwater can be removed. Foundation drains and horizontal drains in rock require some type of drilling method in order to provide an avenue to relieve water pressure from a foundation.

a. *Drain Drilling Methods.*—Numerous drilling methods are used to install drains in varying geologic conditions that range from hard, unweathered rock to soft, intensely weathered rock to uncemented sediments and soils. Drilling methods commonly used for completing drains include auger and rotary. Other drilling methods, including cable tool, jetting, and reverse rotary, are available for special applications. No single drilling method is best for all conditions, because each job is site specific with its own individual characteristics. Access issues may preclude some drilling methods. Therefore, the drilling operation should use the most suitable method and equipment for the specific job, considering the access, geologic, and groundwater conditions that will be encountered and general site work conditions.

In practical terms, a suitably sized hole must be drilled by mechanically breaking or cutting rock or loosening uncemented sediments, and the broken or loosened material (cuttings) must be cleaned from the hole. The hole cleaning method, whether the cuttings are cleaned from the borehole by mechanical or fluid methods (air, water, bentonite, etc.), must be considered when selecting the drilling method for installing drains. For example, drilling with bentonite drill fluid may effectively remove cuttings from the borehole, but could also reduce the effectiveness of the borehole as a drain. After completion of drilling, a clean and open hole can be used as the drain, or perforated or slotted pipe (steel, PVC, or plastic) and/or filter material can be inserted to keep the borehole from collapsing in loose materials. Some of the more common drilling methods are listed with advantages and disadvantages in the following paragraphs. Section C-4 in appendix C also provides more information on portable drills used in restricted or confined spaces.

1. *Auger Methods*.—Augers can be inexpensive and fast in dry soils, weekly cemented soils, unconsolidated formations, and very soft rock. In optimum conditions, auger drilling can advance holes to several hundred feet deep. The auger method generally uses continuous helical flights driven by a top-drive rotary machine that mechanically carries cuttings to the ground surface. There are two basic types of augers, solid stem and hollow stem. Solid stem augers must be removed from the hole before installation of any casing or pipe in the hole, leaving the hole unsupported. Hollow stem augers have the advantage of a hollow center tube or stem. The hollow center tube supports the hole while casing or pipe is inserted. Another advantage of the hollow stem system is that when hard material is encountered that cannot be drilled with the conventional auger, it is relatively simple to convert to rotary diamond coring using the hollow stem of the auger as casing. The major disadvantage of augers is they do not perform well in hard or bouldery materials. Auger methods are usually limited to a maximum hole size of 24 to 36 inches.

2. *Rotary Methods*.—Rotary drilling methods are any form of drilling which makes a hole by turning the bit at the bottom of the hole. Rotary drilling methods include diamond coring, roller rock bits, plug bits, and can also use either a top hole hammer or a down hole hammer to drill through very firm or tough materials. Rotary methods are usually limited to a maximum size hole of 24 inches. Figures 44 and 45 show rotary drilling equipment installing drains on a dam abutment and within a drainage adit.

Rotary methods use fluid or air circulation to clear the cuttings from the drill hole. The drilling fluid can be water, bentonite "mud", man-made muds or additives (engineered water- or oil-based polymers), or air. The liquids or air travel down the interior of the drill

Figure 44.—View of rotary drilling operations to install high-angle drain holes in bedrock from a platform on the downstream right abutment of Horse Mesa Dam, Arizona. The drill rig is a Craelius Diamec 260.

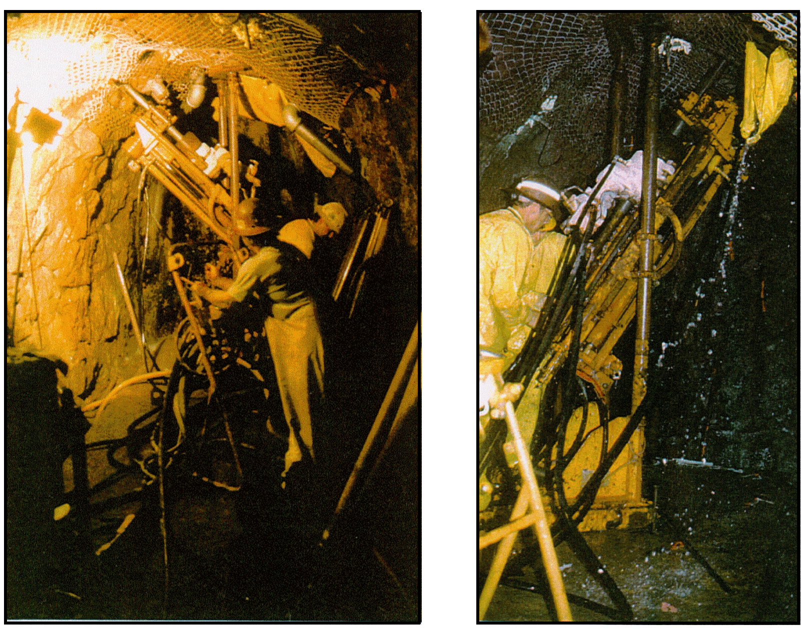


Figure 45.—Views of rotary drilling operations to install high-angle drain holes in bedrock in a drainage adit in the downstream right abutment of Horse Mesa Dam, Arizona. The drill rig is a Craelius Diamec 260. The drill is mounted on a stand when drilling high-angle drain holes.

rods, remove cuttings from the borehole and heat from the drill bit, and return to the surface via the annulus between the drill rods and borehole wall. Advantages and disadvantages of the various drill fluids have to be considered for each particular job. For example, natural or man-made muds (bentonite, polymers) can stabilize borehole walls and improve drilling conditions and drilling rates, but can also mask aquifers or intrude into an aquifer or water-bearing feature, causing damage or plugging, which defeats the purpose of the drainage. Air normally requires greater volume of circulation than fluids in order to properly cool the drill bit. Higher air volumes are accompanied by increased air pressure, which can damage formations, structures (embankment or concrete), and slopes.

Rotary methods have the advantage of relatively rapid penetration rates in all material types, usually minimal casing required during the drilling operation, and rapid mobilization and demobilization. Disadvantages of rotary methods are the high cost of equipment, high maintenance costs, use of drill fluids that may plug formations (if something other than water is needed), and high level of experience required for operators.

3. *Cable Tools*.—Cable tools are one of the oldest methods of drilling. The cable tool works by repeatedly dropping tools suspended from a cable to crush or break material into small fragments which are then mixed with water and bailed or pumped from the hole. Cables tools can achieve depths of thousands of feet. The advantages of the cable tool method is that it is inexpensive to purchase and maintain, and easy to operate. The main disadvantage of the cable tool method is that it is extremely slow.

4. *Jetting*.—There are two methods of jetting. Both methods involve a high velocity stream of water. One of these jetting methods uses a drill bit with chopping action and high pressure nozzles at the bit to clean and loosen material. Large diameter wells over 1,000 feet deep have been drilled in this manner. The second method uses only the washing action of the water jet to remove material. The second method is good only in sand for shallow depths of a few tens of feet. The advantages of jetting methods are that they are relatively easy to use and can achieve fast penetration in unconsolidated materials. The disadvantage of jetting methods is the rods often become stuck and require large hoisting equipment, and large supplies of water are required in permeable materials.

5. *Reverse Circulation*.—Reverse circulation equipment is similar to rotary rigs except the equipment is larger in order to drill larger diameter holes, and the direction of the drilling fluid is reversed when compared to rotary methods. The drilling fluid travels down the annulus between the drill rods and borehole wall, and returns to the surface via the interior of the drill rods. The advantages of the reverse circulation method are that the porosity and permeability of the formation are relatively undisturbed when compared to other drilling methods, large diameter holes can be drilled quickly, and washouts are less frequent because of the low velocity of the drill fluid. Disadvantages of the reverse

circulation method are that a large supply of drill fluid is required, equipment is expensive compared to other methods of drilling, large mud pits are required to dispose of cuttings, some sites are inaccessible because of the size of equipment, and more operators are required when compared to other methods of drilling.

b. *Additional Considerations*.—Material types, cementation, grain size, permeability, and fabric or structure (orientation and spacing of discontinuities) are major lithological factors which influence drain effectiveness. These same factors may also influence the drilling method selected to install drains. The most effective drain installations are those which have been designed and installed on the basis of the site geology.

Drilling programs for installation of drains should be carefully planned to ensure compliance with all Federal, State, and local laws, regulations, and ordinances relating to the performance of the work. All required permits, certifications, and licenses should be acquired. A plan for drilling and safety, and potential sampling and testing, should also be prepared prior to initiating work. In addition, a schedule of drilling drain holes including sequence, method, depth, angle (either in degrees from vertical or horizontal, and bearing or azimuth), and any special instructions should be prepared for the driller.

Legible, permanent copies of drilling logs and records should be maintained in a central filing system for future reference. The drilling logs should record zones of water loss, cavities, rod jerks, rough drilling and other unusual or nonordinary drilling experiences that might illuminate the nature and extent of any fracturing and abnormalities. All such records should be recorded during the actual performance of the drilling. The following minimum information should also be included on the logs or in the records for each hole:

- Hole number or designation and elevation at the top (or collar) of the hole
- Hole diameter
- Depth of bottom of hole
- Depth at which groundwater is encountered initially and when stabilized
- Depths at which drill fluid is lost and regained and amounts

Refer to the *Engineering Geology Field Manual* [31] for the format and required data for a final geologic log.

Whether a drain fails or underperforms can usually be traced to improper construction or drilling methods. An inspector with experience in drain installations should be on site during construction to ensure the proper installation of drains.

The installation of horizontal or up-angled drain holes in slopes, tunnels, or other applications requires a specialized drill rig capable of orienting and drilling in the required direction. In addition, installation of drain holes in tunnels or drainage adits may require electric, instead of hydraulic, systems to comply with environmental or safety requirements. Section C-4 in appendix C provides a discussion of portable drills used in confined or restricted spaces.

3-2. Subsurface Installation of Drains in Soils.—An example of this type of drain installation would be the installation of toe drains for embankment dams. Drains of this type are typically installed by backhoe or continuous trenching machine. The pipe for a toe drain will usually be corrugated HDPE pipe but could also be PVC pipe. A drain envelope is an integral part of the toe drain. The drain envelope prevents movement of soil particles into the drain, improves drain efficiency by providing a material surrounding the drain that is more permeable than the surrounding soil, provides a structural bedding for the drain to protect and improve the strength of the pipe, and stabilizes the soil in which the drain is placed.

The drain envelope includes bedding material and backfill for the HDPE and PVC pipe. The bedding material and the compacted material placed below the drain pipe provide structural support for these flexible pipes and ensure that the pipe retains its shape and is able to achieve its full structural capacity. The bedding material and compacted backfill on top of the pipe must be properly designed and constructed to ensure the integrity of HDPE and PVC pipes. Constriction loads and traffic loads after construction must also be carefully evaluated to ensure that the structural capacity of the drain pipe is not exceeded.

Toe drains should be inspected with a video camera upon backfilling of the trench and completion of the installation. This will ensure that the drain was properly installed and not damaged during the installation and that no obstructions exist in the drain. For drains with large amounts of backfill over the top of them, consideration should be given to an intermediate video inspection, in addition to the final video inspection. This will make it easier to correct any problems with the installed pipe.

a. *Backhoe*.—With this method, drains are constructed by excavating a trench with a backhoe, placing the pipe and envelope material and backfilling the trench. This method is referred to as "open trench" construction and an example is shown in figure 46. The use of a trench box as shown in figure 47 is highly recommended for open trench construction. It provides for a safer installation and better control of the envelope material. Care must be

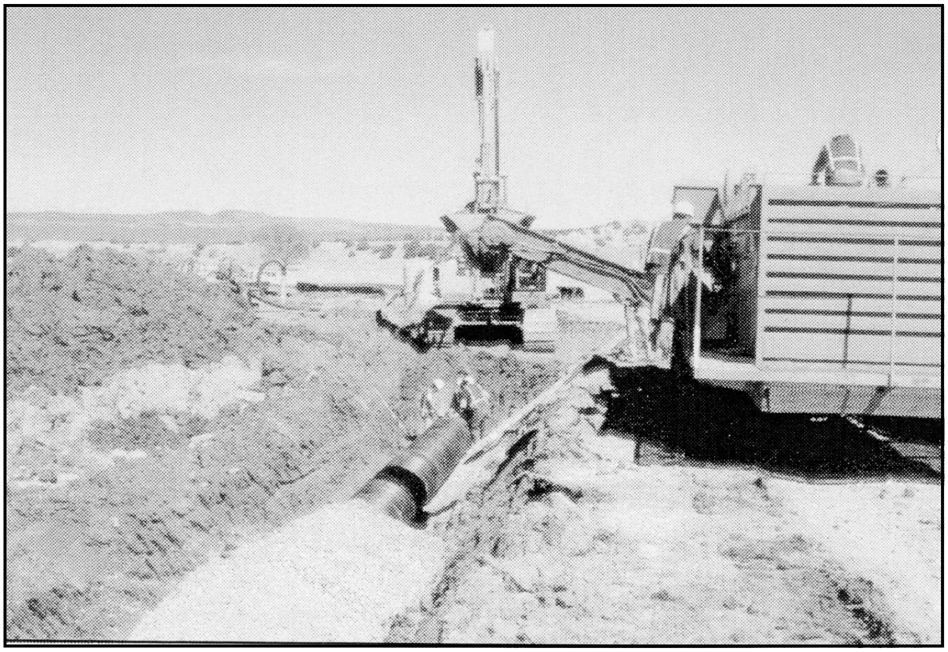


Figure 46.—Open trench excavation without a trench box.

Figure 47.—Open trench excavation with a trench box.

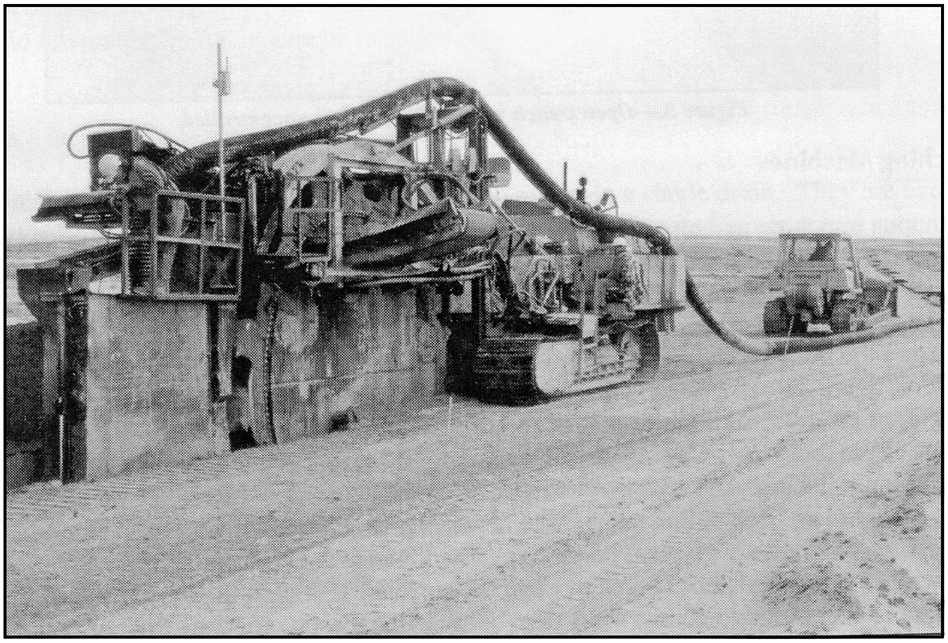


Figure 48.—Wheel trencher.

taken when moving the trench box to avoid strain or displacement of the drain pipe. Dewatering with open trench construction is not required, but is recommended if the water table is more than a few inches above the invert of the trench [32].

b. *Trenching Machines*.—Another method for installing subsurface drains in soils is the use of a continuous trenching machine, with laser plane grade control and a boot for placing the pipe and the drain envelope material. A trenching machine will excavate the trench, lay the pipe within the drain envelope and sometimes backfill the trench in one continuous operation. Trenching machines use three different methods for excavating the trench. Trenching machines can be wheel trenchers (fig. 48), ladder trenchers (fig. 49) or chain trenchers, sometimes referred to as "high speed trenchers" (fig. 50). Dewatering is usually not required because the pipe and envelope material are place only seconds after the trench is excavated. A typical boot arrangement is shown in figure 51, which allows pipe to be placed into the trench through the round chute, while the drain envelope material is fed through the surrounding opening [32].

c. *Grade Control*.—Grade control for pipe drains is important, since these drains rely on gravity flow and may have flat slopes (minimum slope of 1 ft per 100 ft for toe drains). Lightweight HDPE or PVC pipe may be difficult to install at the proper grade. Great care should be excercised during the installation/compaction of these types of drains to ensure the pipe remains on grade. Usually a laser plane is used for grade control. A revolving laser

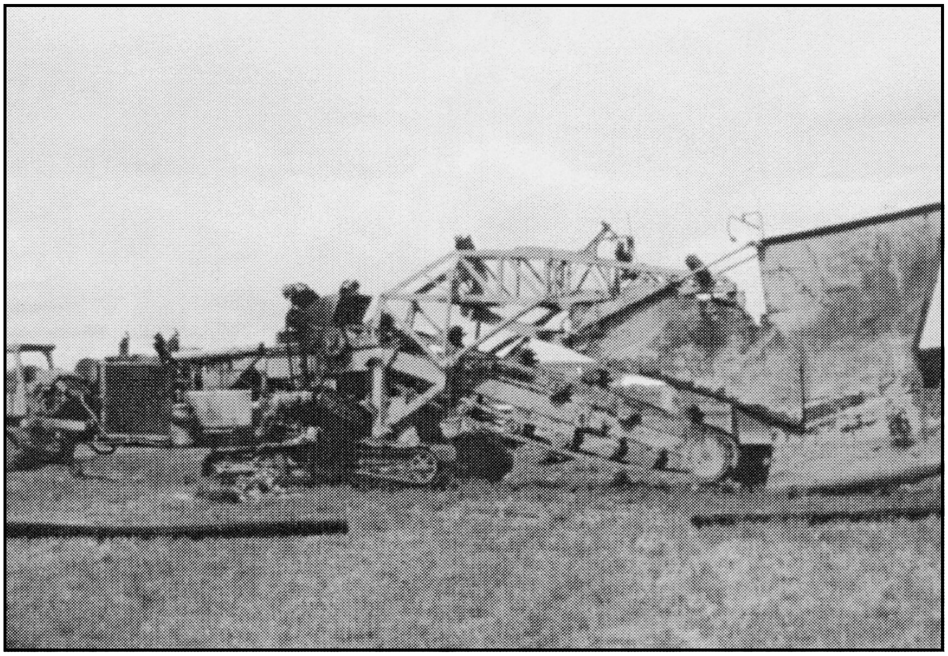


Figure 49.—Ladder trencher.

sender generates a plane of laser light over the work area. With open trench construction, hand receivers are placed on the pipe to check grade. When trenching machines are used, receivers mounted on the trenching machines follow the plane as the trencher moves forward. The receivers send commands to the hydraulic system to raise or lower the digger and the boot to keep the drain on grade. Figure 52 shows the equipment for a laser plane system.

3-3. Subsurface Installation of Drains in Rock.—An example of this type of installation would be a foundation underdrain system for a spillway or outlet works chute or stilling basin, where the foundation consists of rock. A common method of installation would be to excavate trenches in the rock for the grid of underdrains and then place the drain and drain envelope material in the trench. The concrete floor slab would then be placed on top of the foundation and the drain trenches. If the rock foundation is hard and competent, controlled blasting may be required to excavate the drain trenches. Care should be taken to prevent damage to the rock surrounding the drain trenches. If the jointing and bedding in the rock will result in significant overbreak outside of the drain trenches or if a significant area of competent rock foundation would be removed to install drain trenches, drilled drains into the rock foundation and a collector system either on top of the rock foundation or embedded in the concrete slab should be considered. An example of this type of installation can be found in chapter 2 (fig. 31).

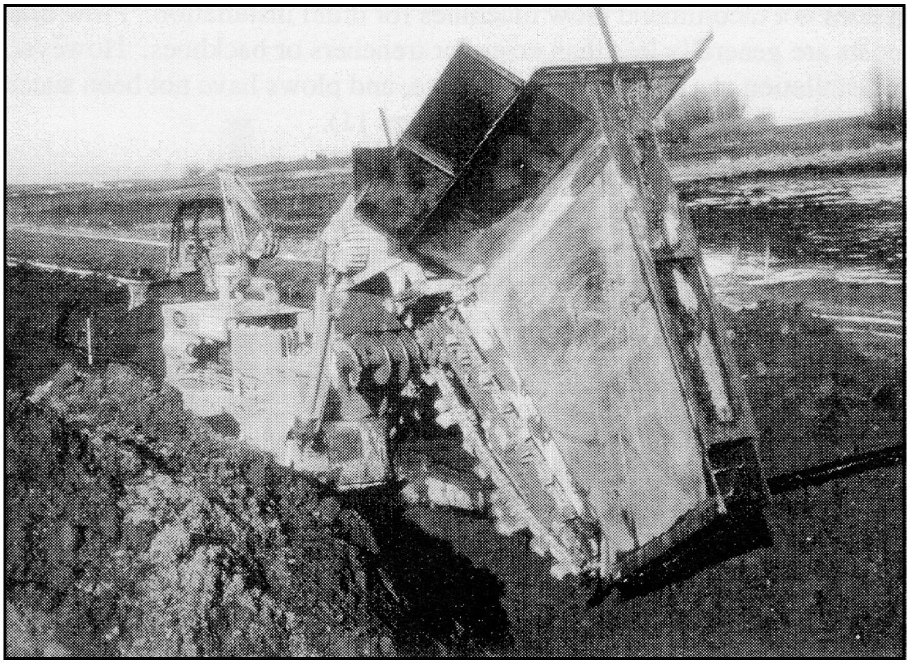


Figure 50.—Chain trencher.

3-4. Installation of Drains at Concrete Interfaces.—An example of this type of drain would be drains installed at the interface between existing concrete and a new concrete overlay. Typically a flat drain or a split round drain is used to provide drainage at this interface. Provisions must be made to prevent wet concrete from the new placement from entering the drains. Geotextile, burlap or some sort of filter fabric is effective for accomplishing this. The drains also must be anchored to the existing concrete to prevent movement of the drains during concrete placement.

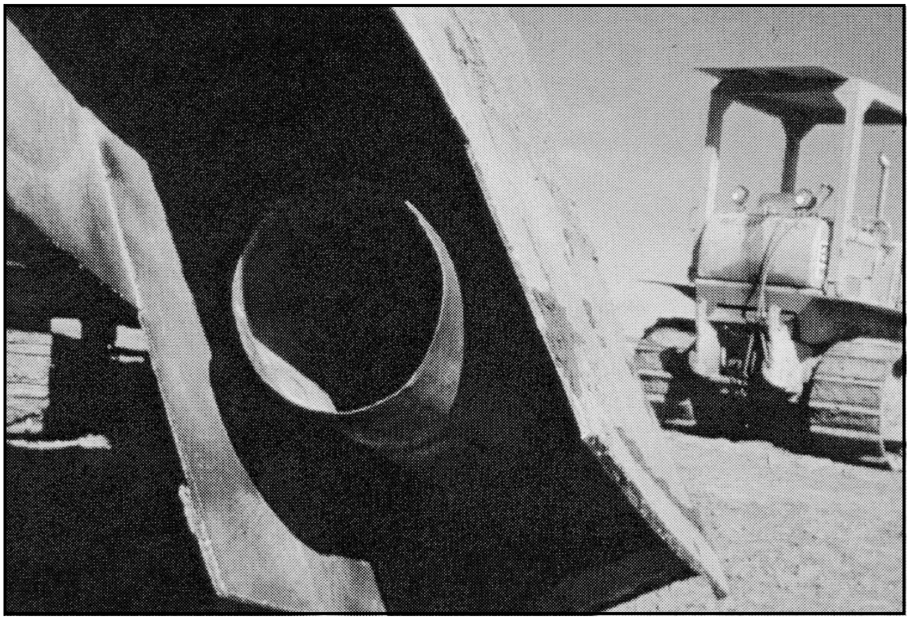


Figure 51.—Trencher boot arrangement.

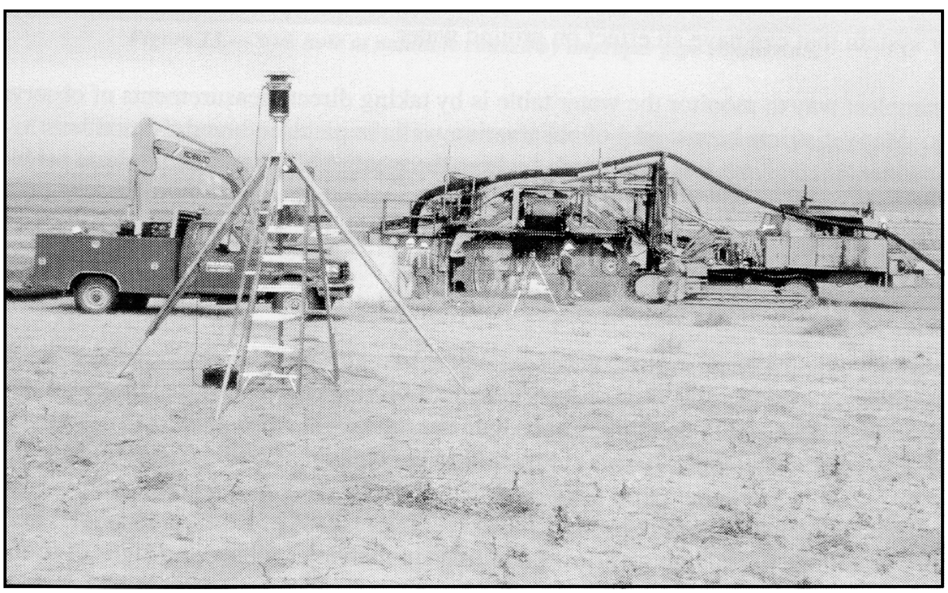
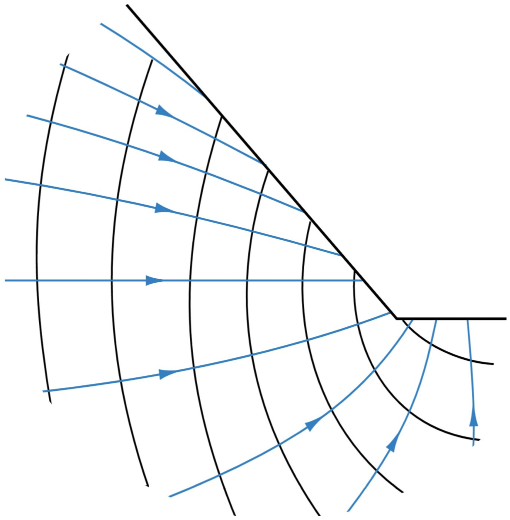


Figure 52.—Laser plane system.



Chapter 4

DRAIN PERFORMANCE

It is common for drains to experience some degree of plugging (i.e., loss of efficiency or effectiveness) during their operational life. A number of mechanisms can cause plugging of drains—calcium carbonate (probably the most common plugging mechanism at Reclamation structures), slime-producing bacteria, deposition of fines or sands in the drains, tree or plant roots, and collapse of the drain (from the collapse of either the foundation material, or the manmade materials forming the drain). These plugging mechanisms can affect a variety of drainage systems, including toe drains, drilled foundation drains in concrete dams, formed drains within the body of a concrete dam, horizontal slope drains, horizontal underdrains for spillway or outlet works structures, or relief wells. Fourteen case histories are presented in this chapter, in which drainage systems have become plugged and a drain cleaning program was initiated. The case histories were chosen to provide a representative cross section of plugging mechanisms, drain types, and cleaning methods.

Table 4 provides a quick reference for each of the case histories and its key parameters. The case histories can be used to help determine the best cleaning equipment for a given drain system.

Table 4.—Plugged/cleaned drain case histories

Case history	Drained feature	Type of drain	Plugging mechanism	Cleaning method
Brantley Dam	Concrete dam foundation	Drilled foundation drains	Iron bacteria, calcium carbonate, silt deposition	Low pressure water flushing, chopping bits, wire brush
Folsom Dam	Dam foundation	Drilled foundation drains	Calcium carbonate	Chemical treatments, mechanical abrader, high pressure water jetting
Friant Dam	Dam foundation	Drilled foundation drains	Calcium carbonate	High pressure water jetting
Grand Coulee Dam	Riverbank stabilization	Relief wells	Iron-reducing bacteria	Chemical treatments (chlorination and acid)
Horizontal Drains	Slopes	Drilled horizontal drains	Mineralization, fines deposition, roots	High pressure water jetting
Ochoco Dam	Embankment toe drain	Concrete bell and spigot toe drain	Failed pipe, sediment deposition	Video inspection; replacement
Senator Wash Dam	Embankment foundation	Relief wells	Fine sand and silt deposition	Surge block
Sherman Dam	Embankment toe drain	12-inch perforated asbestos cement pipe	Unknown	High pressure water jet (1,500-lb/in^2) sewer-cleaning tool
Upper Stillwater Dam	Dam foundation	Drilled foundation drains	Sand deposition, iron bacteria	Redrilled new drains
Concrete gravity dam	Dam foundation	Drilled foundation drains	Calcium carbonate/silt	Reaming, high pressure water jetting
Tuttle Creek Dam	Embankment foundation	Relief wells	Iron bacteria	Boiling water and chlorine

Table 4.—Plugged/cleaned drain case histories (cont'd)

Case history	Drained feature	Type of drain	Plugging mechanism	Cleaning method
Davis Creek Dam	Embankment toe drain	8-inch and 12-inch perforated corrugated polyethylene pipe	Collapsed pipe; sedimet deposition	Replacement; high pressure water jetting; video inspections
Keechelus Dam	Embankment toe drain	Rock drain with outfall pipes	Sediment and gravel	Water jetting; video inspection; installation of new toe drain with filter
Summary of toe drain inspections	Embankment toe drains	Various materials	Various mechanisms	Video inspection

4-1. Brantley Dam—Foundation Drain Cleaning.—

a. *Abstract*.—Brantley Dam, located on the Pecos River in New Mexico, has experienced an increase in uplift pressures under block 2 since 1992. The increases in pressure are approximately 8 to 10 percent. These unexpected uplift pressure increases resulted in the initiation of a program to evaluate the foundation drains at the dam to determine the cause of the increased foundation pressures. Several drain holes were selected as test holes to be evaluated as a precursor to an extensive drain cleaning program. Each test drain hole was inspected with a borehole camera before and after cleaning. The drain cleaning used a variety of methods—low pressure water flushing, chopping bits, wire brush, and combinations of these methods. During this program, three possible drain-plugging mechanisms were identified—biological fouling, mineral incrustation, and siltation. This test program led to a proposal by the Albuquerque Area Office to try several different cleaning methods to maintain all the drain holes at Brantley Dam.

b. *Background*.—Brantley Dam, the major feature of the Brantley Project, has operated satisfactorily since its construction completion in early 1988. It is located on the Pecos River, 15 miles upstream of Carlsbad, New Mexico. The reservoir is a replacement for McMillan reservoir, which was located 5.5 miles upstream.

Brantley Dam is a composite structure consisting of a zoned earthfill embankment with a concrete gravity midsection (fig. 53). The crest length is 20,850 feet, including the 760-foot long concrete gravity section. The crest elevation of the embankment section is 3308.0 feet; the crest elevation of the concrete gravity section is 3308.5 feet. The crest width of the

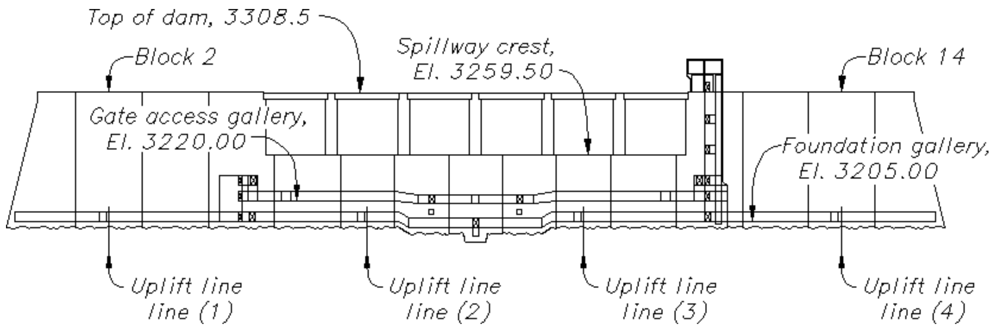


Figure 53.—Profile of the concrete portion of Brantley Dam showing the approximate location of the uplift pressure monitoring lines.

earthfill embankment is 24 feet; the crest width of the concrete gravity section is 15 feet. The dam impounds a reservoir with a total capacity of 381,748 acre-feet at the top of exclusive flood control (El. 3283.0), including 106,950 acre-feet allowance of 100-year sediment deposition. The spillway for Brantley Dam consists of six 50-foot-wide by 25-foot-high radial gates with a total ogee crest length of 300 feet and a 52-foot radius roller-bucket stilling basin that is common to both the spillway and the outlet works. The outlet works is a high pressure gate-controlled structure.

The foundation of Brantley Dam consists of sound and competent layers of dolomite, sandstone, and siltstone. Excavation for the concrete section was about 30 feet into firm foundation. A 30-foot thickness of consolidation grouting was placed beneath the concrete section.

The foundation drains are "NX" sized (2.97-inch diameter) core holes angled approximately 5 to 10 degrees from vertical in the downstream direction from the foundation gallery of the concrete portion of the dam. The top 5 feet of the drain holes is within 3½-inch diameter steel pipe embedded in the concrete; the remainder of the hole is completed uncased in the foundation material. In general, the holes are about 50 feet deep.

c. *Drain Cleaning History*.—Review of Brantley Dam uplift pressure pipe installations indicated that by 1995, the dam was experiencing a slight but constant increase (about 8%) of uplift pressures in block 2 and to a lesser extent in block 14 (2%). Figure 54 illustrates the typical layout of uplift pressure monitoring lines under blocks 2 and 14. The increase in block 2 appears to have begun in 1992. The uplift pressures beneath block 2 were increasing at approximately 2 to 3 percent a year (fig. 55).

The rise in uplift pressures prompted a program of cleaning the foundation drains in 1994. The drain cleaning program consisted of flushing and probing of the foundation drains using

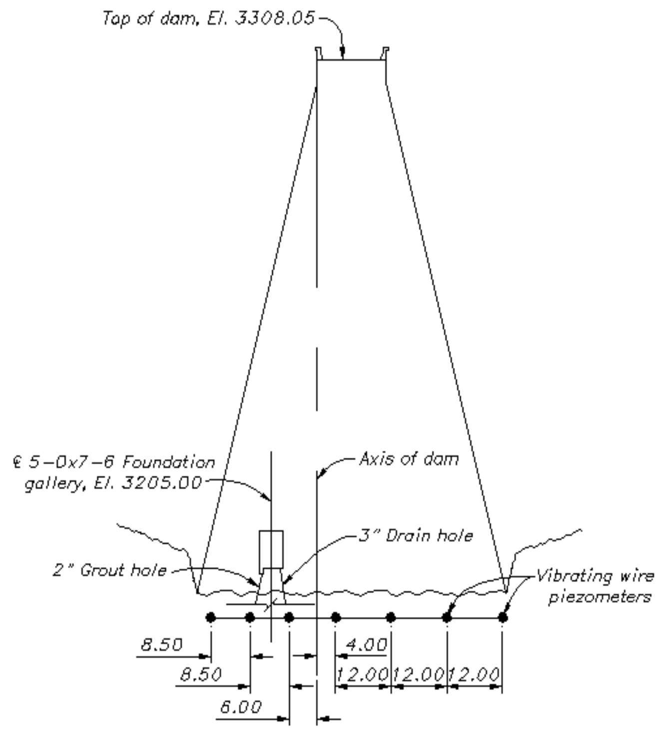


Figure 54.—Typical layout of the uplift pressure monitoring lines under blocks 2 and 14.

a 1-inch diameter PVC pipe and a low head centrifugal pump, rated at 41 gal/min and powered by an electric motor. The PVC pipe was inserted to the bottom of the drain, and water from the gallery gutter was circulated through the drain until discharge cleared, which took from 15 to 20 minutes. Higher pressure methods, such as a jetting tool, were not employed because of the friable nature of the foundation material. This was borne out when some of the drains were video taped with a borehole camera, which showed foundation drain hole erosion.

During the flushing operations, it was noticed that some holes were partially blocked by a white precipitate, later tested to be predominantly calcium and magnesium carbonate with some sulfates also present. These holes were cleaned with a chopping blade mounted on metal pipe. Also present in the foundation drain holes was iron bacteria, which were removed by flushing. This cleaning program provided some relief for the uplift pressures, but only for a short time, as documented by the reduction of uplift pressures. However, the reduced uplift pressures only lasted a short time.

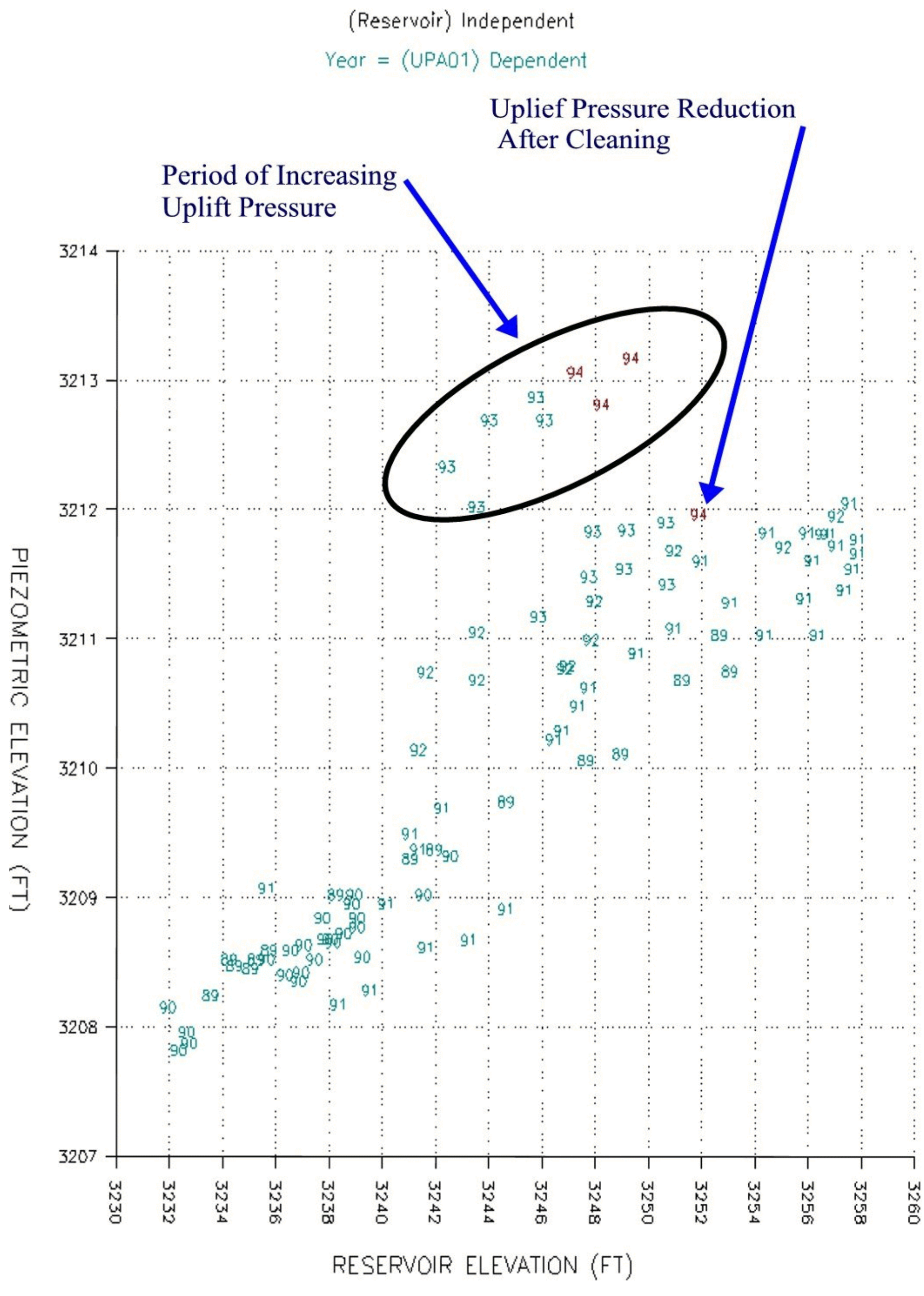


Figure 55.—Scatter plot of uplift pressures under block 2 between January 1990 and March 1994.

Because of the short-lived success of the flushing method to keep the majority of the drains open, a field team consisting of personnel from the Bureau of Reclamation and Carlsbad Irrigation District was assembled in February 1996 to evaluate the foundation drains at Brantley Dam. Down hole video inspection was determined to be the first step toward determining the condition of the foundation drains.

The following 12 foundation drains were selected as part of the video inspection program:

Block	Drain number(s)
1	81
2	71, 74, 75, 76
3	67, 69, 70
4	64
6	62
9	34
11	26

Drain number 62 at station 88+65 in block 6 was designated as the "control" foundation drain because of an 8-percent decrease in uplift pressures in block 6 since 1989. Prior to the video inspection by the borehole camera, the selected foundations drains were cleaned by flushing and probing. Drain 69 had a floating mat of iron-related bacteria that had to be skimmed away prior to video inspection.

Listed in table 5 are the foundation drains in the order in which they were inspected, along with depth reached.

The video inspection identified three possible processes that could contribute to the decline of drain effectiveness—biological fouling, mineral incrustation, and siltation:

- *Biological Fouling.*—Iron-related bacteria were observed in all the foundation drains inspected with the camera. The species *Gallionella ferruginea* was confirmed to be present in foundation drain water samples by the New Mexico Department of Health's scientific laboratory. This genus is commonly found in iron-bearing waters and soils, and its growth causes problems in water wells. *Gallionella* is a stalked iron bacterium characterized by a small bean-shaped cell with a long slender twisted stalk. It was once thought that the stalks were entirely inorganic extrusions of ferric hydrate; current research indicates the stalk may contain protein to which ferric hydrate is bound [33].

  The video inspection showed iron bacteria in all the foundation drains in seemingly equal concentrations with perhaps the greatest accumulations in drain 34, block 9. Using

Table 5.—Foundation drains inspected at Brantley Dam.

Foundation drain number	Station	Block	Approx. drain depth (ft)	Camera total depth (ft)
81	87+09	1	49.2	37.8[2]
76	87+55	2	48.7	48.6
75	87+65	2	49.9	49.7
74	87+75	2	50.7	50.7
71	88+05	2	53.3	40.3[2]
70	88+10	3	49.2	37.4[2]
69	88+20	3	48.8	48.7
67	88+40	3	51.8	51.6
64	88+65	4	49.2	49.2
62	88+85	6	49.7	49.5
34	91+50	9	50.2	50.1
26	92+20	11	50.4	50.1
71	88+05	2	53.3	53.0[3]
74	87+75	2	50.7	50.1[4]

[1] Approximate total depth of hole. Numerous depths for the same hole are given on the cleaning reports. Some of the depths are acknowledged to be off "by a coupling length or two." The depths given here are either rounded "eyeball" averages or the value that most closely agrees with the depth determined with the camera.

[2] Camera stopped by mineral incrustation (deposits) in hole.

[3] Hole cleaned 2/21/96 with the single blade (e - by 2½-inch) sharpened chopping bit on a mixed string of 1-inch and ¾-inch diameter by 5-foot long joints of galvanized steel pipe, followed by about 30 minutes of washing.

[4] Inspection with radial view lens. Actual depth was 50.1 feet. Mirror motor occupies lower 0.6 ft of hole.

the flushing method to clean the drain improved drain effectiveness where iron bacteria were the predominant clogging mechanism.

The method of flushing drains, in which the wash pipe is moved from drain to drain, as well as the manner in which the drains are plumbed into a common gallery gutter suggest that iron bacteria have been able to develop throughout the drainage system.

- *Mineral Incrustation*.—The camera could not be lowered to the bottom of the hole in drains 81 (block 1), 71 (block 2), or 70 (block 3) due to mineral incrustation (deposits) that occluded most in the borehole. These deposits are probably combinations of calcium and magnesium carbonates and calcium sulfate [34]. X-ray diffraction techniques are needed to determine the specific mineral. The remaining opening was only sufficient to allow passage of the 1-inch diameter PVC wash pipe. These blockages and others that the camera was able to pass by correspond closely to “plugs” or “partial plugs” reported on the cleaning report. An unsuccessful attempt was made to clear the plug in drain 71 using a 2½-inch diameter flue brush on d-inch fiberglass rods. The blockage was then chopped out with the single blade, steel chopping bit on steel pipe. From the cleaning report, “plugs” were not exclusively a problem in block 2.

- *Siltation*.—Using the borehole camera’s radial lens revealed that fine grained debris was building up in the horizontal joints/fractures in drain 74. Since this was the only hole this lens was used in, the extent of this type of buildup is not known. These deposits may be the result of the flushing process. Siltation is not considered to be a viable explanation of decreased drain performance.

d. *Conclusion*.**—**Experience gained by the field team in evaluating the foundation drain holes at Brantley Dam has provided some suggestions for future cleaning programs. The field team has made a general suggestion for cleaning drains for each of the specific types of plugging processes:

- *Biological Fouling*.—Disinfecting the drains with one of the chlorine compounds is an effective means of destroying a wide variety of bacteria. Acid treatment is effective to dissolve the ferric hydrate deposited by the organisms. It should be noted that disinfecting with both chlorine and acid compounds provides only temporary relief from fouling. There are also environmental considerations when using these types of chemicals.

- *Mineral Incrustation*.—Mechanical cleaning by chopping, or alternatively, drilling can keep the borehole clean. If these deposits are adjacent to an active groundwater flow zone (likely) and invade the formation (likely), opening the borehole will do little to restore drain performance. Acid treatment is effective to dissolve deposits in the drain hole and, to a limited extent, out into the formation material. It should be noted that acid treatment may not be practical for all locations because of environmental restrictions.

- *Siltation*.—More energy is needed in the flushing process to keep material from settling out of the water column. Top to bottom agitation with a jetting tool on a hose that directs wash water out into the formation, followed by pumping from the drains should remove the sediment. Since the drains may produce very little water during the pumping phase, a means of providing (disinfected) make-up water at the top of the hole may be needed.

This case history demonstrates the need to establish a foundation drain cleaning program to maintain the effectiveness of foundation drains. The cleaning program initiated at this site showed an immediate reduction of uplift pressures after cleaning. The inspection of selected foundation drains indicated that the buildup of deposits from biological fouling, mineral incrustation, and siltation led to the increase in foundation pressures under block 2. Although three processes were identified, none of the three are conclusive causes for the increase in uplift pressures. It appears that the primary cause was from the buildup of deposits from biological fouling and mineral incrustation. As described above, the drain cleaning program reduced uplift pressures at Brantley Dam for only a short period of time, which demonstrates the need to periodically maintain the foundation drains.

4-2. Folsom Dam—Foundation Drain Cleaning.—

a. *Abstract*.—Folsom Dam, located on the American River about 20 miles northeast of Sacramento, California, has a concrete gravity section flanked by earth embankment wing dams. The concrete gravity dam has a structural height of 340 feet and a crest length of 1,400 feet. A single line of 3½-inch diameter foundation drains (139 drains total) is provided in the gravity section to relieve foundation uplift pressures. The foundation drains are affected by calcium carbonate deposition within the drains and have been cleaned several times. In 1983, the foundation drains were cleaned with high pressure water jetting (pressure and flow rates unknown). In 1985, an unsuccessful attempt was made to clean several foundation drains by chemical removal of calcium carbonate deposits. In 1987, several foundation drains were cleaned with ultrahigh pressure water jetting (up to 36,000 lb/in^2, 0.6 gal/min). Also in 1987, a mechanical abrader (Roto-Rooter) was used to clean one foundation drain. In 1988, several foundation drains were cleaned with high pressure water jetting (up to 10,000 lb/in^2, 20 gal/min). Cleaning of selected foundation drains continued periodically through the early 1990s using a 3,000-lb/in^2 water blaster that was not able to effectively remove the calcium carbonate.

b. *Background*.—Folsom Dam is a Reclamation structure located on the American River in the Central Valley of California about 20 miles northeast of Sacramento. Folsom Dam consists of a concrete gravity section across the river channel flanked by earth embankment wing dams extending from the concrete section to high ground on either side of the river. Bedrock at Folsom Dam is quartz diorite, a hard granite-like rock.

Folsom Dam construction was completed in 1956. The concrete gravity dam section has a structural height of 340 feet and a crest length of 1400 feet. A single line of 3½-inch diameter foundation drains (139 drains total) is provided in the gravity section to relieve foundation uplift pressures. The drains range in depth from 11 to 150 feet and average 125 feet. The foundation drains were drilled on 10-foot spacings to relieve uplift pressures that may develop under the dam and potentially cause stability problems. Since construction of the dam, the foundation drains have experienced a gradual loss of effectiveness through deposition of calcium carbonate within the drain holes, inducing increased uplift pressures.

c. *Drain Cleaning History*.—In 1978, the foundation drains at Folsom Dam showed signs of normal seepage. By 1980, the examination report suggested that the foundation drain and discharge pipes be probed and cleaned where plugged, and efforts to probe and clean the foundation drains were initiated. The 1983 examination report stated that work on the drain probing and cleaning recommendation was incomplete but that partial work had been done and would continue until finished. Drain cleaning efforts continued at Folsom Dam with the chronology of drain cleaning events listed below:

1. *High Pressure Fluid Jet*.—The Industrial Hydropower Company of Placerville, California volunteered to demonstrate their high pressure water-blasting system on the 3½-inch diameter drain holes at Folsom Dam in April 1983. They demonstrated its effectiveness by removing some exposed hard calcium carbonate within the upper portion of the drain holes.

In May 1983, Industrial Hydropower returned to Folsom Dam for a more extensive demonstration, cleaning two drain holes selected by Reclamation. Each of the holes was inspected before and after the demonstration with a borehole camera. The demonstrations showed this equipment can access tens of feet into drain holes and is capable of removing hard calcium carbonate deposits.

2. *Chemical Removal of Carbonate Deposits*.—In 1985, Reclamation studied the potential for chemical removal of hard, thick calcium carbonate deposits that had reduced casing and drain hole diameters. Granular and pelletized forms of sulfamic acid were applied to a sampling of drains in quantities equivalent to 2 to 8 percent of the unobstructed drain volumes. An immediate vigorous reaction was observed at the drain opening, when the granular form of the acid reached the point of obstruction. (Obstructions near the upper ends of the drains were detected by probing.) The pellets dissolved slowly, providing acidification at the bottom of the hole over an extended period.

Follow-up inspection of the drains indicated no evidence of improvement in drain function. An odor was detected in and around the treated areas, and due to concern for safety of personnel, acid treatments were discontinued. Possibilities for the sulfamic acid treatment to dissolve the calcium carbonate obstruction remain, but the problem of gases generated needs to be studied. In addition, environmental issues of introducing acid into the groundwater table may preclude the use of acid treatment. The sulfamic acid treatment may be best utilized as a deterrent to calcium carbonate buildup on a preventative maintenance basis.

3. *Ultrahigh Pressure Fluid Jet*.—In 1987, Power Master, Inc., demonstrated an ultrahigh pressure (UHP) water jet method of cleaning the foundation drains at Folsom Dam. The contractor, using a flow rate of 0.6 gallons per minute and nozzle configurations of 45°, 30°, and 20° had little success penetrating a hard calcium carbonate plug located at a depth of 35 feet. The contractor also tried a nozzle tip with jets designed to cut through the center of the plug. This tip also failed to show satisfactory success. The contractor then increased discharge pressure to an estimated 36,000 pounds per square inch with a 60° nozzle tip. This tip cut through approximately six feet of the calcium carbonate plug. A borehole camera lowered into the drains showed satisfactory cleaning. However, the low flow rate would not flush the cuttings from the jet cleaning from the hole, leading to eventual plugging of the drain.

The equipment used to clean the foundation drain holes required 440-volt power source, was very bulky, hard to maneuver, and prone to breakdown. Modifications to the 1987 vintage equipment are required to adapt the UHP cleaning equipment to clean foundation drains.

4. *Roto-Rooter*.—In 1987 a local Roto-Rooter franchise demonstrated the use of an electrically driven, rotary, interior pipe cleaner to break though a plugged foundation drain using a variety of cutting edges. The drain hole was plugged from 16- to 25- and 40- to 53-foot depths. The borehole was opened to 129 feet in six to seven hours. Flow rate from the drain hole increased from no flow to 1.6 gallons per hour. The foundation drain hole was inspected with a borehole camera and found to be free of calcium carbonate.

5. *High Pressure Fluid Jet*.—In 1988, Donco Industries, Inc., demonstrated high pressure fluid jet methods. The equipment was most effective at a working pressure of 10,000 lb/in^2 and a flow rate of 20 gal/min. System pressure losses were 150 lb/in^2 per 50 feet of ½-inch-inside-diameter (I.D.) supply hose and a loss of 3,300 lb/in^2 for 25 feet of ¼-inch-I.D. flexible, nylon steel, lance hose. The heads available for use were:

- Seven-sixteenths-inch flexible lance, with 25 feet of ¼-inch-I.D. nylon steel hose, with one hole straight forward and 18 holes pointing forward 30°

- One-half-inch molehead, with 5-foot long, ½-inch-I.D. steel shaft, and one hole straight forward, three holes at 45° forward, and three holes at 35° aft

- Two-inch molehead, with 5-foot long, ½-inch-I.D. steel shaft, and several different nozzles that could be arranged as needed

- Two-and-one-half-inch rotating molehead, with 5-foot long, ½-inch-I.D. steel shaft, one hole straight forward, two holes at 45° forward, and two holes at 45° aft

- Three-inch diameter carbide bit with high pressure water jets protruding forward, adding 5-foot shafts as necessary to advance the hose and cutter head to required depths

A ½-inch-I.D., 30,000-lb/in^2 capacity hose was used to convey flow from the pump. As a safety feature, a dump-load device with a foot pedal was used to regulate pressure to the molehead or lance.

In drain hole 12-D-4, a solid calcium carbonate plug was encountered from 50 to 80 feet, and cleaning was continued 130 feet deep. After cleaning, the drain hole was flushed with water. Inspection with a borehole camera showed approximately 60 percent of the borehole circumference was clean. The contractor then used the 2½-inch rotating molehead to reclean between 50 and 60 feet deep in 5 minutes. A recheck with the borehole camera showed no significant change.

The contractor then used the 2-inch-diameter molehead to reclean between 50 and 60 feet deep. A recheck with the borehole camera at 55-feet-depth where the cleaning was concentrated showed calcium carbonate on 30 to 40 percent of the borehole wall. The other 60 to 70 percent of the borehole wall was clean.

In drain hole 12-D-5, a solid calcium carbonate plug was encountered 14 feet deep. Using the flexible lance, the contractor attempted to cut through the plug for five minutes with no success. An attempt to break through the plug with the 2½-inch rotating molehead was also unsuccessful. The contractor switched to the ½-inch molehead and penetrated the plug in a few minutes; the plug was only a few feet in length. The contractor then used the flexible lance to clean the drain hole between 17 and 140 feet deep in 17 minutes. While cleaning drain hole 12-D-5, the contractor did not rotate the lance. The borehole was then inspected with the borehole camera. Streaks were present, indicating the lance should be rotated to ensure complete removal of deposits.

6. *Other Cleaning Methods.*—Other cleaning methods used by Reclamation O&M staff at Folsom Dam include installing an inflatable packer at the collar, and bubbling carbon dioxide into drain hole 12-D-1. The pressure on the carbon dioxide could not be maintained and the carbon dioxide escaped through interconnected discontinuities into adjacent drain holes. An additional cleaning method includes using a 2½-inch diameter, pointed brass nozzle (that looks similar to a plumb bob) with holes in the side that induce

the nozzle to spin. The plugged drain holes were initially opened by dropping a 5-foot length of ¾-inch diameter steel pipe attached to a hose onto the plug in an attempt to break the plug. The rotating nozzle was then lowered into the drain to flush cuttings from the hole. Soft plugs (bacterial) are adequately removed using this method.

d. *Conclusions*.—The most effective method to clean obstructed foundation drain holes at Folsom Dam was a high pressure fluid (water) jet. High pressure fluid jet equipment and methods improved through the 1980s. The improvements included downsizing of equipment, increasing reliability of equipment, improving power sources, and improvements in nozzle configurations, jet orifice attack angles, and rates of lowering and raising the jet in the drain holes. High pressure fluid jetting demonstrated its effectiveness by removing hard calcium carbonate within the drain holes, improving drainage flows, and reducing uplift pressures.

While the Roto-Rooter method was successful, other methods used by Reclamation to clean foundation drain holes either failed or were only marginally effective.

Chemical removal of calcium carbonate obstruction that reduces drain hole effectiveness is not proven effective. However, chemicals may be best utilized as a deterrent to buildup on a preventative maintenance basis if environmentally acceptable.

4-3. Friant Dam—Foundation Drain Cleaning.—

a. *Background*.—Friant Dam is a Reclamation concrete gravity dam, located approximately 25 miles northeast of Fresno, California on the San Joaquin River. The dam, which was completed in 1942, has a structural height of 319 feet and a crest length of about 3,500 feet. Formed drains within the dam, as well as foundation drains are provided at Friant Dam to relieve uplift pressures. Five-inch diameter formed drains are provided at 10-foot centers within the dam. Five-inch diameter foundation drains at 10-foot centers are provided with depths into the foundation from 12 to 100 feet. Figures 56 and 57 provide the layout and locations of the foundation drains. The foundation drains discharge into the grouting and drainage gallery, which is located in the lower portion of the dam. Seepage is collected and measured from blocks 21-28 (SM-Area 1), 29-34 (SM-Area 2), 34-49 (SM-Area 3) and 49-57 (SM-Area 4). A V-notch weir is used to measure the flow in the drainage channel at each of the four collection points. The locations of the seepage weirs are shown on figure 58.

The foundation drains have become plugged, as indicated by reduced depths that a probe could be inserted into the drain holes and by increased measured uplift pressures within the

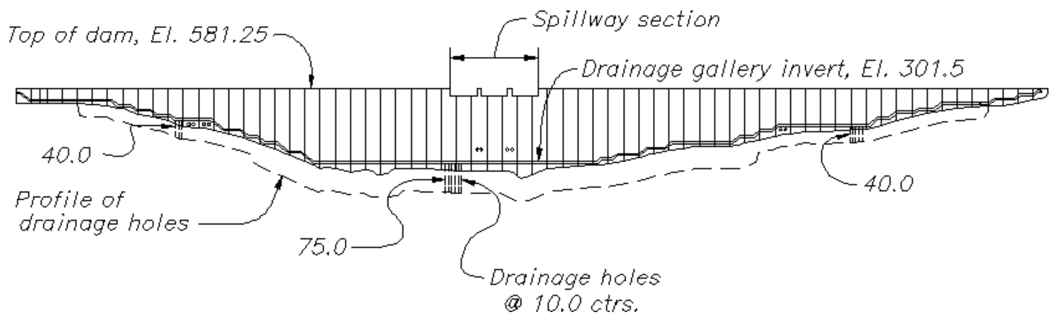


Figure 56.—Profile of Friant Dam and drains.

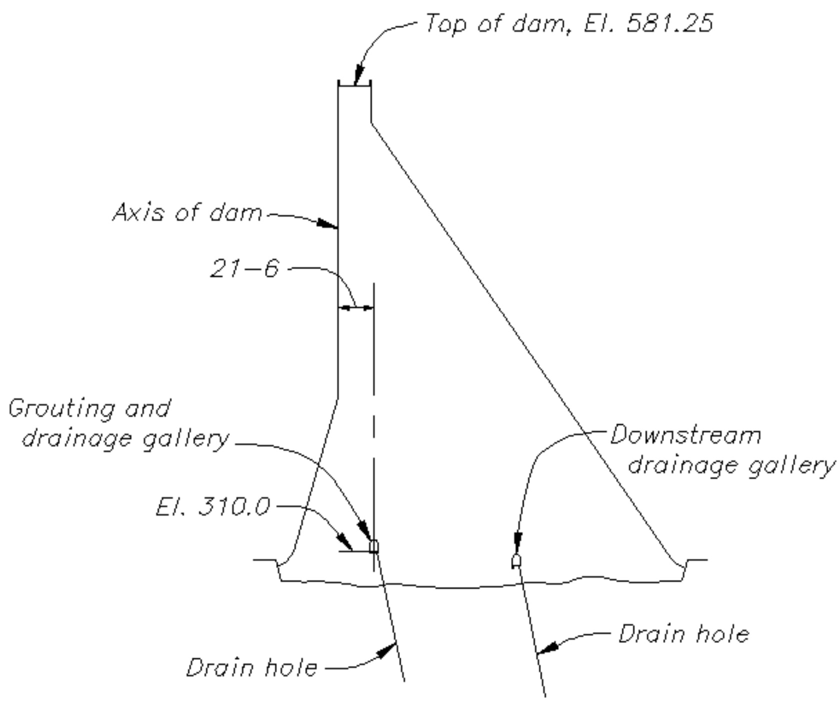


Figure 57.—Cross section of Friant Dam.

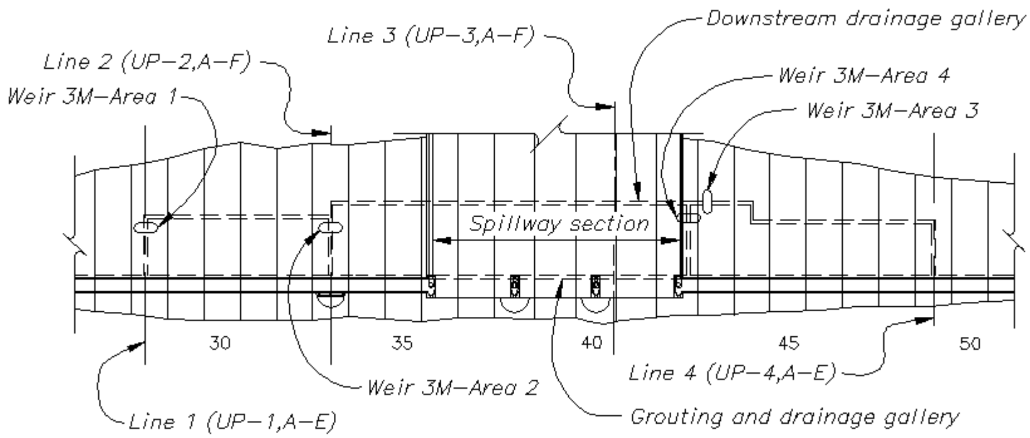


Figure 58.—Plan view of Friant Dam.

dam foundation. During the period of 1987 through 1994, seepage flow values, as measured at weirs in the foundation galleries, decreased dramatically and were very low. The plugging mechanism was determined to be calcium carbonate because of the formation of hard white deposits at the outlet end of the drains. Four lines (oriented in the upstream/downstream direction, five to six pressure pipes per line) of uplift pressure pipes are provided to monitor foundation pressures at the base of the dam. The locations of the uplift pressure lines and pipes are shown on figures 58 and 59. Only one of the lines (line 4) has been responsive to changes in the reservoir elevation, and this line has indicated uplift pressures greater than the assumed design uplift pressures. Figure 60 shows the results of uplift pressure readings on line 1 since 1982, and shows no response to reservoir level or any change over the last 17 years. Lines 2 and 3 show similar results to line 1. All three of these lines showed little or no response to reservoir levels from the time of their original installation. The holes for the uplift pressure pipes were not drilled very deeply into the foundation (only about 3 feet), and it is believed that they may not intersect any discontinuities in the dam foundation, and as a result, do not reflect actual uplift pressures in the foundation. Uplift pressures in lines 1, 2, and 3 have historically been very low and have shown very little response to changes in reservoir levels throughout their history.

Uplift pressures in the Friant Dam foundation are a concern, since measured uplift pressures (along line 4) are greater than the uplift pressures assumed for design. The original uplift assumed during design was full reservoir head at the upstream heel of the dam, one-third of the difference between the reservoir and tailwater head at the drain location, and the tailwater head at the downstream toe of the dam. The uplift was assumed to vary linearly between the three points described above. Monitoring uplift pressures is important. In the

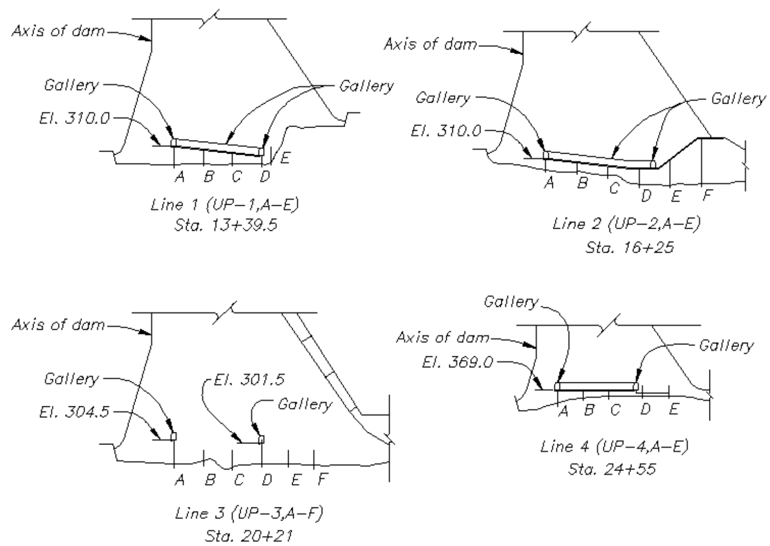


Figure 59.—Profile of Friant Dam and drains.

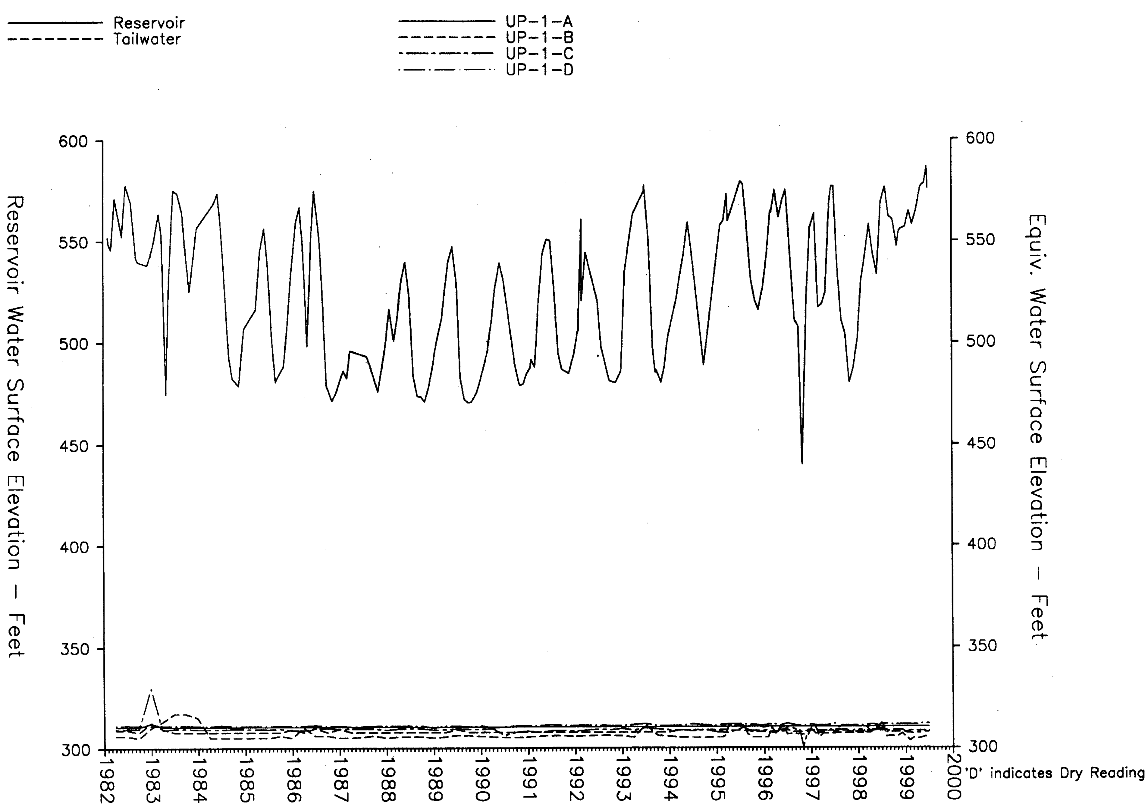


Figure 60.—Friant Dam uplift pressures for line 1, sta. 03+69.5, block 28.

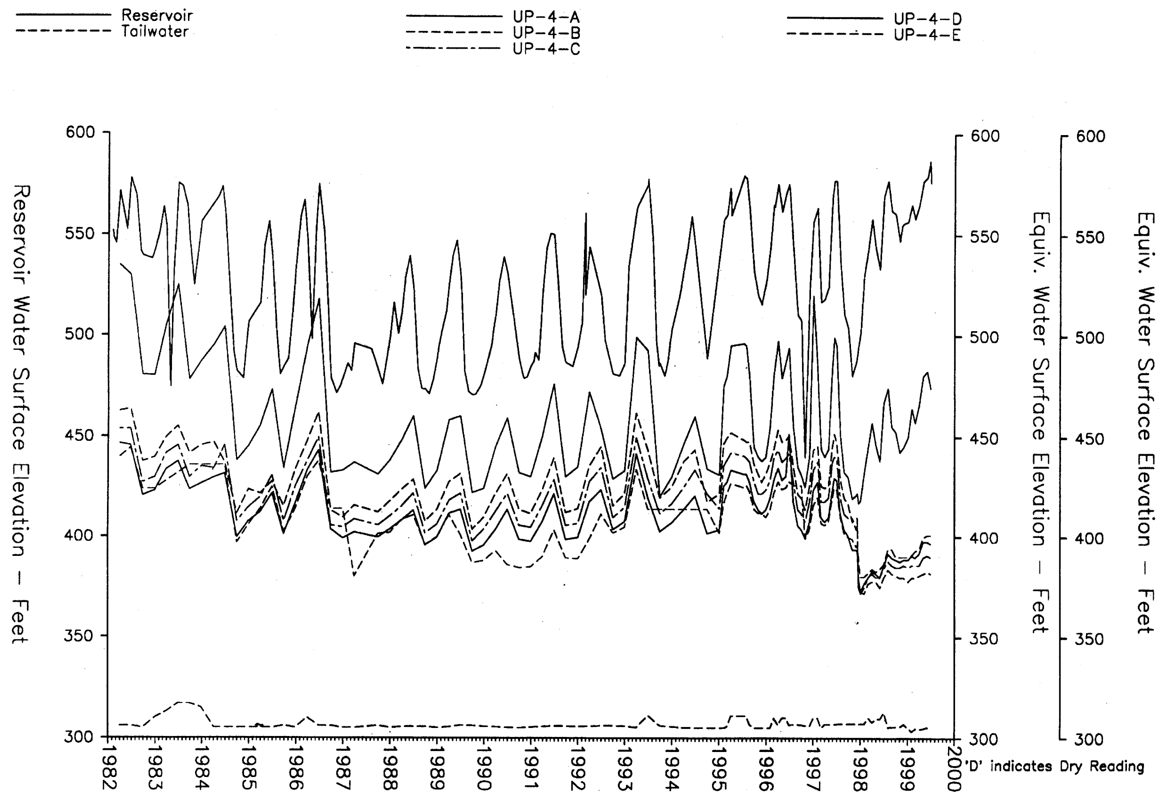


Figure 61.—Friant Dam uplift pressures for line 4, sta. 24+55, block 49.

1998 Performance Parameters for Friant Dam, it was recommended that uplift pressure pipes on lines 1, 2 and 3 be reestablished in an attempt to determine actual uplift pressures in the dam foundation over a wider area than that provided by line 4 only.

b. *Drain Cleaning History*.—In 1993, a contract was issued to clean the foundation drains in blocks 47 through 54, using high pressure water jetting techniques (pressures up to 10,000 lb/in^2 were used). Results of the cleaning effort indicated that most of the drain holes were opened to greater depths (based on probing the holes before and after cleaning), although the pressure gauge readings for line 4 do not reflect an improvement in drainage (see fig. 61 for uplift pressure readings on line 4). However, it was also noted that depths measured after flushing with air and water after cleaning were less than the depths measured by the contractor during cleaning of the drain holes. It was speculated that the water-blasting tool punched through obstructions (made a small hole) but did not fully remove them.

In December 1997, the foundation drains were cleaned again using high pressure water jetting. One hundred ninety eight drains were cleaned in blocks 27 through 65. A 4-inch diameter plumb bob was passed through each drain before and after cleaning to confirm that an acceptable degree of cleaning had been accomplished. In addition, uplift pressures along line 4 were recorded before and after drain cleaning.

A 20,000-lb/in^2, 17-gal/min hydroblast pump was used as the power unit for the 1997 drain cleaning operations. The cleaning tool was a speed-governed reaction-jet rotating mole with a proprietary head design. The mole with its rotating head and diverging nozzles is capable of cutting through solid blockages and scrubbing the walls clean at the same time. A portable derrick and electric winch was used to control raising and lowering the mole within the drain holes. A 4-inch centralizing cable support system was also provided for the mole.

The results indicate that the drain cleaning was successful. Prior to cleaning the foundation drains, very little water was flowing in the drainage channels of the drainage gallery. After cleaning, water was flowing at a much higher rate through the channels (based on a visual assessment). Seepage readings were not made immediately after the cleaning, because the contractor was working in drainage areas 3 and 4 and also working on the sump in the gallery. Instrumentation readings shortly after the cleaning show an increase in seepage flows, but the reservoir level was also rising. Pressures were recorded at the uplift pressure pipes in line 4 before and after the cleaning. Table 6 summarizes the readings, which indicate the cleaning was very effective in reducing uplift pressures, at least in the portion of the dam foundation near line 4.

Table 6.—Results of 1997 drain cleaning, uplift pressures—line 4.

Location		Pressure head (ft) 12/9/97	Pressure head (ft) 1/2/98
Line 4, sta. 24+55, block 49	A	52	48
	B	26	10
	C	36	2
	D	23	3
	E	40	3

The uplift pressure profile from the January 1998 readings matches closely with the original design assumption—some points are slightly above the design profile and some points are slightly below.

Hole depths before and after the drain cleaning indicated that the hole depths increased for all but one foundation drain hole (block 58, No. 3, 21-7). Table 7 provides the hole depths before and after cleaning for selected drain holes, indicating that the drain cleaning was effective.

Table 7.—Results of 1997 drain cleaning, probed depths of drains.

Block No.	Drain No.	Depth before 1997 cleaning (ft)	Depth after 1997 cleaning, feet
32us	5	37	90
33us	1	49	90
33us	5	55	86
35us	2	66	83
39us	4	18	73
39us	6	29	84
43us	2	36	86
45us	4	60	89
48us	4	67	90
48us	5	67	94
50	1	13	71
50	2	12	71
58	2	37	80

Although the uplift pressure data is only at one location in the dam foundation, other evidence, including increased depth of probed holes and visual indication of increased seepage, indicates that the cleaning was effective in increasing the efficiency of the drains across the dam foundation. Figure 61 provides a plot of uplift pressure readings along line 4 and indicates that the uplift pressures stayed at a reduced level after the 1997 cleaning.

Foundation seepage data since the 1997 drain cleaning is somewhat puzzling (see fig. 62). The total drain flows measured at the four weirs have been reduced significantly since the cleaning (contrary to the initial visual reports that the drain flows increased immediately after cleaning). The expectation would be that drain flows would increase with cleaning.

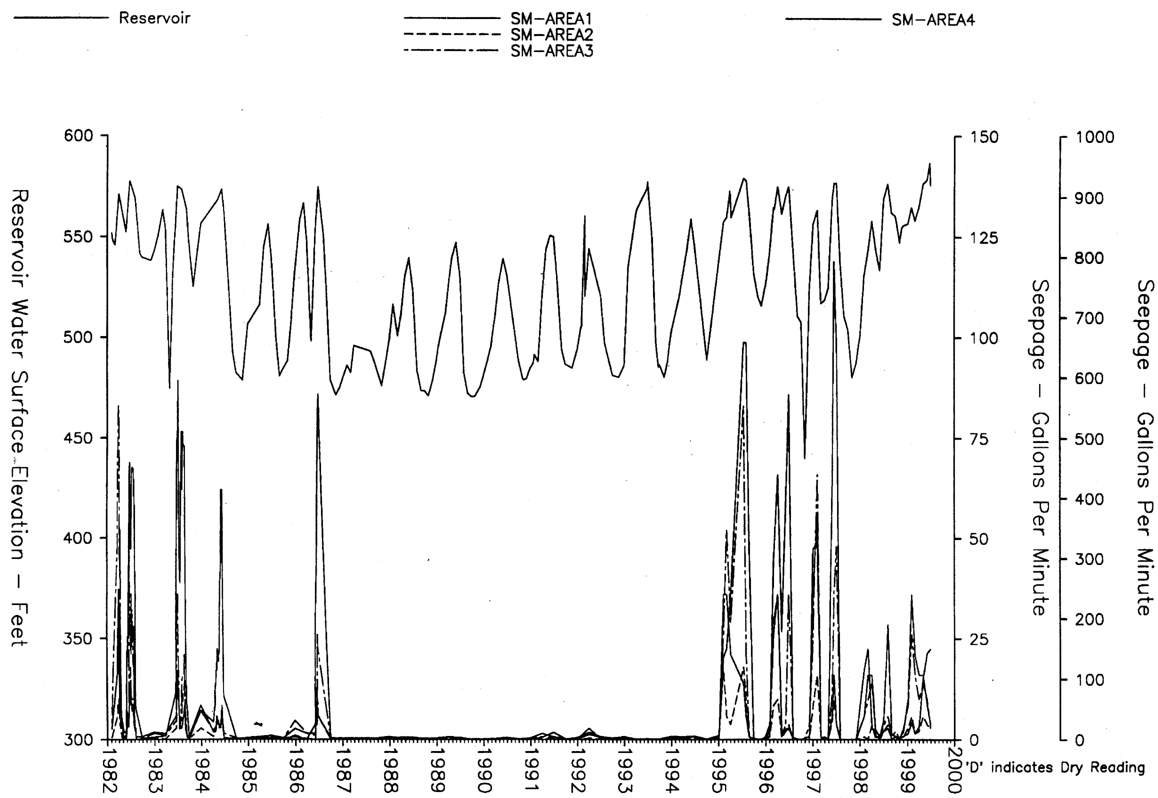


Figure 62.—Friant Dam drain flows.

c. *Conclusions.*—The drain cleaning performed in 1997 was the second drain cleaning effort at Friant Dam, the first being in 1993. While the 1993 cleaning only showed a slight reduction in uplift pressures along line 4 (see fig. 61), the 1997 results showed a significant reduction in uplift pressures. An improvement in cleaning methods from 1993 to 1997 was the cleaning of the holes to their original diameters as opposed to just opening a path through the plugged portions of the drains.

The importance of instrumentation to verify the effectiveness of drains and their ability to reduce uplift pressures was also demonstrated at Friant Dam. Only one line of uplift pressure pipes appears to provide valid foundation uplift pressures. Since this line only provides information over a limited portion of the dam foundation and since the data along this line has indicated uplift pressures which exceed design uplift pressures, plans have been made to reestablish uplift pressure pipes at other locations in the dam foundation.

4-4. Grand Coulee Riverbank Stabilization Well Rehabilitation for Iron-Reducing Bacteria.—About 45 pumped wells were installed in the mid 1980s as part of a stabilization process along the riverbanks of the Columbia River in a 6-mile reach downstream of Grand Coulee Dam. The wells were installed to reduce the groundwater level in the vicinity of the riverbanks. Other features of the stabilization effort included large diameter shafts with horizontal drains. The riverbank slopes were also reshaped and covered with armor riprap to reduce erosion.

Iron-reducing bacteria infested the relief wells, which clogged the screens as well as the gravel pack surrounding the screens. Personnel from the Grand Coulee Project Office developed procedures for rehabilitating the screens. These procedures evolved over a number of years based on the project's experiences with the wells. Project personnel found that an extreme change in the pH of the well water, which is achieved by an acid treatment followed by a chlorine treatment, is more effective in killing the bacteria than using only a chlorine solution or only an acid solution.

Each well is equipped with a water meter, which records the well discharge. When a significant decrease in discharge rate occurs, the wells are rehabilitated using the procedures outlined in this case history.

a. *Results*.—This method is very effective in restoring the production rate in wells with iron-reducing bacteria encrustation. Figures 63 through 65 are plots of well discharge. These figures show the big jump in well production that occurred after the well cleaning procedure. Well production prior to the cleaning procedure was typically about 50 percent of the production after the procedures. In the case of drain well No. 33, the production rate prior to the cleaning procedure was only 30 percent of the rate after the procedure.

Many types of iron-reducing bacteria clog wells, with varying degrees of encrustation. The procedures described here have been effective at Grand Coulee. It may be necessary, through trial and error, to alter the concentrations at other sites to get the best results. Personnel at Grand Coulee recommended using the procedure frequently, rather than waiting until production rate has dropped off by 50 percent or more.

b. *Well Cleaning Procedure*.—

1. *Initial Preparation*.—

1. The well depth, design, screen size, and static water level need to be determined so that the proper amounts of chemicals can be calculated.

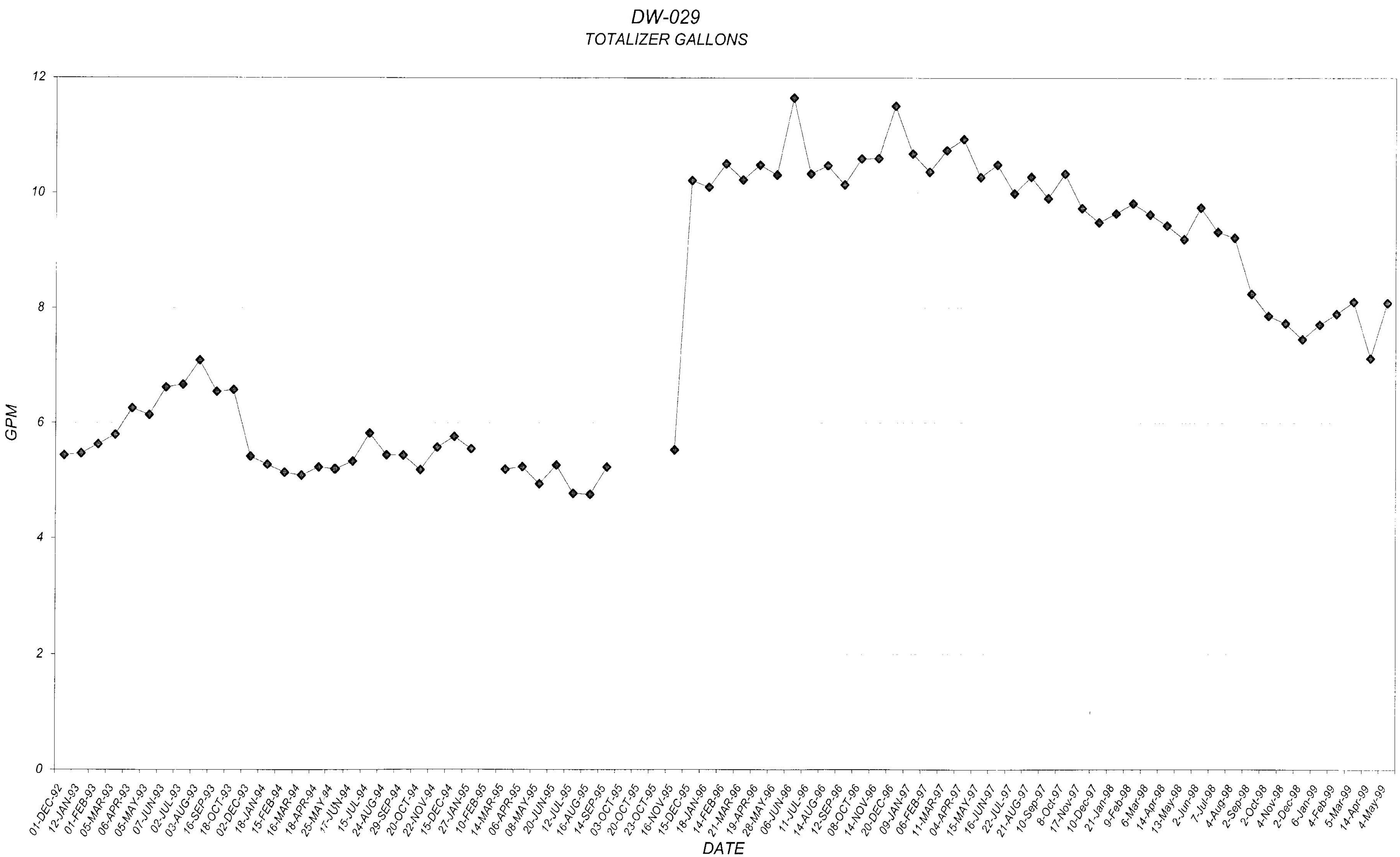


Figure 63.—Discharge from drain well No. 29, showing a large increase in production after cleaning.

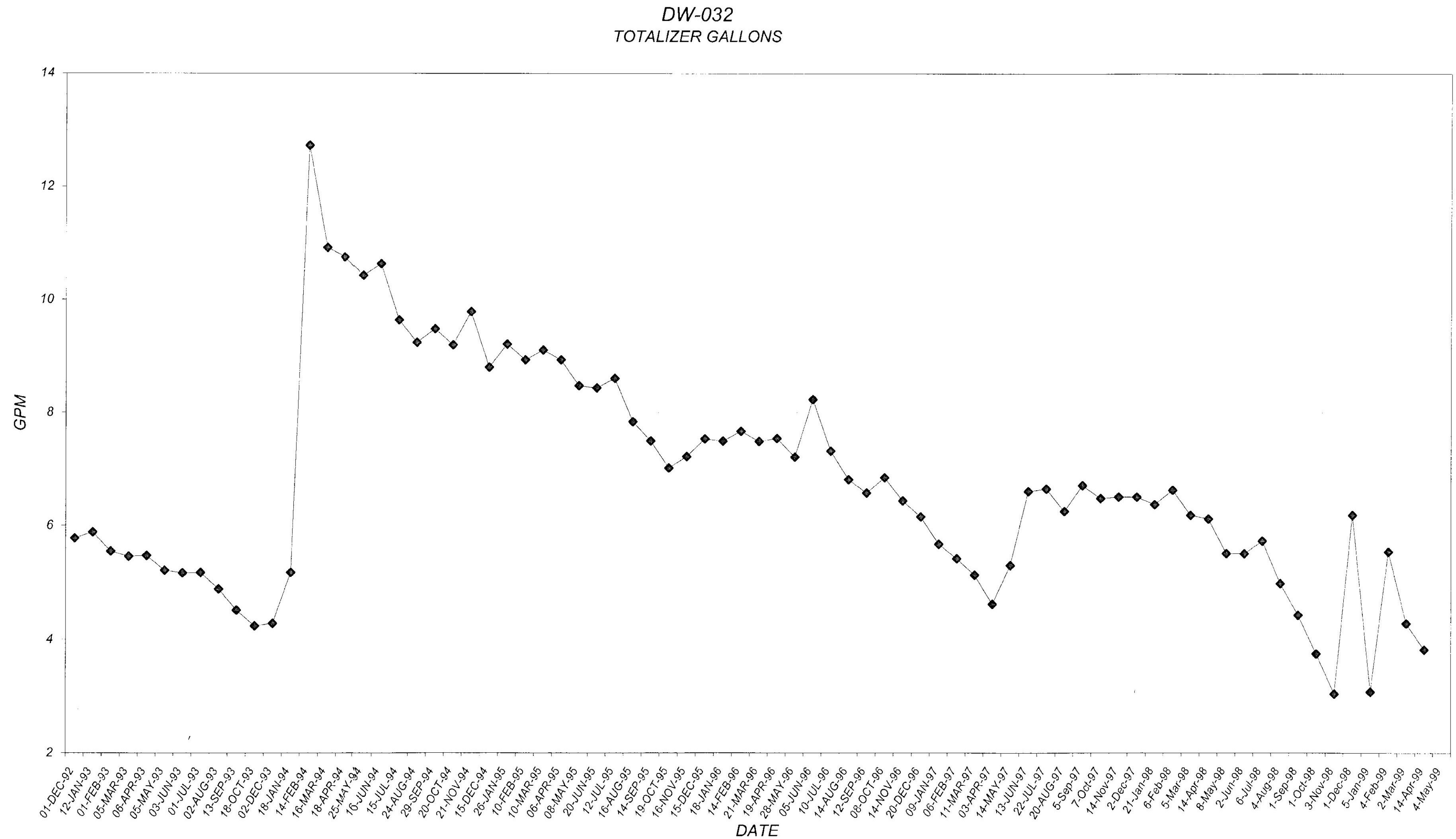


Figure 64.—Discharge from drain well No. 32, showing a large increase in production after cleaning.

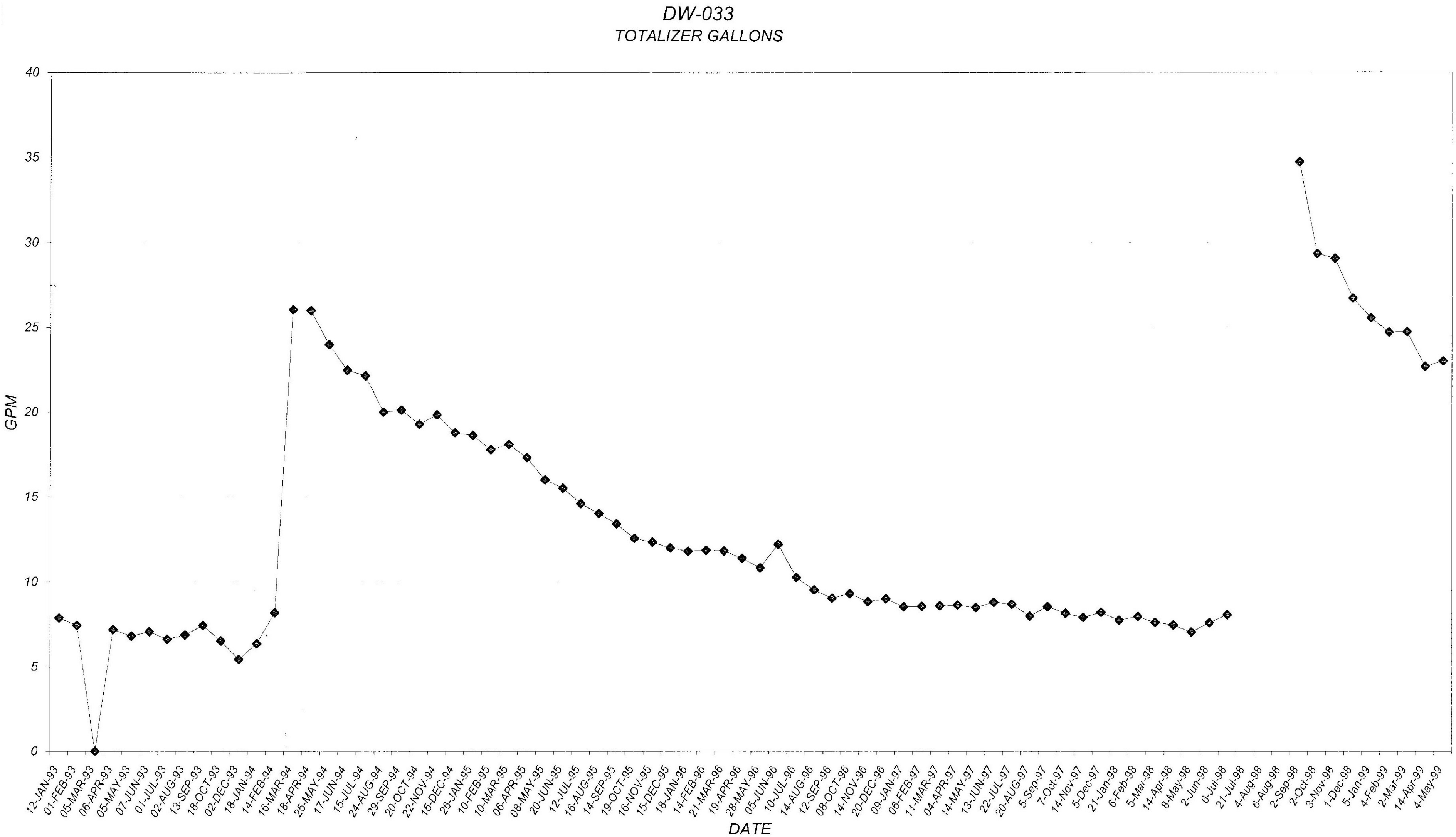


Figure 65.—Discharge from drain well No. 33, showing a large increase in production after cleaning.

2. Remove the well pump. For effective cleaning, any inner screen or casing should also be removed to allow chemicals and the mechanical cleaner to be in contact with the outer screen.

3. Air lift the well for about 2 hours to remove debris and suspended particles. An air lift is simply a pipe, with a diameter of usually two inches or less, that has a compressed air line attached in a manner that will allow air bubbles to travel up the inside of the air lift pipe. The bubbling action of the compressed air creates an upward flow of water in the pipe, and the air lift acts like a vacuum cleaner.

4. Near the end of the 2-hour air lift process, take a water sample and test the pH of the well water. Also, measure and record approximately how much water the well is producing with the air lift system. This information will be needed later.

5. After the air lift process is complete, the entire length of the well screen and the solid casing in the vicinity of the pump intake should be video taped using a down hole camera. The air lift process will improve the clarity of the well water, which benefits the video taping. The type of treatment required to rehabilitate the well will be determined from the video information.

2. *Chlorination Treatment* .—Both the chlorination and the acid treatment require handling of hazardous chemicals. Special precautions and protective equipment are required. Safety considerations are summarized in *Personal Safety Procedures* (p. 119). The safety requirements could vary from case to case; therefore, the procedures for each job should be reviewed by qualified safety personnel.

1. This procedure should be started in the morning to allow a full day of chlorine treatment.

2. Chlorine should be added to the well as required to bring the concentration in the well to about 1,000 ppm of chlorine by weight. Protective clothing must be worn while working with the chlorine. Approximately 1 gallon of water will dissolve 1 pound of calcium hypochlorite pellets/granules. The chlorine solution should be mixed in a plastic or steel container, **not** a galvanized container. The granules take about 30 minutes to go into solution, and after the solution is mixed, it must be gently agitated to keep the chlorine in suspension while the mix is pumped into the well.

3. The chlorine solution is pumped into the well from the bottom up. The solution is dispersed evenly through the entire water column with the use of a tremie pipe. A plastic or black iron pipe should be used for the tremie operation.

4. After the chlorine solution has been evenly dispersed in the well water column, it is gently agitated and surged with a surge block (see the case history on Senator Wash Dam on page 134 for a description of a surge block and how it is operated). Starting at the top and working to the bottom, the surge block is operated in 5-foot intervals. The surging action will gently force the chlorine through the well screen and out into the sand/gravel pack. After the surging operation is complete, the chlorine solution remains in the well overnight.

5. After the chlorine solution has been in the well for about 24 hours, the surge block is once again operated in 5-foot intervals, starting at the top of the well screen. After surging each interval for approximately 10 minutes, the air lift is used to remove the water and debris from the well. The surge and air lift process is repeated several times until the well water discharging from the air lifts clear. Then the surge block is moved down to the next 5-foot interval. Periodically, the surge block should be lifted out of the well to inspect the rubber or leather discs (whichever type is used on the surge block). It is important to replace worn discs, because if they wear down to the wooden or metal block, the well screen could be damaged.

6. Although there should be very little residual chlorine left at this point, caution should be taken to not discharge the well water directly to any water body or near woody plants or trees. If the waste water cannot be discharged into an area adjacent to the well in an environmentally safe manner, pump the well discharge into a tank and transport to a suitable site.

3. *Acid Treatment*.—It is important that all of the chlorine has been removed from the well before beginning the acid treatment. The acid treatment requires handling of hazardous chemicals. As a result, special precautions and protective equipment are required. Safety considerations are summarized in the section on *Personal Safety Procedures* (p. 119). However, the safety requirements could vary from case to case; therefore, the procedures for each job should be reviewed by qualified safety personnel.

1. A product called Nu-Well has been used to prepare the acid solution used to rehabilitate the riverbank stabilization relief wells downstream of Grand Coulee Dam. Nu-Well is a blend of sulfamic and amidosulfonic acid. The acid treatment should be started at the beginning of the shift to allow sufficient time for the chemicals to dissolve and to allow for several hours of surging before the shift ends.

2. The procedures for the acid treatment will depend on the amount and type of encrustation. The video tape which was taken during the initial well preparation should reveal the amount of encrustation. For treatment of a slightly encrusted screen or soft bacteria slime, the Nu-Well by itself should reduce the pH in the well to 5 or less. Five

percent of Nu-Well by weight of water in the well is added to the well. Only the weight of the water in the screened portion of the relief well should be used to calculate the weight of the Nu-Well to be added. In addition to Nu-Well, rock salt should also be used when the encrustation is hard and is believed to be of iron or manganese origin (rustlike deposits on screen). The encrustation in the Grand Coulee wells is commonly of iron origin. The rock salt should be added to the well at a rate of 2 pounds of rock salt for every 10 pounds of Nu-Well. Trial and error is sometimes required. If the process is not effective, the concentration of the Nu-Well may have to be increased. In cases where the screen is moderately to heavily encrusted, it may be necessary to increase the weight of Nu-Well to as much as 30 percent.

3. The calculated quantity of Nu-Well pellets is poured into the well casing, and allowed to set in the well for 1 to 2 hours. Then the rock salt, **if needed**, is added to the well and allowed to set for an additional 2 hours.

4. After the chemicals have been in the well for the required amount of time, the surge block/air lift assembly is installed to gently agitate the well water and help dissolve the remaining chemicals (approximately 30 min).

5. After the chemicals are dissolved, the air lift is used to circulate a quantity of well water equal to the volume of the well. During this process, the well water is very acidic; therefore, it is important that the discharge hose on the air lift is inserted far enough into the well casing that no well water splashes out of the well.

6. Next, the surge block is used to gently force the acid solution out through the well screen and into the sand/gravel pack and natural formation.

7. The acid solution is allowed to set overnight in the well. In the case of moderately to heavily encrusted well screens, it will be necessary to surge the well again on the second day and leave the chemical solution in the well for a second night.

8. The acid solution is removed by first surging the well for 30 minutes and then air lifting the solution from the well for 1 hour. This surging and air lifting cycle is repeated until the water is clear. After the water is clear, the pH is checked to see if it is close to the pH that existed in the well prior to the rehabilitation process. Should the water be clean, but still acidic, the surging is discontinued and the well water is air lifted from the well until the desired pH is attained. As long as the waste water from the well is acidic, the water should not be discharged directly into any water body or near woody plants or trees. If a suitable site to discharge the waste water does not exist near the well, the waste water must be pumped into a tank and transported to a suitable site.

4. *Final Chlorination (Disinfection).*—The well should be video taped again before this procedure is started. If bacterial encrustation or slime is still present on the screen, the well is given a second "chemical" treatment. During the video taping, the screen is also checked for damage as a result of the cleaning procedure.

1. This step is performed to disinfect the well, pump, pipes, and discharge line.

2. A quantity of chlorine (calcium hypochlorite) needed to bring the concentration in the well up to 300-500 ppm should be added to the well and mixed using the same procedures used previously in the chlorination treatment.

3. The chlorine solution is circulated in the well with the airlift system until one well volume has been circulated. This will mix the solution throughout the water column. Then the airlift/surge assembly is removed. During the disinfection phase, no water is added to the well and no surging is performed.

4. The pump and pipes are installed and the whole assembly is allowed to soak in the chlorine solution for 2 hours minimum and up to 24 hours if time permits.

5. The chlorine solution is pumped to waste. If the well is to be connected to a domestic supply, it should be pumped to waste for at least 48 hours before connecting to the domestic line.

c. *Personal Safety Procedures.*—Any job involving hazardous chemicals should have a job hazard analysis completed and reviewed by the required safety personnel. Important considerations are listed below, however, these items are not all inclusive for each specific job.

- All personnel at the site of the well must wear appropriate safety equipment while mixing, injecting, purging, or pouring chemicals into the well. As a minimum, the equipment must include:

    N Chemical goggles
    N Chlorine-approved respirator (each employee must be fit tested)
    N Rubber gloves and boots
    N Coveralls

- When chlorine or acid is being mixed, added, or purged from the well, a container (5-gal minimum) of clear, potable water or a portable eye wash station must be on site for use in case of accidental exposure. A gallon of 35-percent hydrogen peroxide should also be on hand to neutralize any concentrated spillage of calcium hypochlorite (1 pint will neutralize 1 lb of calcium hypochlorite).

- Chlorine granules/pellets must be stored in an approved locker. The cabinet must be labeled "Class III Oxidizer," and no other chemicals or material should be stored in this cabinet.

- Only black pipe and/or plastic pipe shall be used when treating the well. **No galvanized pipe** should be used, because the strong acids and oxidizers (chlorine) can severely react with galvanized pipe.

4-5. California Department of Transportation—Horizontal Drains.—

a. *Background*.—The California Department of Transportation (CDOT) experiences slope failures in natural ground, cut slopes, and road embankments on both the California state highway and Interstate systems [28]. The failures are generally initiated by rainfall, with the water lubricating the material, adding weight, increasing pore pressures, and decreasing shear strength until failure occurs.

Horizontal drains, generally installed at an angle slightly above horizontal, remove water from the material in an effort to avoid initiating slope failure. However, the horizontal drains are susceptible to blockage by mineralization, infiltration of fines, and root growth. CDOT determined high pressure water cleaning systems used for unplugging culverts and cleaning sewers can easily be adapted for horizontal drain cleaning by modifying the hose diameter and nozzle configuration.

b. *Case Histories*.—Seven horizontal drain installations were selected for evaluation because they represent the diverse conditions under which horizontal drains perform successfully in California.

1. *Whitmore Maintenance Station*.—The site is located at elevation 5,000 feet in the Sierra Nevada Mountains between Sacramento, California, and Reno, Nevada. Annual precipitation for the area is 67 inches. After exceptionally heavy rainfall over a 4-day period, in February 1963, two large road embankment slides occurred on the newly constructed eastbound lanes of Interstate 80 near the Whitmore Maintenance Station. All three eastbound lanes were affected by the slope failures and traffic had to be rerouted to the westbound lanes.

Correction of the two failures included removal of slide debris, removal of roadway embankment, placement of permeable blankets over the foundation, replacement of the compacted embankments, and installation of horizontal drains into the embankment and volcanic agglomerate and ash bedrock.

Horizontal drains totaling 21,251 linear feet were installed. The 103 drains average 206 feet in length with initial total flow ranging from 300,000 to 500,000 gal/d. The drains are highly responsive to precipitation. Within 24 hours after rainfall, the flow quantity increases considerably. The drains were cased with 2-inch-diameter perforated steel pipe.

A review of the service records indicates that the drains are cleaned about every 3 to 4 years using a high-pressure water system. Small amounts of silt and rust are encountered from time to time, but heavy root growth from willows was found and removed each time from several of the drains. Since the cleaning is usually done in the late fall, little increase in flow was noted except in those drains clogged with roots.

The effectiveness of horizontal drains in dewatering the volcanic bedrock at this installation is excellent. No distress of the roadway or embankments has been noted since installation of the horizontal drains.

2. *San Andreas.*—The site is located in the Sierra Nevada foothills on State Highway 49 at elevation 800 feet. Annual precipitation for the area is 33 inches. Two landslides developed in newly constructed cut slopes near the community of San Andreas during late winter of 1964. A 1,200-foot-long section of an 80-foot-high 1½:1 cut slope failed in sheared and broken serpentine bedrock. A few days later a second slope 3,000 feet north of the first failed. This cut slope was also newly constructed in serpentine on a 1½:1 slope and was approximately 90 feet high. Free water was evident at the toe of both failures and the lower portions of each cut were wet before movement occurred.

Correction of the failures consisted of removal of the failed debris, laying the slopes back to 2:1 ratio in the areas of movement, and installing horizontal drains.

Thirty-three horizontal drains, ranging from 150 to 200 feet long, were installed from the toe of the cut slopes. Grades ranged from 5 to 20 percent (angled upward) and all 5,189 lineal feet of drain were drilled normal to the roadway alignment. Spacing between drains ranged from 20 to 150 feet. Locations for the drains were selected based on the surface geology, evidence of free water on the slopes, and subsurface information obtained during a forensic investigation drilling program. Perforated steel pipe (2-inch-diameter) was used as casing for all the installations, and 2-inch-diameter galvanized steel pipe was used for the exposed outlets and for connecting to the buried 8-inch corrugated metal pipe collector system.

The combined initial flow from the horizontal drains was 119,486 gal/d. Two months after completion of the installation, the combined flow dropped to 68,504 gal/d, or approximately one-half of the initial flow. Examination of the installation approximately 1 year after completion revealed a combined flow of 5,710 gal/d. While the flows are greatly diminished, flows are very responsive to rainfall and increase significantly after a heavy rain.

Cleaning records for this installation are not complete. Apparently, the horizontal drains were cleaned using a high-pressure water system three times during their first 15 years of service, for an average of about once every 5 years. Minor amounts of silt and rust were removed from all the drains, and heavy root growth was removed from four. Little increase in flow was noted after cleaning except for the drains clogged by roots.

The drains are in excellent condition and show little sign of rusting or damage. The four drains previously plugged with roots were again clogged and require cleaning. The remaining 29 drains could easily function another 2 or 3 years without cleaning. With the exception of the four drains, this installation only needs cleaning about every 8 years.

3. *Pacific House*.—The site is located at elevation 3,600 feet on the western slope of the Sierra Nevada Mountains at Pacific House on Trans-Sierra Highway 50. Annual precipitation for the area is 51 inches. Saturated clayey soil and decomposed coarse-grained granitic debris slid onto the eastbound lanes from a 30-foot-high, 3/4:1 slope. The slide mass was fed by springs which may be partially sustained through irrigation of an apple orchard located upslope from the top of the cut.

Ten horizontal drains were installed in bedrock, during October 1969, ranging in length from 156 to 213 feet. Grades varied from 2 to 4 percent (angled upward). Schedule 80, 1½-inch-diameter PVC pipe was used in 4 of the 10 drains, while 2-inch-diameter perforated steel pipe was used in the remaining 6 drains.

The combined initial flow was 26,285 gal/d. Upon completion of the project, the flows totaled 15,438 gal/d.

Ten years later the site remained stable. Wet spots are common around the drains and two of the drains showed more water coming from around the drain pipe than through them. Heavy root growth from willows plugged most of the drains. Approximately 3,000 gal/d flow from the drains. The slope appeared stable and no distress was evident on the highway pavement.

4. *Cloverdale*.—During extremely heavy rainstorms, in February 1941, a large landslide occurred on Highway 101 near Cloverdale, 90 miles north of San Francisco. Portions of a high cut slope and side-hill embankment in poorly bedded, sheared shale with minor lenses and beds of sandstone slid into the Russian River, destroying approximately

1,100 feet of the roadway. Large quantities of water in the form of springs and saturated slide debris were associated with the failure.

Corrective measures included a benched 1½:1 cut slope and a reconstructed 2:1 embankment slope. Ninety-seven horizontal drains were installed using "Hydrauger" equipment with 2-inch-diameter perforated steel casing placed in 4-inch-diameter drilled holes. The loose, broken shale caused difficulty during the installation. The average drain length was only 55 feet. Several holes were abandoned because of caving conditions. Drain outlets and downpipes were connected to corrugated metal pipe buried below the shoulder, preventing periodic inspection of the drains.

A complete record of initial flows is missing although some of the individual drains produced between 150,000 and 200,000 gal/d.

Heavy rains 15 years after construction produced appreciable quantities of water which appeared in various places along the toe of the cut slope. The water was coming from around the horizontal drains which had ceased to function properly because of heavy deposits of rust, gypsum, and root growth. A drain cleaning and restoration program using a high-pressure water system was completed. Forty-nine drains were located. Prior to cleaning, the drains produced a combined flow of 184,250 gal/d with one drain producing 143,000 gal/d of this total. Immediately after cleaning, the drains produced a cumulative total flow of 284,440 gal/d. This flow increase clearly illustrates the value of the drain cleaning work performed (table 8).

In addition to the cleaning and reconditioning of the collector system with easily accessible cleanouts, three new drains were installed in the most critical areas. An average length of 114 feet was used. These three drains produced initial flows totaling 28,000 gal/d. Later in the year, 11 additional horizontal drains, with an average length of 125 feet, were installed in a slide immediately south of the original failure. Initial flows from these 11 drains totaled 261,989 gal/d. An unknown additional number of drains were also installed in a large cut area. There is no record of their initial performance.

Twenty years later, the installation was evaluated. Of the 147 drains examined, approximately 40 percent had flows. Many of the original drains installed 39 years earlier were still functioning, one of which had a flow of 36,000 gal/d. Most of the steel casings were severely rusted. Root growth from willows and other native plants clogged many of the drains and the 8-inch collector system. Sloughing of the weathered slopes buried some of the drain outlets on both the bench and at grade.

Table 8.—Effect of drain cleaning on horizontal flow at Cloverdale, California—1956.

Drain No.	Hole depth (ft)	Depth cleaned (ft)	Flow before cleaning (gal/d)	Flow after cleaning (gal/d)	Remarks
3	65	65	830	1,080	Heavy root growth and rust
6	60	60	143,800	143,800	Gypsum and heavy rust
9	50	50	2,460	4,320	Gypsum and heavy rust
11	75	75	300	43,000	Very heavy root growth
13	80	80	1,440	7,200	Heavy rust
18	45	45	100+	10,790	Heavy root growth and silt
19	30	11	2,880	7,200	Heavy root growth and rust
21	26	26	100+	4,320	Heavy root growth
22	35	6	360	1,080	Heavy root growth
23	35	8	6,640	10,790	Heavy root growth and rust
24	37	37	200+	12,340	Heavy root growth and rust
27	57	57	720	1,540	Heavy rust
29	45	45	300	17,280	Root growth first 20 feet, sand and rust

5. *Nojoqui Grade*.—Approximately 200 feet of the northbound lanes of Coastal Highway 101 was lost or endangered by the sliding of a portion of a sidehill embankment in December 1940. Geologic investigations indicated the presence of a high water table in soft, indistinctly bedded claystone and siltstone. A smaller slide of similar nature occurred 800 feet north of this site at approximately the same time.

Forty-two horizontal drains were installed using "Hydrauger" equipment after the embankments had been reconstructed. They were placed into the hillside from locations immediately below the toe of the fill slope in the saturated foundation area. These drains ranged in length from 72 to 191 feet. Perforated 2-inch-diameter steel pipe was used in 4-inch-diameter holes drilled with the auger. Grades ranged from 2 to 10 percent angled upward. Records of initial flows are not available.

The first record of cleaning was 21 years after installation. Only 27 of the original 42 drains were found. The other drains had been buried by end dumping over the side of the embankment. A heavy accumulation of roots, rust, and silt was evident in most of the drains cleaned. No appreciable increase in flow was noted after cleaning, probably due to cleaning at the end of a hot, dry summer. Flows increase considerably during the wet season.

In late 1974, the embankment was widened and two additional lanes for traffic were constructed. During construction of the roadway, boggy conditions were encountered. Several of the severely rusted but still-functioning, buried drains were uncovered. Drains with exposed outlets had large accumulations (mounds) of iron oxide and algae on the ground below the casing. Additional subdrainage was necessary to assure construction and maintenance of a stable highway and thirty-two new drains were installed, ranging in length from 150 to 450 feet. The combined initial flow was 157,480 gal/d. The combined flow at the completion of the job was 11,350 gal/d.

Four years later the installation was inspected. Of the original 42 drains installed, in 1941, all but 8 were destroyed or replaced during the highway widening project. Of the eight remaining drains, two were dry, one had a drip, two displayed a trickle, two produced 90 gal/d and one had a flow of 720 gal/d. The steel pipe of the nearly 4-decade-old drains were severely rusted and would probably be destroyed if disturbed by a cleaning operation.

6. *La Selva Beach*.—In August 1973, cracking developed during construction of a 120-foot-high, 1,200-foot-long 2:1 cut slope near La Selva Beach, located on the coastal highway south of Santa Cruz, California. Just prior to excavation, a large grove of mature eucalyptus trees located near the top of the planned cut slope was cut down. Elimination of transpiration by the trees caused a rapid rise in the groundwater table, resulting in saturation of the poorly cemented sand foundation and a loss of strength, resulting in cracking of the new cut slope near the top, followed shortly by slope failure and flow of saturated sand near grade.

Corrective measures included removal of the failed slide and excavation of a new 2:1 cut slope down to the grade of an upper 50-foot-wide bench. A 30-inch-diameter vertical well was installed on the bench to a depth of 51.5 feet and five observation wells were installed along the upper bench spaced on 200-foot centers. These wells were drilled through 45 to 50 feet of sandy material to the top of a major clay layer along which slope movement and groundwater flow were occurring. The lower 2 to 12 feet of sandy material in the wells were saturated. The 30-inch well was pumped 24 hr/d for 3 months with average daily discharge of 1,900 gallons. No appreciable drawdown was noted at the observation wells, and the pumping well was abandoned.

Sixty-nine horizontal drain holes from 150 to 200 feet in length were installed in the cut slope. Grades ranged from 6 to 8 percent (angled upward), and the holes were cased with PVC pipe with 0.010-inch slots. The combined initial flow was 169,500 gal/d. Observation well data indicated a slow, steady drawdown of the groundwater table during and following completion of the horizontal drain installation. Based on these positive results, excavation resumed from the upper bench to the lower bench level. An 8-inch-diameter slotted plastic pipe underdrain was placed in an 8- to 12-foot-deep trench on the lower bench. This underdrain was located 1.5 feet below the top of the major clay layer. During placement of the underdrain a crack formed on the slope above the upper bench. Construction accelerated until the system was backfilled and no additional movement has been observed in this area.

After 6 years there were no signs of distress. The horizontal drains have performed extremely well and have been effective in keeping the groundwater at a safe level.

Maintenance forces clean the drains every 2 to 3 years using a high pressure water system. Minor amounts of sand have been removed, but the quantity decreases each time the drains are cleaned.

Slot size was particularly important on this project. In comparison, 2-inch-diameter steel pipe with 3/8-inch perforations was used on a job with similar foundation materials on a project located 3 miles east of the present site. The horizontal drain hole casings filled with sand and the drains lost their effectiveness almost immediately.

7. *York Mountain*.—Failure of a 1½:1, 750-foot-long cut slope occurred in November 1970 during a heavy rainfall near York Mountain along a new highway alignment between Paso Robles and Cambria on the central coast of California. The planned cut for this slope was 140 feet in height. Excavation had been completed to within 50 feet of grade when movement was first observed. Excavation stopped 37 feet above grade as the area of disturbance rapidly enlarged.

Corrective measures included redesigning the cut slope to 2¼:1, with a 20-foot-wide bench 50 feet above grade and a 10-foot-wide debris bench at grade. Cracks appeared at the top of the redesigned slope within a year, and during the wet season movement renewed. Free water occurred along the debris bench at grade, on the lower portion of the slope, and for about 20 feet above the upper bench.

Five 360-foot-long exploratory horizontal drains were installed at grade. All drains were oriented normal to the roadway alignment to intercept as many bedding planes as possible. Grades were 5 percent (angled upward) and spacing between drains ranged from 45 to

100 feet. The combined initial flow was 72,830 gal/d. Somewhat reduced movement continued the next winter.

An additional 18 horizontal drains were installed from grade and from the bench above grade. Lengths ranged from 245 to 360 feet and grades ranged from 5 to 15 percent (angled upward). Orientation of each drain was normal to the steeply dipping shale beds and all drains were completed with 1½-inch slotted PVC pipe. Initial flows ranged from dry to 175,000 gal/d and the combined flow was 332,940 gal/d. At completion of installation, combined flows were 70,954 gal/d. The drains are extremely responsive to rainfall.

The project was inspected 7 years after completion. The drains and collector system were in good condition but have not been cleaned since installation. Some silt and root growth was noted. No additional movement is recorded since the final 18 drains were installed.

c. *Conclusions.*—The seven case histories presented demonstrate the effectiveness of horizontal drain installations in arresting movement and stabilizing serious problem areas.

Several factors influence the effectiveness of a drain system:

- *Lithology.*—Material types, cementation, grain size, permeability, and fabric or structure (orientation and spacing of discontinuities) are major lithological factors which influence drain effectiveness. Most horizontal drains installed were drilled in weathered rock covered by thin surficial layers of soil. Movement of ground water within the rock is largely confined to discontinuities such as joints, fractures, bedding planes, foliation planes, and fault planes. Exceptions are where rock is permeable volcanic agglomerate at the Whitmore Maintenance Station or poorly cemented dune sand at the La Selva Beach project. Horizontal drains installed in these materials must intersect discontinuities or permeable materials so that influent seepage and ground water can be removed.

- *Drain location, orientation and spacing.*—The most effective horizontal drain installations are those which have been designed and installed on the basis of the geology of the site.

  The design of a horizontal drain installation should incorporate the location, orientation, and spacing of discontinuities and confining layers or aquatards. Groundwater barriers such as clayey shear zones or confining beds, such as clay layers, must first be identified and then penetrated by horizontal drains to relieve the impounded water behind them. Spacing of drains in an installation should be based on the location of productive zones where the water occurs rather than an even spacing over a large area. Spacing of joints may vary widely over a short distance, requiring a variable spacing of drains to accommodate change, assuming the ability to identify which joints are carrying water and where spacing changes.

Drain length is governed by the water bearing strata intercepted rather than on a predetermined length. In most cases the initial drains are drilled longer than is considered adequate. Volumes and points where the water is intercepted are recorded during drilling, and subsequent drain lengths are determined from the drill data.

- *Effect of time on drain hole casing and productivity.*—Some of the first horizontal drains installed by the CDOT were cased with 2-inch-diameter, perforated steel pipe. Subsequent inspections showed the steel pipe rusted in 20 to 30 years, in some cases severely. Since about 1970, CDOT drain installations are completed with slotted PVC pipe. Follow up inspections indicate the PVC pipe remains in excellent condition for years and should perform well for decades.

  Generally, PVC pipe with 0.020-inch slots is used in the majority of installations. When poorly-cemented, granular material is encountered, a mechanical analysis is performed to determine the proper slot size. Slotted PVC pipe can be penetrated by roots near the ground surface. While there is no specific correlation of production with time, drains tend to decrease in effectiveness with increasing age. Steel pipe can rust, steel and PVC pipe can shear with new or renewed slope movement, and iron-reducing bacteria, mineralization, sediment, or root penetration can obstruct drain pipe or collector systems. However, if a proper cleaning and maintenance program is performed on a regular basis, a drain installation will function effectively for at least as long the expected life of the structure.

- *Maintenance programs.*—Perhaps the most important factor in long-term performance of horizontal drains is a well developed and executed program of inspection, repair, and cleaning. A drain maintenance program should be established with provisions for annual inspections for cleaning and repair of damaged outlets or collector systems. Most drains need to be cleaned only every 5 to 8 years. Heavy root growth or fine-grained sediments may reduce this period to every 2 years.

  A drain cannot be maintained if it cannot be located. Good records, including plans which show the locations in relationship to survey monuments or permanent landmarks, are required. Otherwise, drain locations are lost because of transfers, retirements, or promotions of personnel who are only aware of the drain locations because they were present during the actual installation. Drains located near the toes of embankments are sometimes lost through the practice of end-dumping waste material over the side of the fill, covering the outlets. The establishment and maintenance of a central file on all drain installations as well as placement of markers such as steel posts or signs is a good practice.

The most effective method of maintaining continuity in a horizontal drain maintenance program is to assign qualified personnel the full-time task of inspecting, cleaning, replacing, or repairing horizontal drain installations on a regional or statewide basis.

Dense growth of water-seeking vegetation around the outlets not only tends to conceal the drains but also to curtail their performance by extensive root growth within the first 10 to 20 feet of the drain pipe. Nonslotted pipe, or galvanized pipe slipped over the PVC pipe and inserted into the drilled hole for the outer 20 feet of each drain, can discourage roots from entering the drains. Selective herbicides can be used around the drain outlets to retard or eliminate undesired vegetation.

Horizontal drains are also lost or damaged because they are not protected against rockfall, particularly in the case of exposed PVC pipe, or because they are vulnerable to snow plows, rock plows, or straying vehicles near the edge of the slope. A good practice is to protect the PVC pipe at the outlet with a galvanized pipe that slips over the PVC pipe and is inserted into the drilled hole approximately 20 feet.

Drain cleaning and maintenance records should be kept for each installation. These records should indicate dates of cleaning and repairs, what was done, flows recorded for each drain prior to and after each cleaning, and damage due to slide movements or external forces.

- *Ground water pH and mineral content.*—Drain performance can be adversely affected by ground water pH or mineral content. Slots in drain pipe and the drain pipe itself can become plugged by precipitation of minerals in solution in ground water.

4-6. Ochoco Dam Video Toe Drain Inspections.—Remote control, track-mounted video cameras were used to inspect existing toe drains at Ochoco Dam. Camera inspections were used to determine the amount and type of debris that may exist inside the drain pipes, as well as the condition of the pipes themselves.

Ochoco Dam is an earthfill structure located 6 miles east of Prineville in central Oregon. The Veterans Farm Administration constructed the dam between 1917 and 1921 and transferred it to Reclamation in 1948. Several modifications have been made to the structure due to seepage concerns. In 1949 and 1950, the east and west drains were added to the dam. As a result of video inspections performed in 1998, it was determined that both of these drains should be rehabilitated. During examination of the east drain, a previously unknown segment of the drain was discovered.

a. *Ochoco Dam Video Inspections*.—The original east and west drains at Ochoco Dam were constructed of concrete bell and spigot pipe. In 1998, video examinations were performed on all existing east and west drains. The video camera was supplied and operated by a subcontractor that specialized in inspections of sewer lines. The camera, which had the capability to view left, right, up, and down, was mounted on a motorized, tracked vehicle. Typical inspection cameras are shown in figures 66 and 67.

During the video inspections, breaks were found in the drain pipe, and sediment was also observed inside the pipe. These conditions were considered unacceptable, and the decision was made to modify the drains. However, the video inspection also revealed sections of the drains that were in good condition and could remain in place. The inspection also revealed a previously unknown segment of the east drain. Early discovery of the unknown segment had the added benefit of giving Reclamation enough lead time to address this segment in the modification.

The modified drains were constructed of double-walled HDPE corrugated pipe with smooth interior walls. The pipe diameter ranged from 10 to 18 inches. Figure 68 shows the locations of the modified drains. Video examination was also performed on the completed drains, as required by the modification specifications. The postmodification video examination revealed an accumulation of debris in the drain which came from construction activity. Although care was taken to keep the drain pipes clean during construction, some material was introduced into the pipes. The drain segments containing the debris were cleaned with a sewer cleaner, and another video examination was performed to verify that cleaning had been successful.

Since access to the original drains had been difficult, it was desired to have easier access to the modified drain system. This was achieved by including cleanouts and inspection wells (manholes) in the drain design. For the cleanouts, the drain pipe was daylighted by using two 22½° elbows in the drain pipe. A single 45° elbow is too tight for some types of video and cleaning equipment. A protective galvanized pipe with locking lid was then installed at the ground surface. Figure 69 shows a typical cleanout.

Inspection wells, which were installed at the junction of drains and/or outfalls, were constructed of 8-foot diameter concrete pipe. The inspection wells are especially useful in gaining access to bends in the east drain. A typical inspection well is shown in figure 70.

b. *Conclusions*.—Video inspections were very successful in evaluating the conditions of the toe drains and outfall pipes at Ochoco Dam.

Video inspections are valuable as a documentation tool after drains have been cleaned or after drains have been installed to verify that the work has been performed satisfactorily. It

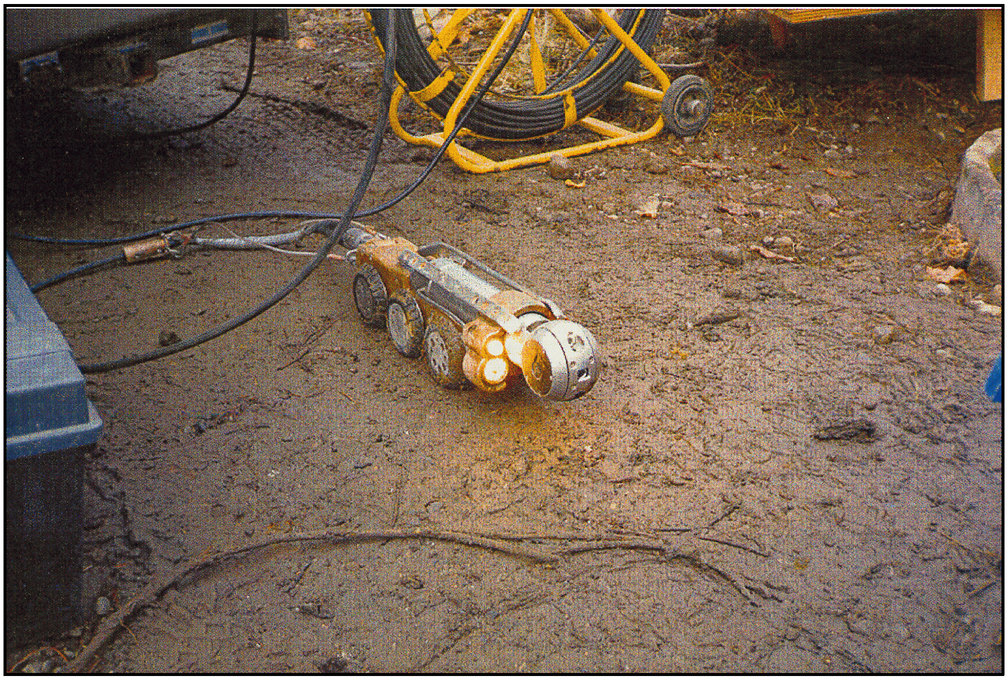


Figure 66.—Self-propelled camera.

Figure 67.—Sled-mounted camera (the cell phone provides scale).

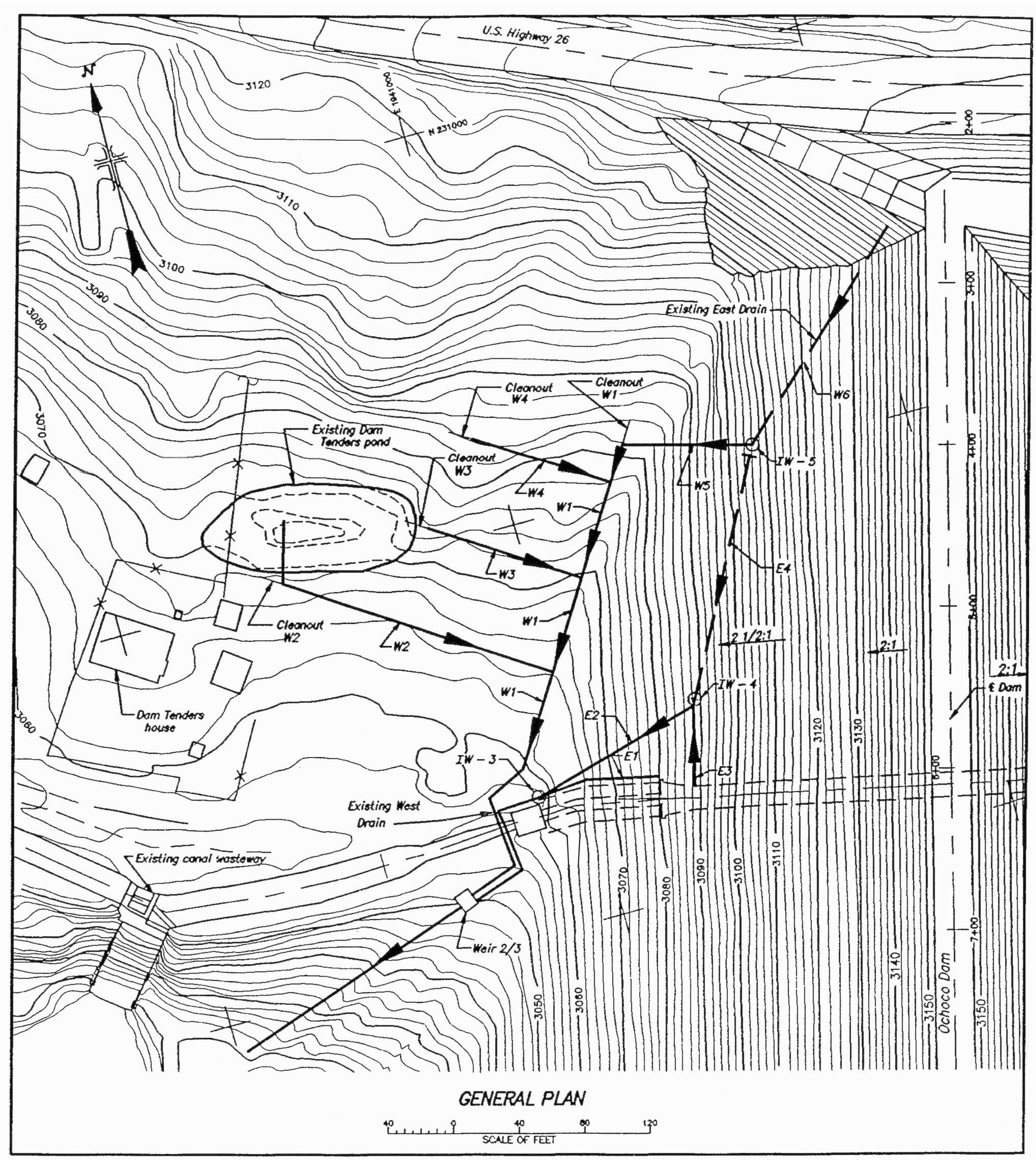


Figure 68.—Ochoco Dam plan view showing east and west drains.

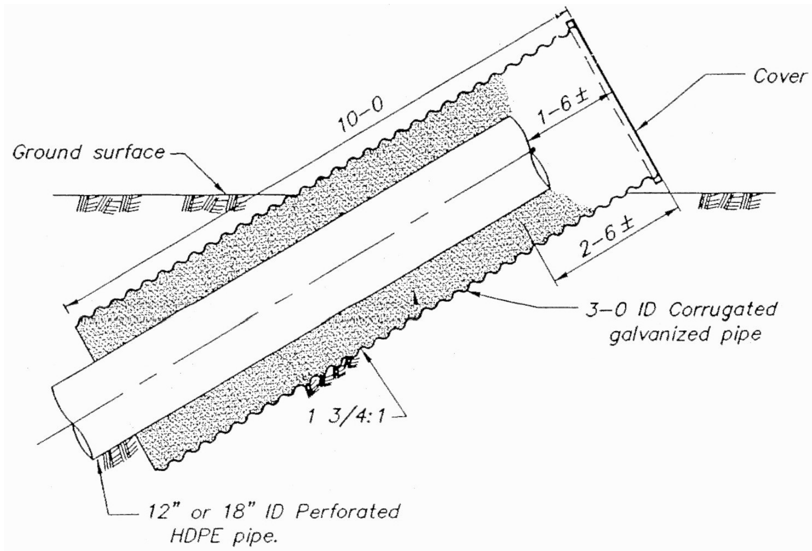


Figure 69.—Typical drain cleanout.

Figure 70.—Typical inspection well used to gain access to outfall pipes.

is advisable to specify postconstruction video inspections to determine if debris has been introduced into the drain pipes during construction. The specifications should also require that any damage to the pipe as a result of installation or the existence of debris introduced during construction be rectified at the contractor's expense. This should include the cost to reinspect the drain pipe after the contractor has rectified any construction-related problems. Recent video inspections for the HDPE toe drains installed at San Justo and Davis Creek Dams have revealed collapsed portions of these drains. The damage likely occurred during installation of these drains. For more information on remote controlled video inspctions, see appendix C.

New drain designs should include inspection wells and cleanouts to make drains more accessible for inspection and cleaning tools.

4-7. Senator Wash Dam Relief Wells.—Senator Wash Dam is located in California just west of the Colorado River and about 15 miles north of Yuma, Arizona. In 1989, 20 relief wells with stainless steel screens were installed at the downstream toe of the dam to control seepage uplift pressure. Figure 71 is a site map of Senator Wash Dam.

a. *Background*.—The well screens at Senator Wash Dam were silting up, resulting in reduced efficiency of the relief wells. Discharge rates had been steadily dropping since well installation in 1989. By 1997, the well discharge rates on some of the relief wells had dropped to about 10 percent of the original discharge rates. As examples, figures 72 and 73 respectively show discharge rates for RW-14 and RW-15. On the average, about 2 feet of fine soil particles had collected in the bottom of the well screens. Little or no mineral deposits or bacteria built up on the well screens. Even though a filter pack had been placed around the outside of the well screens; some of the native soil, which is predominantly fine sand and silt, and some of the filter material had passed through the screens and caused the clogging. Periodically, most relief wells need to be redeveloped to maintain efficiency. In this instance, a surge block was used. The surge block is one of the oldest and most effective methods of well development.

b. *Relief Well Cleaning Operation*.—The surge block process forces water back and forth through the well screen. The back-and-forth action loosens the clogged particles and eventually forces most of the particles through the screen and into the well.

Surge blocks come in many varieties. Figure 74 shows an example of a vented surge block. Some surge blocks are solid, and others are vented by drilling a number of holes through the body. The top of the vented body is fitted with rubber to act as a flap valve. This seals the holes on the upward stroke and permits water to move through the holes on the downstroke. The *Ground Water Manual* [45] shows additional drawings of surge blocks.

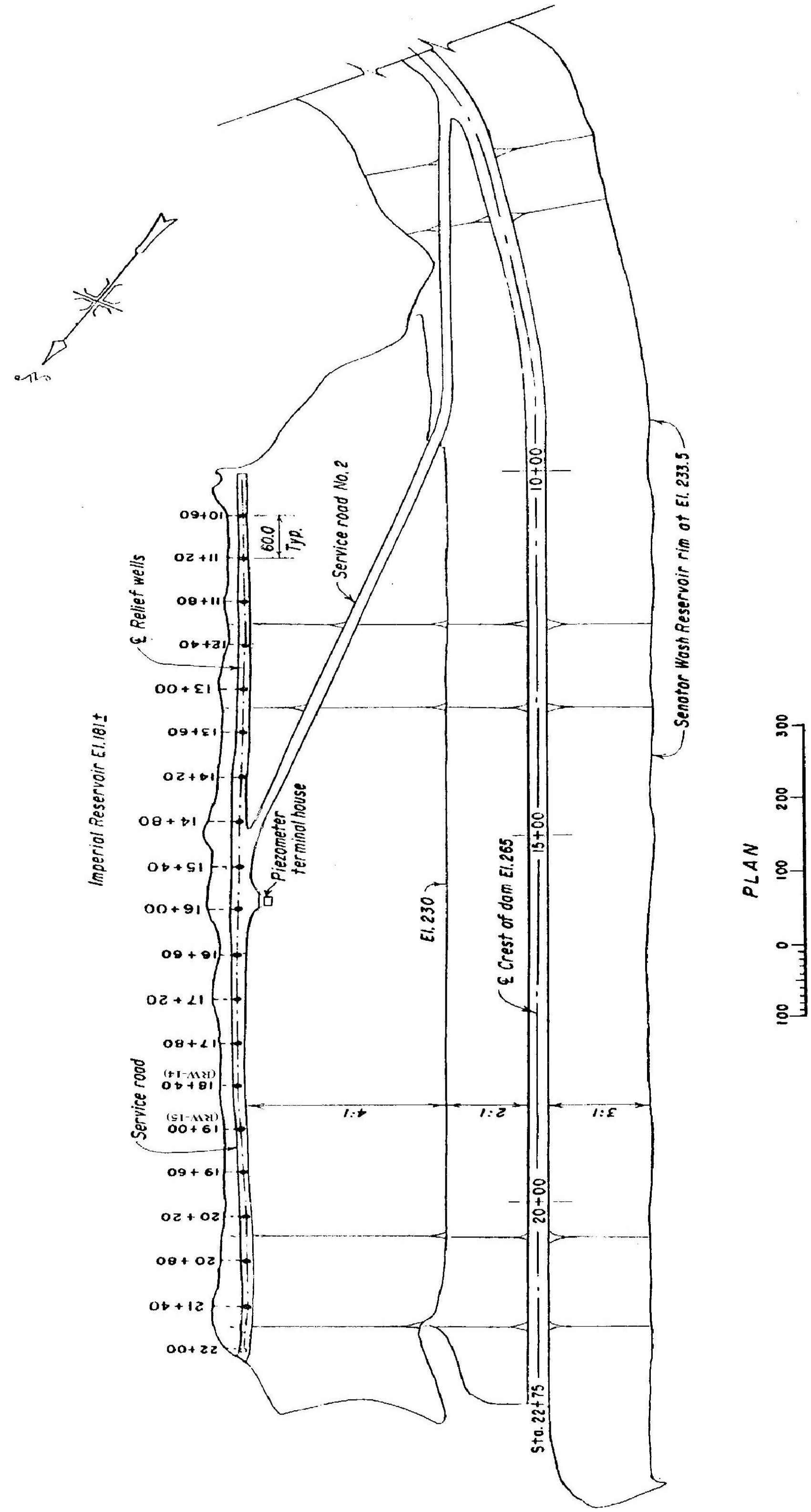


Figure 71.—Site map of Senator Wash Dam.

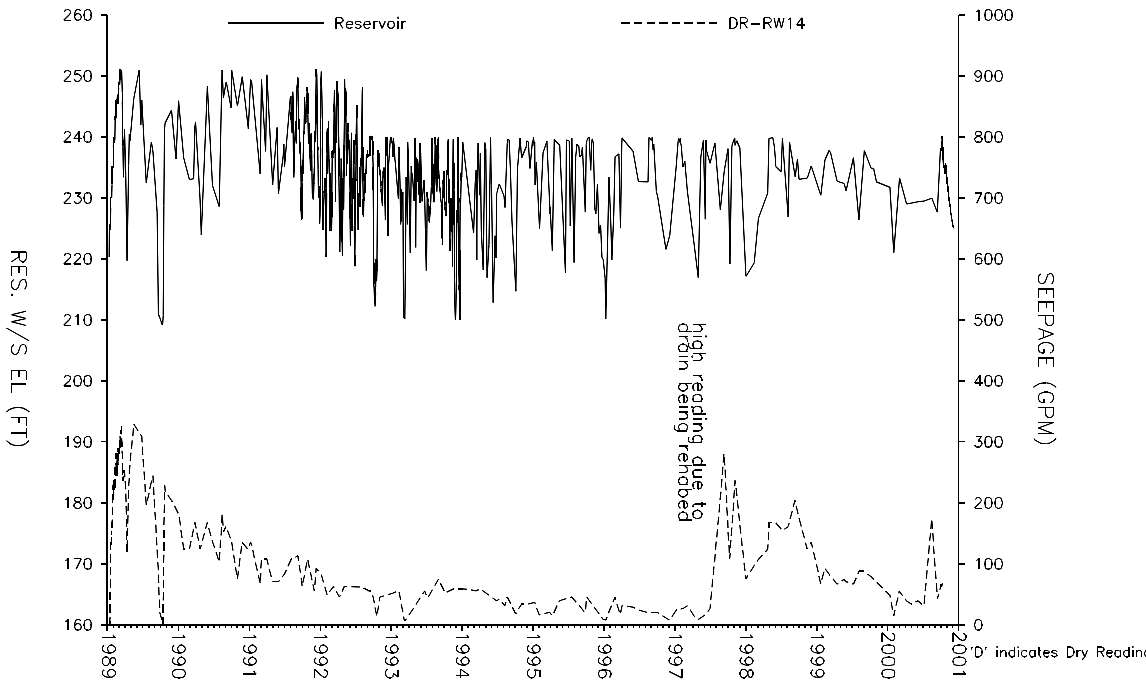


Figure 72.—Discharge rates for relief well 14 at Senator Wash Dam.

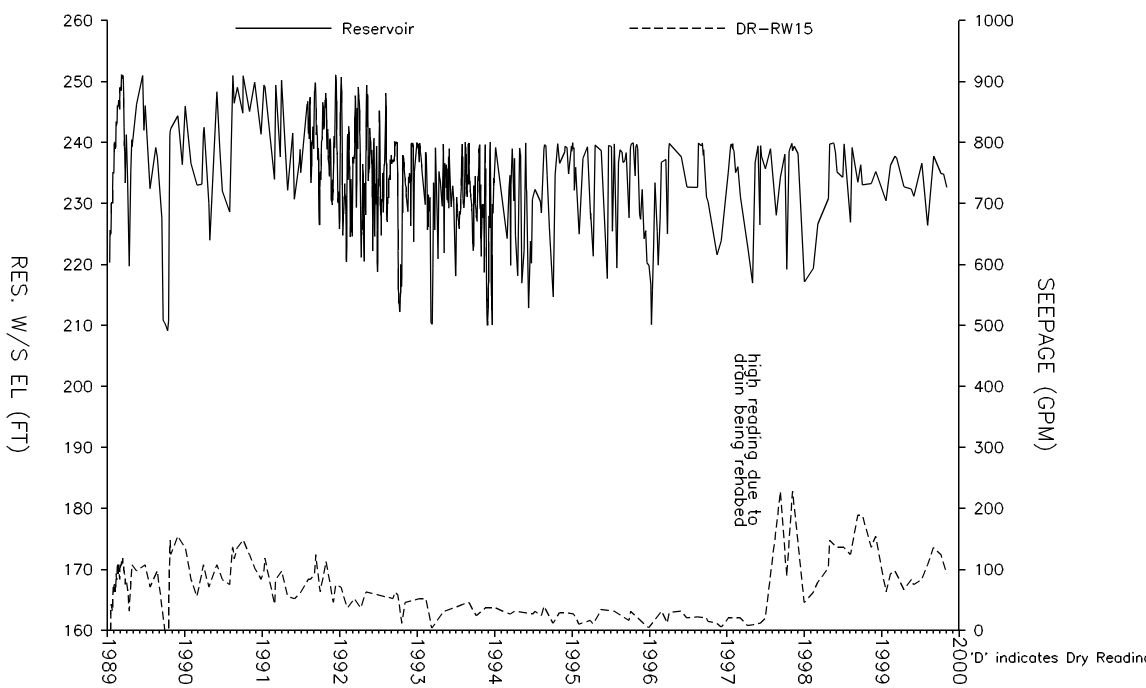


Figure 73.—Discharge rates for relief well 15 at Senator Wash Dam.

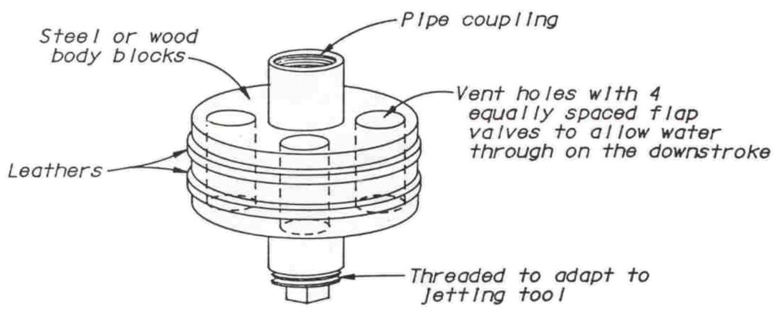


Figure 74.—Vented surge block.

Much of this information on surge blocks was obtained from the *Ground Water Manual*, which also has information on other well development techniques.

The solid and vented blocks consist of a body block which is 1 to 2 inches smaller in diameter than the well screen. The body block is fitted with as many as four ¼- to ½-inch thick disks called leathers, which have a diameter the same as the inside diameter of the well screen. Rubber can also be used for the disks. The block is attached to a metal rod so it can be cycled up and down in the relief well. At Senator Wash Dam, a cable tool rig was used to cycle the surge block during the well redevelopment process. The surge block was cycled about once per second over a 4-foot distance. Before and after the surging, fine sand and silt should be cleaned from the bottom of the well.

Surging should be started above the screen to bring in the initial flow of soil particles, thus minimizing the hazard of sand locking the block in the screen. After the initial surging above the screen is complete, the surge block is lowered to the bottom of the screen, and the cyclic action is performed in an upward direction over the entire length of the screen. This process is repeated several times over the entire length of the screen. During the first upward pass, the strokes are slower. Then the rate of the cycling is increased for each successive upward pass by the surge block. Each time the entire length of screen has been surged, the depth of sediment in the bottom of the well should be checked. The sediment should be bailed or air lifted if it is encroaching on the well screen.

Discharge data for two typical wells are noted here from both before and after the surge block operation:

- Relief well 14 (fig. 72), was making about 330 gal/min in 1989, then slowly dropped off to 25 gal/min by 1997 before the well was surged. After surging, the discharge increased to about 280 gal/min, then dropped to 220 in 1998 and 90 gal/min in 1999. The reservoir did not reach maximum reservoir level in 2000, making discharge rates unavailable.

- Relief well 15 (fig. 73), was originally making 120 gal/min in 1989 and actually increased to 150 gal/min in 1990 before slowly dropping off to 25 gal/min in 1997 before surging. After the well was surged, the discharge rate increased to 230 gal/min, then dropped to 190 gal/min in 1998 and 130 gal/min in 1999. As with relief well 14, discharge rates were unavailable for 2000.

c. *Conclusions.*—The discharge rate for typical wells increased by an order of magnitude after the wells were redeveloped with the surge block technique. This indicates the effectiveness of the surge block method for redevelopment of relief wells. The surge block method is only recommended for cases where soil particles are plugging the well screen. This method is not recommended for bacterial plugging or mineral encrustation. Plugging mechanisms for relief wells can be determined by video inspections of the wells. If the plugging mechanism cannot be clearly identified, sampling and testing can be performed (see setion C-5 in app. C).

In the case of the Senator Wash relief wells, the discharge rates dropped off fairly quickly. This was true not only after the wells were originally installed, but also after the rehabilitation. This rapid dropoff is somewhat unusual, and may be due to the type of native soil in which the wells have been installed. It could also be related to the gradation of the gravel pack.

With any relief well system, the time interval at which the wells should be redeveloped depends not only on the decrease in well discharge but also on the increase in piezometric level in the foundation. The piezometric level is typically measured at the midpoint between relief wells. For each dam, the allowable piezometric level should be determined, and as that level is approached, relief well redevelopment should be initiated. Even though the piezometric level may be acceptable, U.S. Army Corps of Engineers experience suggests that after a certain period of time, relief wells cannot be redeveloped.

4-8. Sherman Dam Toe Drain Rehabilitation.—Sherman Dam, located about 40 miles northwest of Grand Island, Nebraska, is 98 feet high and 4,450 feet long. The dam is a homogeneous earth structure, constructed between 1959 and 1962. The toe drains were replaced in 1980. Toe drain No. 3 is a 12-inch perforated asbestos-cement pipe with sealed joints and has a length of about 1,200 ft. Sherman Dam has four toe drains, but No. 3 is the only one with significant flow.

Toe drain No. 3 has had a steady decrease in discharge rate over the past 14 years. In 1998, the irrigation district used a standard sewer cleaner with a jetting pressure of about

1,500 lb/in^2 in an attempt to improve the efficiency of the toe drain. The discharge rate changed from about 4 gal/min before the cleaning process to 15 gal/min immediately after the process. The rate dropped to 12 gal/min at the next reservoir peak. The 12-gal/min rate is about 40 percent of the discharge rate observed in 1986.

The early readings obtained from toe drain No. 3 are difficult to explain. The discharge rate increased steadily between mid-1980 and mid-1982 from 8 gal/min to 70 gal/min. Then the rate leveled off until mid-1983, when it started climbing to about 95 gal/min in March 1984. Normally, when a new toe drain is installed at a dam where the reservoir has been full or nearly full for several years, as is the case with Sherman Dam, the discharge from the toe drain would not take 4 years to peak. This suggest that either the readings were inaccurate or something other than reservoir seepage contributes to the flow in the toe drain.

After March of 1984, the discharge rate started dropping rapidly until it was only 29 gal/min in August of 1984. The drop in discharge was unrelated to changes in the reservoir. Again, this indicates that something other than the reservoir level was influencing the toe drain discharge, or else the readings were in error.

Between 1986 and 1998, toe drain No. 3 had a gradual decrease in discharge (see fig. 75). The gradual decrease in discharge was similar to the change that might be expected if the toe drain were slowly plugging off. By 1998, the discharge rate was fluctuating between 2 and 5 gal/min, which was down considerably from the 29-gal/min rate in 1986.

Some observation wells such as OW-84-1 and OW-84-3 indicated gradual increases in groundwater level of 5 to 6 feet between 1986 and 1998 (see fig. 76). These increases could be the result of decreasing efficiency in toe drain No. 3 during the same time period. Figure 77 shows the location of the observation wells with respect to the dam and the toe drain outfall.

a. *Drain Cleaning Operation*.—The irrigation district used a Meyer sewer cleaning tool to rehabilitate drain No. 3. The tool has a 2-inch diameter nozzle that can generate a pressure of about 1,500 lb/in^2. Some jets on the nozzle are angled forward; but most of the jets are angled backward, allowing the nozzle to pull the supply hose up the drain pipe. The pressure from the jets also has a self-centering effect, keeping the nozzle approximately in the center of the drain pipe.

During this operation, the sewer cleaning unit works its way up the toe drain at its own rate. Most of the cleaning occurs as the unit is pulled slowly out of the toe drain. The nozzle advanced 600 ft up the drain pipe, which was the extent of the cleaning operation. The operator estimates that the nozzle was pulled out at a rate of about 1 ft/s. The cleaning tool was run up and down the 600-foot section of the toe drain three times.

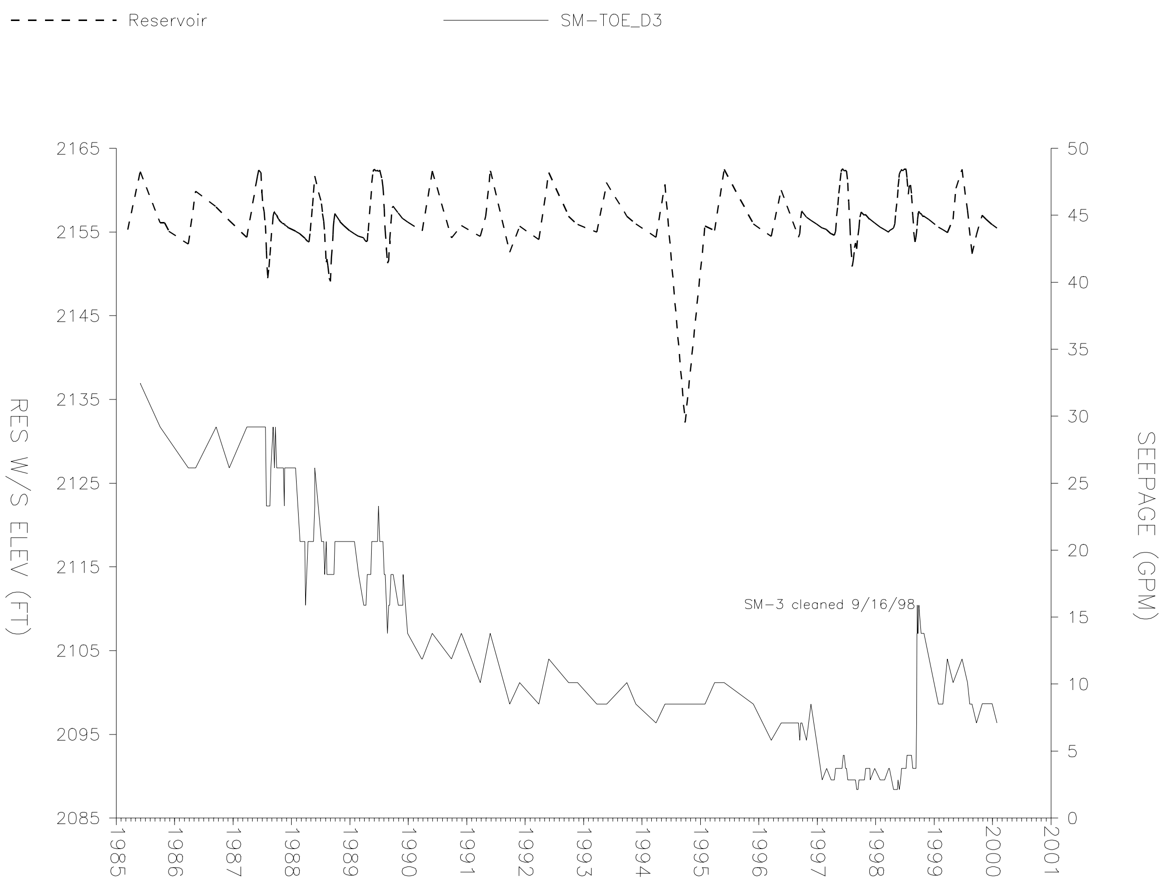


Figure 75.—Seepage flows from toe drain No. 3.

Unfortunately, no video of the inside of the drain pipe was obtained before or after the cleaning. This prevented a comparison of before and after conditions inside the pipe. Also, without the video information, the irrigation district was not able to determine the plugging mechanism.

The discharge rate, after cleaning, was measured at 16 gal/min (see fig. 75). This may have included some return water from the cleaning operation, because the next time the reservoir peaked in June of 1999, the discharge rate was 12 gal/min. The 12 gal/min rate is about 2.5 times the discharge rate prior to cleaning and about 40 percent of the discharge rate measured in 1986. The water levels in observation wells OW-84-1 and OW-84-3 dropped back to about the same levels recorded in 1986.

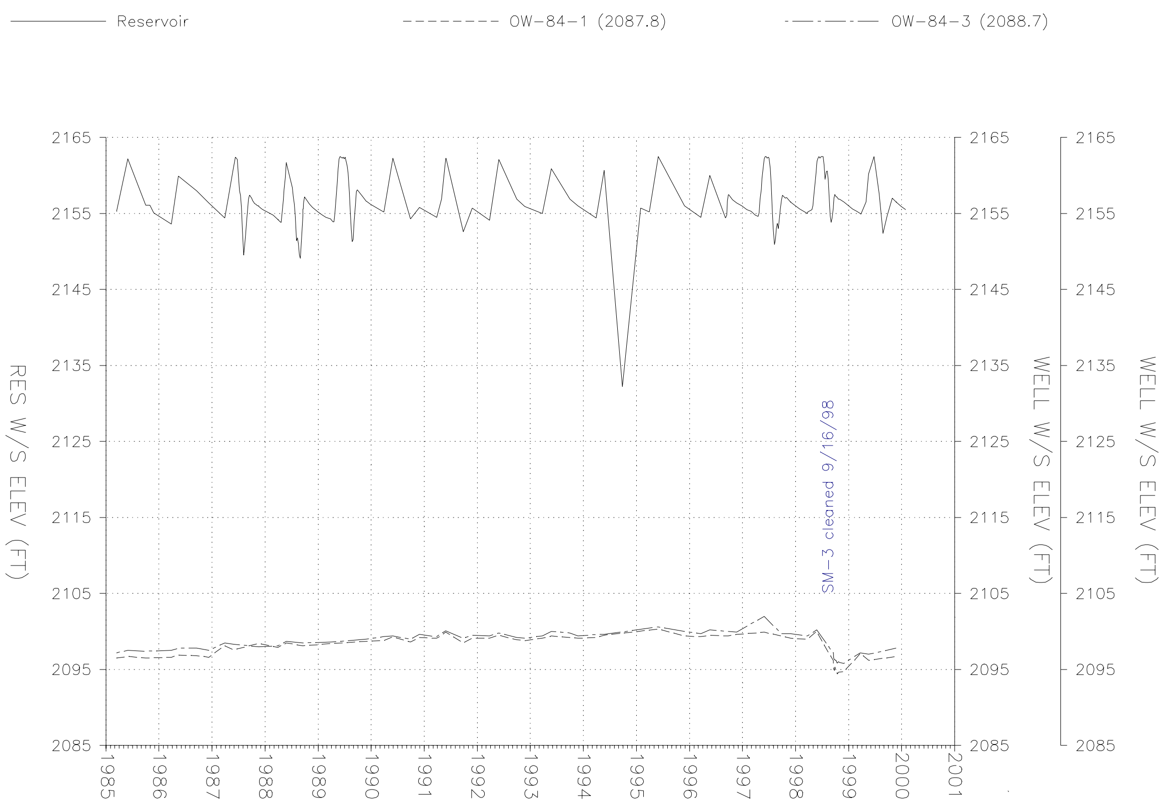


Figure 76.—Readings of groundwater levels from observation wells near toe drain No. 3.

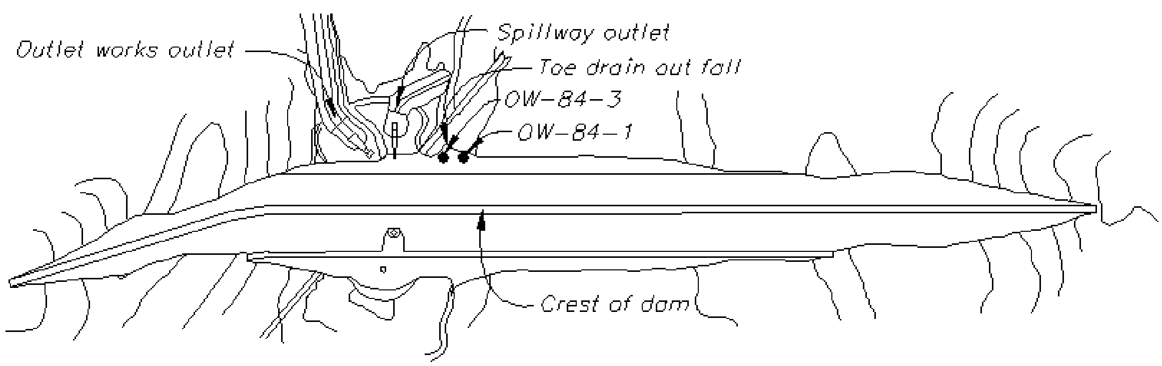


Figure 77.—Locations of observation wells with respect to the dam and the toe drain outfall.

b. *Conclusions*.—The sewer cleaner significantly improved the short term efficiency of toe drain No. 3. Because it has only been a short time since the cleaning process was completed, long term efficiency of the cleaning operation has not been evaluated. It should be noted that only half the length of the drain was cleaned. If the entire length were cleaned, the discharge rate might have improved even more.

Cleaning the toe drain several years earlier, and perhaps more often, might have resulted in the ability to maintain a higher discharge rate.

If the plugging mechanism was the result of biological fouling (such as iron bacteria) or mineral incrustation (such as calcium carbonate), the cleaning process would likely be more effective if an additive such as bleach or sulfumic acid were mixed with the cleaning water.

Video of the toe drain taken before cleaning would be useful in trying to determine the cause of the toe drain plugging. Such information could influence decisions regarding the use of chemicals in the cleaning water. Video of the toe drain taken after the cleaning operation would be useful in evaluating how effectively the perforations were cleaned, and determining whether or not the pressure jetting did any damage to the inside of the drain pipe.

4-9. Upper Stillwater Dam—Foundation Drain Rehabilitation.—

a. *Background*.—Upper Stillwater Dam, constructed from 1983 to 1987, is a Reclamation RCC gravity dam located 31 miles northwest of Duchesne, Utah. The dam has a structural height of 292 feet and a crest length of about 2,650 feet. Three-inch diameter foundation drains were drilled at 10-foot centers and at least 75 feet into the dam foundation, with the intent of penetrating the unit L argillite layer. The drains were provided to reduce uplift pressures acting at the base of the dam and within the dam foundation. Figure 78 provides the layout of the foundation drains. The drains are located in a 6-foot wide gallery, with the gallery centerline located 20 feet from the upstream face of the dam. The gallery extends through the dam from one abutment to the other. The invert of the gallery is at two different elevations, elevation 7992 through most of the dam (lower gallery) and elevation 8042 at the left abutment (upper gallery). Foundation grouting was also performed from within the gallery. Adits, which connect to the foundation gallery, were also excavated into the abutments. Upward and downward drain holes were drilled from the adits. Figure 79 provides details of the foundation gallery. The foundation for Upper Stillwater Dam consists of Pre-Cambrian sandstone and argillites with nearly horizontal bedding planes. It was recognized that fine sand and silt infillings were present in some joints and bedding plane partings.

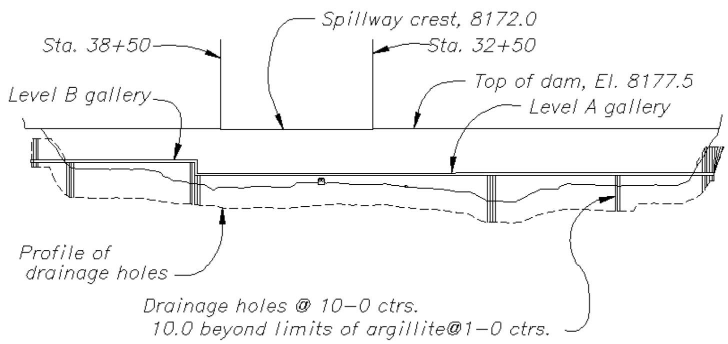


Figure 78.—Profile of Upper Stillwater Dam.

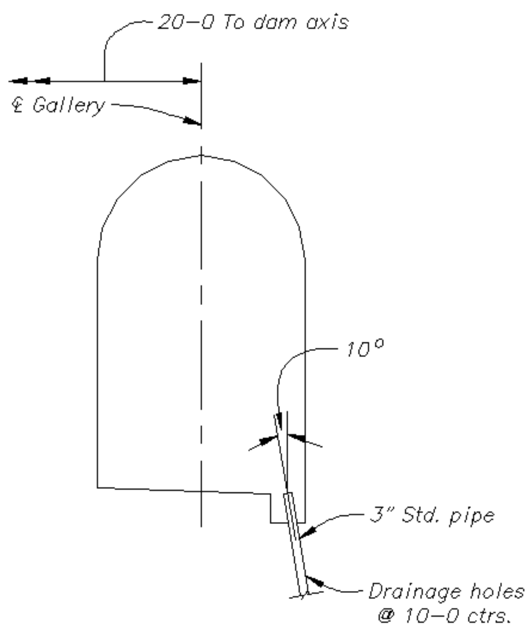


Figure 79.—Cross section of foundation gallery.

After the dam RCC was placed, a single row grout curtain was constructed from the gallery and also from abutment adits. Holes were drilled as deeply as 150 feet into the foundation rock, inclined from vertical by 5 degrees upstream and by 30 degrees toward the nearer abutment. Downstream of the grout curtain, a drainage curtain was constructed from the gallery and abutment adits by drilling holes into the foundation. A gutter system in the gallery collects water from the foundation drains, and three 12-inch diameter steel pipes

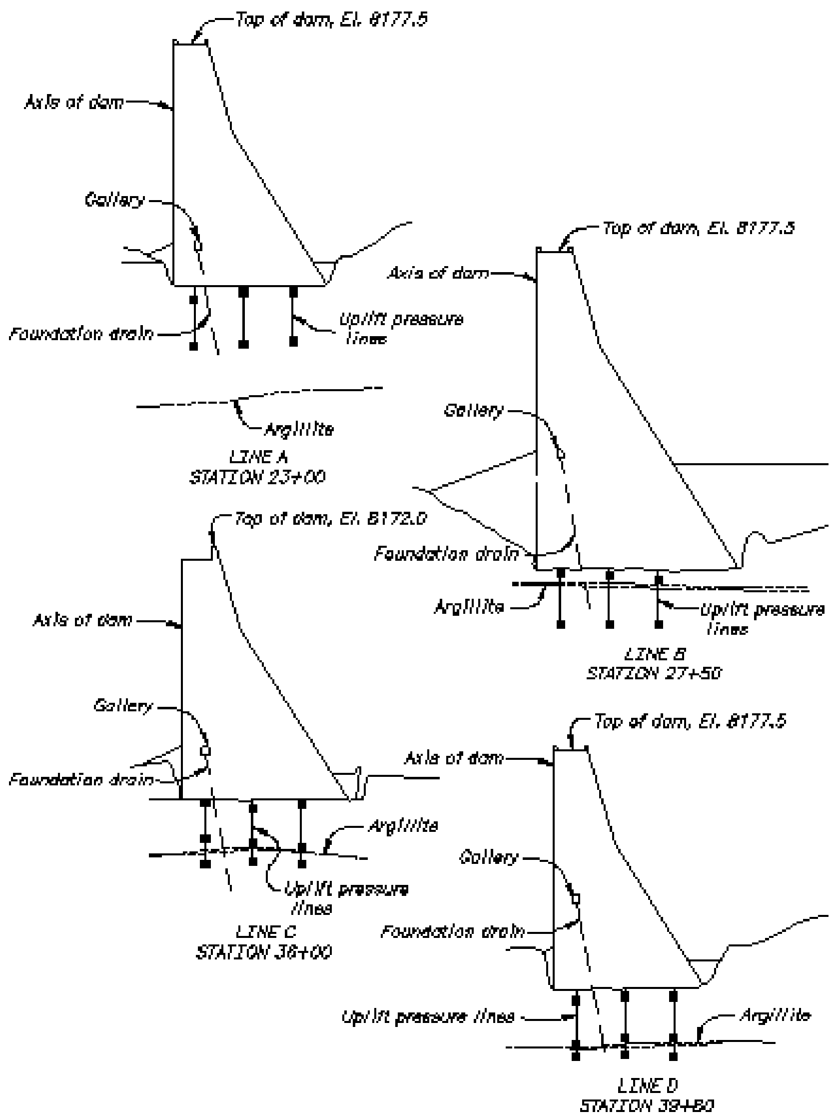


Figure 80.—Uplift pressure lines.

carry water from the gutter to below the water surface in the spillway stilling basin. Drain flows can be measured individually at each drain, or total drain flows can also be measured by weirs in the foundation gutter, where the gutter flows dump into the 12-inch pipes.

Four lines of piezometers are provided within the dam foundation to provide upstream/downstream profiles of foundation uplift pressures. The four lines are located at stations 23+00 (line A), 27+50 (line B), 32+00, (line C) and 38+70 (line D). Figure 80 provides cross sections at each of the uplift pressure lines.

During first filling of the reservoir, flowing sand from some of the drains and filling of some of the drains with sand (which reduced the effectiveness of the drains) was noted. It was believed that certain areas of the foundation had developed complex interconnected paths formed by the intersection of joints, shears and bedding planes. Sand was transported into the drains from these paths over the first 5 years of reservoir operation.

Slotted PVC pipe wrapped in filter cloth was installed in some of the drains to filter the migrating sand. Most of the filter-wrapped drains plugged completely due to the presence of iron-fixing bacteria, and the installation of the filters was discontinued. Tests using filter socks indicated that the plugging of the filter fabric typically occurred in a few hours.

The source of the sand is backfill material placed at the upstream toe of the dam flowing through cracks in the RCC and sand-filled joints in the dam foundation. Concerns over clogged drains was that the factor of safety for sliding on shallow beds would be reduced, and washing of sand from foundation joints in large quantities could lead to settlement, cracking, and ongoing maintenance problems for the dam.

b. *Remedial Measures*.—In 1992 and 1993, remedial action was undertaken to address the migration of sand into the drains. A limited grouting and drainage program was initially planned, but the program was expanded to include grouting and drain remediation across the entire foundation due to the high grout takes that occurred at the start of the program. The treatment from the gallery included grouting upstream, redrilling the downstream drains, and installing slotted drain pipe, some surrounded by a filter pack. The *Technical Report of Construction* [36] provides details of the program.

The grouting program drastically reduced the rate and distribution of sand infiltration into the drains and the drain flows. Probe data of the drain holes from 1993 indicated a definite reduction in sand infiltration rates with the reservoir at maximum elevation. In a 1995 report [37], caution was urged in installing 1- or 1.5-inch slotted PVC pipe in the 4-inch drains until it could be proven that the iron bacteria problem has been eliminated and that there is a real need for filter installation. Iron bacteria is not a problem in the 4-inch open drains, because the drains can be readily cleaned.

Some of the piezometers in the dam foundation indicated increases in foundation uplift pressures after the grouting. Most of the increased pressures were upstream of the drainage curtain, with only one increase occurring downstream of the drainage curtain. Some of the piezometers indicate pressures that exceed Reclamation criteria, and all of these are upstream of the drainage curtain. The pressures that exceed Reclamation criteria act over relatively small areas and do not present a significant concern [37]. See figure 81 for the plot of piezometer data along line B.

Figure 82 provides total seepage data before and after the foundation treatment program. The plot indicates a dramatic decrease in drain flows, and hence sand inflow, as a result of the foundation grouting program. It should be noted that the urethane grout used to seal cracks in the RCC deteriorated with time, and flow rates again increased.

c. *Conclusions.*—Foundation drains at Upper Stillwater Dam began filling with sand during first filling of the reservoir. A remedial grouting and drainage program greatly reduced the amount of sand infiltration. The probed depths of drains indicate that the sand infiltration has stopped at most drains since 1993 (see figs. 83 and 84 for plots of typical drain probed depths). Drain depths continue to be monitored.

The experience at Upper Stillwater Dam revealed the potential for filter fabric becoming plugged from iron bacteria. While the intent of the fabric was to filter sand and prevent potential movement of foundation materials, it had the effect of reducing the effectiveness of drains, increasing foundation pressures, and increasing the potential for sliding within the foundation.

4-10. Concrete Dam Foundation Drain Cleaning.—

a. *Background.*—This case history involves a concrete gravity dam located in the Pacific Northwest that is owned and operated by an agency other than the Bureau of Reclamation. The dam is 256 feet high and 3,791 feet long. Bedrock at the dam is hard basalt. The basalt is comprised of successive series of basalt flows separated by thin (several inches to several feet thick) interlayers of volcanic ash and tuff. The ash and tuff layers are typically hard, being baked by deposition of the overlying basalt flow.

The dam was constructed in 1969. Five hundred and forty foundation drains were drilled to 80-foot depths to relieve uplift pressures that may develop under the dam and potentially cause stability problems. Since construction of the dam, the foundation drains experienced gradual loss of effectiveness, inducing increased uplift pressures.

b. *Drain Cleaning History.*—Foundation drains in the dam experienced gradually reduced flows and increased uplift pressures that caused concern for static and dynamic stability of the structure. The drains became plugged with calcium carbonate and silt that entered through fractures intersecting the drain holes in the basalt bedrock. Since no natural, geological source of calcium carbonate occurs in the area, the source of the calcium carbonate is believed to be the concrete dam and/or the cement grout curtain. The source of the silt is sedimentation in the reservoir.

In 1992, conventional drilling methods were used to clean the silt and calcium carbonate from the foundation drains. The boreholes were reamed using diamond bits to clean the borehole walls, increasing the borehole size by an additional 3/32 inch.

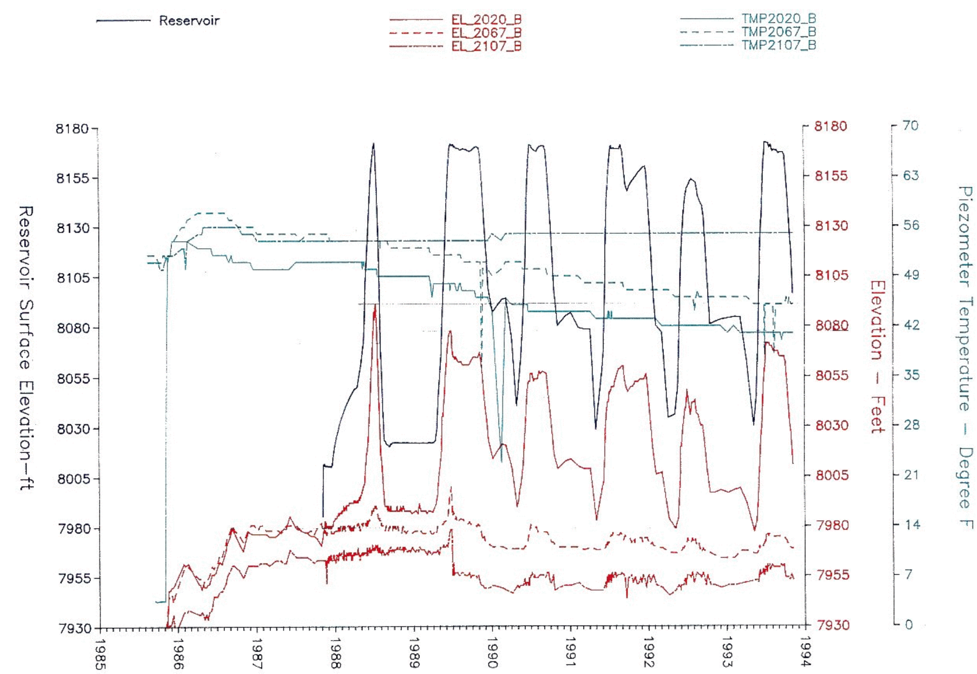


Figure 81.—Plot of piezometer data along line B at Upper Stillwater Dam.

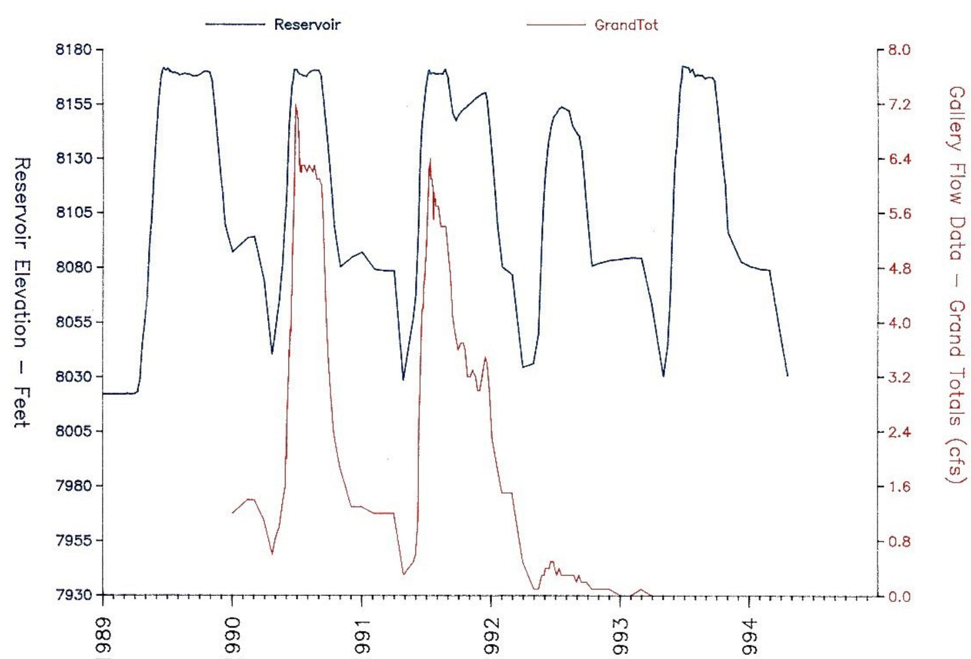


Figure 82.—Total seepage before and after the foundation treatment program.

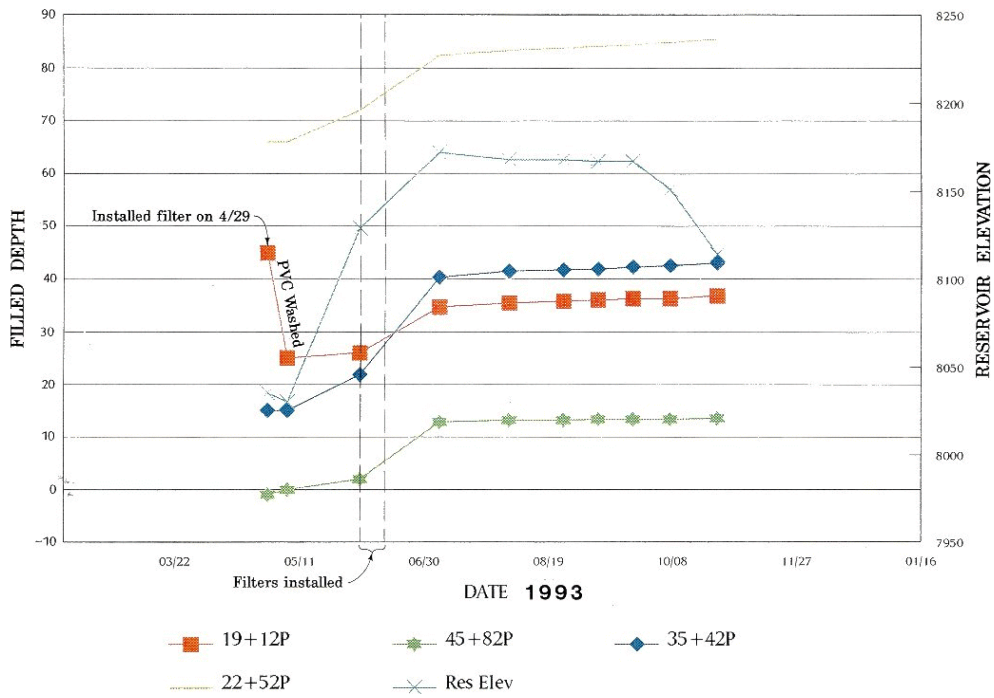


Figure 83.—Filling of drains containing PVC and gravel filter.

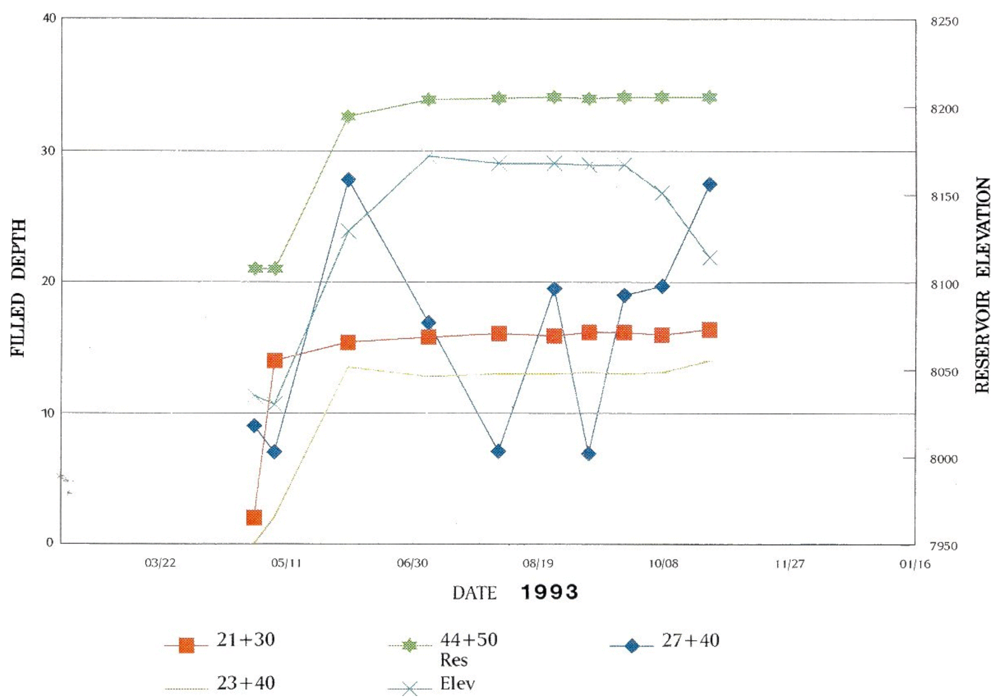


Figure 84.—Filling of drains without PVC.

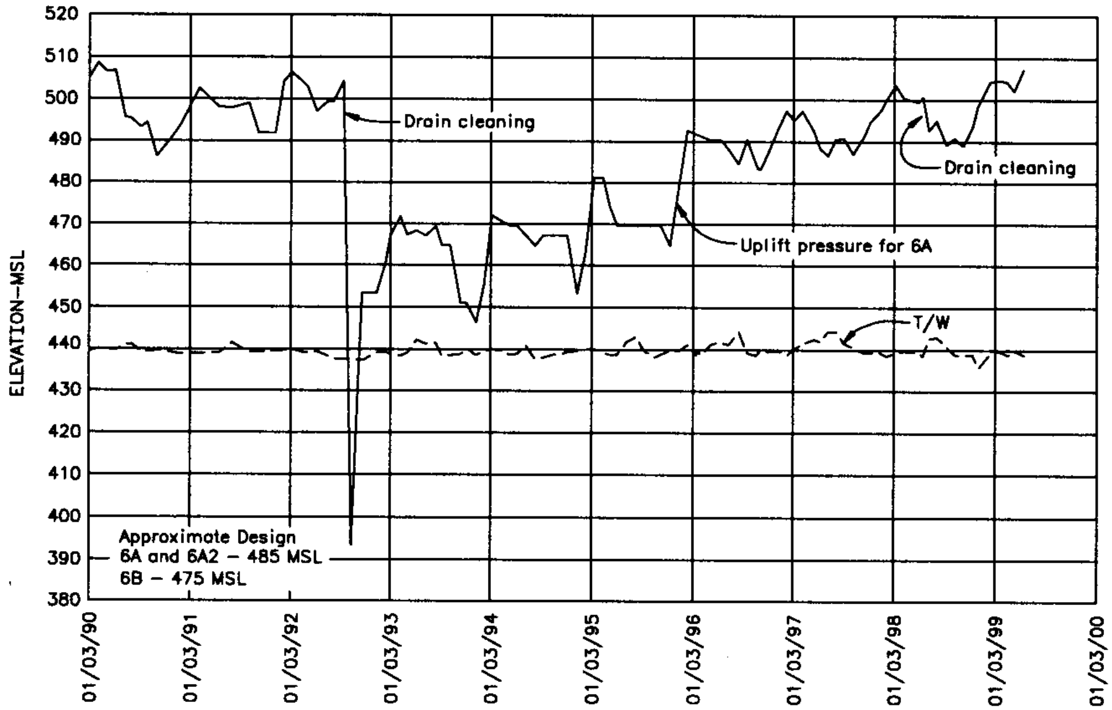


Figure 85.—Uplift pressures response to drain cleaning.

Figure 85 shows the immediate and significant response to the 1992 drain cleaning operation. Measured uplift pressures were immediately reduced by 50 feet of pressure. Figure 85 also shows a gradual decrease in drain effectiveness between mid-1992 and 1998 until uplift pressures nearly returned to precleaning levels.

In 1998, a contract was issued for drain cleaning of the same drains using high pressure fluid (water) jet methods. The contract allowed the contractor to determine the pressure and nozzle configuration needed for the cleaning operation. High pressures of 12,000 to 13,000 lb/in^2, and low flow rates of 15 gal/min were used.

c. *Conclusions.*—The following conclusions were reached from rehabilitating the drains with conventional drilling methods:

- The use of conventional drilling methods in 1992 to clean the drain holes was effective but expensive.

- The borehole walls were cleaned of any calcium carbonate coating during the process of reaming and enlarging the borehole diameter $^3/_{32}$ inch.

- The drilling method used clean water for drilling fluid that thoroughly washed the borehole, lifting and removing calcium carbonate and basalt cuttings and silt from the borehole, resulting in renewed effectiveness of the drain holes.

- The conventional drilling action may also have broken and removed calcium carbonate for a short distance into the fractures that intersect the borehole, removing blockages in the seepage path and enhancing flow into the boreholes.

The following conclusions were reached from rehabilitating the drains with high pressure fluid jet methods:

- The use of high pressure fluid jet methods in 1998 to clean the drain holes was marginally effective.

- Insufficient pressure may have been applied to induce scouring/etching of the calcium carbonate from the borehole walls.

- The water jet was raised within the borehole too rapidly to adequately induce scouring/etching of the calcium carbonate on the borehole walls.

- The water jet nozzles were not properly oriented and/or sized to induce scouring/etching of the calcium carbonate on the borehole walls.

- The water jet did not have sufficient flow to lift loosened calcium carbonate deposits or silt from the borehole. After the water jetting process is completed, a wash tube should be inserted to the bottom of the hole to lift/wash loosened debris from the hole.

- If the drain holes are open sufficiently to allow a borehole camera to be lowered, the condition of the drain hole walls should be determined prior to cleaning, and then after the cleaning process. Inspection of the drain hole walls prior to initiating cleaning operations could identify specific areas of calcium carbonate deposits and identify potential problem areas such as fractured rock or soft zones in the rock. Water jet cleaning in the potential problem areas could be avoided so that caving or erosion of the drain hole walls is not induced. Inspection of the drain hole walls after the cleaning operation could identify areas of calcium carbonate requiring remedial cleaning, if additional material adheres to the drain hole walls.

4-11. Tuttle Creek Dam—Blended Chemical and Heat Treatment for Bacteria in Relief Wells.—The U.S. Army Corps of Engineers (USACE), Kansas City District, has used a process referred to as a blended chemical and heat treatment (BCHT) process to combat iron, sulfate-reducing, and slime bacteria in relief wells. Environmental restrictions often preclude the use of acids. The hot water treatments (well water temperatures greater than 130 degrees) are environmentally safe, and they have been found to be effective in killing bacteria in relief wells. The process has been used on at least seven dams in the Kansas City District, most notably Tuttle Creek Dam.

a. *Background*.—Tuttle Creek Dam, which the USACE constructed and operates, is located 5 miles north of Manhattan, Kansas on the Big Blue River. The dam consists of a rolled earth and rockfill embankment with a gated, concrete chute spillway on the left abutment, and an outlet works with intake tower near the right abutment. The embankment has a maximum height of 157 feet and a crest length of 7,500 feet. Construction of the dam was completed in 1959. Figure 86 is a photograph of the dam at the maximum section with the outlet channel in the foreground. Bacterial infestation of the relief well screens was identified as a problem at Tuttle Creek Dam, and the USACE has been using the BCHT process to treat the wells.

b. *Relief Well Treatment Process*.—Prior to treating the wells, the water is sampled and tested for bacterial types. Three main categories of bacteria are treated with the BCHT

Figure 86.—Tuttle Creek Dam.

Figure 87.— Hotsy unit used in the BCHT process.

process. These include sulfate-reducing , iron, and slime bacteria. The wells are pump tested prior to the treatment to determine the specific capacity for comparison with the value obtained after the treatment process. An air lift (see sec. 5-4 for a description of the air lift) is used to remove any silt or organic material from the bottom of the well. If soil particles as well as bacterial growth are responsible for clogging, or if there is uncertainty about the cause of clogging, the relief well should be surged with a surge block as part of the rehabilitation process. Surge blocks are discussed in section 5-7.

In the BCHT process, boiling water is pumped into the relief well through a pipe with ½-inch diameter water jets. The water jets direct boiling water through the well screen and out into the gravel pack. The top of the well is packered off to force more of the hot water to flow out through the screen, and in ideal situations the hot water may also penetrate the aquifer.

Figure 87 is a photograph showing the "Hotsy" unit that the USACE uses to heat and control the boiling water to be injected into the relief wells. The unit is basically a steam cleaner with a high capacity boiler for heating the water.

Typically, the well is treated in 10-foot vertical intervals. A steel plate with a rubber gasket is used at the top of the 10-foot interval to impede hot water from rising past the top of the interval. The steel plate does not need to create a tight fit to be effective; but to verify that water in the treated interval is hot enough, a thermal sensor is used. The temperature should be at least 130 °F. For unusually difficult bacterial infestation problems, it may be necessary to reduce the treated interval to 5 feet. Initially, a trial-and-error process might be required to determine the best length for the interval. Once the water temperature in the interval reaches 130 °F, the interval is treated for about 15 minutes. As each interval is completed, chlorine (household bleach) is added to the jetting fluid to increase the effectiveness of the treatment. Approximately ½ to 1 gallon of household bleach is used per interval. The casing above the well screen is also jetted to kill any bacteria that may be attached. However, the process goes faster for the unslotted casing, because the hot water heats up faster when it doesn't pass through the screens into the aquifer.

When the BCHT process is complete, the relief well is again air lifted to remove any organic debris and/or soil in the bottom of the relief well. A pump test is performed to determine the specific capacity of the well after treatment. The specific capacity is typically greater than the value obtained before the treatment. However, the goal of the BCHT process is to achieve at least 80 percent of the specific capacity that was measured when the well was originally constructed.

Generally, the time period between treatments is 1 to 2 years. However, there have been cases where the bacterial infestation increases significantly within a few months. This is believed to be the result of nutrient sources that naturally exist in some aquifers.

c. *Conclusions.*—The USACE Kansas City District has used the blended chemical and heat treatment process for relief wells on at least seven dams, including Tuttle Creek Dam. This method has often been effective in reestablishing a specific capacity equal to 80 percent or more of the original specific capacity of the relief wells. The method is also environmentally safe, because strong acids are not required as part of the treatment process.

4-12. Davis Creek Dam Toe Drain Rehabilitation and Cleaning.—

a. *Background.*—Davis Creek Dam was completed in 1992 and is located about 6 miles southeast of North Loup in Central Nebraska. The dam is a homogenous earthfill embankment with a structural height and crest length of approximately 110 feet and 3,000 feet, respectively. The toe drain system consists of two toe drains, one to the right of the outlet works centerline and another to the left of the outlet works centerline. The right and left toe drains consist, respectively, of approximately 1,200 feet of 8-inch and 1,440 feet of 12-inch diameter perforated, corrugated polyethylene pipe. Flow from the right toe drain is measured by a V-notch weir, located about 30 feet to the right of the outlet works centerline in inspection well No. 7 (see fig. 88). Flow from the left toe drain is measured by a V-notch weir located at the end of a weir box. The weir box is on the ground surface several

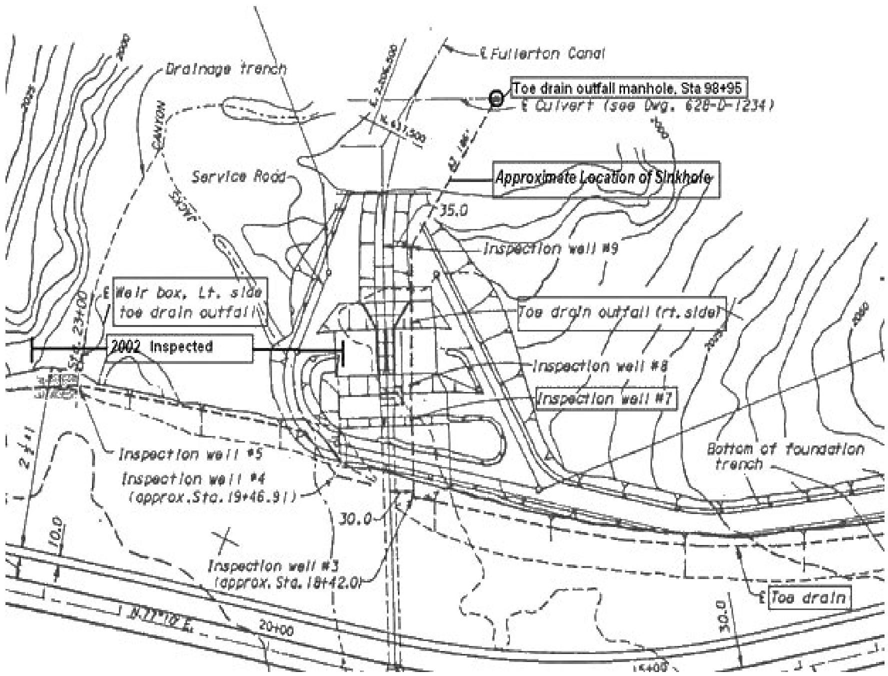


Figure 88.— Locations of observation wells and cleaned reaches.

hundred feet to the left of the outlet works centerline. The toe drains meet at the location of the toe drain outfall manhole, station 98+95, where they flow into the Jack Canyon drain pipe. The Jack Canyon drain pipe was constructed to carry toe drain discharges and surface runoff. The Jack Canyon drain pipe extends for about 1,100 feet and consists of 18-inch-diamter perforated, corrugated polyethylene drain pipe.

In the spring of 1994, a sinkhole 8 to 10 feet deep and approximately 20 feet wide developed above the 12-inch nonperforated, corrugated polyethylene outfall pipe. The sinkhole was located along the right outfall about midway between inspection well No. 9 and the Jack Canyon diversion drain culvert outlet transition. Drain rehabilitation in the fall of 1994 and the spring of 1995 consisted of replacing the 12-inch outfall drain pipe with a 12-inch perforated pipe placed within a gravel envelope.

b. *Inspections and Cleanings.*—In November of 2000, Reclamation performed a video inspection of the toe drains at Davis Creek Dam as part of routine drain maintenance. Observations from the video inspection showed areas of pipe failures, other potentially damaged areas of pipe, and sediment deposition. The photo in figure 89, taken during the inspection at an unknown location, shows the typical amount of sediment deposition that was seen in the toe drain pipe.

Figure 89.—The typical amount of sediment deposition observedin toe drain pipes in a November 2000 inspection.

Based on this video inspection, selected reaches of the Davis Creek toe drains were cleaned in January of 2002. The drains were cleaned by the Twin Loups Irrigation District which used the Farwell Irrigation District's sewer cleaning equipment. The reaches cleaned were located in the left toe drain pipe from stations 19+46.91 to 23+00 and from stations 23+00 to 26+00; however, care was taken not to wash out any of the materials from the locations where the pipe was damaged.

On February 12, 2002, Reclamation inspected the cleaned reaches, including the short reach of the Davis Creek toe drain outfall replacement pipe and stations 98+95 to 99+12 of the Jack Canyon drain. The video inspection consisted of viewing the interior surfaces of drain pipe using Reclamation's inline MicroTrac camera-crawler. The inspection of the left toe drain at Davis Creek was within the 12-inch diameter pipe. The camera-crawler was inserted into the manhole at station 23+00, and then it proceeded downstream to station 19+46.91. The camera-crawler was then backed out and turned around in order to proceed upstream. The camera-crawler proceeded upstream to approximately station 23+25. The reaches of the left toe drain pipe that were inspected are outlined in figure 88. Originally, it was intended to inspect the entire cleaned reach to station 26+00, but the camera-crawler was unable to proceed when it came across damaged pipe that was previously reported during the 2000 inspection. Figure 90 shows the results of toe drain cleaning and the pipe damage

Figure 90.—A damaged left toe drain pipe at approximately sta. 23+25 stopped the camera-crawler. A cleaning removed fine materials from the pipe invert.

that halted the camera-crawler. This photograph was taken in the Davis Creek toe drain at approximately station 23+25. From the photograph, it can be seen that the fine materials previously seen on the pipe invert have been removed.

For the short reach of the Davis Creek toe drain outfall replacement pipe, the crawler went upstream in the manhole at station 98+95 and was halted at approximately station 98+78, where the camera-crawler encountered 2- to 3-inch diameter rocks. The 2- to 3-inch diameter rock might be a result of construction activities for the rehabilitation. Next, the camera-crawler was sent downstream of the manhole into the 18-inch diameter Jack Canyon drain. The camera-crawler was able to inspect the drain from stations 98+95 to 99+12.

Typically, flows in the left toe drain range from 30 gal/min to a maximum of 253 gal/min with the flow record paralleling the reservoir level. On the other hand, the right toe drain is typically dry or flows are limited to less than 20 gal/min. Toe drains typically show a decrease in flow leading up to the cleaning and then an increase in flow after the cleaning. A graph of toe drain seepage and reservoir elevation data with respect to time is given as figure 91. From the graph, it can be seen that flow did increase a few months after the cleaning; however, the reservoir elevation also increased. Since both the reservoir and the

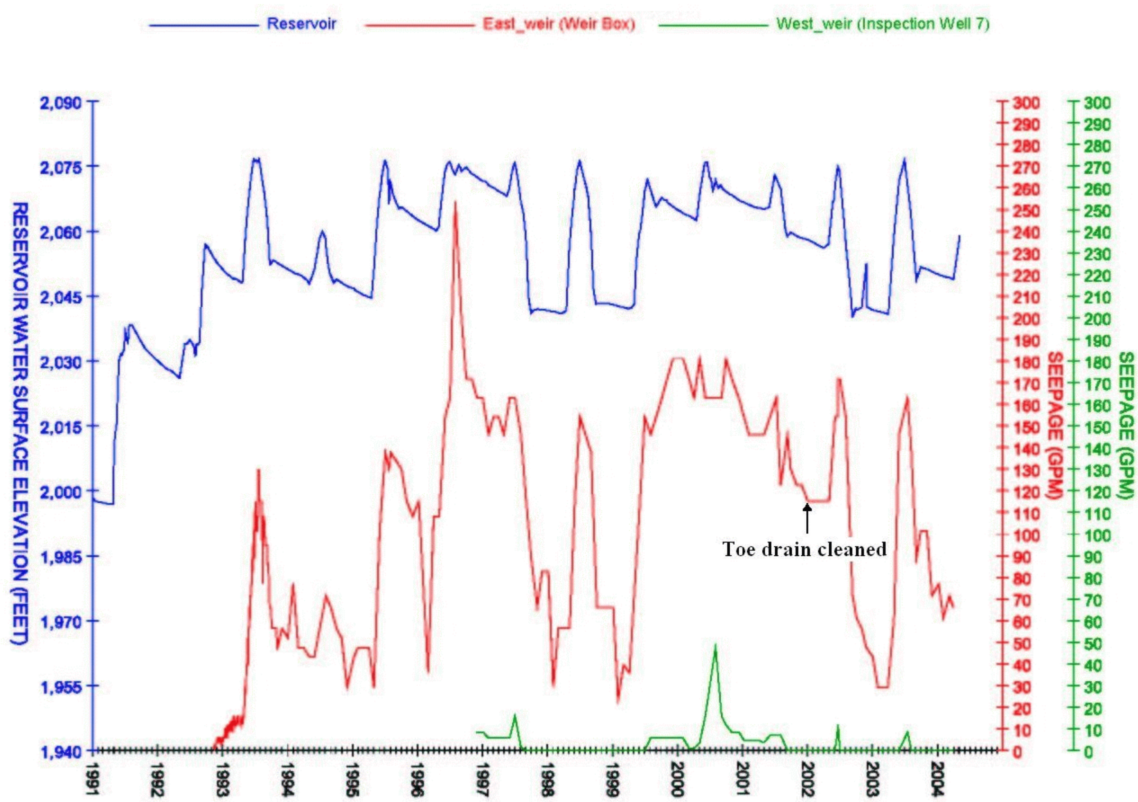


Figure 91.—Readings of left and right toe drain seepage relative to reservoir level.

toe drain seepage increased, it is hard to determine if any increase in the toe drain seepage was a result of the cleaning.

c. *Conclusions.*—The long-term efficiency of the cleaning operation is unknown, since the 2002 video inspection was completed so soon after cleaning. In the short term, the sewer cleaner was effective in removing most of the deposited sediments within the cleaned reaches. It was also seen that no damage occurred inside the drain pipe because of the pressure jetting. Decreases in toe drain flows were not seen before cleaning, nor were higher flows seen immediately following cleaning. It should be noted that only a portion of the toe drain was cleaned. If the entire length were cleaned, the discharge rate might have increased. It is also possible that the sediments plugging the toe drain are not controling toe drain flows.

In a 1994 field examination, it was concluded that the sink hole developed from material being transport into an open, collapsed pipe. It was thought the collapse of the pipe could have occurred either from equipment load during construction or from earth pressure on the outside of the pipe. A video inspection immediately following or during construction would have been helpful in pinpointing the cause of the pipe failures. Even though the cause of the sinkhole could not be pinpointed, the video inspections in 2000 and 2002 were helpful in viewing the condition of the toe drain pipe and supporting the conclusion from the 1994 exam. Both inspections noted some pipe failures that could facilitate the development of sink holes.

4-13. Keechelus Dam—Piping into Drainage System.—Keechelus Dam is located in central Washington about 20 miles northwest of the town of Cle Elum. The dam, which was constructed between 1913 and 1917, has a structural height of 138 feet and a crest length of 6,550 feet. At the top of active conservation pool, the structure impounds 157,000 acre-feet of water.

a. *Background*.—During construction, a rock drain was placed at the downstream toe of the dam. This drain did not contain a drain pipe, but it did have at least 17 known discharge pipes to serve as outfalls for the rock drain. These outfall pipes had a variety of diameters and compositions. None were slotted, but some were open jointed. The rock drain is not filter compatible with the majority of the embankment and foundation soils that were adjacent to the drain. In addition, the earthfill and foundation materials were considered to be internally unstable with regard to piping. Over the years, sinkholes or depressions have been observed on the upstream face of the dam that were believed to be the result of internal instability of the embankment materials.

In June 1998, a void in the crest of the dam was discovered during excavation for a cable trench. At the same time, the Pacific Northwest Regional Office was in the process of inspecting toe drains at several dams including Keechelus. Reclamation was not aware of all of the outfall pipes that existed at Keechelus Dam when the inspection process started. However, the inspection was expanded to include a search for other unknown outfalls that might exist.

b. *Video Investigation and Monitoring*.—Initial video inspections were performed in September and October of 1998. Although the outfalls were generally in good condition with only a limited number of cracks, silt to gravel sized debris (see figs. 92 and 93) was present in most of the outfall pipes. Many of the pipes had little or no flow, but water marks observed during video inspection indicated higher flows had occurred in the past.

The majority of outfall pipes that had been discovered by the time of the September and October inspections were cleaned with a water jetting tool. This work occurred in December 1998 and May 1999. The pipes were reinspected by video camera after the cleaning process. The next step was to inspect the outfall pipes again after the 1999 reservoir filling cycle to determine if material from the rockfill toe drain was moving into the

Figure 92.—Clay tile outfall pipe with fines in the invert.

Figure 93.—CMP outfall pipe with coarse grained material.

outfall pipes. The reservoir reached its highest level of the year on July 13, 1999. Only seven outfall pipes flowed with water during the filling cycle. Therefore, the inspections were limited to those seven outfalls. Many of the drains may have remained dry, because the reservoir level was under restriction.

The reinspection of the seven outfall pipes was performed on September 9, 1999. The video inspection revealed that silt to sand sized material was transported into the pipes by seepage flows in six of the seven drains. In some cases, gravel sized material was also found in the drain outfall pipes. The gravel was able to enter through the upstream open end of the outfall pipes, which were exposed to the rock drain. The sediment sumps or weir boxes at the downstream end of the outfall pipes were also inspected and found to contain sediment. Unfortunately, the sediment traps were not covered during the year and outside debris may have been introduced from sources other than the outfall pipes. As part of the September 1999 work, the sediment traps were secured with covers to prevent outside soil from being introduced.

The same seven outfall pipes experienced flow during the 2000 reservoir cycle and were inspected again on October 23, 2000. Silt to sand sized sediments with some gravel were again found in six of the seven pipes. The volume of the soil in the sumps was measured, and in the worst case, the sump for outfall SM-D5 North had two gallons of fines. More typically, sediment volumes were a quart or less. The sumps and weir boxes were cleaned and covered after the October 2000 inspection.

On September 19, 2001 the drains were once again video inspected. The maximum reservoir elevation was about 22 feet lower in 2001 compared to 2000. As a result, only four toe drain outfalls had seepage flows in 2001. All four outfalls indicated fines were transported from the rock drain by seepage flows. In the worst case, SM-D5 North had three quarts of silt to gravel sized soil in the sump. The common sediment sump for SM-D11 and SM-D12, which was cleaned in October 2000, had ½ inch of fines and iron bacteria in the bottom of the sump (see fig. 94) when inspected in September 2001.

c. *Conclusions.*—As a result of the video inspections made in 1999, 2000, and 2001, Reclamation concluded that piping of fines into the outfall pipes was a frequent occurrence that could ultimately result in a dam safety issue. Consequently, the decision was made to install a completely new toe drain system. The new system contained slotted pipe enclosed within an engineered two-stage filter to prevent piping of soils into the toe drain.

Figure 94.—Sediment sump for SM-D11 and -D12 showing fines that accumulated in one season.

4-14. Summary of Toe Drain Inspection Using Closed Circuit Television.— The Bureau of Reclamation's Technical Service Center (TSC) has been performing closed circuit television (CCTV) inspection of toe drain systems as part of their dam safety program since about 2000. CCTV has also been used to perform inspections of wall drains, structural underdrains, pressure relief wells, siphons, pipelines, outlet works and spillway conduits, gates, and valves. The TSC has provided CCTV inspection services to many federal and state agencies.

CCTV inspection equipment consists of a video camera attached to a self-propelled transport vehicle (crawler). The transport vehicle and camera are commonly referred to as a camera-crawler (see fig. 95). An operator remotely controls both the transport vehicle and camera. The camera can provide both longitudinal and circumferential views of the interior of the pipe being inspected. Video images are transmitted from the camera to a television monitor, from which the operator can view the conditions within the pipe. The video images are recorded onto videotape, compact disc, or digital versatile disc (DVD) for technical evaluation and documentation (Report of Findings). The operator can add voice narrative and alphanumeric captions or notations as the inspection progresses. Section C-1 in appendix C provides more details of CCTV inspection equipment.

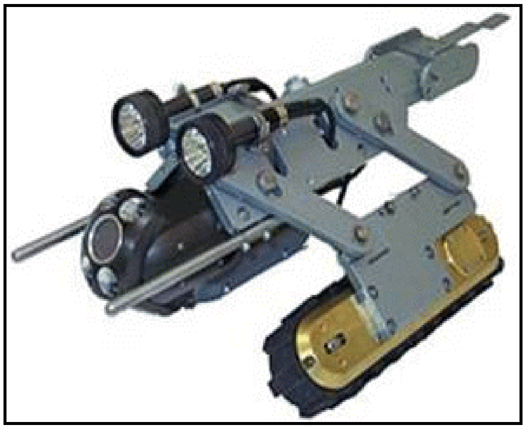


Figure 95.—Camera-crawler used for CCTV inspection.

The TSC performed a series of tests in 2002 to evaluate the performance capabilities using camera-crawlers in double-walled HDPE pipe. The results of the performance tests served as the basis for the development of design guidance on acceptable pipe diameters and bends, invert slopes, and distances between manholes or access entry locations (see app. C) required to accommodate CCTV inspection. The design guidance is generally applicable for use with other pipe materials.

Sometimes a toe drain pipe is so small that a camera-crawler cannot be used, or obstructions or invert conditions exist within the pipe that prevent the transport vehicle from traversing the pipe. For these types of situations, small color cameras (1.5 to 3 inches in diameter) can be attached to metal or plastic poles (often referred to as push poles) and manually pushed up the pipe. Push poles are normally used for straight sections of pipe. The use of push poles for advancement is generally limited to about 400 feet of pipe length. If bends exist in the pipe, a flexible snake device (spring steel wire, coiled wire, or flexible polypropylene-jacketed fiberglass push rod) can be used instead of the push poles. The color cameras are connected to a video cassette recorder and to a television monitor. Snake devices are generally limited to about 75 to 200 feet of pipe length.

a. *Common Pipe Material Types*.—Reclamation has used a variety of pipe materials to construct toe drain systems. In some instances, Reclamation has used combinations of pipe materials. The most common pipe materials are (the numbers in parentheses indicate the percentage of use based solely on toe drain systems inspected with CCTV):

- Clay tile (29%)
- HDPE (25%)

- CMP (22%)
- Concrete (16%)
- PVC (4%)
- Asbestos cement (3%)
- Iron (1%)

Based on the toe drain systems inspected, clay tile, concrete, and CMP pipe were frequently used in older dams (1910 to about 1980), and HDPE pipe has been used in newer dams (1980 to present).

b. *Common Obstructions to CCTV Inspection*.—Toe drain systems can contain a variety of obstructions that may limit the success of a CCTV inspection. These obstructions include (the numbers in parentheses indicate frequency of occurrence):

- Sediments, gravels, and rocks (40%)
- Sharp bends and tee sections (22%)
- Shape deformation and failure (11%)
- Roots (8%)
- Adverse invert slopes (5%)
- CCTV cable tether limitations (5%)
- Joint offsets and separations (3%)
- Pipe diameter constrictions (3%)
- Other (3%)

The type and location of any obstruction encountered affects the overall success of the CCTV inspection. The typical range of completion for toe drain inspection is (percentage is based on the total linear feet of toe drain pipe inspected divided by the total linear feet of toe drain system):

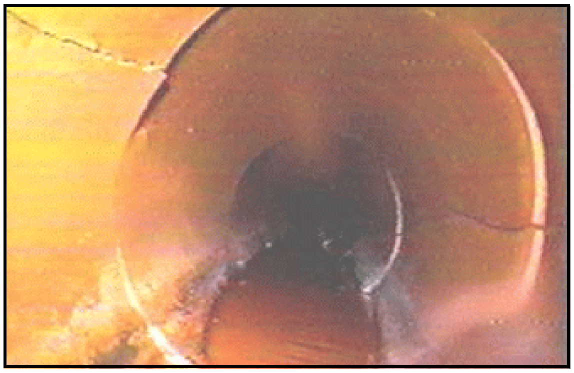


Figure 96.—Extensive longitudinal cracking within a clay tile pipe.

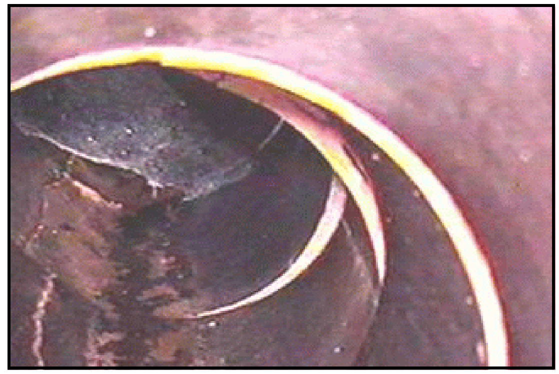


Figure 97.— Extensive joint offsetting within a clay tile pipe.

Inspection completion	Percentage of occurrence
0 to 24%	(49%)
25 to 49%	(15%)
50 to 74%	(13%)
75 to 100%	(23%)

Due to obstructions encountered in the pipes, most toe drain systems cannot be fully inspected. Other options need to be considered to provide more complete inspections, such as high pressure jet washing to clear debris from the pipes, additional access to provide alternate pipe entry locations for CCTV equipment, and future designs made to accommodate CCTV equipment.

c. *Common Defects Cbserved during Inspections.*—Some pipe materials are more prone to specific defects developing over time. The following summarizes instances of specific defects observed inside of the most common toe drain pipe materials.

1. *Clay Tile.*—Longitudinal and transverse cracking was observed in 24 percent of all clay tile pipes. Cracks ranged from hairline to extensive. Figure 96 shows a clay tile pipe that has experienced extensive longitudinal cracking.

Joint offsets and separations were observed in 67 percent of all clay tile pipes. Joint offsets and separations ranged from minor to extensive. Figure 97 shows a clay tile pipe that has experienced extensive joint offsetting.

Figure 98.— Clay tile pipe experiencing inward collapse of the crown.

Figure 99.— HDPE pipe experiencing extensive shape deformation.

Figure 100.—HDPE pipe experiencing failure at the crown. Materials surrounding the pipe have entered through the failure.

Figure 101.—An HDPE pipe joint has experienced extensive separation. Materials surrounding the pipe have entered through the separated joint.

Shape deformation and failure was observed in 24 percent of all clay tile pipes. Shape deformation ranged from minor to extensive. Figure 98 shows a clay tile pipe experiencing a failure of the crown.

2. *HDPE*.—Shape deformation and failure was observed in 56 percent of all HDPE pipes. Shape deformation ranged from minor to extensive. Figure 99 shows an HDPE pipe experiencing extensive shape deformation. Figure 100 shows an HDPE pipe that has failed.

Joint offsets and separations were observed in 11 percent of all HDPE pipes. Joint offsets and separations ranged from minor to extensive. Figure 101 shows an HDPE pipe joint that

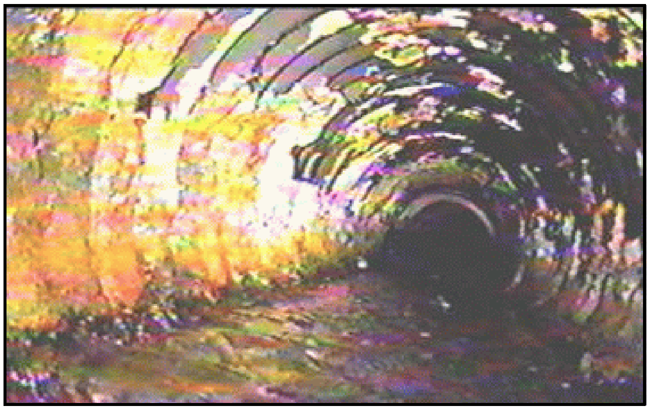


Figure 102.— Extensive deterioration existing within a CMP pipe.

Figure 103.—CMP pipe experiencing extensive loss of surface coating due to delamination.

has experienced an extensive separation and has allowed materials surrounding the pipe to enter through the separated joint.

3. *CMP.*—Deterioration was observed in 75 percent of all CMP pipes. Deterioration ranged from minor to extensive. Figure 102 shows a CMP pipe that has experienced extensive deterioration.

Some CMP pipes have the interior surfaces coated with asbestos bonded or bituminous coatings. Instances of loss of surface coating due to delamination was observed in about 69 percent of all surface-coated CMP pipes. Loss of surface coating ranged from minor to extensive. Figure 103 shows a CMP pipe that has experienced extensive loss of surface coating due to delamination.

4. *Concrete.*—Joint offsets and separations were observed in 58 percent of all concrete pipes. Joint offsets and separations ranged from minor to extensive. Figure 104 shows a concrete pipe that has experienced extensive joint offset and separation at a bend in the pipe.

Cracks were observed in 42 percent of all concrete pipes. Cracks ranged from hairline to extensive. Figure 105 shows a concrete pipe that has experienced extensive transverse cracking.

5. *Asbestos Cement, PVC, and Iron.*—A few cracks and joint offset/separation observations were noted within asbestos cement, PVC, and iron pipe. Figure 106 shows a PVC pipe that has experienced extensive transverse cracking.

Figure 104.—Concrete pipe experiencing extensive joint offset and separation at a bend in the pipe.

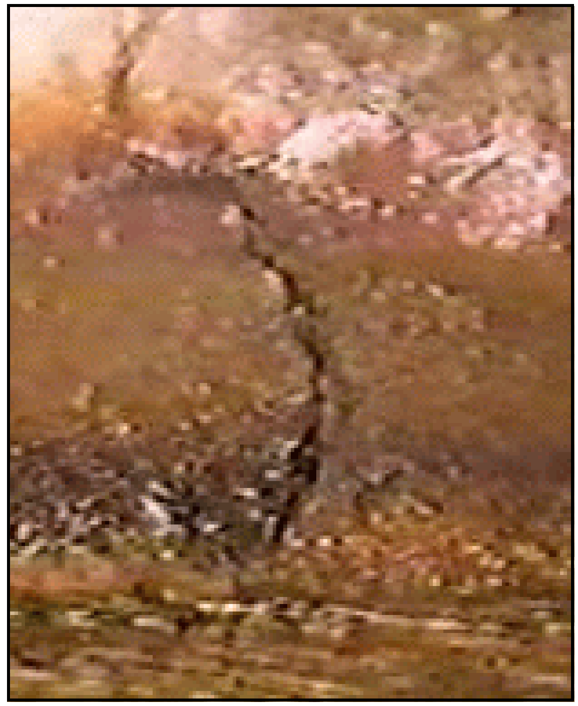


Figure 105.— Extensive transverse cracking within a concrete pipe.

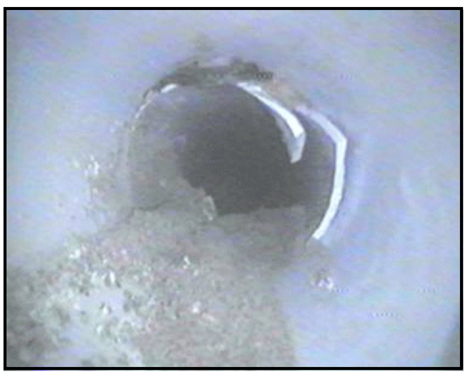


Figure 106.— PVC pipe experiencing extensive transverse cracking.

d. *Plugging Mechanisms.*—Plugging mechanisms can affect the performance of pipe perforations and slots and also the conveyance of collected seepage water. The most common plugging mechanisms encountered were (the numbers in parentheses indicate frequency of occurrence):

- Sediments, gravels, and rocks (36%)
- Biofouling (23%)
- Mineral incrustation (23%)
- Roots (18%)

Figures 107 through 110 show examples of toe drain pipes which have experienced plugging due to sediments, biofouling, mineral incrustation, and roots, respectively.

Figure 107.—Accumulation of sediments has resulted in plugging of the pipe.

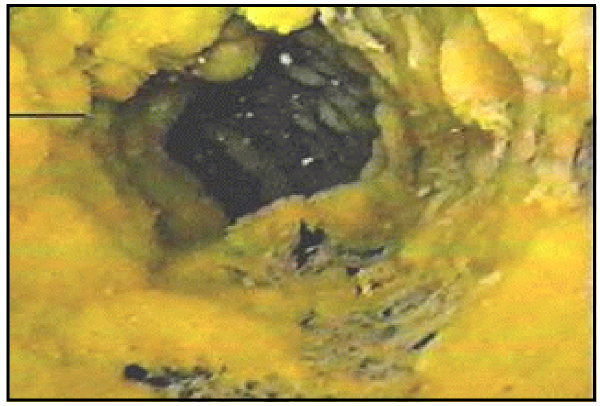


Figure 108.— Biofouling has resulted in plugging of the pipe.

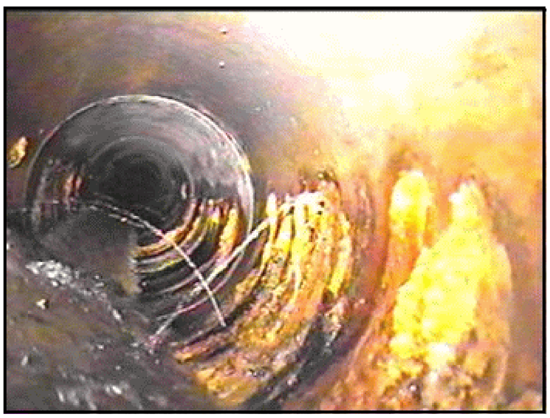


Figure 109.—Mineral encrustation has resulted in plugging of a number of the pipe perforations.

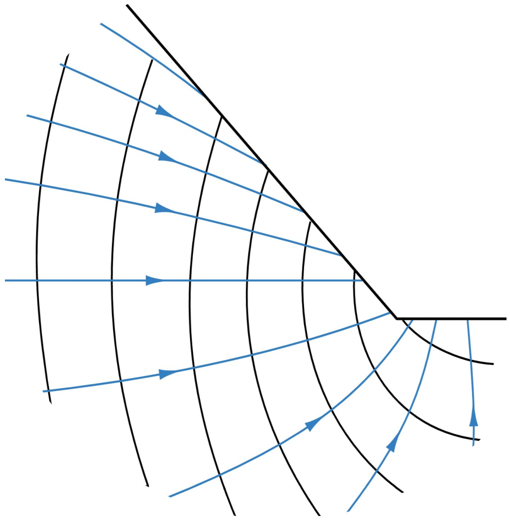
Figure 110.— Root growth has resulted in partially plugging the pipe.

e. *Conclusions.*—Clay tile pipe was frequently used in the construction of early toe drain systems. The common practice of laying clay tile pipe with open joints has allowed sediments, gravels, and rocks to enter toe drain systems. Entry of these materials has resulted in obstructions for inspection and plugging mechanisms. Clay tile pipes are also prone to joint offsets and separations either from improper installation during construction, backfill loadings, or foundation conditions.

HDPE pipe has been used in many toe drain systems constructed or modified after about 1980. HDPE pipe, while lightweight and easily handled and installed, has experienced a significant number of shape deformation and failure instances. Some of the HDPE pipe failures may be related to stress cracking. Stress cracking is a failure mechanism that develops over time at stresses less than the yield strength. In the past, HDPE pipe resins have differed in the amount of stress crack resistance (SCR). Proper installation of HDPE pipe requires good compaction and quality control of the backfill to ensure good support under the haunches. If the pipe is not well supported by the backfill, the pipe will deflect excessively and stresses will be concentrated at the crown, invert, or springline. These stress concentrations can lead to premature failure, especially if the pipe does not have sufficient SCR. Other failures could be the result of isolated point loads from construction loading, such as equipment crossings. When using HDPE for toe drain applications, a preliminary CCTV inspection should be performed when 3 to 5 feet of backfill has been placed over the pipe. The purpose for this inspection would be to identify and repair any abnormalities, cracks, bulges, etc. early before construction is completed. Another CCTV inspection should be performed when the final backfill loading over the pipe is completed. CCTV inspection should be performed prior to the contractor pulling the torpedo-shaped plug or pig through the pipe and prior to any cleaning. The purpose for this inspection would be to identify any abnormalities, cracks, bulges, etc. that may have developed since the preliminary inspection. CCTV inspection could replace the need for pulling the plug or pig through the pipe.

Most CMP pipes have experienced deterioration ranging from minor to extensive. The rate of deterioration varies, depending on chemical and physical properties of the soils and water, and exposure to the environment. Where corrosion has occurred, it is a continuous and irreversible process. Interior surface coatings have been somewhat effective in extending the service life of CMP. However, most CMP pipe with surface coating has experienced some loss of coating from delamination.

Biofouling and mineral encrustation are frequent plugging mechanisms that can affect the long-term performance of pipe perforations and slots. In a few cases, cleaning using high pressure jet washing has been performed after identification during the initial CCTV inspection. Follow-up CCTV inspection has shown that the biofouling and mineral incrustation was generally removed from the interior surface. Some improvement of discharge from the toe drain pipe is typically observed. However, no determination could be made as to the extent of the plugging mechanism remaining in the backfill materials surrounding the pipe.



Chapter 5

MONITORING AND MAINTAINING DRAINS

Most drain systems will require maintenance over time. The biggest cause for maintenance is plugging mechanisms, which reduce the effectiveness of drains by making it more difficult for water to enter or exit. Drains with reduced effectiveness can allow pressures to increase in a structure or foundation, to the point where the stability of a structure is reduced, or cause seepage to move to unprotected areas, where piping would start to develop undetected. Even if the reduced stability has no immediate effect on a structure, the reduced stability could become critical during an extreme loading condition, such as a large earthquake or flood.

There are a number of ways to evaluate if drains are becoming plugged. Often, visual evidence will exist, such as a calcium carbonate plug at the outlet end of a drilled foundation drain for a concrete dam, or a drain that is dry but had previously been flowing under the same reservoir elevations. Drains can also be probed to determine if the full length or depth of the hole is open and unobstructed. If the full depth of the drain cannot be probed, plugging is likely. Remote video cameras can also be used to determine if drains are plugged, and to determine the location and extent of plugging. Finally, instrumentation may provide indirect evidence that drains are becoming plugged. Reduced drain flow measurements (under comparable reservoir water surface elevations) or increased uplift pressure or piezometric readings (under comparable reservoir water surface elevations) are indicators of potentially plugged drains.

If it is determined that drains are plugged or they are becoming plugged, a drain cleaning program should be initiated. Regular monitoring of drains is a key component of a drain maintenance program. Deferred maintenance of drains will result in continued degradation

Figure 111.—Calcium carbonate deposit.

of the stability of the structure or foundation and will make any future cleaning of the drains more difficult and expensive. In some cases, drains cannot be restored once they have degraded to a certain point. A number of different cleaning methods can be used, depending on the type of drain, the nature of the plugging mechanism, and the extent of the plugging.

This chapter on maintaining drains addresses plugging mechanisms, site-specific considerations, methods for evaluating drain effectiveness, guidelines for drain maintenance, information on establishing a drain cleaning program, and drain cleaning methods.

5-1. Drain-Plugging Mechanisms.—A variety of mechanisms can plug drains, including calcium carbonate deposits, bacterial deposits, collapse of the drains, and the deposition of fines, sand, or gravel particles in drains. The following is a summary of the different plugging mechanisms:

a. *Calcium Carbonate Precipitation*.—Solid deposits are often found at the seepage drain emergence points and at other locations where drainage water evaporates. These deposits often contain calcium carbonate ($CaCO_3$), which commonly precipitates out of solution as the mineral calcite. Figure 111 shows an example of a calcium carbonate deposit at the outlet end of a formed drain in a concrete dam. Calcite will precipitate out of solution and form deposits when the calcium ion and bicarbonate and carbonate ion concentrations in the water increase to the point where they exceed the solutioning capacity of the water. This situation may occur when drainage water evaporates at outfalls (thereby increasing the concentrations of calcium and hydrogen carbonate [Ca and HCO_3]), or when lower concentration waters have an alkaline pH above 8.3.

Calcium in seepage water can come from a variety of sources. Concrete structures and grout curtains used to control seepage underneath dams and some foundation rocks or soils (containing limestones) provide a steady supply of calcium. Factors that affect calcium carbonate precipitation are a steady supply of calcium, pH (an alkaline environment, created at grout curtains and concrete structures, promotes calcite precipitation), and evaporation (deposits occur at locations where seepage water evaporates). The primary source of Ca in seepage is water dissolution of limestones present in rock formations and in soil. Concrete will contribute some Ca, but the larger effect on calcite precipitation is from the high pH (> 9.5) usually caused by dissolution of sodium oxides/hydroxides associated with the cement. While calcite is most frequently observed at locations where seepage water evaporates (at the outfall of foundation drains, at formed drains within a concrete dam, or along the sidewalls of a toe drain, subjected to intermittent flows), calcium carbonate can precipitate underwater if the pH is high and a steady supply of calcium is available.

Precipitated calcium carbonate usually forms a white solid deposit, which may mineralize and harden with time. Calcium carbonate deposits may be colored by small amounts of manganese, iron, or other impurities. Evaporative deposits will contain other salts, depending on the proportions of dissolved ions in solution. These deposits will be complex mixtures of several hydrated minerals, including: the chlorides, sodium chloride, potassium chloride, and manganese chloride ($NaCl$, KCl, and $MgCl_2$, respectively); gypsum/anhydrite, $CaSO_4$; along with the carbonate salts, calcite, dolomite, and magnesite. Calcite is the least soluble of these minerals and will precipitate before sulfates and well before chlorides drop out of solution. If a deposit is suspected to be calcium carbonate but cannot be confirmed in the field, samples should be submitted to a qualified laboratory for analysis. Section C-5 in appendix C provides information on sampling and testing for calcium carbonate.

b. *Biological Plugging*.—Biological plugging results from life process activities of certain bacteria, which obtain energy for their existence from the conversion of sulfates to sulfides, iron to ferric oxides, and manganese to manganese oxides. Bacterial deposits are common and can develop under a variety of conditions. Bacterial growth can occur anaerobically (without oxygen) or aerobically (with oxygen). Energy sources can be organic materials or other carbon-containing substances. Bacteria require a steady supply of dissolved iron, manganese, or sulfate, depending on the type of bacteria. Most of the time, bacterial deposits are soft and easily removed, but some can become hard and mineralized.

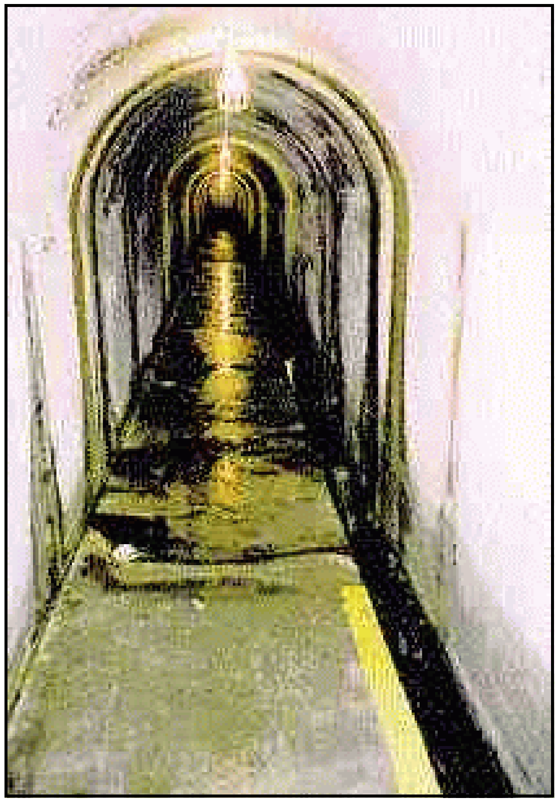


Figure 112.—Iron sulphide deposits in foundation gallery.

Aerobic bacterial growth can also create a hazardous condition by depleting the oxygen in the air of a confined space.

Iron bacteria deposits are the most common type of bacterial deposits found in drain systems. Iron bacteria form rusty slime deposits as ferrous iron is converted to ferric oxide or what is also known as iron oxide. Sulphur bacteria and moderate levels of iron may result in the formation of iron sulphide. Iron sulphide deposits in drainage systems are usually observed as black tarlike sticky substances. Section C-5 in appendix C provides information on testing for bacterial deposits.

Figure 112 shows iron sulphide deposits from drain flows in a concrete dam foundation gallery. Shock chlorination is an effective method for preventing iron bacteria growth. The Grand Coulee Dam case history (in ch. 4) provides an example of the use of this technique. Using hot water or steam to kill the bacteria has also shown promise as a preventive measure.

Figure 113.—Plug of organic material in toe drain (also shown is camera used to inspect drain).

c. *Deposition of Fines, Sands and/or Gravels/Cobbles* .—The deposition of fines, sand and/or gravel/cobble particles in drains can also reduce the effectiveness of drains. The source of these particles can be foundation materials, backfill, or embankment materials, and their deposition is generally the result of no filter, or a poorly designed filter surrounding the drain. They can be blown in by the wind or transported by drainage flows.

d. *Other*.—Other mechanisms for plugging drains include roots from trees or bushes that penetrate the drains and cause obstructions; the accumulation of inorganic fibers (fig. 113); collapse of drain hole in either rock or soil; plugging from animals or humans (vandalism); debris plugs (from back flooding); and deterioration of the drain pipe material (i.e., corrosion of metal pipes) which may lead to a collapse of the pipe. In some installations, drain pipes are damaged during installation, and the damage is not detected, because a video inspection of the pipe is not performed immediately after backfilling and installation of the drain is completed. On Brantley Dam, for instance, a video inspection of a drain in 1991 showed that a lath had been driven through the plastic pipe to hold it in place and never removed. Recent video inspections of the HDPE toe drains installed at San Justo and Davis Creek Dams have revealed collapsed portions of the drains. The damage likely occurred during the installation of these drains.

5-2. Site-Specific Considerations.—Geology and specific drain installation details (steel pipe, PVC pipe, size of screen openings, diameter of perforations, etc.) are major factors which influence drain cleaning effectiveness. Drain flows may decrease, or cease entirely, because of heavy deposits of rust, iron-reducing bacteria, mineralization, sedimentation, or root growth. Alternating layers of hard and soft material (sandstone and shale) can also render drain cleaning difficult, with the soft layers providing material for sedimentation in the drain hole, while the hard layers may develop hard mineralization encrustation. Cleaning of drains in this type of material will also be difficult, especially if high pressure water jetting is used. Pressures required to clean the sandstones will likely erode the shales. Drain cleaning, or the degree of rehabilitation that is possible, depends on these and other conditions. For example, several-decades-old steel pipe in horizontal drains and metal collector systems may be severely rusted and would probably be destroyed if disturbed by a cleaning operation, while the newer PVC pipe installations may remain in excellent condition and should perform well for decades.

While individual drain systems tend to lose effectiveness at varying rates, most drains tend to decrease in effectiveness with increasing age. Steel pipe can rust, steel and PVC pipe can fail with new or renewed slope movement, and iron-reducing bacteria, mineralization, sediment, or root penetration can obstruct drain pipe or collector systems. However, if a proper cleaning and maintenance program is performed on a regular basis, a drain installation is likely to function effectively for at least as long the expected life of the structure that it protects.

a. *Importance of Drains to Dam/Structure Stability.*—Maintaining an effective drainage system (whether it is toe drains or relief wells for an embankment dam, foundation and formed drains for a concrete dam, or drains for a tunnel outlet works) is very important to ensure that the structure behaves as designed and originally constructed (i.e., drainage systems are a primary "line-of-defense" in maintaining structural stability/integrity of many features). Good drainage may be particularly important under unusual loadings, such as floods or earthquakes. Given this, it is imperative that drainage features for all dams and appurtenant structures be adequately designed, constructed, monitored, and maintained. When choosing a drain cleaning method, consideration must be given to the type of drain, drain materials and foundation materials and conditions.

Reclamation has constructed and/or operates about 60 concrete dams and over 210 embankment dams with a structural height of 50 feet or taller. Most of these dams have some form of drainage system, and most dams have appurtenant structures that also rely on drainage systems to help maintain intended performance. Reclamation's structures are aging; therefore, monitoring, analyzing, and maintaining existing dam drainage systems is as critical as ever.

Reclamation has implemented numerous programs and activities that facilitate monitoring and maintaining drainage systems. As part of the safety of dams program, Reclamation performs periodic reviews/examinations of its high- and significant-hazard structures, including the annual examination, the periodic facility review and the comprehensive facility review (CFR). Additionally, visual and instrumentation performance parameters have been developed for each high- or significant-hazard dam. What is noteworthy about these activities includes:

- Many potential failure modes typically include a foundation-related failure, which has some element of uplift and/or seepage associated with it.

- Failure of a concrete dam and/or foundation tends to be very rapid, associated with a large peak breach outflow (i.e., potential of limited warning time, significant property damage, and loss of life).

- For some Reclamation concrete dams, a review and subsequent analysis of uplift pressures indicated an apparent loss of drain efficiency (higher uplift pressures) from the efficiency present after initial reservoir filling. It should be noted that limited uplift measurements are available, and even if higher pressures are not measured where instruments are located, they could be increasing elsewhere.

- During several CFR examinations, blockage of foundation and/or abutment drains was observed at dams without a program of periodic probing/inspection and/or cleaning of drains. These situations have resulted in a CFR recommendation to establish a drain monitoring/cleaning program.

- Designers' Operating Criteria for many dams recommend periodic drain cleaning (every 5 to 10 years). However, these recommendations have been rountinely overlooked.

Perhaps the most important factor in long-term performance of drains is a well developed and executed program of inspection, repair, and cleaning. A drain maintenance program should be established with provisions for annual inspections for cleaning and repair of damaged outlets or collector systems. Most drains need to be cleaned only every 5 to 8 years. Rapid accumulation of mineral or bacterial deposits, heavy root growth, or fine-grained sediments may reduce this period to every year or every other year.

An effective method of maintaining continuity in a drain program would be to permanently assign qualified personnel the full-time task of inspecting, cleaning, replacing, or repairing drain installations on a regional or area office basis.

Drain cleaning and maintenance records should be kept for each installation. These records should indicate dates of cleaning and repairs, flows, and pressures (if available), recorded for each drain prior to and after each cleaning, and any damage that has occurred to the drains.

b. *Access to Drains.*—To inspect and maintain drain systems, good records are required, including plans and cross sections showing the locations of drains in relationship to survey monuments or permanent landmarks. A permanent record of specifications drawings or as-built drawings showing the locations of drains is also very important. Otherwise, drain locations are lost because of transfers, retirements, or promotions of personnel who are only aware of the drain locations because they were present during the actual installation. In some cases, the original design and layout of the drain system precludes access to much of the system, but any information that will allow access even to portions of the system is valuable.

Drains located on slopes or near the toes of embankments are sometimes lost through the practice of end-dumping waste material over the side of the fill or road grading, covering the outlets. Sloughing of weathered slopes has buried many drain outlets both on benches and at grade at several sites. Horizontal drains are lost or damaged because they are not protected against rock fall, particularly in the case of exposed PVC pipe, or because they are vulnerable to snow plows, rock plows, or straying vehicles near the edge of the slope. A good practice is to protect the PVC pipe at the outlet with a galvanized pipe that slips over the PVC pipe approximately 20 feet. The establishment and maintenance of a central file on all drain installations as well as placement of markers such as steel posts or signs, is a good practice.

Drains located in difficult access areas are challenging to maintain or rehabilitate. Examples of difficult access areas include drilled drains or formed drains in drainage galleries within concrete dams, spillways, and powerplants. The drainage galleries usually are small (5 by 8 feet), providing limited work area for equipment and crews. Access into these areas requires mobilization of equipment up and down stairs and/or spiral staircases, and possibly through maintenance facilities, power generation facilities, etc. If rehabilitation of drains involves using conventional drilling methods, mobilization of equipment and safety considerations such as clean air supply, noise, and oxygen levels can be significant. A source of clean water is typically required for drain cleaning operations. Depending on the closeness of the drains to the reservoir at Reclamation dams or to another water source, a lengthy delivery system may be required.

5-3. Evaluating Drain Effectiveness with Instrumentation/Visual Monitoring.— Along with the importance of maintaining drains at Reclamation structures comes the importance of evaluating the effectiveness of these drains with the passing of time. Numerous methods can be used to evaluate drainage systems. The most common methods to evaluate the effectiveness of drain systems at dams are monitoring changes in water pressures or water levels, drainage or seepage flows, depth of drain holes, and physical and visual inspection of drainage systems themselves.

a. *Site Review*.—Prior to evaluating the performance of a drainage system, the site geology should be reviewed. This review should include the material type of the foundation, and orientation and spacing of discontinuities. Likewise, foundation material type can also indicate where drain plugging might be more extensive. For example, foundations containing limestone or carbonaceous materials are susceptible to calcium carbonate type of plugging. Joint infillings, weak shale layers, and other soft materials can also erode and plug drains.

In addition to the type of foundation material, records such as those for construction grouting should also be reviewed. Special attention should be given to areas along the foundation where grout takes were greater than the average. Areas of large grout takes would be areas where the foundation drains might be more prone to calcium carbonate plugging because of leaching of carbonaceous materials or prone to plugging due to loose foundation conditions, where drain holes might collapse.

b. *Concrete Dams/Foundations*.—

1. *Water Pressures*.—Increased water pressures (that do not reflect reservoir fluctuations) are an indirect indication that the drainage system is becoming plugged. Regular review of the uplift pressure readings would alert one to this condition. The most obvious indications that the drainage system has become less efficient due to plugging are when uplift pressures exceed historical performance under the same reservoir head. Also, gradual increases in uplift pressures over several years at similar reservoir and tailwater elevations are another indication that the drainage system is becoming plugged. This type of trend is difficult to identify until there is a notable increase in expected values. Uplift pressures that are greater than those assumed in the original design of the concrete dam are a concern, since they will likely reduce safety factors assumed in the design.

Performance parameters have been prepared for all of Reclamation's major dams. These documents identify the most likely failure modes for each dam and identify the instrumentation used to monitor for these failure modes. Acceptable limits for each instrument or set of instruments are provided, and this information should be used when evaluating uplift pressure readings and other instrumentation data.

2. *Drainage Flows*.—Decreased drainage flows under constant conditions are usually a direct indication that the drainage system is becoming plugged although reservoir siltation may be another explanation. Constant total drainage flows for a dam may not be a

clear indication that the drainage system is fully functional. As drain holes lose their effectiveness and become plugged, the water may migrate toward surrounding holes with minor or no change in total flows. Therefore, a change in total flows may not materialize until large portions of the drain system have become nonfunctional. Monitoring multiple points along a drainage collection system will assist with identifying changing flows from one point source to another point within the system. This should include an accounting of which drain holes are flowing. The performance parameter document for a given dam should identify the expected behavior and the conditions for performing a closer evaluation of the drainage system.

3. *Drain Hole Depths*.—Decreasing diameter or depth of drain holes (due to deposition of materials within drains) is a direct indication that the drainage system is becoming clogged. Probing of the drain holes on a regular basis (at least every 6 years and more frequently if potential drain plugging is indicated by visual evidence or instrumentation data) is a proactive maintenance program to effectively monitor drain hole plugging. For dams with a large number of deep foundation drains, probing the drains may be time-consuming and expensive. Probing of a representative sample of drain holes and/or video inspections may be worth considering under these circumstances. The performance parameter document for a given dam should identify the expected behavior and the conditions for performing a closer evaluation of the drainage system.

4. *Visual Observations*.—Visual inspection is another useful tool for evaluating drain holes. Visual inspections should consist of three parts. The first part is visual inspection of drain hole exit conditions, looking specifically at the exit of the drain hole for buildup of deposits (most commonly calcium carbonate or iron bacteria deposits) that are restricting flows. The second part is a general visual inspection of the exposed portions of the entire drainage system, looking for changing seepage flow points; transfer of seepage flows from one drain hole to another or new seepage points appearing; seepage occurring at joints; etc. These observations indicate that some of the drain holes might be plugged and that seepage paths could be changing or that new preferential seepage paths have developed. The third part of visual inspection is using a camera to inspect the interior of the drain hole. If visual inspections indicate plugging is likely occurring, then the affected drain holes should be cleaned.

The performance parameter visual inspection checklist should include items relative to a visual inspection of the drains (parts one and two). The recommended interval or conditions for performing a remote video inspection of the drains should also be included in the performance parameter document.

c. *Embankment Dams/Foundations/Horizontal Drains*.—

1. *Seepage Flows*.—Decreasing seepage flows at similar reservoir and tailwater elevations are a direct indication that the drainage system (typically toe drains) may be becoming clogged. Increasing flow, particularly if sediment is detected as well, may indicate that the drainage system has a failure at some location (loss of filter, collapse of drain pipe, pipe joint separation, etc.).

Performance parameters have been prepared for all of Reclamation's major dams. These documents identify the most likely failure modes for each dam and identify the instrumentation used to monitor for these failure modes. Acceptable limits for each instrument or set of instruments are provided, and this information should be used when evaluating uplift pressure readings and other instrumentation data.

2. *Water Levels*.—Changing water levels monitored by observation wells and piezometers are indirect indicators that the drainage system (typically toe drains or relief wells) may be clogging. Therefore, water level monitoring data can become important indicators that the drains may be plugged. The performance parameter document for a given dam should identify the expected behavior and the conditions for performing a closer evaluation of the drainage system.

3. *Visual Observations*.—Visual inspection of drains indicates drain effectiveness. The appearance of sediment, organic material, and other deposits at the outlet indicates a drainage system problem. The most practical method of performing a visual inspection of a toe drain is by probing with a rod or stick and using a mobile or remote camera to inspect the length of the drain. If a toe drain or horizontal drain has not been inspected since its original installation, a video inspection of the drain should be considered. Recent inspections of toe drains at Reclamation dams revealed collapsed sections of the HDPE pipe (see case history in ch. 4) forming the toe drain. However, the appearance of new seepage; wet areas; the appearance of material transport in seepage and leakage; and settlements, depressions, or collapse or formation of sinkholes in areas close to toe drains would be other visual signs that drains may not be functioning as designed and may be a more serious indicator that an internal failure erosion mode is in progress. If anything indicates that a toe drain may have become plugged or has potentially experienced a failure, a video inspection of the toe drain should be performed. Appendix C provides information on remotely controlled video inspections. Reclamation has equipment which may be used for these types of inspections.

The performance parameter visual inspection checklist should include items relative to a visual inspection of the drains. The recommended interval or conditions for performing a remote video inspection of the drains should also be included in the performance parameter document.

d. *Relief Wells*.—

1. *Flow Rates*.—Reduced flow rates are a direct indication that the drainage system is becoming plugged. Decreasing flow amounts can signal that the relief wells are becoming plugged.

2. *Water Levels*.—Increasing water levels monitored by observation wells and piezometers indicate indirectly that the relief well system is becoming plugged. Therefore, water level monitoring points close to the wells can become important indicators that the relief wells are becoming plugged.

3. *Visual Observations*.—Visual inspection of relief wells directly indicates drain effectiveness. A method of performing a visual inspection of a relief well is by remote video borehole camera. This type of equipment is inserted and lowered into the well. Appendix C contains additional information on remotely controlled video inspections. If areas of plugging are found, the well screen should be cleaned. However, the appearance of new seepage; changes of existing seepage; the appearance of material transported by seepage and leakage; and the collapse or formation of sinkholes in areas close to the relief wells would be visual signs that they may not be functioning as designed, and more seriously, that an internal erosion failure mode is in progress.

4. *Pump Tests*.—Pump tests should be performed on relief wells that flow infrequently. The tests should be performed to verify that screens are not plugged and the design drawdown can still be achieved.

e. *Conclusions*.—Many methods can be used to evaluate drainage systems. The most common methods to evaluate the effectiveness of a drainage system are monitoring changes in uplift pressures, water levels, drainage flows, seepage flows, or depth of drain holes; and visually inspecting the condition of the drainage system.

Certain types of drainage systems, such as foundation underdrain systems for spillway crest structures and chutes, have traditionally not received much attention in the form of inspection and monitoring. With the improvements in remote video inspection capability (see app. C), consideration should be given to performing video inspections on the systems. Drainage flows should also be visually monitored wherever possible.

A drainage system is an important part of the monitoring system at a site, since it provides information about the health of the structure. Therefore, maintaining such a system warrants as much attention as the monitoring instruments at the site. In addition, a properly maintained monitoring system (including instrumentation data as well as data from the drainage system) is useful in conjunction with other O&M activities to support the decisions to schedule drainage cleaning activities at a specific site.

5-4. Drain Cleaning Methods.—Depending on accessibility to drains, the type of plugging mechanism, and the application of a given method, success in reestablishing drain flows may vary from site to site. Several methods have been successfully used by Reclamation to clean plugged drains and restore their efficiency (see ch. 4 for case histories). Pressures and forces proposed for mechanical cleaning methods and high pressure and ultrahigh pressure water jetting should be determined before their use to avoid damage to drains and the surrounding materials. Cleaning methods include:

a. *Rodding.*—A steel rod or similar device is used to break through the plugged encrustation deposit. In some cases, a metal object such as a star drill has been attached to a line and dropped down the foundation drain to break through the blockage. Flushing of the hole should be performed after rodding. Rodding does not completely clean the drain hole walls. It is most effective where plugs high in the drains must be removed but where lower areas of the drain still allow good flow through the drain hole walls. It would not be a good method where extensive drain plugs exist. This is an economical method that uses simple equipment.

b. *Flushing and Air Lifting.*—Soft and loose deposits (iron bacteria, organics, sediments, etc.) can be flushed out of drain holes (typically vertical) by placing the end of a water line at the bottom of a drain and using water pressures of up to 250 lb/in^2 and flows up to 60 gal/min to loosen the deposits and flush them out of the hole. Air lifting is done in a similar manner, but uses compressed air to force debris out of the drain. This could be an effective method for bacterial deposits and other loose deposits that have not hardened but would not be effective in removing hard deposits such as calcium carbonate.

c. *Reaming, Drain Enlargement or Drilling of New Holes.*—For foundation drains in rock, the existing drain holes can be reamed up to the original diameter using a drill to remove obstructions and coatings on the borehole walls. The effectiveness of this method will be reduced if the fractures and joints around the perimeter of the drain are plugged. Correct alignment of the drill and drill bit is critical to successfully using this method.

Drain enlargement is another method that utilizes drill equipment to restore the efficiency of drains. For foundation drains in rock, the existing drain holes are redrilled to enlarge the original diameter of the hole by ¼ to 1 inch. The first cleaning using drain enlargement appears to achieve the most improvement in drain efficiency. Subsequent redrilling of the same hole often does not result in the same improvement. This can occur if joints and fractures become plugged in the areas immediately surrounding the perimeter of the hole, blocking off pathways for water to enter the drains.

As an alternative, new drain holes are sometimes drilled to replace the old ones, if the desired efficiency cannot be economically achieved with drain enlargement or reaming. All of the drilling methods are usually very effective in improving drain efficiency, but these methods are some of the more costly methods of drain rehabilitation. Any of the drilling methods should only be used after less expensive alternatives have been ruled out.

d. *Rotary Tube Cleaners or Mechanical Abraders.*—This method cleans the deposits from the inside surfaces of the foundation drain, restoring the original diameter of the hole. These devices typically have a rotating cutting head on the end of a flexible rod or hose. The effectiveness of this method will be reduced if the fractures and joints around the perimeter of the drain are plugged. A Roto-Rooter device, a common commercial method for cleaning sewer drains, is a device in this category that has been successfully used in drains at Reclamation facilities.

e. *Ultrahigh Pressure Water Jet System.*—A typical ultrahigh pressure water cleaning system delivers a flow of 3 to 10 gal/min at pressures between 20,000 and 50,000 lb/in^2. A high pressure pump is connected to a filtered water supply. Hoses are provided from the pipe to the hole being cleaned, and a tripod is used to lower the equipment in and out of the hole. A jetted nozzle attached to a flexible lance is lowered into the hole and removed slowly during the cleaning operation. Typically, a number of different heads with different nozzles can be used. This method should be used with extreme care for drains in embankment dams and in areas of soft foundation rocks or in other areas where surrounding materials could be fractured or washed away.

f. *High Pressure Water Jet System.*—This method delivers a flow of 10 to 20 gal/min at a pressure typically between 6,000 and 10,000 lb/in^2. Other than the pressure and flow rates, the equipment and methods for these systems are similar to the ultrahigh pressure water jet systems. Figures 114 and 115 show equipment used for high pressure water jet cleaning. Extreme care should be exercised when using this method for drains in embankment dams and in areas of soft foundation rocks or in other areas where surrounding materials could be fractured or washed away. Under these conditions, a good knowledge of the foundation/drain conditions is required, along with the experience and expertise to adjust the equipment for the site conditions. Both the ultrahigh pressure water jet system and the high pressure water jet system are very effective, cost-efficient methods when used by trained personnel. These methods are very effective for drains installed in concrete or rock.

Figure 114.—High pressure water jet nozzle and hose being lowered into foundation drain.

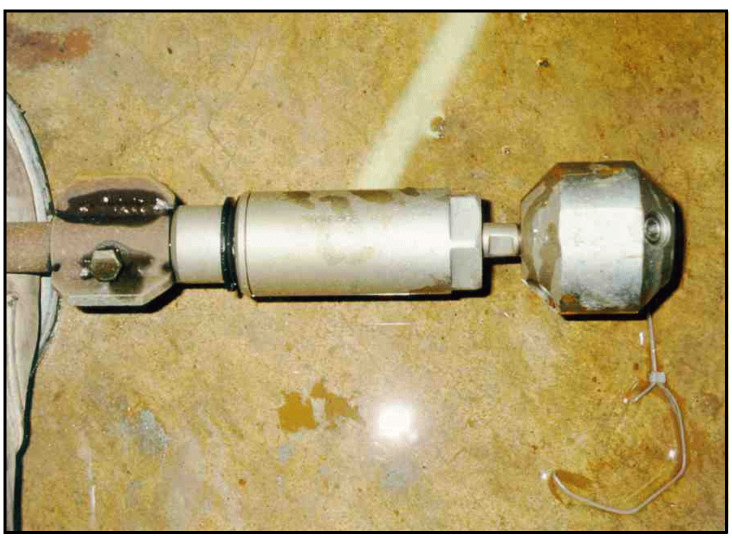


Figure 115.—High pressure water jet nozzle.

g. *Chemical Treatments.*—Sulfamic, sulfuric, and hydrochloric acids have been used to dissolve deposits in drains. Sulfamic acid has been field tested at Folsom Dam to chemically dissolve calcium carbonate in clogged foundation drains. Granular and pelletized forms of the acid are applied to drains in quantities equivalent to 2 to 8 percent of the unobstructed volume. Other acids have been used in liquid form with limited success due to dilution and health problems related to their use. Acids have not been effective in clearing fully plugged drains. Acids seem most effective when used as a preventive maintenance procedure for controlling the buildup of mineral deposits in drains.

Relief wells downstream of Grand Coulee Dam have been successfully maintained and major plugging of the wells avoided through the use of both bleach and sulfamic acid. The solutions are alternated in the wells and the rapid change in pH has been effective in controlling bacterial deposits.

Another chemical method that has been used is adding carbon dioxide under pressure to drains plugged with calcium carbonate. Typically the zone being treated is isolated with packers. This process has the potential for dissolving calcium carbonate, since the carbon dioxide can acidify water in the drain, however, an attempt to use this method at Folsom Dam was unsuccessful because joints in the foundation rock made it impossible to pressurize the holes.

h. *Overpumping.*—This method consists of pumping a relief well at a discharge rate considerably higher than the design capacity of the well. This method is only recommended for thin, relatively uniform grained, permeable aquifers. The pump is normally set above the top of the screen, and development is primarily concentrated in the upper one fourth to one half of the screen length. As long as pumping continues and water is moving from the outside to the inside of the well screen, stable bridging of the sand grains occurs. When pumping is stopped, the water in the column pipe drops back into the well, causing a reverse flow which destroys the bridging. When the well is again pumped, sand will enter the well until bridging is reestablished. For a thorough discussion of overpumping, see the *Ground Water Manual* [45].

i. *Surge Block Technique.*—This technique is used to maintain the efficiency of relief wells. Surge blocks are usually solid or vented and consist of a body block 1 to 2 inches smaller in diameter than the well screen and fitted with as many as four ¼- to ½-inch disks of belting, rubber or other tough material having the same diameter as the inside diameter of the well screen. Solid surge blocks have a solid body, while vented surge blocks have a number of holes drilled through the body, parallel to the block axis. The top of the body is fitted with a rubber flap valve, that seals the holes on the upward stroke and allows water to flow through the holes on the downstroke.

As a solid surge block is moved up and down in the well screen, a surging action is imparted to the water, which is about equal in both directions. The gentler downstroke of a vented surge block causes only sufficient backwash to break up any bridging that may occur, and the stronger upstroke pulls in the sand grains freed by the destruction of the bridging. The solid surge block is usually most effective in dirty sands containing large percentages of clay, silt and organic matter. A vented surge block is more effective in cleaner sands.

A figure showing a vented surge block, and more details on this technique can be found in the case history for Senator Wash Dam (in ch. 5). For a thorough discussion of the surge block technique, see the *Ground Water Manual* [45].

j. *Excavation and Replacement*.—Excavation and replacement is a method of removing a damaged or plugged section of a drain when other techniques are not possible or are unsuccessful. This is typically used on embankment dam toe drains when the exit of the drain is damaged or where collapse of the toe drain or infilling through openings at joints along the pipe has caused plugging or a restriction in the drain. Prior to excavating any drain, technical specialists should be consulted to ensure that the safety of the structure is not compromised.

k. *Slip Lining*.—Slip lining can be used at damaged or unfiltered toe drains or at other unfiltered drains to prevent piping or migration of foundation or embankment materials. Slip lining may also be useful to prevent collapse of drains in soft or unstable materials. Slip lining consists of installing a smaller diameter perforated drain pipe into the original drain.

Table 9 provides a summary of the drain methods described above, and information that can be used when selecting a drain cleaning method.

5-5. Reclamation Drain Cleaning Program.—

a. *Reclamation Guidelines for Maintaining Drains*.—Drains are an important feature at dams and their associated structures. They reduce the water pressures within structures, within structure foundations, and at structure/foundation contacts. Elevated water pressures reduce the stability of structures by reducing the sliding and overturning resistance of the structure. Drains also control seepage to prevent piping. While a structure may perform adequately under static loading conditions, the lack of drains in a structure or plugged drains can reduce the factors of safety for the overall stability of the structure for operational, flood, and earthquake loading conditions. This could become especially critical if unforeseen loading conditions or higher loads under seismic or hydrologic conditions occur beyond what was originally expected. Because drains increase the stability of structures and foundations and because they are a significant investment which would be

Table 9.—Drain-cleaning methods

Method	Most Effective Conditions	Ineffective Conditions	Cost	Access
Rodding	Plugs are fairly soft and occur near drain outlet, good flow in remainder of drain. Typically used for concrete dam foundation drains	Holes where drain hole walls are plugged over significant lengths or plugs are fairly hard	Low	Simple, easily transported equipment, holes need to be flushed
Flushing/Air Lifting	Removing soft and loose deposits	Removing mineralized or hardened deposits	Low	Air or water compressor can be located remotely from drain holes, with hoses run to hole
Reaming/Drain Enlargement	Drains installed in rock or concrete (typically concrete dam drains); Removing mineralized or hardened deposits and/or plugs; Other cleaning methods have been ineffective; Plugging mechanism influence extends into fractures and joints outward from hole perimeter (drain enlargement)	Removing soft and loose deposits; Successive attempts using this method may provide reduced effectiveness	High	Drilling equipment will have to be transported to each drain location; access difficulties associated with size of drilling equipment
Drilling New Holes	Drains installed in rock or concrete (typically concrete dam foundation drains); Zone around drain hole has fractures and joints plugged so water can't enter drains; Last resort method—other methods tried and failed	Limited access	High	Drilling equipment will have to be transported to each drain location; access difficulties associated with size of drilling equipment
Rotary Tube Cleaners or Mechanical Abraders	Smooth walled drain holes; removing mineralized or hardened deposits; removing tree or plant roots that have infiltrated toe drains or horizontal drains	Not effective in removing deposits within joints and fractures around perimeter of holes	Low	Equipment is generally easy to transport.

Table 9.—Drain-cleaning methods (cont'd)

Method	Most Effective Conditions	Ineffective Conditions	Cost	Access
Ultrahigh Pressure Water Jet System	Good general method; Removes soft deposits as well as mineralized and hardened deposits. Effective in removing deposits in joints and fractures to limited depths (more so than high pressure water jets). Has been effective in cleaning well screens	Removing fines or sands that have settled into a drain; Foundation drains in rock where foundation is soft and erodible; pressure losses can be high in long hoses, limiting the accessibility to some drains; rotation of the nozzle is required to cover the entire circumference of a drain hole.	Mod-erate	Water compressor can be located remotely from drain holes, with hoses run to hole
High Pressure Water Jet System	Good general method; removes soft deposits as well as mineralized and hardened deposits; Somewhat effective in removing deposits in joints and fractures (to limited depths); relatively low pressure losses allow the use of long hoses to extend from the pump to the drain being cleaned. Has been effective in cleaning well screens	Removing fines or sands that have settled into a drain; Foundation drains in rock where foundation is soft and erodible	Mod-erate	Water compressor can be located remotely from drain holes, with hoses run to hole
Chemical Treatments	Effective as a preventative measure and for controlling the buildup of deposits (calcium carbonate and iron bacteria); often used for relief wells; has been used successfully for horizontal agricultural drains with significant calcite deposits	Not effective in fully plugged drains; More complicated method—requires trained personnel to be effective and to avoid health problems associated with some materials; environmental considerations may limit use.	Low	Equipment is limited and access should not be an issue; Ventilation may be required depending on chemicals used

Table 9.—Drain-cleaning methods (cont'd)

Method	Most Effective Conditions	Ineffective Conditions	Cost	Access
Temperature Treatments	Effective as a preventative measure and for controlling the buildup of deposits (iron bacteria?); often used for relief wells	Not effective for plugging mechanisms other than iron bacteria	Low	Equipment is small and access should not be an issue
Overpumping	Used for relief wells in the initial development of the well and for restoring the efficiency of the well; only recommended for thin, relatively uniform grained, permeable aquifers and where soil particles are plugging the well screen.	Not applicable outside of relief wells; not recommended for cases where mineral encrustation or bacterial plugging.	Low	Utilizes existing pump; no additional equipment needed
Surge Block Technique	Used for relief wells to redevelop the well; only recommended where soil particles are plugging the well screen.	Not applicable outside of relief wells; not recommended for cases where mineral encrustation or bacterial plugging.	Low	Surge block is a very simple device that is easily transported and installed in relief well.
Excavation and Replacement	Primarily used for toe drains and drains that are easily accessible and installed in a soil trench.	Not practical for foundation underdrains under a concrete slab.	Moderate to High*	Excavation required.
Slip Lining	Useful for damaged or unfiltered toe drains or for other unfiltered drains where liner can provide some filtering capacity. May also be used in drains susceptible to collapse.	No real benefits in drains where additional structural capacity or filtering capacity is not needed.	Low to Moderate	Limited access may require frequent couplers to maintain liner continuity.

* Depends on extent of drain that is replaced.

expensive to replace, drains should be maintained. If drains are not maintained, the stability of the structure could be jeopardized under critical loadings of the structure. Also if drains are not maintained for a period of time and then an attempt is made to rehabilitate them, it may not be possible to restore the drains to their full effectiveness. This could occur if the plugging mechanism (e.g., calcium carbonate) plugged a significant zone within the drain foundation and extended outward from the perimeter of the drain. While the inside opening of the drain could be reestablished, the drain effectiveness could be drastically reduced, because the zone immediately around the drain had been made impermeable.

Drains at Reclamation dams have received varying amounts of attention. In most cases, drains that were obviously plugged have been detected during examinations of the dam, and a cleaning program was initiated. Cleaning programs that were initiated resulted in varying degrees of success as indicated by the case histories in chapter 4. In other cases, instrumentation data have provided a strong indication that drains are plugged, and appropriate action was taken. In some cases, drains have become plugged without the plugging being detected for a period of time, because the drains could not be easily accessed, and visual evidence did not strongly suggest that the drains were plugging.

This manual hopefully will provide a heightened awareness of the importance and benefits of drainage systems and the need for continual monitoring and maintenance of these systems. Rather than providing strict requirements for a regular drain cleaning program, this manual is intended to provide guidelines for evaluating drainage systems and determining the need for initiating a drain cleaning program. The current CFR and periodic facility review processes provide for a systematic evaluation of each of Reclamation's dams. This provides an ideal opportunity to evaluate drainage systems, considering the instrumentation data, physical condition of the exposed portions of the drainage system, and the need for a comprehensive inspection of the drainage system, typically through the use of a remote video camera. The Performance Parameter documents now available for all major Reclamation dams also are a valuable tool for assessing the performance of drainage systems. If not already provided, Performance Parameter documents should indicate acceptable levels for drain flows and water pressures at a specific dam, and levels at which a further evaluation of a drainage system or a drain cleaning program should be initiated. The Performance Parameter document can also be used to identify a regular interval for probing or inspecting drains with remote video cameras. With experience in maintaining a drainage system at a given dam, a regular schedule of drain cleaning can possibly be established. This will make programming of O&M funds easier and ensure that cleaning can be performed when necessary.

Drain inspection or cleaning should be considered under the following conditions:

- Total drain flows from the entire drainage system have decreased from the original drain flows, or since the most recent drain cleaning exercise. Drain flows should be compared at similar reservoir levels or consistent conditions for slope drains where the water source is groundwater and not a reservoir. If conditions are not consistent for drain flows, it will be difficult to determine if drain effectiveness has changed.

- Uplift pressures, piezometer readings or observation wells indicate an increase from the original installation values or since the most recent drain cleaning exercise. Readings should be compared under consistent conditions as discussed above.

- Significant plugging of the drains has occurred from a visual or video inspection of the drains. Significant plugging would consist of the outlet end of the drains being sealed or nearly sealed off by drain deposits, the inside surface of the drains being coated or sealed by drain deposits, or a blockage within a drain that prevents or nearly prevents portions of the drain from discharging.

As a further tool to evaluate the effectiveness of a drainage system and to establish a maintenance program for drains, the following information should also be considered:

- Information on the drain performance, historical drain flows, and historical instrumentation data providing evidence of drain effectiveness—drain flows, uplift pressure, piezometer and observation well data. In most cases, instrumentation data will need to be evaluated over a long period of time and the general trends evaluated. Changes from year to year may be small and inconclusive. All relevant instrumentation should also be considered as a complete set of data, to provide the best interpretation of drain performance.

- Original design assumptions and analysis results regarding the drains, indicating whether they were considered an integral part of the design, or strictly a defensive measure, and indicating assumptions regarding drain effectiveness.

- For those situations where functioning drains were counted on in the design, an updated analysis should be performed for the case of nonfunctioning drains, if evidence exists that the drainage system has been compromised, and these results should be documented in a Technical Memorandum.

b. *Checklists for Drain Cleaning Program.*—In order to conduct an effective drain cleaning program, monitoring of key parameters before and after the cleaning should be conducted. Two checklists are provided—one for vertical or near vertical drilled foundation drains in rock foundations (primarily for concrete dams) and one for horizontal drains, which would apply to toe drains, slope drains, and other horizontal installations. The following is a checklist of recommended procedures for ensuring a successful drain cleaning program for vertical drains in a rock foundation. This checklist could also be adapted to address cleaning of vertical formed drains in a concrete dam.

- Measure drain flows prior to cleaning, measuring individual drains if possible

- Take uplift pressure readings and piezometer readings prior to cleaning.

- Probe holes with rods or a plumb bob to determine depth of open hole prior to cleaning. Compare this measurement to as-built depths, if available.

- Use a borehole camera to inspect borehole walls on selected drains prior to cleaning. Drains to be inspected should be selected to represent distinct zones in the foundation. Note changes in rock type, deposits, and other variables along length of hole. The borehole may require washing/flushing to allow access with the camera or to provide a clean column of water for viewing with the borehole camera. The borehole inspection of the borehole walls, as well as other geologic information should be used to identify any soft zones in the foundation. The presence of soft zones and the general condition of the borehole walls should be used to establish maximum cleaning pressures, where high pressure water jet cleaning is used.

- Initiate cleaning of the holes. If using water jetting methods, record nozzle details, nozzle orientations, cleaning rate (ft/min) and pressures used during cleaning. Also, document specifics of equipment used (catalog sheets and devices used to centralize the nozzles in the hole).

- Flush holes thoroughly with water after cleaning.

- Measure drain flows and record uplift pressures periodically during drain cleaning to identify incremental effects.

- Probe holes with rods or plumb bob to determine depth of open hole after cleaning. Compare this measurement to precleaning depths (if available).

- Use a borehole camera to videotape borehole walls of the same drains previously inspected after cleaning. Note any remaining deposits or partial plugs along depth of hole, as well as any evidence of erosion or caving of sidewalls from cleaning. If deposits still remain, additional cleaning efforts should be considered. If possible, determine if deposits were cleaned from fractures/joints intersecting the borehole walls.

- Measure drain flows after cleaning, measuring individual drains if possible.

- Take uplift pressure readings and piezometer readings after cleaning.

- Summarize cleaning activities in a report, including, photographs, graphs and tables that readily portray before and after conditions and demonstrate effectiveness of cleaning. Integrate postcleaning flow rates and/or uplift pressures and piezometer readings with historical instrumentation records for the structure. Some of the case histories in chapter 4 provide plots showing before and after instrumentation data.

The following is a checklist of recommended procedures for ensuring a successful horizontal drain cleaning program:

- Measure drain flows and pressures prior to cleaning, measuring individual drains if possible.

- Probe drains with rods to determine depth (or length) of open holes prior to cleaning. Compare this measurement to as-built depths, if available. If possible, use a camera to videotape the walls/casing/drain pipe interior before cleaning. Note any deposits or partial plugs along depth of drain, as well as any evidence of erosion or caving of sidewalls in noncased holes, or damage to casing or drain pipe. The inspection of the drain walls as well as other information on the drain pipe materials and the material surrounding the drains should be used to identify any vulnerable areas along the drains. The presence of vulnerable areas along the drains and the general condition of the drain should be used to establish maximum cleaning pressures, where high pressure water jet cleaning is used.

- Initiate cleaning of the drains. The drain may be cleaned using only washing/flushing with water. If using water jetting methods, record nozzle details, nozzle orientations, cleaning rate (ft/min), and pressure used during cleaning. Also, document specifics of equipment used (catalog sheets and devices used to center nozzles in the hole, size and length of rods/pipe used to wash/flush the holes, flow rates, photographs, etc.).

- Record type of material cleaned from the drains (rust, roots, sediment, mineral encrustation, fragments of drain pipe, etc.). Knowledge of the type of material cleaned from the drains may be useful during future cleaning operations.

- Flush drains thoroughly with water after cleaning.

- Probe drains with rods to determine depth of open hole after cleaning. Compare this measurement to precleaning depths (if available).

- If possible, use a camera to videotape the walls/casing/drain pipe interior after cleaning. Note any remaining deposits or partial plugs along depth of hole, as well as any evidence of erosion or caving of sidewalls from cleaning noncased holes, or damage to casing or drain pipe from cleaning.

- Measure drain flows and pressures periodicallly during cleaning to identify incremental effects.

- Measure drain flows and pressures after cleaning, measuring individual drains if possible.

- Summarize cleaning activities in a report, including graphs, tables, and photographs that readily portray before and after conditions and demonstrate effectiveness of cleaning. Integrate postcleaning flow rates with historical instrumentation records for the structure. Some of the case histories in chapter 4 provide plots showing before and after instrumentation data.

Checklists for other types of drains, such as relief wells, can also be developed using the same concepts of inspecting the drains and monitoring instrumentation before cleaning, cleaning the drains, and then reinspecting the drains and monitoring instrumentation after cleaning.

Bibliography

[1] Douglas, K.J., M. Spannagle, and R. Fell, *Analysis of Concrete and Masonry Dam Incidents*, UNICIV Report No. R-373, The University of New South Wales, Sydney, Australia, August 1998.

[2] Bureau of Reclamation, *Guidelines, Foundation and Geotechnical Studies for Existing Concrete Dams*, September 1999.

[3] U.S. Army Corps of Engineers, Engineer Manual 1110-2-2007, *Structural Design of Concrete Lined Flood Control Channels*, April 30, 1995.

[4] Casagrande, A., "Control of Seepage through Foundations and Abutments of Dams," First Rankine Lecture, *Geotechnique*, Vol. II, June 1961, pp 159-180.

[5] Brahtz, J.H.A., *Pressures due to percolating water and their influence upon stresses in hydraulic structures*, Trans. 2nd Congress Large Dams, Washington, 5:43-71, 1936.

[6] Muskat, M., *The flow of homogeneous fluids through porous bodies*, McGraw-Hill, New York, 1937.

[7] Goodman, R.E., B. Amadei, and N. Sitar, "Uplift Pressure in Crack Below Dam," *Journal of Energy Engineering*, Vol. 109, No. 9, American Society of Civil Engineers, December 1983.

[8] Powell, Chris, Technical Memorandum No. HM-8312-2, *Right Abutment Stability Improvement for Horse Mesa Dam, Salt River Project, Arizona*, October 1997.

[9] U.S. Army Corps of Engineers, Engineering Manual No. 1110-2-1914, *Engineering and Design; Design Construction and Maintenance of Relief Wells*, Washington D.C., May 29, 1992.

[10] U.S. Army Corps of Engineers, Engineering Manual No. 1110-2-1942, *Engineering and Design; Inspection, Monitoring and Maintenance of Relief Wells*, Washington D.C., September 25, 1998.

[11] Cedegren, H.R., *Seepage, Drainage and Flow Nets*, John Wiley and Sons, New York, 1967.

[12] Bureau of Reclamation, Design Summary, *Big Sandy Dam Spillway Modification, Eden Project, Wyoming*, March 1987.

[13] Jones, C.W., D.G. Miedema, and J.S. Watkins, Report REC-ERC-82-17, *Frost Action in Soil Foundations and Control of Surface Structure Heaving*, Bureau of Reclamation, June 1982.

[14] Robinski, E.I., and K.E. Bespflug, Paper, "Design of Insulated Foundations," *ASCE Journal of the Soil Mechanics of Foundations Division*, September 1973.

[15] Aiken, G.W., and R.L. Berg, Special Report 122, *Digital Solution of Modified Berggren Equation to Calculate Depths of Freeze or Thaw in Multilayered Systems*, U.S. Army Corps of Engineers, October 1968.

[16] Departments of the Army and the Air Force, Technical Manual 5-852-6, *Calculation Methods for Determination of Depths of Freeze and Thaw in Soils*, January 1966.

[17] Berg, R.L., Special Report 76-3, *Thermoinsulating Media Within Embankments on Perennially Frozen Soil*, U. S. Army Corps of Engineers, May 1976.

[18] Smith, C.D., and Zirong Gui, Paper, *Pressure Relief Under Hydraulic Jump Stilling Basins*, Hydraulic Engineering 93, 1993.

[19] Bureau of Reclamation, Draft Specifications, *Spring Creek Debris Dam Enlargement*, December 1994.

[20] Hepler, T.E., and P.L. Johnson, Paper, *Analysis of Spillway Failures by Uplift Pressures*, ASCE Conference, August 1988.

[21] Bureau of Reclamation, Engineering Monograph No. 41, *Air-Water Flow in Hydraulic Structures*, December 1980.

[22] Bureau of Reclamation, *Glossary of Technical Terms*, web site: http://www.usbr.gov/cdams/index.html.

[23] Bureau of Reclamation, Assistant Commissioner - Engineering and Research (ACER) Technical Memorandum (TM) No. 9, *Guidelines for Controlling Seepage Along Conduits Through Embankments*, April 1987.

[24] Hoek, E., and J.W. Bray, *Rock Slope Engineering*, Third Edition, The Institute of Mining and Metallurgy, London, England, 1981.

[25] Houlsby, A.C., *Construction and Design of Cement Grouting—A Guide to Grouting in Rock Foundations*, John Wiley and Sons, Inc., 1990.

[26] Bureau of Reclamation, Specifications Document 1-553.01-1, Amendment No. 3, *Costilla Dam Modifications*, May 23, 1988.

[27] Bureau of Reclamation, Construction Considerations, *Big Sandy Dam Spillway Modification, Eden Project Wyoming*, June 1984.

[28] California Department of Transportation, *The Effectiveness of Horizontal Drains*, Report No. FHWA/CA/TL-80/16, Sacramento, California, June 1980.

[29] *Drilling, The Manual of Methods, Applications, and Management*, Australian Drilling Industry Training Committee Limited, 1997.

[30] Driscoll, F.G., *Groundwater and Wells*, Second Edition, 1986.

[31] Bureau of Reclamation, *Engineering Geology Field Manual*, Second Edition, Volume I, 1998.

[32] Sanders, Glen D., "Subsurface Drainage of Irrigated Land," *Water Operation and Maintenance Bulletin*, No. 188, Bureau of Reclamation, June 1999.

[33] Hackett, Glen, and Lehr, J.H., *Iron Bacteria Occurrence Problems, and Control Methods in Water Wells*, National Water Well Association for U.S. Army Corps of Engineers, 1985.

[34] Sisneros, Thomas, Memorandum to Robert Love, from Thomas Sisneros, subject: *Brantley Dam Drain Hole Samples, Alamosa Field Division, Colorado*, August, 3, 1995.

[35] Day, Gregg, and Todd Rutenbeck, *An Evaluation of Water Jetting for Bureau of Reclamation Use in Cleaning Concrete Dam Foundation Drains*, Bureau of Reclamation, Technical Service Center, Denver, Colorado, April 1999.

[36] Bureau of Reclamation, Technical Report of Construction, *Grouting and Drainage Remediation, Upper Stillwater Dam—Strawberry Collection System, Bonneville Unit, Central Utah Project, Utah*, November 1993.

[37] Shaffner, Pete, Technical Memorandum No. US-3620-3, *Evaluation of 1992/1993 Grouting and Drain Remediation Program, Upper Stillwater Dam*, Technical Service Center, Bureau of Reclamation, September 1995.

[38] Baumgarten, Robert A., and James A. Lundeen, *Foundation Drain Maintenance Methods*, Report No. GR-89-1, Geotechnical Services Branch, Research and Laboratory Services Division, Bureau of Reclamation, Denver, Colorado, January 1989.

[39] Bureau of Reclamation, Piezometer Installation Program, Davis Dam, Phoenix Area Office, Phoenix, Arizona, May 21, 1997.

[40] Bureau of Reclamation, Design Standards No. 13—Embankment Dams, Chapter 4, *Static Stability Analysis*, Denver, Colorado, August 3, 1987.

[41] Bureau of Reclamation, Design Standards No. 13—Embankment Dams, Chapter 5, *Protective Filters*, Denver, Colorado, November 24, 1994.

[42] Bureau of Reclamation, Design Standards No. 13—Embankment Dams, Chapter 8, *Seepage Analysis and Control*, Denver, Colorado, March 20, 1987.

[43] Bureau of Reclamation, *Design of Small Dams*, Third Edition, Denver, Colorado, 1987.

[44] Wong, J., and A. Kumar, *Drain Practices at B.C. Hydro Dams*, B.C. Hydro, September 1994.

[45] Bureau of Reclamation, *Ground Water Manual*, Second Edition, Denver, Colorado, 1995.

[46] Bureau of Reclamation, *Drainage Manual*, Third Edition revised reprint, Denver, Colorado, 1993.

[47] Bureau of Reclamation, Specifications Design Data, *Big Sandy Dam Spillway Modification*, May 1982.

[48] Advanced Drainage Systems, Inc. (ADS), *ADS Culvert Pipe. ADS Sewer Pipe Engineered for Tough Drainage and Storm Sewer Applications*, (promotional literature), ADS, Columbus, Ohio, 1986.

[49] Bureau of Reclamation, *Hydraulic and Excavation Tables*, Eleventh Edition, 1957.

[50] Agard, S.S., *Preliminary Geologic Map of the Yellowtail Dam Quadrangle, Big Horn, County, Montana*, USGS Miscellaneous Field Studies Map MF-2039, 1988.

[51] Allen, D.U., *Final Construction Engineering Geology Report, Yellowtail Dam— Montana*, Bureau of Reclamation, 1966.

[52] Bergantino, R.N., *Geologic Map of the Hardin 1 Degree by 2 Degrees Quadrangle, Southeastern Montana*, Montana Bureau of Mines and Geology Montana Atlas 2-A, 1980.

[53] Bureau of Reclamation, Technical Record of Design and Construction, *Yellowtail Dam and Powerplant*, 1975.

[54] Bureau of Reclamation, Solicitation No. 6-SI-60-02010, *Foundation Drain Cleanout, Yellowtail Dam and Powerplant, Yellowtail Unit—Lower Bighorn Division, Pick-Sloan Missouri Basin Program, Montana*, 1986.

[55] Bureau of Reclamation, *1995 Periodic Facility Review Report—Yellowtail Dam, Pick-Sloan Missouri Basin Program, Great Plains Region*, 1995.

[56] Feuerstein, V., *Safety for the Hi-Pressure Drain Cleaning Operations*, Bureau of Reclamation Safety, 1999.

[57] Jepson, D.H. and W.J. Witherspoon, *Explorations, Yellowtail Damsite*, Bureau of Reclamation, 1948.

[58] Mann, D., *A Report on the Water Testing of Cavities and Drill Holes in Tunnels "A" and "C" at Yellowtail Damsite*, Bureau of Reclamation, 1949.

[59] Patterson, C.G., et al., *Mineral Resources of the Pryor Mountains, Burnt Timber Canyon, and Big Horn Tack-On Wilderness Study Areas, Carbon County, Montana, and Big Horn County, Wyoming*, USGS Bulletin 1723, 1988.

[60] Perry, E.S., *Geology and Ground-Water Resources of Southeastern Montana, Montana*, Bureau of Mines and Geology Memoir No. 14, 1935.

[61] Taucher, G., *Engineering Geology Report on Seepage Control Drill Holes, Spring S-10 and Spring S-24-R Area*, Bureau of Reclamation, 1969.

[62] Thom, W.T., et al., *Geology of Big Horn County and the Crow Indian Reservation - Montana*, USGS Bulletin 856, 1935.

[63] Witherspoon, W.J., *A Special Preliminary Report on the Foundation Exploration and Geology of Yellowtail Damsite*, Bureau of Reclamation, 1948.

[64] Witherspoon, W.J., Final Construction Report, *Drilling of Holes for Ground-Water Investigations, Yellowtail Dam Site*, Bureau of Reclamation, 1953.

[65] Bureau of Reclamation, Technical Record of Design and Construction, *Canyon Ferry Dam and Powerplant*, 1957.

[66] Scott, Gregg A., *Report of Findings Comprehensive Facility Review for Canyon Ferry Dam*, Bureau of Reclamation, 1998.

[67] Wilson, R.E., *Final Construction Report on the Geology of Canyon Ferry Dam, Missouri Basin Project, Canyon Ferry Unit, Canyon Ferry, Montana*, Bureau of Reclamation, 1953.

[68] Bureau of Reclamation, *SEED (Safety Evaluation of Existing Dams) Report, Canyon Ferry Dam, Pick-Sloan Missouri Basin Program, Montana, Missouri Basin Region*, 1987.

[69] Water and Power Resources Service, *Project Data*, 1981.

[70] Grenoble, B. Alex, and Amadei, Bernard, *Evaluation of Uplift Pressure for Concrete Gravity Dams Founded on Jointed Rock: Analytical Results*, Dam Foundation Engineering, 10th Annual Lecture Series, New Orleans, LA, March 1990.

Index

A

B

C

D

E

F

G

H

I-L

M-N

O

P

R

S

T

U-Y

Bibliography

[1] Douglas, K.J., M. Spannagle, and R. Fell, *Analysis of Concrete and Masonry Dam Incidents*, UNICIV Report No. R-373, The University of New South Wales, Sydney, Australia, August 1998.

[2] Bureau of Reclamation, *Guidelines, Foundation and Geotechnical Studies for Existing Concrete Dams*, September 1999.

[3] U.S. Army Corps of Engineers, Engineer Manual 1110-2-2007, *Structural Design of Concrete Lined Flood Control Channels*, April 30, 1995.

[4] Casagrande, A., "Control of Seepage through Foundations and Abutments of Dams," First Rankine Lecture, *Geotechnique*, Vol. II, June 1961, pp 159-180.

[5] Brahtz, J.H.A., *Pressures due to percolating water and their influence upon stresses in hydraulic structures*, Trans. 2nd Congress Large Dams, Washington, 5:43-71, 1936.

[6] Muskat, M., *The flow of homogeneous fluids through porous bodies*, McGraw-Hill, New York, 1937.

[7] Goodman, R.E., B. Amadei, and N. Sitar, "Uplift Pressure in Crack Below Dam," *Journal of Energy Engineering*, Vol. 109, No. 9, American Society of Civil Engineers, December 1983.

[8] Powell, Chris, Technical Memorandum No. HM-8312-2, *Right Abutment Stability Improvement for Horse Mesa Dam, Salt River Project, Arizona*, October 1997.

[9] U.S. Army Corps of Engineers, Engineering Manual No. 1110-2-1914, *Engineering and Design; Design Construction and Maintenance of Relief Wells*, Washington D.C., May 29, 1992.

[10] U.S. Army Corps of Engineers, Engineering Manual No. 1110-2-1942, *Engineering and Design; Inspection, Monitoring and Maintenance of Relief Wells*, Washington D.C., September 25, 1998.

[11] Cedegren, H.R., *Seepage, Drainage and Flow Nets*, John Wiley and Sons, New York, 1967.

[12] Bureau of Reclamation, Design Summary, *Big Sandy Dam Spillway Modification, Eden Project, Wyoming*, March 1987.

[13] Jones, C.W., D.G. Miedema, and J.S. Watkins, Report REC-ERC-82-17, *Frost Action in Soil Foundations and Control of Surface Structure Heaving*, Bureau of Reclamation, June 1982.

[14] Robinski, E.I., and K.E. Bespflug, Paper, "Design of Insulated Foundations," *ASCE Journal of the Soil Mechanics of Foundations Division*, September 1973.

[15] Aiken, G.W., and R.L. Berg, Special Report 122, *Digital Solution of Modified Berggren Equation to Calculate Depths of Freeze or Thaw in Multilayered Systems*, U.S. Army Corps of Engineers, October 1968.

[16] Departments of the Army and the Air Force, Technical Manual 5-852-6, *Calculation Methods for Determination of Depths of Freeze and Thaw in Soils*, January 1966.

[17] Berg, R.L., Special Report 76-3, *Thermoinsulating Media Within Embankments on Perennially Frozen Soil*, U. S. Army Corps of Engineers, May 1976.

[18] Smith, C.D., and Zirong Gui, Paper, *Pressure Relief Under Hydraulic Jump Stilling Basins*, Hydraulic Engineering 93, 1993.

[19] Bureau of Reclamation, Draft Specifications, *Spring Creek Debris Dam Enlargement*, December 1994.

[20] Hepler, T.E., and P.L. Johnson, Paper, *Analysis of Spillway Failures by Uplift Pressures*, ASCE Conference, August 1988.

[21] Bureau of Reclamation, Engineering Monograph No. 41, *Air-Water Flow in Hydraulic Structures*, December 1980.

[22] Bureau of Reclamation, *Glossary of Technical Terms*, web site: http://www.usbr.gov/cdams/index.html.

[23] Bureau of Reclamation, Assistant Commissioner - Engineering and Research (ACER) Technical Memorandum (TM) No. 9, *Guidelines for Controlling Seepage Along Conduits Through Embankments*, April 1987.

[24] Hoek, E., and J.W. Bray, *Rock Slope Engineering*, Third Edition, The Institute of Mining and Metallurgy, London, England, 1981.

[25] Houlsby, A.C., *Construction and Design of Cement Grouting—A Guide to Grouting in Rock Foundations*, John Wiley and Sons, Inc., 1990.

[26] Bureau of Reclamation, Specifications Document 1-553.01-1, Amendment No. 3, *Costilla Dam Modifications*, May 23, 1988.

[27] Bureau of Reclamation, Construction Considerations, *Big Sandy Dam Spillway Modification, Eden Project Wyoming*, June 1984.

[28] California Department of Transportation, *The Effectiveness of Horizontal Drains*, Report No. FHWA/CA/TL-80/16, Sacramento, California, June 1980.

[29] *Drilling, The Manual of Methods, Applications, and Management*, Australian Drilling Industry Training Committee Limited, 1997.

[30] Driscoll, F.G., *Groundwater and Wells*, Second Edition, 1986.

[31] Bureau of Reclamation, *Engineering Geology Field Manual*, Second Edition, Volume I, 1998.

[32] Sanders, Glen D., "Subsurface Drainage of Irrigated Land," *Water Operation and Maintenance Bulletin*, No. 188, Bureau of Reclamation, June 1999.

[33] Hackett, Glen, and Lehr, J.H., *Iron Bacteria Occurrence Problems, and Control Methods in Water Wells*, National Water Well Association for U.S. Army Corps of Engineers, 1985.

[34] Sisneros, Thomas, Memorandum to Robert Love, from Thomas Sisneros, subject: *Brantley Dam Drain Hole Samples, Alamosa Field Division, Colorado*, August, 3, 1995.

[35] Day, Gregg, and Todd Rutenbeck, *An Evaluation of Water Jetting for Bureau of Reclamation Use in Cleaning Concrete Dam Foundation Drains*, Bureau of Reclamation, Technical Service Center, Denver, Colorado, April 1999.

[36] Bureau of Reclamation, Technical Report of Construction, *Grouting and Drainage Remediation, Upper Stillwater Dam—Strawberry Collection System, Bonneville Unit, Central Utah Project, Utah*, November 1993.

[37] Shaffner, Pete, Technical Memorandum No. US-3620-3, *Evaluation of 1992/1993 Grouting and Drain Remediation Program, Upper Stillwater Dam*, Technical Service Center, Bureau of Reclamation, September 1995.

[38] Baumgarten, Robert A., and James A. Lundeen, *Foundation Drain Maintenance Methods*, Report No. GR-89-1, Geotechnical Services Branch, Research and Laboratory Services Division, Bureau of Reclamation, Denver, Colorado, January 1989.

[39] Bureau of Reclamation, Piezometer Installation Program, Davis Dam, Phoenix Area Office, Phoenix, Arizona, May 21, 1997.

[40] Bureau of Reclamation, Design Standards No. 13—Embankment Dams, Chapter 4, *Static Stability Analysis*, Denver, Colorado, August 3, 1987.

[41] Bureau of Reclamation, Design Standards No. 13—Embankment Dams, Chapter 5, *Protective Filters*, Denver, Colorado, November 24, 1994.

[42] Bureau of Reclamation, Design Standards No. 13—Embankment Dams, Chapter 8, *Seepage Analysis and Control*, Denver, Colorado, March 20, 1987.

[43] Bureau of Reclamation, *Design of Small Dams*, Third Edition, Denver, Colorado, 1987.

[44] Wong, J., and A. Kumar, *Drain Practices at B.C. Hydro Dams*, B.C. Hydro, September 1994.

[45] Bureau of Reclamation, *Ground Water Manual*, Second Edition, Denver, Colorado, 1995.

[46] Bureau of Reclamation, *Drainage Manual*, Third Edition revised reprint, Denver, Colorado, 1993.

[47] Bureau of Reclamation, Specifications Design Data, *Big Sandy Dam Spillway Modification*, May 1982.

[48] Advanced Drainage Systems, Inc. (ADS), *ADS Culvert Pipe. ADS Sewer Pipe Engineered for Tough Drainage and Storm Sewer Applications*, (promotional literature), ADS, Columbus, Ohio, 1986.

[49] Bureau of Reclamation, *Hydraulic and Excavation Tables*, Eleventh Edition, 1957.

[50] Agard, S.S., *Preliminary Geologic Map of the Yellowtail Dam Quadrangle, Big Horn, County, Montana*, USGS Miscellaneous Field Studies Map MF-2039, 1988.

[51] Allen, D.U., *Final Construction Engineering Geology Report, Yellowtail Dam— Montana*, Bureau of Reclamation, 1966.

[52] Bergantino, R.N., *Geologic Map of the Hardin 1 Degree by 2 Degrees Quadrangle, Southeastern Montana*, Montana Bureau of Mines and Geology Montana Atlas 2-A, 1980.

[53] Bureau of Reclamation, Technical Record of Design and Construction, *Yellowtail Dam and Powerplant*, 1975.

[54] Bureau of Reclamation, Solicitation No. 6-SI-60-02010, *Foundation Drain Cleanout, Yellowtail Dam and Powerplant, Yellowtail Unit—Lower Bighorn Division, Pick-Sloan Missouri Basin Program, Montana*, 1986.

[55] Bureau of Reclamation, *1995 Periodic Facility Review Report—Yellowtail Dam, Pick-Sloan Missouri Basin Program, Great Plains Region*, 1995.

[56] Feuerstein, V., *Safety for the Hi-Pressure Drain Cleaning Operations*, Bureau of Reclamation Safety, 1999.

[57] Jepson, D.H. and W.J. Witherspoon, *Explorations, Yellowtail Damsite*, Bureau of Reclamation, 1948.

[58] Mann, D., *A Report on the Water Testing of Cavities and Drill Holes in Tunnels "A" and "C" at Yellowtail Damsite*, Bureau of Reclamation, 1949.

[59] Patterson, C.G., et al., *Mineral Resources of the Pryor Mountains, Burnt Timber Canyon, and Big Horn Tack-On Wilderness Study Areas, Carbon County, Montana, and Big Horn County, Wyoming*, USGS Bulletin 1723, 1988.

[60] Perry, E.S., *Geology and Ground-Water Resources of Southeastern Montana, Montana*, Bureau of Mines and Geology Memoir No. 14, 1935.

[61] Taucher, G., *Engineering Geology Report on Seepage Control Drill Holes, Spring S-10 and Spring S-24-R Area*, Bureau of Reclamation, 1969.

[62] Thom, W.T., et al., *Geology of Big Horn County and the Crow Indian Reservation - Montana*, USGS Bulletin 856, 1935.

[63] Witherspoon, W.J., *A Special Preliminary Report on the Foundation Exploration and Geology of Yellowtail Damsite*, Bureau of Reclamation, 1948.

[64] Witherspoon, W.J., Final Construction Report, *Drilling of Holes for Ground-Water Investigations, Yellowtail Dam Site*, Bureau of Reclamation, 1953.

[65] Bureau of Reclamation, Technical Record of Design and Construction, *Canyon Ferry Dam and Powerplant*, 1957.

[66] Scott, Gregg A., *Report of Findings Comprehensive Facility Review for Canyon Ferry Dam*, Bureau of Reclamation, 1998.

[67] Wilson, R.E., *Final Construction Report on the Geology of Canyon Ferry Dam, Missouri Basin Project, Canyon Ferry Unit, Canyon Ferry, Montana*, Bureau of Reclamation, 1953.

[68] Bureau of Reclamation, *SEED (Safety Evaluation of Existing Dams) Report, Canyon Ferry Dam, Pick-Sloan Missouri Basin Program, Montana, Missouri Basin Region*, 1987.

[69] Water and Power Resources Service, *Project Data*, 1981.

[70] Grenoble, B. Alex, and Amadei, Bernard, *Evaluation of Uplift Pressure for Concrete Gravity Dams Founded on Jointed Rock: Analytical Results*, Dam Foundation Engineering, 10th Annual Lecture Series, New Orleans, LA, March 1990.

Appendix A

Design/Analysis Examples

Appendix A provides design/analysis examples for various types of drain systems:

Example A1—Drainage "Rules of Thumb" for Appurtenant Structures.—

- *Location of drainage provisions for hydraulic appurtenant structures (such as spillways and outlet works).*—Generally limited to downstream of water barriers in dams and/or dikes. Drainage provisions downstream of the water barriers tend to be continuous, such as an underdrain system that collects and conveys seepage to the exit channel and/or river. Any drainage provisions upstream of the water barriers, such as weepholes, do not cross the water barriers. Their intent is to relieve localized hydrostatic pressures due to reservoir fluctuations. Water barriers include grout curtains (in concrete, embankment, and rockfill dams/dikes) and impervious cores (in embankment, and rockfill dams/dikes).

- *Riprap bedding (i.e., filter material) thickness (associated with excavated or filled channels).*—Half the riprap thickness. Bedding material is typically well graded crushed rock, gravel, or concrete aggregate.

- *Size of perforated/slotted PVC collector pipes (such as those used in trench-type drainage provisions).*—Minimum of 6-inch diameter, maximum range of 8- to 10-inch diameter. For perforated/slotted PVC pipe inserts (such as those used in drilled/formed weep holes), the minimum size should be 2-inch diameter.

- *Spacing/number of lateral perforated/slotted PVC collector pipes (such as those used in trench-type drainage provisions).*—Generally located at or adjacent to floor joints (i.e., contraction, control, and/or expansion joints), which typically have spacing of 25 to 50 feet for concrete appurtenant structures.

- *Spacing/number of longitudinal perforated/slotted PVC collector pipes (such as those used in trench-type drainage provisions).*—When the width (lateral) of appurtenant structures is less than 30 feet, collector pipes (longitudinal direction) located at the outside edges of the appurtenant structures are generally adequate. When the width of appurtenant structures is greater than or equal to 30 feet, intermediate collector pipes (longitudinal), spaced between the outside edges collector pipes, should be considered.

- *Spacing/number of drilled/formed weep holes for walls and slabs—Longitudinal and lateral, vertical and horizontal.*—Range of 5 to 25 feet.

- *Depth of drilled/formed weep holes for walls and slabs.*—Greater than or equal to depth of anchor bars and/or rockbolts.

Example A2—Pervious Backfill Requirements Adjacent to Appurtenant Structures.—Pervious backfill applications can be adjacent to spillway and outlet works inlet structure walls (upstream of dam and/or dike); adjacent to spillway and outlet works chute and terminal structure walls (downstream of dam and/or dike); enveloping outlet works conduits (downstream of dam and/or dike); adjacent to outlet works control structures (downstream of dam and/or dike); and adjacent to other appurtenant structures when control of seepage and ice is important. Its primary purposes include augmenting drainage provisions, and controlling the development of ice lenses and subsequent "frost heave". As an illustration, figure 1 shows types of soil materials that have varying potentials for frost heave.

a. *Material Requirements*.—A basis for design and development of specifications paragraphs is summarized in the following text. Pervious backfill is similar to embankment zone 2 (sand and gravel mixture), which is selected materials, reasonably well graded to 6-inch maximum size, except that occasional fragments larger than 6 inches may be used if well distributed in the backfill, given that no material larger than 6 inches will be allowed adjacent to the concrete appurtenant structure. This is to mitigate the potential for a stress point developing through the larger rock fragment into the concrete. The pervious backfill shall not contain more than 5 percent, by weight, of materials passing a United States Standard No. 200 sieve. The pervious backfill is provided from any "approved" borrow or commercial source. The pervious backfill may require washing or other processing to remove excess fines. Care is required to protect the pervious backfill from contamination, such as silt, clay, cement, organic materials, or other materials that would reduce the in-place permeability and density or increase its in-place cohesion. Also, during stockpiling of pervious backfill, remixing may be required if segregation occurs due to inclement weather (such as wind or rain). The specific gravity of the pervious backfill larger than a United States No. 4 sieve shall be 2.56 or greater. Clean pervious backfill with a specific gravity of less than 2.56 will be allowed farther than 5 feet from the concrete [26, 27]. Figure 1 presents frost susceptibility of a wide range of soil materials. As one can see, sands and gravels are the least susceptible, while silts are the most susceptible to frost formation.

The pervious backfill shall be handled and placed in such a manner as to prevent segregation. Pervious backfill placement on either side of an appurtenant structure shall be kept approximately at the same level as the placing of the backfill progresses. Pervious backfill shall be placed and roughly leveled off in layers not more than 24 inches thick. For most cases, pervious backfill is not compacted. Where compaction is deemed needed, relative densities in the range of 70 to 75 percent are typical.

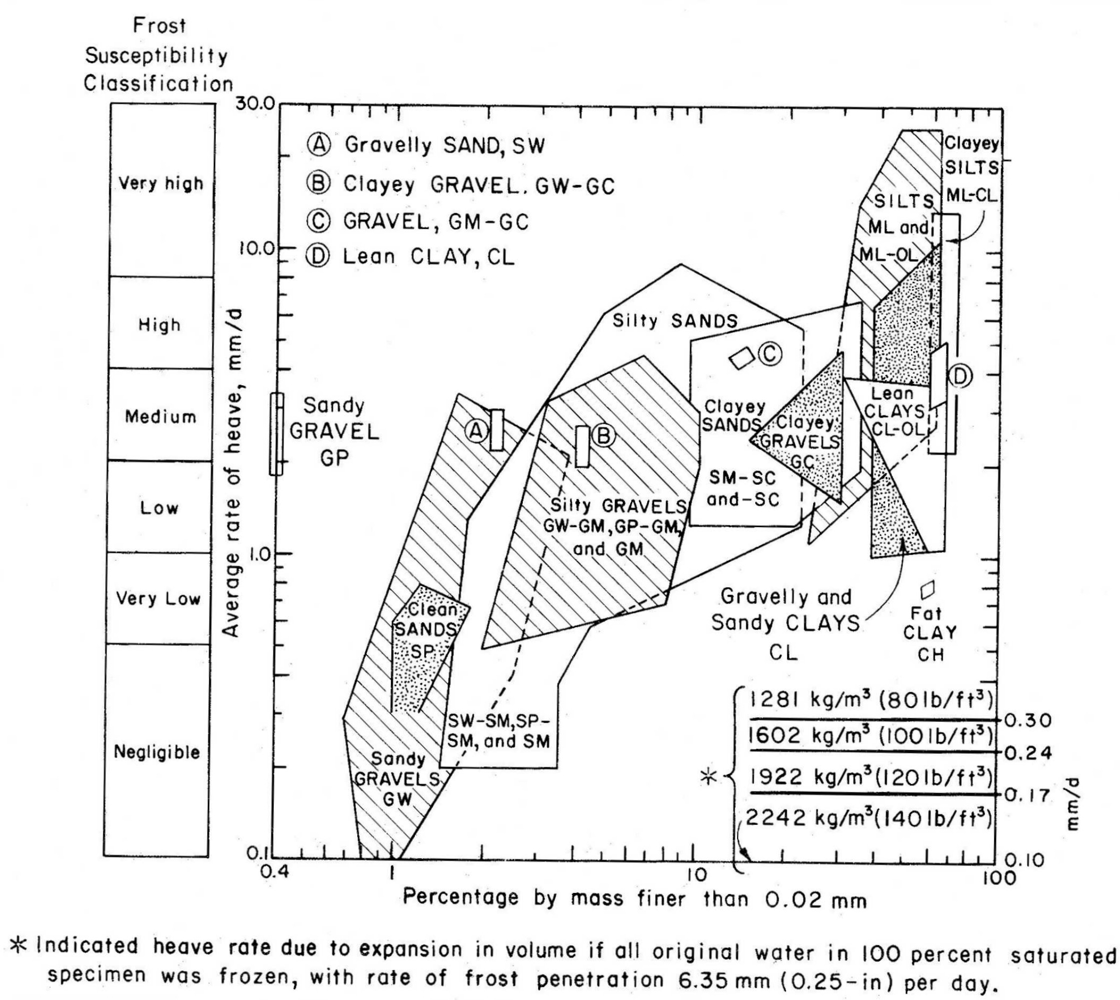


Figure 1.—Summary of average rate of heave versus percentage of natural soil finer than 0.02 mm [13].

b. *Example*.—The following example is the actual design for estimating the thickness of pervious backfill needed for the Big Sandy spillway modification [12].

1. *Design Data*.—

- Design surface freezing index, °F—1,800 degree-days [13]
- Average wall thickness for battered cantilever walls—2 feet [12]

- Material properties of pervious backfill—unit weight, (, 120 lb/ft^3 and 15 percent moisture [12]

2. *Analysis, Design, and Results*.—Using figure 2 to determine the thickness of non-frost-susceptible backfill behind concrete walls, the estimated minimum thickness is 5 feet.

3. *Conclusions*.—Although only a 5-foot thickness of pervious backfill was estimated, the actual thickness specified was 8 feet. This increase was due to constructability considerations (i.e., minimum size of placing equipment). Note: Although figure 3 shows a zoned backfill, in other cases it may be more cost effective and more constructable to simply specify one type of backfill.

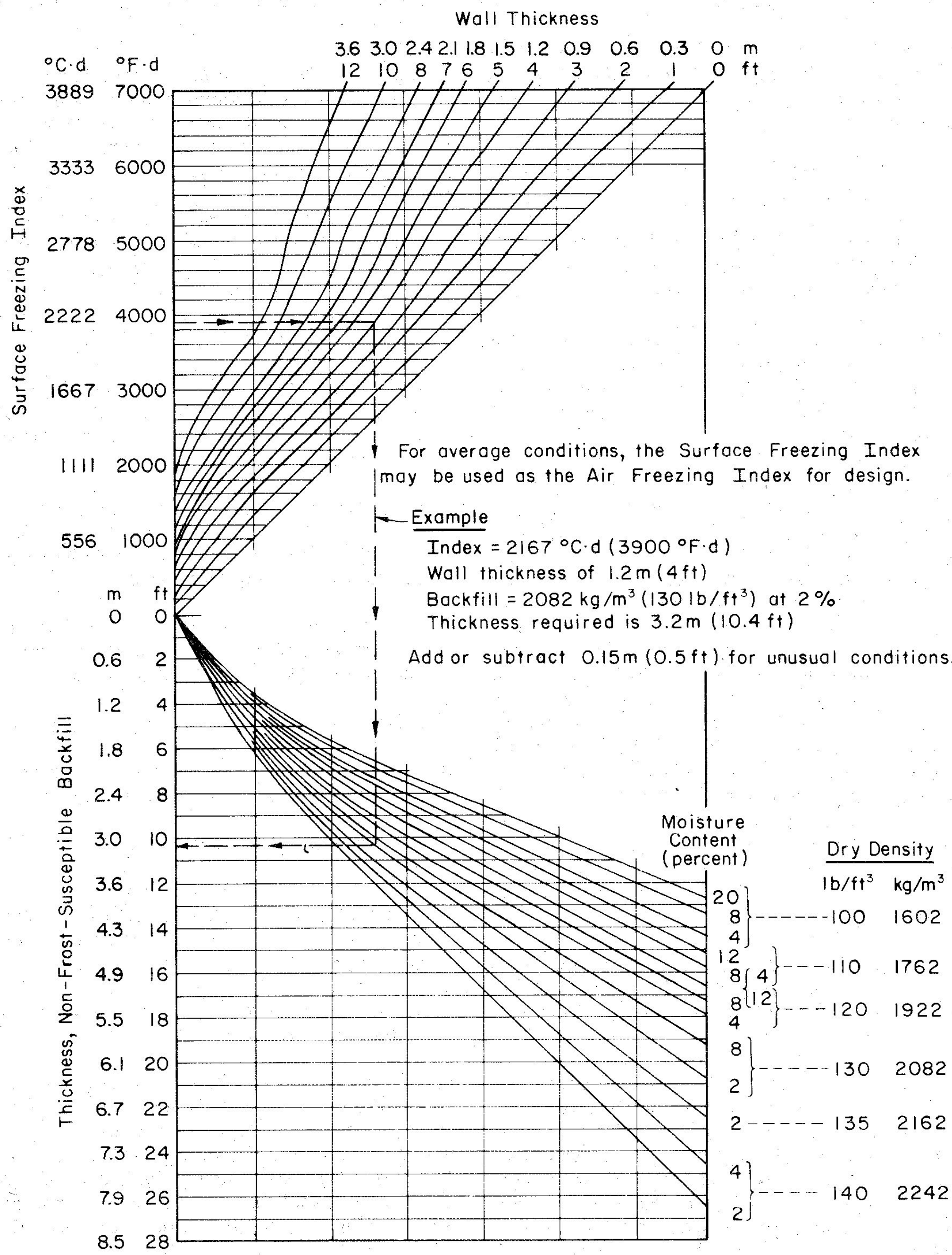


Figure 2.—Thickness of non-frost-susceptible backfill behind concrete walls [13].

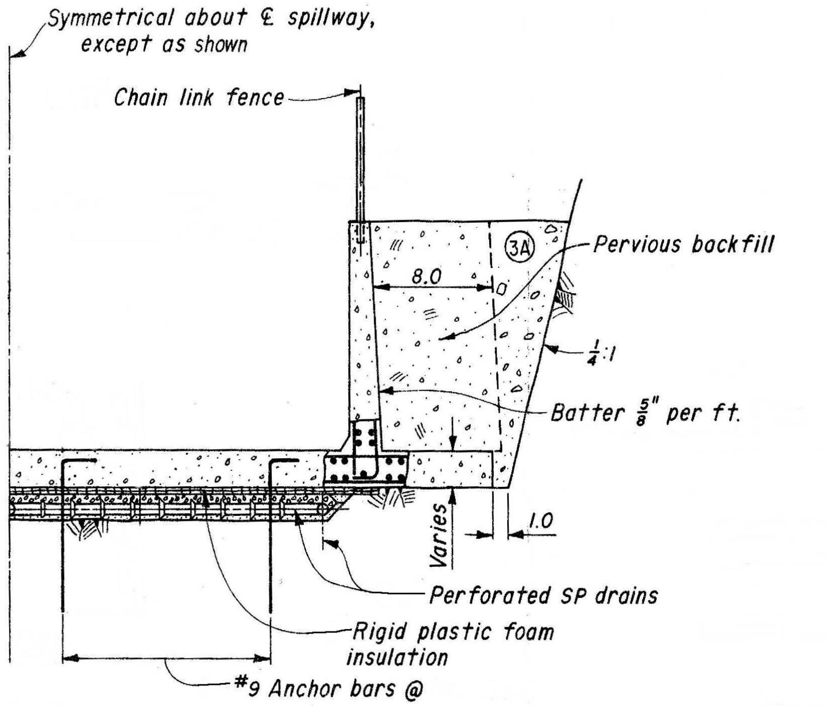


Figure 3.—Big Sandy Dam spillway modification, Wyoming. Pervious Backfill adjacent to wall [12].

Example A3—Insulation Requirements for Underdrains.—The example presented is the design of the spillway underdrain insulation for Big Sandy Dam modification [12] (figs. 4 and 5).

a. *Design Data.*—

1. *Site Conditions.*—

- Design air-freezing index, °F—2,000 degree-days [13]
- Surface correction factor (pavement free of snow and ice), n—0.9 [16]
- Design surface freezing index, nF—1,800 degree-days [13]
- Mean annual temperature, MAT—36.3 °F [47]
- Length of freezing season (November through April), f—160 days [47]

2. *Thermal Properties of Materials.*—

Property	Concrete	Insulation	Gravel*	Source
Unit weight, ((lb/ft^3)	145	2	120	[47]
Percent moisture, W	5	0.25	15	[47]
Coefficient of thermal conductivity, K (Btu/ft-hr-°F)	1	0.021	2.1	[16]
Volumetric heat capacity, C (Btu/ft^3-°F)	30	0.54	34	[16]
Volumetric latent heat, L (Btu/ft^3)	1044	0	2592	[16]

*Typically, rigid plastic foam insulation is used.

b. *Analysis/Design.*—Using the multilayer solution for the modified Berggren equation, (1) frost depth through the concrete slab on gravel bedding, without insulation, is determined, and if excessive, (2) the thickness of insulation and gravel are determined. A simple spreadsheet was developed and used in the following example. Also, the width of insulation (from the centerline of the feature/area being protected) is equal to or greater than the frost depth estimate (e.g., if the frost depth is estimated to be 30 inches, the lateral insulation cover width would be at least 30 inches from the centerline of the feature/area being protected). Note: For larger diameter pipes, consideration should be given to extending the insulation from the edge of the pipe rather than the centerline of the pipe.

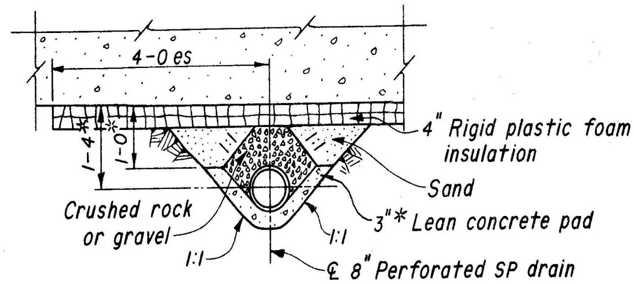


Figure 4.—Big Sandy Dam spillway, Wyoming—Typical longitudinal underdrain provisions on firm formation (filter and insulation requirements).

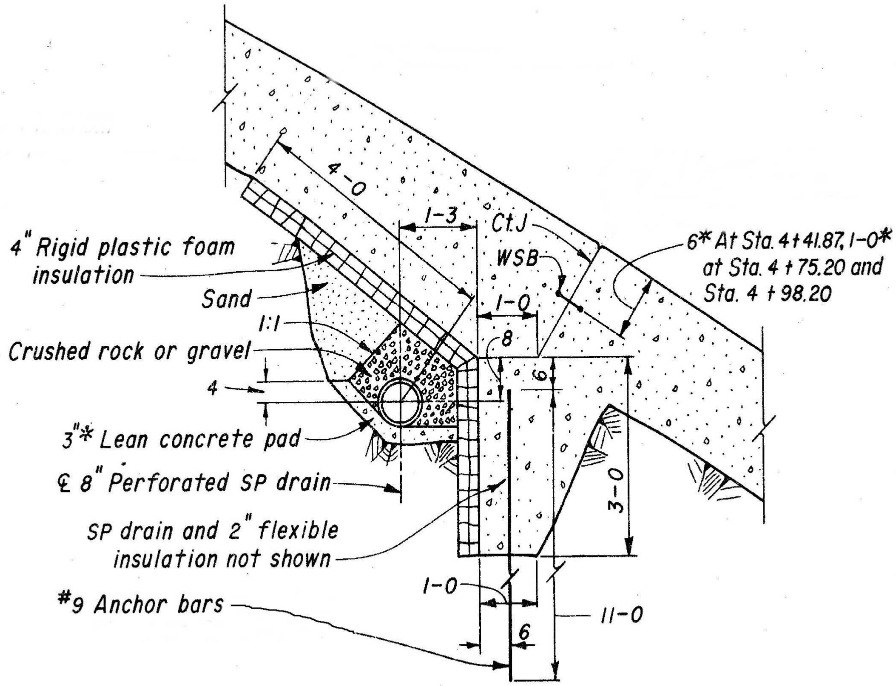


Figure 5.—Big Sandy Dam spillway, Wyoming—Typical lateral underdrain provisions on firm formation (filter and insulation requirements).

a). Estimating Frost Penetration Beneath Concrete Slab - Multilayer Solution using Modified Berggren Equation

Given:

Mean annual temperature (MAT):	36.3	oF
Air-freezing index (F):	2000	degree-days
Surface-freezing index (nF):	1800	degree-days
Length of Freezing season (Nov thru Apr):	160	days
Concrete slab thickness:	1.0	feet

Thermal properties of materials:

Properties	Concrete	Gravel
Unit weight, gamma (lbs/ft3):	145.0	120.0
Percent moisture, w (%):	5.00	15.00
Thermal conductivity, K (btu/ft-hr-oF):	1.000	2.100
Coefficient of volumetric heat capacity, C ((btu/ft3-oF):	30.00	34.00
Volumetric latent heat, L ((btu/ft3):	1044.0	2592.0

Surface freezing index (vs) = nF/(length of freezing season) = 1800/160 = 11.25 oF

Soil temperature difference (vo) = MAT - 32 = 36.3 - 32 = 4.30 oF

Thermal ratio (alpha) = vo/vs = 4.3/11.25 = 0.38

Procedure:

1. Assume thickness of gravel (dg).
2. Solve for sum(nF) and compare to given nF.
3. If sum(nF) equal to or slightly greater than given nF, frost penetration depth has been identified.
4. If sum(nF) is less than given nF, repeat steps 1 thru 3 until sum(nF) equals or slightly greater than given nF.

Assumed dg: **40.4 inches** **3.36 feet** **Given nF:** **1800 degree-days (given information which was determined from historic temperature data)**
Calculated nF: **1800.15 degree-days (calculated from column 22)**

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
Layer	**gamma** (lb/ft3)	**w** (%)	**d** (ft)	**sum(d)** (ft)	**C** (btu/ft3-oF)	**K** (btu/ft-hr-oF)	**L** (btu/ft3)	**Ld** (4) x (8) (btu/ft2)	**sum(Ld)** (btu/ft2)	**bar(L)** (10) / (5) (btu/ft3)	**Cd** (4) x (6) (btu/ft2-oF)	**sum(Cd)** (btu/ft2-oF)	**bar(C)** (13) / (5) (btu/ft3-oF)	**mu** vs[(14) / (11)]	**lambda** Figure 6	**lambda^2**	**Rn** (4) / (7) (ft2-ft-hr/btu)	**sum(Rn)** (ft2-ft-hr/btu)	**sum(Rn) +** (18)/2 + (19) (ft2-ft-hr/btu)	**nF** (20) [(9)/24(17)] (degree-days)	**sum(nF)** (degree-days)
Concrete	145.0	5.00	1.00	1.00	30.00	1.000	1044	1044.00	1044.00	1044.00	30.00	30.00	30.00	0.323	0.85	0.72	1.00	1.00	1.50	90.31	90.31
Gravel	120.0	15.00	3.36	4.36	34.00	2.100	2592	8715.60	9759.60	2237.16	114.33	144.33	33.08	0.166	0.85	0.72	1.60	2.60	3.40	1709.84	1800.15

Results:

Therefore, estimated frost depth is 40 inches below the 12-inch thick concrete slab (assuming a gravel sub-base) or a total frost depth of 52 inches from the concrete slab surface.
Since it would be impractical to place an underdrain system at this depth (to minimize freezing potential), insulation should be considered.

b). Estimating Insulation and Gravel Thicknesses for Underdrains - Multilayer Solution using Modified Berggren Equation

Given:

Mean annual temperature (MAT):	36.3	oF
Air-freezing index (F):	2000	degree-days
Surface-freezing index (nF):	1800	degree-days
Length of Freezing season (Nov thru Apr):	160	days
Concrete slab thickness:	1.0	feet

Thermal properties of materials:

Properties	Concrete	Insulation	Gravel
Unit weight, gamma (lbs/ft3):	145.0	2.0	120.0
Percent moisture, w (%):	5.00	0.25	15.00
Thermal conductivity, K (btu/ft-hr-oF):	1.000	0.021	2.100
Coefficient of volumetric heat capacity, C ((btu/ft3-oF):	30.00	0.54	34.00
Volumetric latent heat, L ((btu/ft3):	1044.0	0.0	2592.0

Surface freezing index (vs) = nF/(length of freezing season) = 1800/160 = 11.25 oF

Soil temperature difference (vo) = MAT - 32 = 36.3 - 32 = 4.30 oF

Thermal ratio (alpha) = vo/vs = 4.3/11.25 = 0.38

Procedure:

1. Assume thickness of insulation (di) and gravel (dg).
2. Solve for sum(nF) and compare to given nF.
3. If sum(nF) equal to or is greater than given nF, assumed thicknesses di and dg are OK (i.e., frost penetration would not extend through the gravel to the underdrain pipe).
4. If sum(nF) is less than given nF, repeat steps 1 thru 3 until sum(nF) equals or is greater than given nF (i.e., frost penetration could extend through gravel to the underdrain pipe).

Assumed di: 4.0 inches 0.33 feet Given nF: 1800 degree-days (given information which was determined from historic temperature data)
Assumed dg: 9.0 inches 0.75 feet Calculated nF: 1953.31 degree-days (calculated from column 22)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
Layer	**gamma** (lb/ft3)	**w** (%)	**d** (ft)	**sum(d)** (ft)	**C** (btu/ft3-oF)	**K** (btu/ft-hr-oF)	**L** (btu/ft3)	**Ld** (4) x (8) (btu/ft2)	**sum(Ld)** (btu/ft2)	**bar(L)** (10) / (5) (btu/ft3)	**Cd** (4) x (6) (btu/ft2-oF)	**sum(Cd)** (btu/ft2-oF)	**bar(C)** (13) / (5) (btu/ft3-oF)	**mu** vs[(14) / (11)]	**lambda** Figure 6	**lambda^2**	**Rn** (4) / (7) (ft2-ft-hr/btu)	**sum(Rn)** (ft2-ft-hr/btu)	**sum(Rn) +** (18)/2 + (19) (ft2-ft-hr/btu)	**nF** (20) [(9)/24(17)] (degree-days)	**sum(nF)** (degree-days)
Concrete	145.0	5.00	1.00	1.00	30.00	1.000	1044	1044.00	1044.00	1044.00	30.00	30.00	30.00	0.323	0.85	0.72	1.00	1.00	1.50	90.31	90.31
Insulation	2.0	0.25	0.33	1.33	0.54	0.021	0	0.00	1044.00	783.00	0.18	30.18	22.64	0.325	0.85	0.72	15.87	16.87	24.81	0.00	90.31
Gravel	120.0	0.25	0.75	2.08	34.00	2.100	2592	1944.00	2988.00	1434.24	25.50	55.68	26.73	0.210	0.87	0.76	0.36	17.23	17.41	1863.00	1953.31

Results:

Therefore, 4 inches of insulation and 9 inches of gravel placed beneath a 12-inch thick concrete slab will minimize the potential of freezing the underdrains.

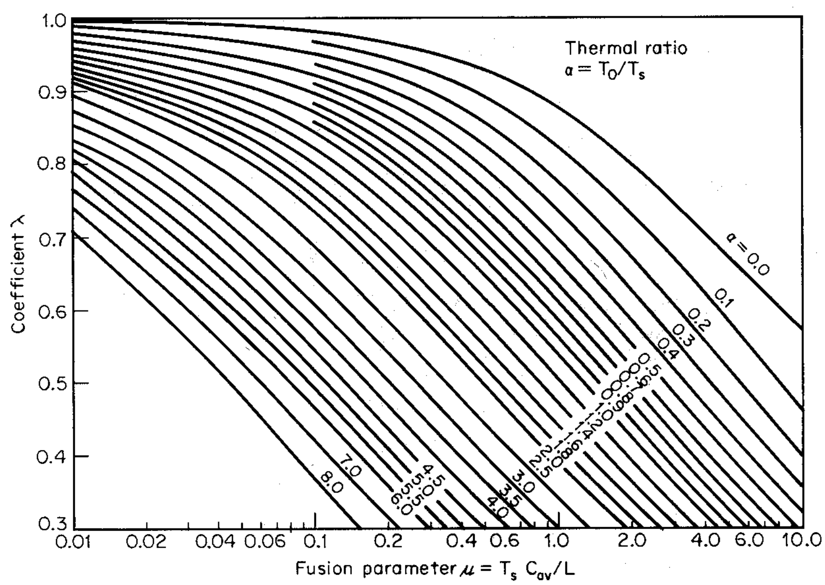


Figure 6.—Coefficient 8 for modified Berggren equation. T_0—difference between mean annual site temperature and 0 °C (32 °F); T_s—surface freezing or thawing index divided by length of freezing of thawing season; C_{av}—average volumetric heat capacity = $0.5(C_u + C_f)$; L—latent heat of fusion of soil.

c. *Results.*—Based on the assumed thermal properties for concrete, insulation, and gravel, and using a surface freezing index of 1,800 degree-days, 4 inches of insulation and 9 inches of gravel are required beneath a 12-inch thick concrete slab. The details of the procedure for estimating these thicknesses are described more fully on the following spread sheets.

d. *Conclusions (Final Layout).*—For frost penetration calculations, it is assumed that the effective thickness of gravel may be measured from the bottom of the insulation to the centerline of the drain pipe. (Note: For larger diameter pipes, consideration should be given to measuring the gravel layer thickness from the top of the pipe.) Therefore, for the 6-inch diameter collector pipe, the minimum effective gravel layer thickness would be 9 inches. For a 9-inch effective gravel thickness, a 4-inch insulation thickness is required. The width of insulation required depends on insulation depth, frost depth, and pipe diameter. The estimated frost penetration below the 1-foot thick concrete slab is 40 inches. As a rule-of-thumb, the insulation should prevent freezing of the pipe and therefore, the width of the insulation will be at least 80 inches (40 inches on either side of the centerline of the pipe). For constructability purposes, the actual width will be set at 96 inches (48 inches on either side of the centerline) to use full sheets of insulation (manufactured in 4-foot widths). Note: Until approximately 1990, Reclamation used sewer pipe for trench-type drainage provisions. Since then, PVC pipe has been almost exclusively used (stronger and more durable).

Example A4—Design Procedures to Mitigate Back Pressure in Drainage Provisions for Chutes and Hydraulic Jump Stilling Basins.—The following procedure was outlined in the published proceedings of the Hydraulic Engineering '93 Conference, in a paper titled *Pressure Relief Under Hydraulic Jump Stilling Basins*, by C.D. Smith and Zirong Gui [18]. See figure 7.

1. Develop tailwater curve(s) and/or table(s) for the range of discharges anticipated for the appurtenant hydraulic structure.

2. Determine the range of hydraulic jump profiles associated with the range of discharges anticipated for the appurtenant hydraulic structure.

3. Based on the location of the upstream extent of the hydraulic jump (i.e., where the hydraulic jump begins), for the range of discharges, a determination of instability (i.e., the potential for hydrostatic head beneath the floor slab being greater than the weight of the water jet and slab just upstream of the hydraulic jump) can be made. From this effort, a range of instability along the chute and stilling basin for the range of discharges can be developed.

4. Based on information from step 3, identify locations of "eductors" (i.e., aspirators), based on location where uplift approaches the weight of the concrete slab. They may be at multiple locations, from intersection of the chute and stilling basin to an upstream location on the chute which is at or just downstream of the beginning of the hydraulic jump profiles for minimum releases (which could result in instability). Whenever possible, specific locations will be at lateral collector drains, just upstream of the floor joints.

5. Suggested design features include:

    - One 50-mm (2-in) eductor for each 8 meters (26 ft) of lateral collector drain is recommended.

    - Vertical standpipes are located at each end of the lateral drain, which are usually embedded in the chute walls. Vertical standpipes serve two purposes: (1) acting as air vents to the lateral drain and eductors, whenever the pressure at the eductors drops below atmospheric pressure, which prevents suction of the drain provisions beneath the floor; and (2) acting as surge pipes when the eductors are subjected to high transient pressures, such as those that occur when the eductors are submerged.

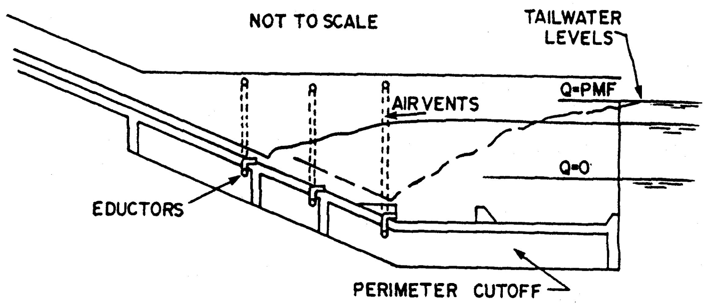


Figure 7.—Profile through chute and hydraulic jump stilling basin, illustrating locations of eductor, which mitigate high backpressure beneath the chute and stilling basin and subsequent damage or failure.

- The standpipe diameter is 100 mm (4 in) for chutes up to 40 meters (130 ft) wide; 150 mm (6 in) for chutes between 40 and 90 meters (130 and 295 feet) wide; and 200 mm (8 inches) for chutes wider than 90 m (295 ft).

- The horizontal portion of the eductor running from the underdrains to the chute floor should be galvanized steel and should be placed horizontally through the chute.

- As a guide, the crown of the eductor should extend at least $^1/_{15}$ of the jet flow depth above the floor surface (to induce a pressure drop when flow passes over the eductor).

- Provisions should be incorporated into the design to allow for inspecting and cleaning the drains.

Example A5—Toe Drain Pipe Sizing.—The procedure below can determine the diameter needed for a drain pipe to convey, for example, 3 ft^3/s of water:

a. *Manning's n.*—From the specifications and technical data for Advanced Drainage Systems, Inc. (ADS) for corrugated polyethylene pipe less than 15 inches in diameter, n is 0.018 for a corrugated interior, and 0.010 for a smooth wall interior [48].

b. *Slope.*—The minimum slope of the drain must be determined. In this case example, assume the minimum slope occurs in the area from the toe of the existing dam to the flow measurement boxes, which is 0.0106 feet/foot

c. *Pipe Diameter.*—Perform a trial calculation. Assuming a pipe diameter of 12 inches and substituting values into Manning's equation:

$$Q = \left(\frac{1.49}{n}\right) r_H^{2/3} \sqrt{s} A$$

where:

r_H is the hydraulic radius, defined as the wetted flow area divided by the wetted perimeter, $r_H = A/P$. For a 12-inch diameter pipe flowing ¾ full, $r_H = 0.317$ ft.

A = flow area = 0.6319 ft^2 for a 12-inch diameter pipe flowing ¾ full.

These values were obtained from tables in Reclamation's *Hydraulic Excavation Tables* [49].

Substituting values into Manning's equation makes it clear that a 12-inch diameter polyethylene pipe with a smooth wall interior will pass the required flow:

Corrugated interior:

$$Q = \left(\frac{1.49}{n}\right) r_H^{2/3} \sqrt{s} A = \left(\frac{1.49}{0.018}\right) 0.317^{2/3} \sqrt{0.0106}\ 0.6319 = 2.5\ \mathrm{ft}^3/\mathrm{s}$$

Smooth wall interior:

$$Q = \left(\frac{1.49}{n}\right) r_H^{2/3} \sqrt{s} A = \left(\frac{1.49}{0.010}\right) 0.317^{2/3} \sqrt{0.0106}\ 0.6319 = 4.5\ \mathrm{ft}^3/\mathrm{s}$$

Example A6—Filter Design.—

a. *Design Criteria*.—The design of the filter/drainage blanket zone should be in accordance with Reclamation's Design Standard No. 13, *Embankment Dams*, Chapter 5, "Protective Filters" [41]. Figure 8 shows a simplified version of the filter criteria for different base soils. The primary design requirement is to size the material to prevent migration and piping of materials, and to have sufficient permeability to permit seepage to escape freely and thus provide a high degree of control over seepage forces and hydrostatic pressures.

b. *Material Gradation*.—The materials to be protected (base soils) generally consist of the new or existing embankment materials and foundation materials. Gradation curves of the materials can be obtained from preconstruction borrow area investigations, construction quality control testing, and postconstruction sampling and testing. For this design example, it is assumed there is a fine grained material (core) against which a sand and gravel shell material is to be placed. The flow of water is from the core (fine grained) material into the shell (sand and gravels). Figure 9 shows the assumed gradations of the two types of materials.

c. *Check of Filtering Capability of Shell Against the Core*.—The base soil is the material which is to be protected from piping and in this case, is the core material. The base soil contains about 85 percent finer than the No. 200 sieve and, as shown on figure 8, is considered a category 1 soil, so the filter criterion is:

$D_{15}F$ # 9 x $D_{85}B$

where: $D_{15}F$ is the 15-percent size (15 percent is finer) of the filter material in mm
$D_{85}B$ is the 85-percent size (85 percent is finer) of the base material in mm, and is 0.074

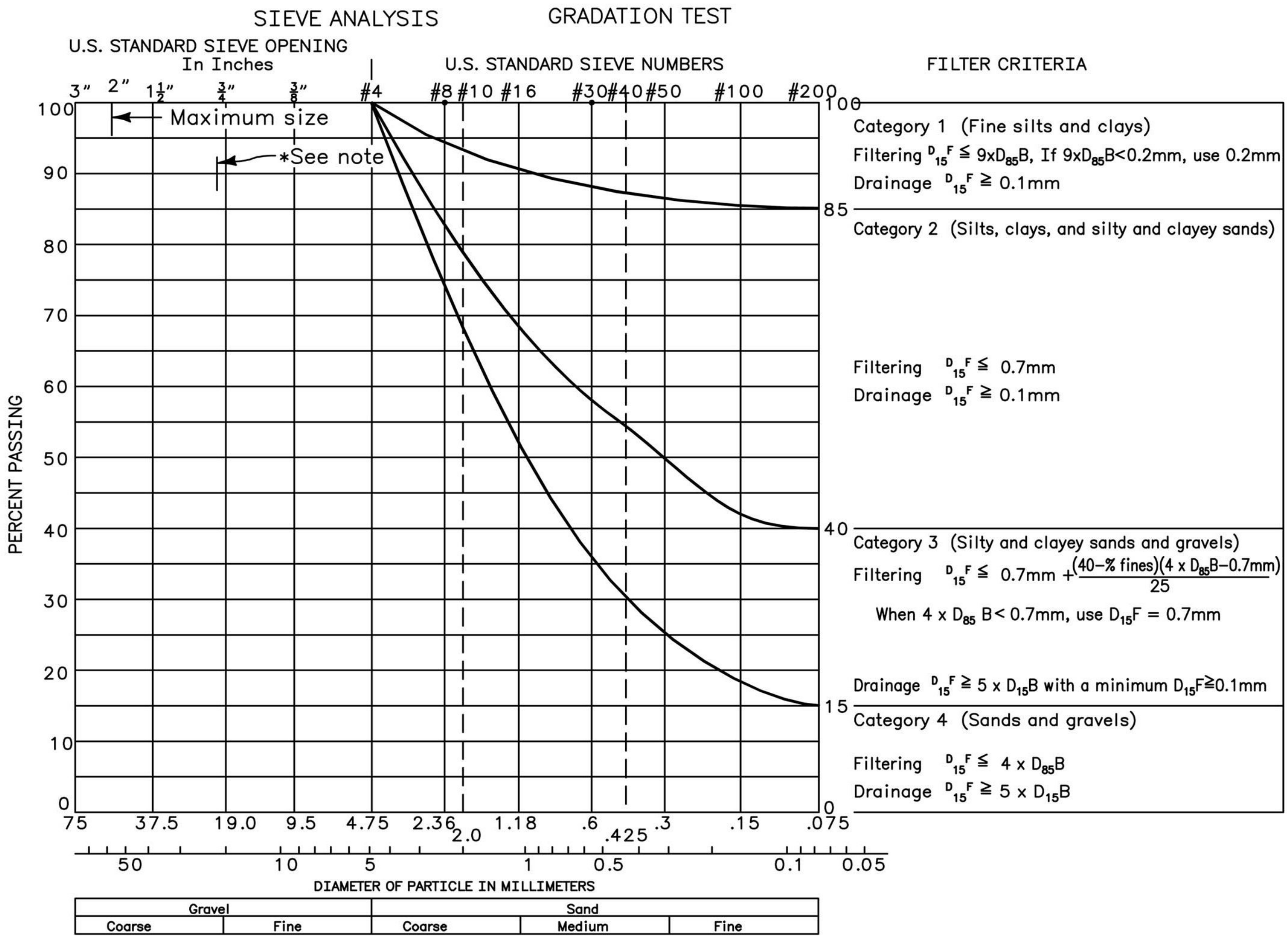
Therefore, $D_{15}F$ # 0.67 mm

D_{15} of the shell is 1.18 mm, which is greater than the maximum $D_{15}F$ to protect against piping; therefore, a filter is required.

d. *Filter Design*.—Again, the base soil contains about 85 percent fines and is considered a category 1 soil. The filter criteria are:

1. $D_{15}F$ # 9 x $D_{85}B$; therefore, $D_{15}F$ # 0.67 mm

2. To ensure sufficient permeability:



1. For all gap–graded and all internally unstable, broadly–graded soils, category designation 1, 2, 3, and 4 is determined from a gradation curve of the base soil which has been adjusted to 100 percent passing the No. 4 (4.75mm) sieve.
2. For all soils containing particles larger than the No. 4 (4.75mm) sieve, including gap–graded and internally unstable broadly–graded soils, category designation 1, 2, and 3, is determined from a gradation curve of the base soil which has been adjusted to 100 percent passing the No. 4 (4.75mm) sieve.
3. For all soils not gap–graded and not internally unstable broadly graded, category 4 designation is determined from a gradation curve of the natural grain–size distribution of the base soil.
4. Filter criteria is applied to the base soil gradation (natural or regraded) used to determined the category of the base soils.

* For $D_{90} > 20$ mm, the ratio of D_{90}/D_{10} is limited in accordance with the following table:

Min. D_{10}/F (mm)	Max D_{90}/F (mm)
<0.5	20
0.5-1.0	25
1.0-2.0	30
2.0-5.0	40
5.0-10	50
10-50	60

Figure 8.—Graphical representation of categories of base soils.

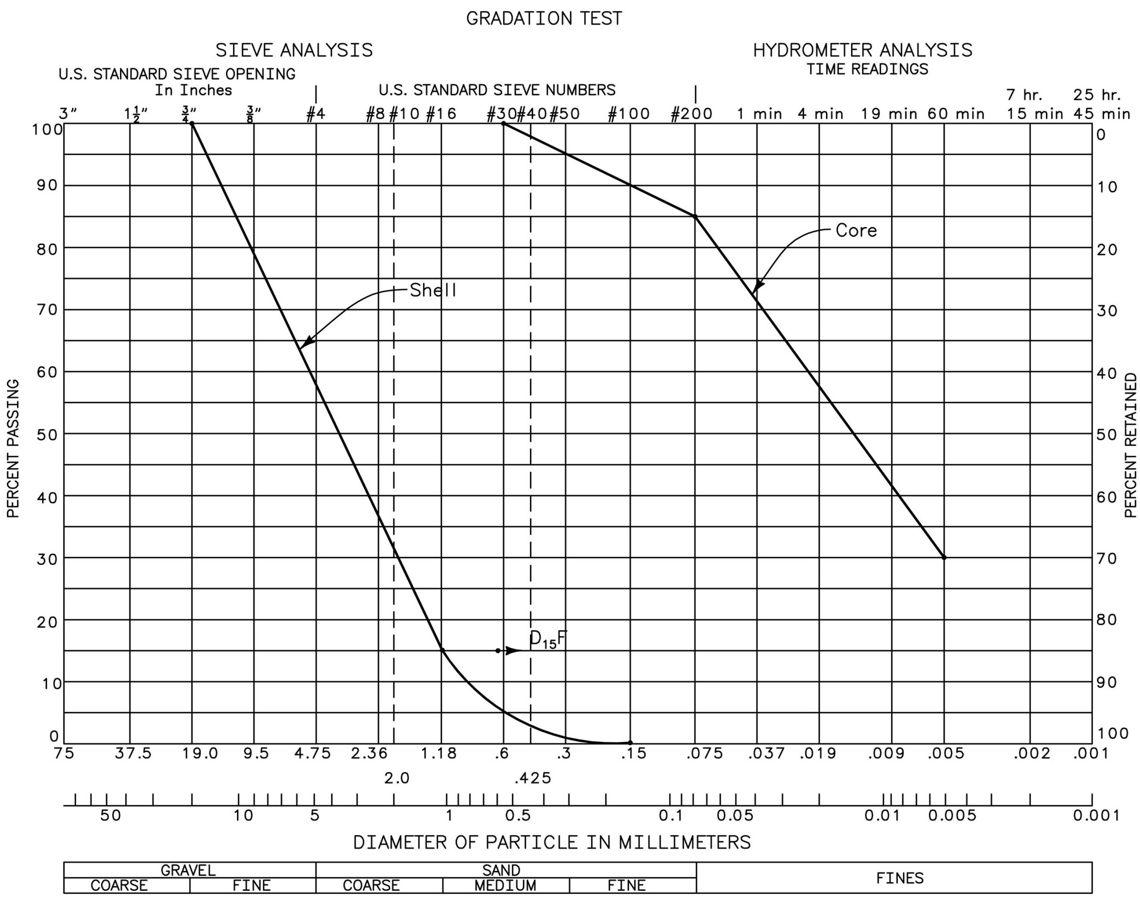


Figure 9.—Gradation of existing materials.

$D_{15}F$ \$ 5 x $D_{15}B$, but no less than 0.1 mm

Since $D_{15}B < 0.005$, $D_{15}F$ \$ 0.1 mm controls

See subsection h.2 below for further discussion.

3. Maximum particle size of 2 inches (50 mm) for the filter material

4. Maximum passing the No. 200 (0.074-mm) sieve of 5% for the filter material

5. $D_{10}F$ and $D_{90}F$ limits to prevent segregation:

Since $D_{10}F < 0.5$mm, then, $D_{90}F$ # 20 mm

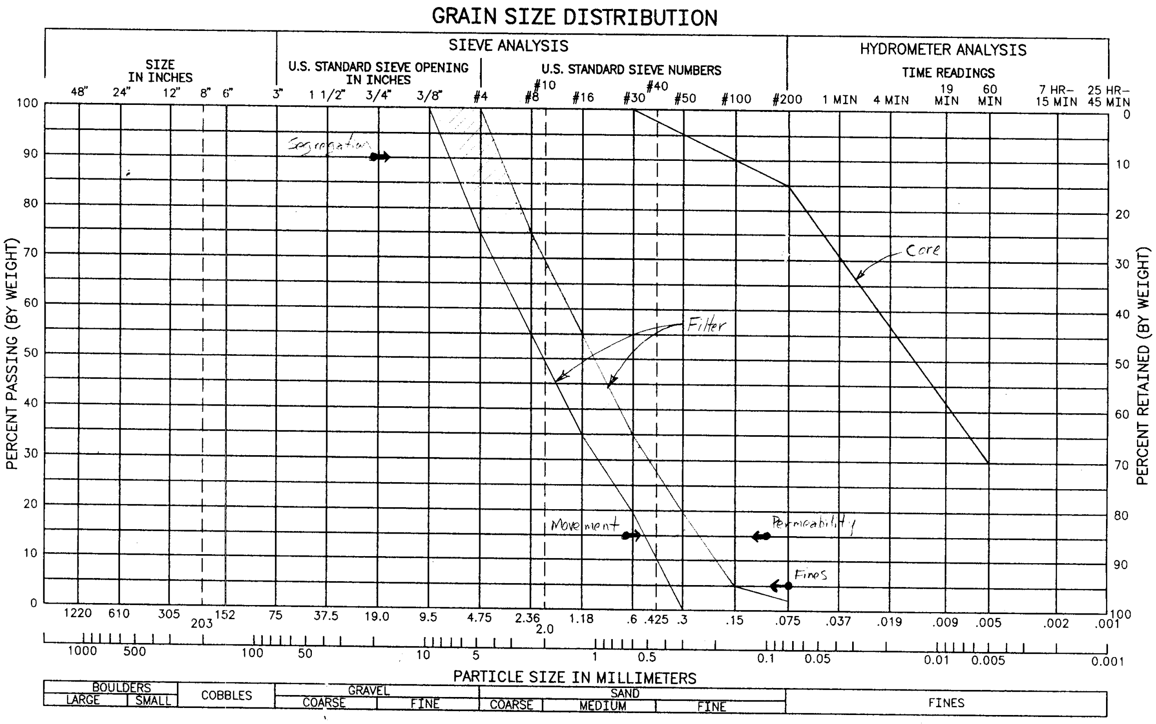


Figure 10.—Design of filter material for core.

The gradation limits of the filter zone are shown on figure 10. The gradation of the material used for the filter should be based on materials available from borrow sources near the site or a commercial source. Borrow source material should be evaluated to determine what is available and what can be economically screened to meet the filter requirements. If a small volume of material is required or processing of local materials will be difficult or expensive, commercial sources should be considered. If a commercial source is to be used for the material, a standard gradation that the supplier may already be producing for other uses may be considered to facilitate production and reduce the cost. However, filter design requirements should not be compromised just because of material availability.

e. *Check of Filtering Capability of Shell Against the Filter*.—A check also needs to be made to determine if the filter material has the potential to pipe into the shell. The filter material is now considered the base material. The new base material has a range of gradation, and the gradation that contains the largest amount of fines (the curve furthest to the right) should be used in the filter criteria evaluation. Since this gradation curve contains no gravels, the gradation curve does not require adjusting. The base soil contains less than 15 percent fines and is considered a category 4 soil. The filter criteria are:

1. $D_{15}F \leq 4 \times D_{85}B$

   where: $D_{15}F$ is the 15-percent size (15 percent is finer) of the filter material in mm
   $D_{85}B$ is the 85-percent size (85 percent is finer) of the base material in mm, and is approximately 3 mm

   Therefore, $D_{15}F \leq 12$ mm.

2. To ensure sufficient permeability:

   $D_{15}F \geq 5 \times D_{15}B$ but no less than 0.1 mm

   $D_{15}B$ is approximately 0.25 mm; therefore $D_{15}F \geq 1.3$ mm

3. As a check on segregation potential:

   Since $D_{10}F < 0.5$mm, then, $D_{90}F \leq 20$ mm

A second stage filter is not required, since the shell meets these filter requirements, as shown on figure 11. (Note that permeability is only marginal for the first base material but will obviously satisfy most of the gradation range).

f. *Check of Compatibility of Filter with Pipe Slot Sizes.*—Perforated pipe is often placed within the filter or drain material to collect and convey the seepage water. The material placed against the perforated pipe must be large enough to prevent washing of materials through the holes into the pipe, leading to a piping failure or causing the pipe to become clogged. The criterion used for sizing materials placed against perforated pipe is $D_{85} \geq 2 \times$ (slot size). Perforated pipe can be purchased with a standard circular hole size of ⅜-inch (9.52mm), or ⅛ inch (3.18 mm) by 2½-inch (64-mm) slots. For this example pipe, ⅜-inch perforations will be used; therefore:

$D_{85} \geq 2 \times 9.52 = 19$ mm

Since in this example, D_{85} of the filter material is about 3 to 7 mm, a second layer of material or an envelope of gravel material about the drain pipe is required.

g. *Gravel Envelope Gradation.*—

1. The filter material is a category 4 material and the filter criterion is:

   $D_{15}F \leq 4 \times D_{85}B$

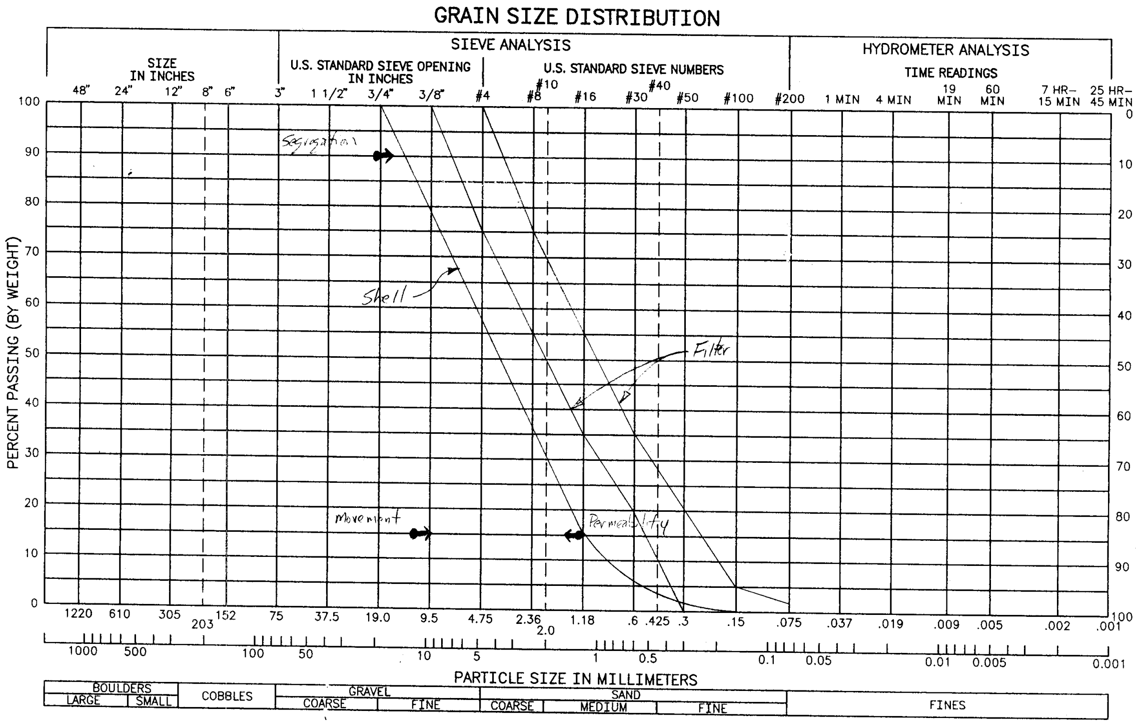


Figure 11.—Filter criteria check between filter material and shell.

where: $D_{15}F$ is the 15-percent size (15 percent is finer) of the gravel envelope material in mm
$D_{85}B$ is the 85-percent size (85 percent is finer) of the base (filter zone) material in mm, and is about 3

Therefore, $D_{15}F$ # 12 mm.

2. Permeability:

$D_{15}F$ $ 5 x $D_{15}B$ but no less than 0.1 mm

$D_{15}B$ < is about 0.25 mm; therefore $D_{15}F$ $ 1.3 mm

3. Check of gravel envelope against pipe slot size:

D_{85} $ 2 x (slot size) = 2 x 9.52 = 19 mm

D_{85} gravel envelope about 25 to 50 mm, which is > 19 mm

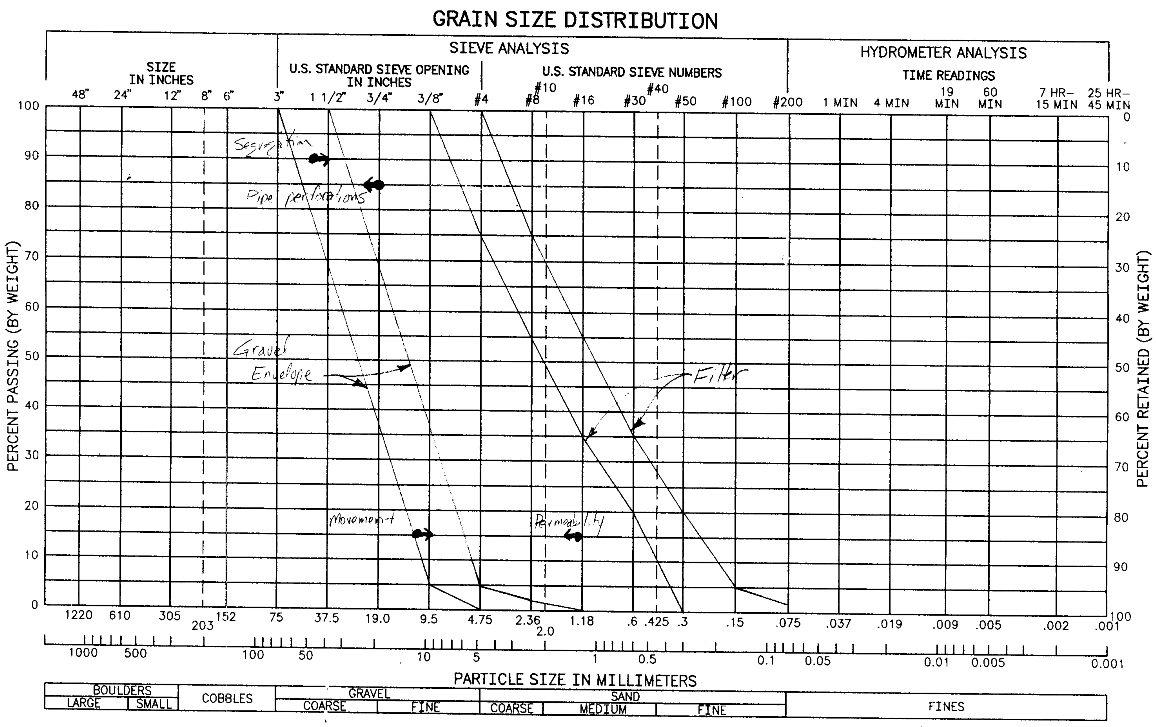


Figure 12.—Gradation of gravel envelope.

4. As a check on segregation potential:

   $D_{10}F$ will be between 5 to 10 mm, so $D_{90}F$< 50mm

The gradation limits of the gravel envelope are shown on figure 12. Note that all criteria could not be completely satisfied. However, the critical cases are the "movement potential" and "perforation size" for a gravel envelope. In this example, only the segregation criteria, which can be easily monitored and controlled, is not fully satisfied on the course end. (However the "average" gradation would meet the requirement.)

h. *Check of Permeability.*—

The permeability of the filter or drain needs to be checked to ensure that the filter or drain has the capacity to carry the expected flows from seepage and other sources, so the filter should have sufficient carrying capacity to handle any seepage emerging from the core.

1. *Permeability of Fine Grained Material*.—Field tests, laboratory tests, and seepage analyses can be used to estimate permeability of materials. For this example, it is assumed a permeability test was performed and the permeability of the core material was estimated at $K_h = 4.8 \text{ X } 10^{-6}$ cm/s and $K_v = 1.0 \text{ X } 10^{-6}$ cm/s.

2. *Filter Zone*.—The permeability of the filter zone can be estimated as:

$$K = 0.35(D_{15}F)^2$$

where:

K is the coefficient of permeability in cm/s

$D_{15}F$ is the 15-percent size (15 percent is finer) of the filter material in mm

For the finer side of the filter material, the permeability is approximately:

$$K = 0.35(D_{15}F)^2 = 0.35\ (0.25)^2 = 0.02 \text{ cm/s}$$

This permeability is much greater than the core material permeability.

The gradation for the filter and drain shown in this example is just one of many that could meet filtering to prevent piping. The preference is to use a gravel against the drain pipe and use the coarsest filter that meets filter criteria to reduce the potential for clogging and failing. Also, coarser filters are desirable for secondary lines of defense for plugging cracks that could develop from seismic shaking or other causes.

Example A7—Design of Foundation Drains for Concrete Dams.—Foundation drains are an important feature to reduce uplift pressures in concrete arch and gravity dam foundations. Without foundation drains, the assumed uplift distribution could vary linearly from full reservoir head at the upstream heel of the dam to tailwater head at the downstream toe of the dam, or even higher if adverse geologic conditions existed. When drains are provided, it is assumed that (1) full reservoir head exists at the upstream heel of the dam, (2) the uplift at the line of drains is reduced to a third of the difference between the reservoir head and the tailwater head, and (3) the uplift at the downstream toe of the dam is equal to the tailwater head, based on empirical evidence from uplift measurements at a number of Reclamation dams. Linear variations are assumed between these three points. Measurements of uplift along sections of existing concrete gravity dams have shown that uplift is typically less than what is assumed for design purposes. Drainage curtains for concrete dams have typically been designed using rules of thumb. However, this has resulted in drains that are too shallow for some high structures, and a more rigorous approach is proposed here.

In a landmark paper by Casagrande [4], equations are presented for designing and evaluating the performance of foundation drains in concrete gravity dams. Key parameters in the design of foundation drains are:

- the distance of the line of drains from the upstream face of the dam
- the horizontal spacing of drains
- the diameter of the drain holes
- the elevation of the drain outlet (typically dictated by the elevation of the foundation gallery floor)
- the location and the extent of permeable zones in the dam foundation
- the reservoir and tailwater elevations

The general approach and concepts provided in the Casagrande paper have been found to still be reasonable based on more recent studies. Goodman, Amadei, and Sitar [7] evaluated the effectiveness of drains for a crack along the foundation contact at the base of the dam, as opposed to the porous media assumption for the foundation in the Casagrande paper. Grenoble and Amadei [70] evaluated the interaction of drains with joints and other geologic features in the dam foundation. The conclusions of this study, based on a Monte Carlo simulation of two-dimensional joint networks, indicate that on the average, the porous

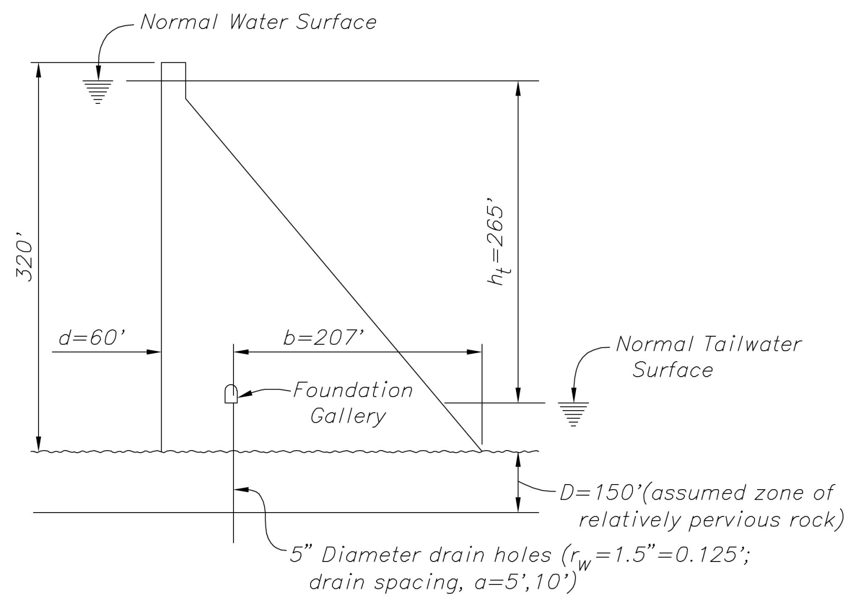


Figure 13.—Dimensions used in drain calculations.

media assumption is valid, but local geologic conditions or drains that are not deep enough can significantly affect the results.

Drain design using this method requires initial layouts, calculation of the results, and then selection of the best design, or performing further adjustments or checks. This design example will evaluate the effectiveness of foundation drains for a hypothetical dam. Figure 13 diagrams the dam and the details of the foundation gallery and drainage system. Foundation drains will be evaluated for two cases—a case where the gallery is below the tailwater elevation (this will require a sump and sump pump to discharge the drain flows), and a case where the gallery is above the tailwater elevation (this layout is less efficient in reducing uplift, but drain flows can be discharged by gravity to the downstream river channel). For each case, two drain spacings were evaluated—5 feet and 10 feet.

a. *Case 1—Foundation Gallery Below Tailwater Elevation.*—

1. *Drains at 10-foot spacings.*—The required water level in the drains (h_w, measured from the tailwater elevation to the discharge elevation of the drains, which is typically in a gutter in the foundation gallery, just below the floor elevation of the gallery), the piezometric level between drains (h_m, measured above tailwater elevation), and the flow to each drain (q_w) are calculated below. The gallery elevation is calculated to provide no flow downstream of the drains.

Water level in drains (below tailwater elevation)

$$\frac{h_w}{h_t} = -0.366\frac{a}{d}\log\frac{a}{2\pi r_w} \text{ (when } \frac{a}{d} < 3 \text{ and } \frac{r_w}{a} < 0.1)$$

$$\frac{h_w}{h_t} = -0.366\frac{10}{60}\log\frac{10}{2\pi(0.125)} = -0.067$$

h_w = - 0.067(265) = - 17.8 feet (below tailwater)

Piezometric surface halfway between drains (measured above tailwater)

$$\frac{h_m}{h_t} = 0.110\frac{a}{d} = 0.110\frac{10}{60} = 0.018$$

h_m = 0.018 (265) = 4.9 feet (above tailwater)

average h = -6.4 feet (below tailwater)

Rate of flow toward each drain

$$q_w = kh_t a\frac{D}{d}$$

assume permeability of granitic dam foundation (from examination of geology and results of packer permeability tests), k = 0.01 cm/s = 0.00033 ft/s

$$q_w = 0.00033(265)\ (10)\frac{150}{60} = 2.19 \text{ ft}^3/\text{s}$$

2. *Drains at 5-foot spacings.*—The required water level in the drains (h_w, measured from the tailwater elevation to the discharge elevation of the drains, which is typically in a gutter in the foundation gallery, just below the floor elevation of the gallery), the piezometric level between drains (h_m, measured above tailwater elevation), and the flow to each drain (q_w) are calculated below. The gallery elevation is calculated to achieve no seepage downstream of the drains.

Water level in drains (below tailwater elevation)

$$\frac{h_w}{h_t} = -0.366\frac{a}{d}\log\frac{a}{2\pi r_w} \quad \left(\text{when } \frac{a}{d} < 3 \text{ and } \frac{r_w}{a} < 0.1\right)$$

$$\frac{h_w}{h_t} = -0.366\frac{5}{60}\log\frac{5}{2\pi(0.125)} = -0.024$$

h_w = - 0.024 (265) = - 6.5 feet (below tailwater)

Piezometric surface halfway between drains (measured above tailwater)

$$\frac{h_m}{h_t} = 0.110\frac{a}{d} = 0.110\frac{5}{60} = 0.0092$$

h_m = 0.0092 (265) = 2.4 feet (above tailwater)

average h = - 2.0 feet (below tailwater)

Rate of flow toward each drain

$$q_w = kh_t a\frac{D}{d}$$

assume permeability of granitic dam foundation (from examination of geology and results of packer permeability tests), k = 0.01 cm/s = 0.00033 ft/s

$$q_w = 0.00033(265)\ (5)\frac{150}{60} = 1.09 \text{ ft}^3/\text{s}$$

In addition to evaluating drains at 5- and 10-foot spacings, drains at 20-foot centers were also evaluated. The results for all three cases are summarized in table 1 below.

Table 1.—Summary of results for gallery located below tailwater elevation (no flow past line of drains)

	Drain spacing		
	5 feet	10 feet	20 feet
Water level in drains, ft*	-6.5	-17.8	-45.3
Piezometric surface halfway between drains, ft*	2.4	4.9	9.7
Average piezometric surface at line of drains, ft*	-2.0	-6.4	-17.8
Flow towards each drain, ft^3/s	1.09	2.19	4.37

* measured from tailwater elevation (negative = below tailwater)

As can be seen in table 1, the water level in the drains (which dictates approximately where the floor of the gallery would be located), is affected significantly by the spacing of the drains. The assumption for this case is that no flow occurs downstream of the line of drains. For the 20-foot drain spacing, significant drawdown is required at the drains to satisfy this requirement. The average piezometric surface, which relates to the average uplift at the base of the dam, decreases with increasing drain spacing. The flow toward each drain is directly related to the drain spacing. The same total flow is captured in all three cases. As the spacing increases, and the number of drains decreases, the flow toward a single drain increases proportionally. While the drain spacing at 20 feet appears to have some benefit over the 5- and 10-foot spacings, the 20-foot spacing would require significantly more pumping effort to lift the drain flows from the gallery floor elevation up to the tailwater elevation in the channel downstream of the dam.

b. *Case 2—Foundation Gallery Above Tailwater Elevation*.—For this case, the uplift at the line of wells (u_w), the total flow through the foundation (q), the flow passing through the wells (q_t), and the ratio of flow passing through the wells to the total flow through the foundation (q_t/q) are calculated below. As part of the calculations, an auxiliary parameter, h_c, is also calculated. The drains are assumed to have a water level 5 feet above the actual tailwater elevation () h_w = 5 feet). The water level in the drains corresponds to approximately the invert elevation of the foundation gallery. The parameter h_c represents the elevation difference between the reservoir elevation and a fictitious tailwater elevation. The fictitious tailwater elevation represents the tailwater for which there is no flow downstream of the drains and such that with a constant gradient, i_t, the new tailwater elevation and the water level in the wells will coincide.

1. *Drains at 10-foot spacings.—*

Auxiliary quantity, h_c

$$h_c = \frac{h_t - \Delta h_w \dfrac{b+d}{b}}{1 + \dfrac{1}{2\pi} * \dfrac{a}{d} * \dfrac{b+d}{b} * \ln \dfrac{a}{2\pi r_w}}$$

$$h_c = \frac{265 - (5)\dfrac{(207+60)}{207}}{1 + \dfrac{1}{2\pi} * \dfrac{10}{60} * \dfrac{267}{207} * \ln \dfrac{10}{2\pi(0.125)}} = 237.8 \text{ feet}$$

Hydraulic gradients i_t (downstream of drain) and i_w (upstream of drain)

$$i_t = \frac{h_t - h_c}{b+d}$$

$$i_t = \frac{265 - 237.8}{267} = 0.102$$

$$i_w = \frac{h_c}{d} + i_t$$

$$i_w = \frac{237.8}{60} + 0.102 = 4.06$$

Uplift at line of wells

$$u_w = b * i_t + \Delta h_w$$

The formula shown above is the formula that appears in Casagrande's paper [4]. It appears that the last term ( ) h_w) is extraneous and should not be included in the equation. Calculations were made without adding this term.

$u_w = 207*0.102 = 21.1$ feet

Seepage: assume packer tests for granitic dam foundation yield an average permeability, k = 0.01 cm/s = 0.00033 ft/s

Total flow: $q = kDi_w = 0.00033(150)(4.06) = 0.20\ \text{ft}^3/\text{s}$

Flow passing wells: $q_t = kDi_t = 0.00033(150)(0.102) = 0.005\ \text{ft}^3/\text{s}$

Ratio passing wells: $\frac{q_t}{q} = \frac{i_t}{i_w} = \frac{0.005}{0.20} = 2.5\%$

2. *Drains at 5-foot spacings.—*

Auxiliary quantity, h_c

$$h_c = \frac{h_t - \Delta h_w \frac{b+d}{b}}{1 + \frac{1}{2\pi} * \frac{a}{d} * \frac{b+d}{b} * \ln \frac{a}{2\pi r_w}}$$

$$h_c = \frac{265 - (5)\frac{(207+60)}{207}}{1 + \frac{1}{2\pi} * \frac{5}{60} * \frac{267}{207} * \ln \frac{5}{2\pi(0.125)}} = 251.0 \text{ feet}$$

Hydraulic gradients i_t *(downstream of drain) and* i_w *(upstream of drain)*

$$i_t = \frac{h_t - h_c}{b+d}$$

$$i_t = \frac{265 - 251.0}{267} = 0.052$$

$$i_w = \frac{h_c}{d} + i_t$$

$$i_w = \frac{251.0}{60} + 0.052 = 4.24$$

Uplift at line of wells

$$u_w = b * i_t + \Delta h_w$$

$$u_w = 207*0.052 = 10.8 \text{ feet}$$

The formula shown above is the formula that appears in Casagrande's paper [4]. It appears that the last term (Δh_w) is extraneous and should not be included in the equation. Calculations were made without adding this term.

Seepage: assume packer tests for granitic dam foundation yield an average permeability, k = 0.01 cm/s = 0.00033 ft/s

Total flow: $q = kDi_w = 0.00033(150)(4.24) = 0.21 \text{ ft}^3/\text{s}$

Flow passing wells: $q_t = kDi_t = 0.00033(150)(0.052) = 0.003 \text{ ft}^3/\text{s}$

Ratio passing wells: $\frac{q_t}{q} = \frac{i_t}{i_w} = \frac{0.003}{0.21} = 1.4\%$

In addition to evaluating drains at 5- and 10-foot spacings, drains at 20-foot centers were also evaluated. The results for all three cases are summarized in table 2 below.

As can bee seen from table 2, doubling the drain spacing, results in almost doubling the uplift at the line of wells. While the net increase in uplift going from drains at 5-foot spacing to drains at 10-foot spacing is about 10 feet, the net increase is about 30 feet when going from 5-foot to 20-foot spacing. The amount of flow captured by the drains is similar for all three cases.

Table 2.—Summary of results for gallery located above tailwater elevation

	Drain spacing		
	5 feet	10 feet	20 feet
Uplift at line of wells, ft*	10.8	21.1	41.4
Total flow, ft^3/s	0.21	0.20	0.18
Flow passing wells, ft^3/s	0.003	0.005	0.010
Ratio passing wells, percent	1.4	2.5	5.4

* measured from tailwater elevation (negative = below tailwater)

When evaluating the results for drains at 5-, 10-, and 20-foot spacings, it appears that the 10-foot spacing (which is the typical spacing provided for concrete dam foundation drains), is a good choice. For the case of the gallery below tailwater, the 20-foot spacing provides the most reduction in uplift but would result in higher pumping costs to discharge the drain flows back into the river channel downstream of the dam. Drains at 10-foot spacing would require some additional pumping costs as well, but this would be offset by reduced costs for drilling the drain holes (half as many drains are required at 10-foot spacing). For the case of the gallery above tailwater, drains at 20-foot spacing result in the highest uplift pressures. Drains spaced at 10 feet would have a good balance between installation cost and reduction in uplift pressures. A refined analysis, considering construction costs, operation and maintenance costs and impact of drain spacing and uplift pressures on dam and foundation stability could be used to more accurately determine the optimum spacing of the foundation drains.

When comparing the first case (foundation gallery below tailwater elevation) to the second case (foundation gallery above tailwater elevation), for drains at 10-foot spacings, the first case results in lower uplift pressures along the dam foundation. The difference at the line of drains is about 32 feet for drains at 10-foot spacing, for a structure over 300 feet high. Sliding stability analyses will indicate whether this is significant. However, if there is significant cohesive bond or intact rock along sliding surfaces, this is not likely to be a significant difference, and designing the gallery to be free-draining and will save on operations and maintenance costs.

While an initial layout of foundation drains should generally provide good reduction of uplift, flow through jointed rock is not always predictable, and ultimately the performance of the drains (as reflected by drain flows and uplift pressures) will provide the best indication of drain adequacy. Based on the monitored performance, additional drains may need to be installed.

Another important consideration for drain design and layout is the geology in the dam foundation. Fault zones or other impervious features in a dam foundation can greatly affect the ability of drains to control uplift pressures. It is important that drains are deep enough (40 percent of the dam height is recommended as a general rule) to intersect the critical discontinuities in the foundation as well as penetrating any impervious barriers in the foundation that might control uplift pressures. Casagrande's paper [2] provides a number of examples of geologic features that can influence and control uplift pressures in a dam foundation.

Example A8—Gravity Dam Analysis Showing the Effects of Functional and Nonfunctional Drainage.—The analysis of a gravity dam is based on a simple free-body diagram where the resultant force is determined from summing the moments of the dam weight, hydrostatic load and uplift load. Safety factors are the ratio of the resisting forces to the driving forces. The two scenarios illustrate the effect on stability at the foundation contact for a gravity dam with nonfunctional foundation drains (fig. 14) and functional foundation drains (fig. 15). Nonfunctional drains increase the uplift load in a dam by about 60 percent, assuming the drains are located 10 percent of the base width from the upstream face and have a drain efficiency of b (the drains, when functioning, reduce the uplift pressure at the drains to a of the differential between full reservoir head and tailwater head). At the same time, nonfunctional drains reduce stability in two ways: first, the resisting force is reduced by the amount of additional uplift, and second, the potential for tensile stress, and subsequent cracking, to develop on the upstream face is increased.

Factor of safety:

$$FS = \frac{cA + (N - U)\tan\theta}{P}$$

where:

c = cohesion
A = area
N = normal force
U = uplift pressure
θ = friction angle
P = pressure force

Without drains:

$$U = \gamma H\left(\frac{B}{2}\right) + \gamma TB$$

FS = 1.1, assuming 0 cohesion and 40° friction angle

With drains:

$$U = \gamma\left(\frac{B}{2}\right) + \lambda\left(\frac{H}{3}\right)\left(\frac{B-b}{2}\right) + \lambda TB$$

where:

γ = density of water
H = height of water
B = base width
T = height of tailwater
b = distance to drains

FS = 1.5, assuming 0 cohesion and 40° friction angle

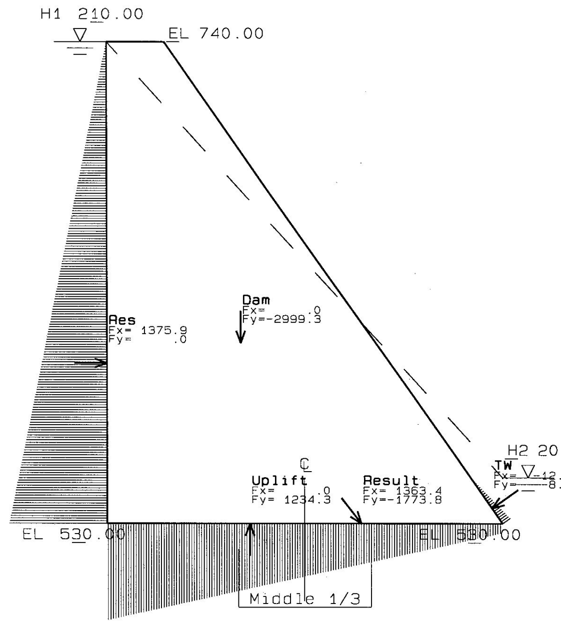


Figure 14.—Loads applied in an analysis of a typical gravity dam section with no foundation drainage.

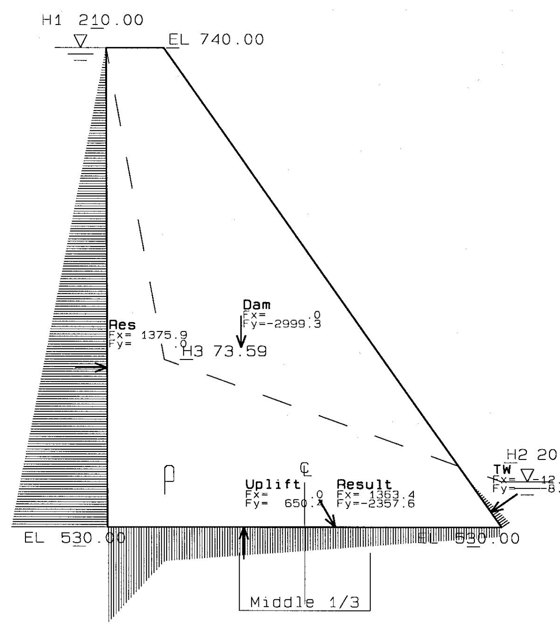


Figure 15.—Loads applied in an analysis of a typical gravity dam section with foundation drainage.

Example A9—Foundation Stability Analysis Resulting in Installation of Drainage Provisions.—Horse Mesa Dam, a feature of the Salt River Project located east of Phoenix, Arizona, is a 305 ft high concrete thin arch dam as shown in plan view in figure 16. The following is a summary of the foundation analysis that prompted modifications to the right abutment of the dam. This description is limited to a single static load case for one foundation block on the right abutment; the complete analysis, which includes a description of the dynamic (earthquake) analysis, is documented in Technical Memorandum No. HM-3620-1.

a. *Abutment Geology*.—The rock that forms the abutments at Horse Mesa Dam consist of dacite porphyry that exhibits continuous joints parallel to flow boundaries. The right abutment bedrock has a distinct stepped structure resulting from the interaction of flow joints which dip moderately toward the river, and high-angle joints. A number of predominant low-angle flow features are present on both abutments. Due to the orientation and obvious continuity of the predominant joints on both abutments, the adequacy of the abutments to resist loading was of concern. Individual joints were mapped from which wedges were defined on the abutments.

b. *Abutment Stability Procedure*.—The stability assessment procedure consists of the following steps:

- Use available discontinuity data to identify the critical foundation blocks.
- Use computer program SAPLOD-M (with output tapes generated by the SAPIV finite element program) to determine for each identified block the external dam load, which is the resultant of the combined influence of gravity, reservoir, and temperature loadings.
- Determine the weight of the critical rock blocks.
- Determine the weight of the spillway structure acting on the critical rock mass (the spillway structures were not included as part of the finite element model).
- Determine the hydrostatic load acting on the spillway structures.
- Develop a differential-head contour map for the damsite based on piezometer and surface seepage data.
- Estimate the hydraulic loads acting on the planes bounding the critical rock mass.
- Estimate the shear strength of the critical discontinuities.

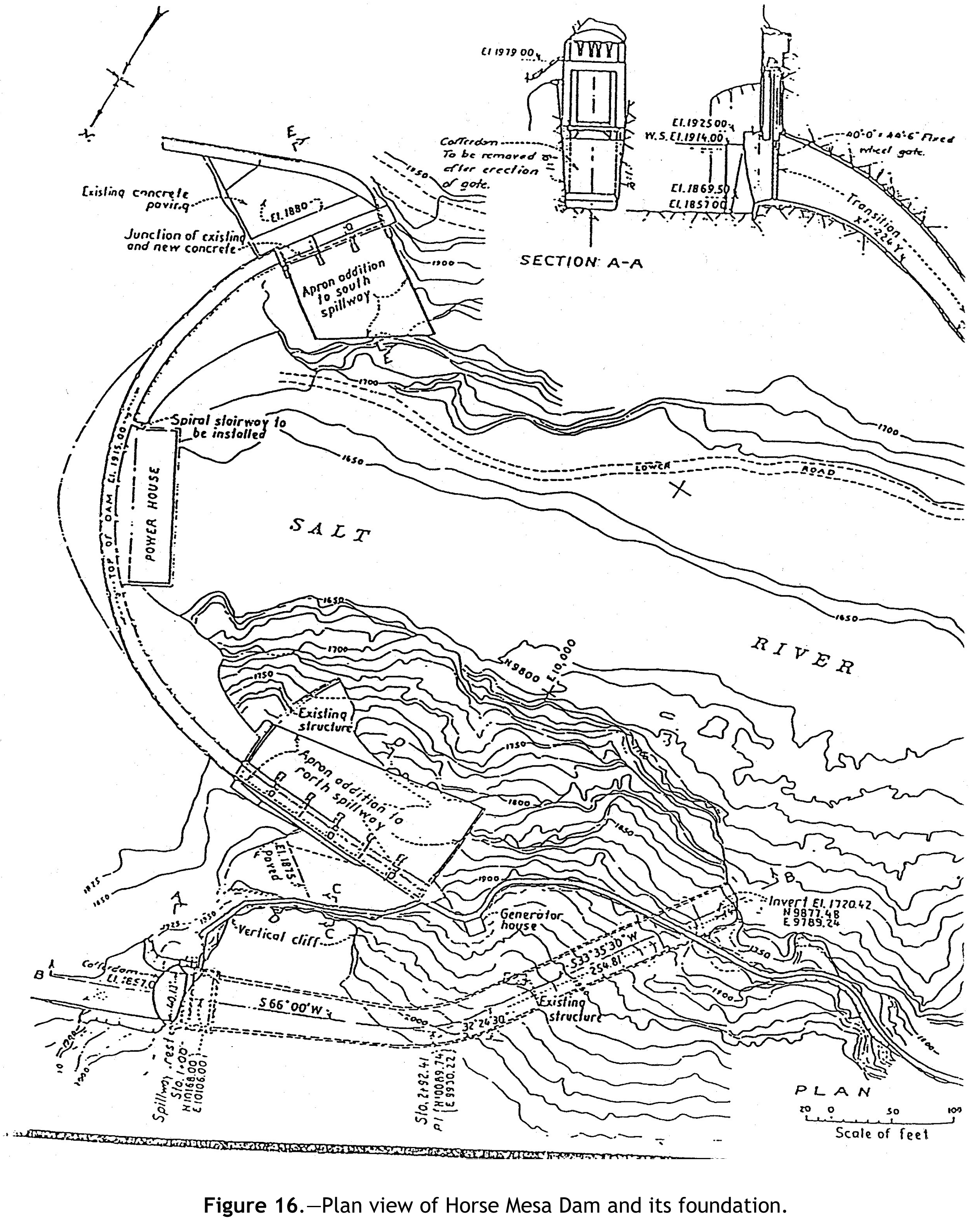


Figure 16.—Plan view of Horse Mesa Dam and its foundation.

- Use computer program RIGID to resolve all forces acting on the critical rock mass into components parallel and normal to planes used to define the critical rock mass, and determine the static factor of safety against sliding.

Due to observable continuity of a number of geologic features in the abutment, a request was made for orientation data for specific joints. Joint orientations obtained along continuous fractures exposed during the 1993 spillway releases represent realistic critical potential foundation wedge/joint combinations. This information was used to define a deterministic critical foundation blocks rather than a statistical representation if joint surveys were used. Wedge planes were defined according to their location and function as:

- Base sliding plane (sub-horizontal)
- Side sliding plane (moderately dipping)
- Release plane (sub-vertical at upstream boundary)

c. *External Dam Load.*—The external dam load is made up of three components:

- Gravity load
- Temperature load
- Reservoir water load

The total external dam force was determined by summing the static loads associated with gravity, normal reservoir, and high temperature loading.

To determine the X, Y, and Z components of the external dam load, computer program SAPLOD-M was used in conjunction with output files generated during Finite Element Model (FEM) studies of the dam. Prior to execution of SAPLOD-M, the wedge for the right abutment was superimposed onto the foundation contact of the FEM, and all dam elements having either a face, an edge, or a node in direct contact with the wedge were identified as load contributors. All foundation nodes within the wedge boundary contribute load to the right abutment wedge.

Once the elements and nodes contributing load to the wedge were identified, a SAPLOD-M run was made for each of the individual loads. Table 3 is a summary of the individual load force components acting on the wedge.

The resulting external dam force components are summarized in table 4 and the resultant force and orientation is summarized in table 5.

Table 3.—Individual force components.

Abutment	Load	Force components X (lb)	Y (lb)	Z (lb)
Right	Gravity (odd cantilevers in contact with wedge)	5,281,000	-8,140,000	80,350,000
	Gravity(even cantilevers in contact with wedge)	226	-288	8369
	Normal reservoir (R.S.=1915)	-67,000,000	247,000,000	25,940,000
	High temperature	-43,200,000	36,460,000	-55,650,000

Table 4.—External dam force components.

Abutment	Load case	Force components X (lb)	Y (lb)	Z (lb)
Right	1	-104918774	275,419,712	50,648,369

NOTE: The positive X-axis is directed into the left abutment and has a bearing of S30.5E, the positive Y-axis is directed downstream and has a bearing of S59.5W, the positive Z-axis is directed downward.

Table 5.—Magnitude and orientation of the external dam force.

Abutment	Load case	Magnitude(lb)	Bearing	Smallest angle with respect to X-axis (degrees)	Plunge (degrees)
Right	1	299,047,194	S80.5W	69	10

d. *Wedge Weight*.—To determine the wedge weight, sections oriented parallel to the direction of sliding were cut at a spacing of 20 feet. Cross sectional areas were determined and the average end area method was used to estimate wedge weight assuming a unit density of 155 lb/ft^3 for the dacite porphyry. The weight and mass of the wedge analyzed is given in table 6.

Table 6.—Wedge weight and mass.

Abutment	Weight (lb)	Mass (lb-s^2/in)
Right	241,200,000	624,224

e. *Spillway Structure Loads*.—The spillway structure was not included as part of the finite element model, therefore, it was necessary to estimate the weight, magnitude, and direction of the water force acting on it. The reservoir water force on the spillway structure acts normal to the orientation of the spillway crest. The magnitude of this horizontal force was determined for a reservoir elevation of 1915 feet. This water force was resolved into X and Y components. Table 7 gives the X, Y, and Z force components associated with the spillway structures.

Table 7.—Spillway structure loads.

Spillway structure	Reservoir elevation (ft)	Spillway force components		
		X (lb)	Y (lb)	Z (lb)
Right	1915	12,352,020	7,869,104	70,000,000

The spillway force components were added to the corresponding external dam force components. The resulting composite force, referred to here as the final external dam force, was applied to the potential wedge during abutment stability analyses. The final external dam forces used for stability analysis is summarized in table 8.

Table 8.—Final external dam force components.

Abutment	Load case	Force components*		
		X (lb)	Y (lb)	Z (lb)
Right	1	-92600000	283,300,000	120,600,000

* Force components have been rounded off to the nearest 100,000 pounds.

f. *Foundation Seepage Hydraulic Loads*.—A differential head contour map was developed for the damsite based on two types of data:

- Foundation uplift measurements taken following construction of the dam
- Recent surface seepage survey

Uplift measurements were taken with the reservoir between elevations 1909 and 1913, and the tailwater between elevations 1652 and 1649, respectively. Pressure measurement holes were angled upstream at angles varying between 45N and 75N with respect to horizontal. Eight surface seeps were located on each abutment.

The percent differential head was computed based on the measured water pressure. Surface seep elevation, drill hole data, and reservoir and tailwater elevations were used to estimate the percent differential head as follows:

1. Assume pressure head equals zero at surface seep location

2. Determine percent differential head using the following relationship:

$$\begin{matrix} \%\mathit{DIFFERENTIAL} \\ \mathit{HEAD} \end{matrix} = \left[\frac{(\mathit{SEEP\ EL.} - \mathit{TAILWATER\ EL.})}{(\mathit{RESERVOIR\ EL.} - \mathit{TAILWATER\ EL.})} \right] \times 100$$

The following assumptions were made prior to development of this water pressure contour map:

- The upstream edge of the foundation contact is taken as the 100 percent differential head boundary condition.

- The normal tailwater contour (elevation 1648) is taken as the 0 percent differential head boundary condition.

- Surface seeps are true indicators of the groundwater surface (if a piezometer was installed at a surface seep it would indicate a water level at the ground surface).

- Uplift pressure measurements made following dam construction represent water pressure in the bedrock at midpoint between the concrete/rock contact and the bottom of the hole.

When drawing pressure contours, an effort was made to ensure that they do not daylight except at known surface-seep locations. The differential head contours were used in conjunction with the wedge plane contours to estimate uplift forces acting on the abutment wedges. The procedure used to estimate uplift force acting on the base, side, and release planes of the right abutment wedge is outlined below:

1. Using a plan view of the abutment, each wedge plane was divided up into smaller triangles such that the vertices of each triangle are located on differential head contours.

2. Wedge plane elevation was determined at each triangle vertex using the wedge plane contours.

3. An average pressure head was determined for each triangle using information from steps 1 and 2, the design reservoir water elevation, and the design tailwater elevation. The following equation was used to determine the average pressure head for each triangle:

$$PH_{AVG} = \frac{\left[\sum_{i=1}^{3} \left[\left(RWE - TWE \right) DHP_i + TWE \right] - WPE_I \right]}{3}$$

where:

PH_{AVG} = Average Pressure Head Acting On Individual Triangle
TWE = Design Tailwater Elevation
WPE = Wedge Plane Elevation at Triangle Vertex
RWE = Design Reservoir Water Elevation
DHP = Differential Head Percentage at Triangle Vertex

4. The uplift force on each wedge plane triangle was calculated using the following equation:

$$F = \frac{PH_{AVG} \left[\frac{(B)(H)}{2} \right] \gamma_w}{COS\ DIP}$$

where:

F = Uplift Force Acting on Individual Triangle
PH_{AVG} = Average Pressure Head Acting on Individual Triangle

B = Base Dimension of Individual Triangle (plan view)
H = Height Dimension of Individual Triangle (plan view)
DIP = Dip of Wedge Plane
γ_w = Unit Weight of Water

5. Forces computed for individual wedge plane triangles were summed resulting in a total wedge plane uplift force.

Table 9 summarizes the uplift forces used for the stability analyses.

Table 9.—Wedge plane uplift forces.

Abutment	Wedge plane	Uplift force (lb) Res. El. = 1915
	Base	77,900,000
Right	Side	69,800,000
	Release	94,300,000

NOTE: A tailwater elevation of 1648 was used with the reservoir elevation. Uplift forces have been rounded off to the nearest 100,000 pounds.

g. *Shear Strength of Critical Discontinuities.*—The "Q-system" of rock mass classification developed by Barton in which the tangent of the friction angle is defined as the ratio of joint roughness number (J_r) to joint alteration number (J_a), or tan $\phi = J_r/J_a$, was used to estimate shear strength of the critical discontinuities.

Assuming rock wall contact along the joints, and taking into consideration joint information included in the MDA geology report, estimates for J_r vary between 1.0 and 3.0 and estimates for J_a vary between 1.0 and 2.0.

Table 10 summarizes estimates for J_r and J_a of specific geologic discontinuities as provided by project field geologists.

Though values of the friction angle could be as high as 63° for the right abutment wedge side plane, a value of 50° was used as a more conservative value. A friction angle of 45° was used for the flow-joint base planes and a friction angle of 50° was used for the release planes. Cohesion on all wedge planes was assumed to be zero for the following reasons:

- Cohesive shear strength is attributed to: shearing through steep asperities; a dilatancy effect; and the presence of rock bridges. A quantitative assessment of the rock bridge effect is very difficult.

- It is not prudent to rely on strength that cannot be determined with a high degree of confidence, especially when evaluating a large dam's ability to withstand earthquake loads.

Table 10.—Discontinuity shear strength

Abutment	Feature	J_r	J_a	n(°)	Wedge plane
Right	C1 joints	2.0	1.0	63	Side
	C2 joints	2.0	1.0	63	
	D1 joints	1.25	1.0	51	Release
	Flow joints	1.5	1.5	45	Base

h. *Static Stability Analyses*.—Computer program RIGID was used to evaluate the static stability of the abutments at Horse Mesa Dam. This program performs a rigid block limit equilibrium analysis. It uses three-dimensional vector techniques to analyze the forces acting on the foundation wedges. Input to the program included: the weight of the wedge (and the mass for the dynamic analysis), the combined loads from the dam and the spillway structures (resolved into x, y, and z components), the hydraulic uplift forces on the individual planes that form the wedge, and the orientation and shear strength of the wedge planes.

Table 11 gives static analyses results using input from tables 4 through 7 and table 10.

Table 11.—Static analyses results.

Abutment	Load case	Factor of safety
Right	1	1.26

i. *Results*.—The computed factor of safety for the selected foundation wedge under the loading analyzed was 1.26. To gain some insight on how the dam and reservoir have impacted the stability of the right abutment, a factor of safety was estimated for the potential foundation wedge with no dam or reservoir loads. The estimated factor of safety is 5.75.

Assumptions regarding the continuity and shear strength of the wedge planes, as well as the hydraulic uplift forces acting on these planes, have a significant effect on the estimated factor of safety. However, the assumptions made during the analyses are not considered overly conservative, therefore, the stability of the right abutment critical potential foundation wedge should be considered a safety of dams deficiency.

A risk analysis indicated that the risk associated with the right abutment at Horse Mesa Dam was sufficiently high to justify action to reduce the risk. Alternative remedial proposals were studied which included anchor bolts, post-tensioned tendons, and several drainage options. The selected alternative was a phased approach in which the first phase would was intended to provide further geologic data while relieving uplift pressures in the abutment. In subsequent phases additional measures would be taken to increase the abutment stability as needed.

j. *Abutment Modifications Resulting from the Stability Analysis*.—The modification to the right abutment was completed in the following steps as documented in Technical Memorandum No. HM-8312-2, *Horse Mesa Dam—Right Abutment Stability Improvement*:

1 Install piezometers in the abutment to measure the existing uplift pressures. Measurements continued throughout the project so that the effectiveness of the modifications could be determined.

2. Pre-drain holes were drilled to relieve uplift pressure in the abutment and reduce seepage in the construction area. Pre-drain holes were orientated to intercept seepage from certain known discontinuities as shown in figure 17. Measurements showed that the pre-drains reduced uplift pressures by between 38 and 90 per cent.

3. An 8- by 10-ft adit was constructed in the right abutment to a length of 175 ft. During the construction of the adit geologic data was collected for use in mapping the abutment and in future stability analyses.

4. Deep drains were drilled into the abutment from the adit. The depth and orientation of the deep drains were based on projections of seepage paths within the abutment as shown in figure 18. Deep drains were found to further reduce uplift pressures by between 13 and 51 per cent.

The stability of the right abutment was evaluated based on the measured reduction in uplift pressures and the additional geologic data collected during construction of the adit. The procedure followed that described above with adjustments made based on the data collected during the construction of the modifications. The minimum factor of safety on the right abutment for static loading was found to be 1.73. The updated stability analyses are documented in Technical Memorandum No. HM-8312-3, *Horse Mesa Dam, Post-Construction Abutment Stability Studies*.

The need for further modifications to stabilize the right abutment was evaluated based on a revised stability analysis and risk analysis. The results of the risk analysis indicated that the risk associated with the right abutment was sufficiently reduced; no further modifications were proposed.

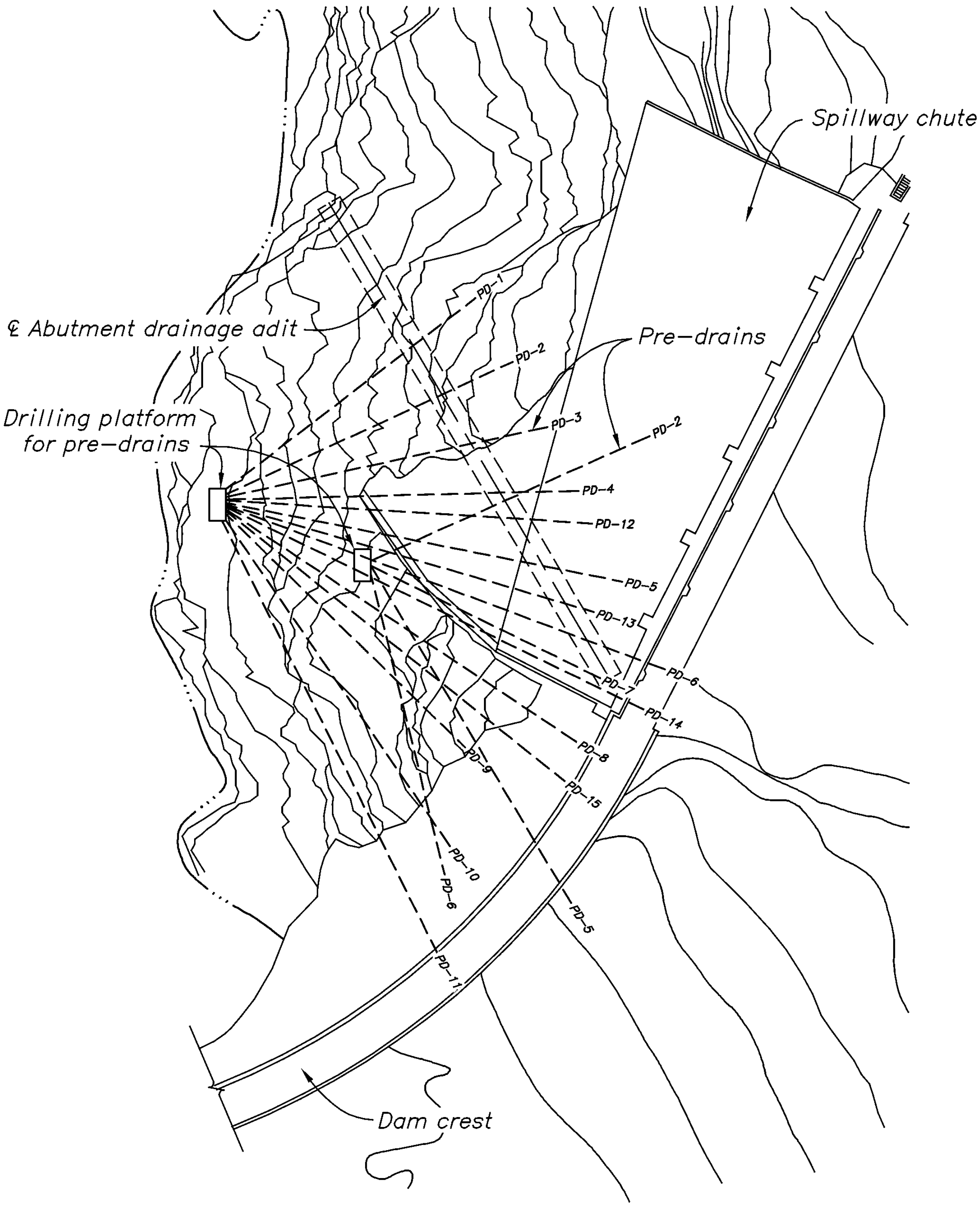


Figure 17.—Plan view of right abutment with predrains and adit.

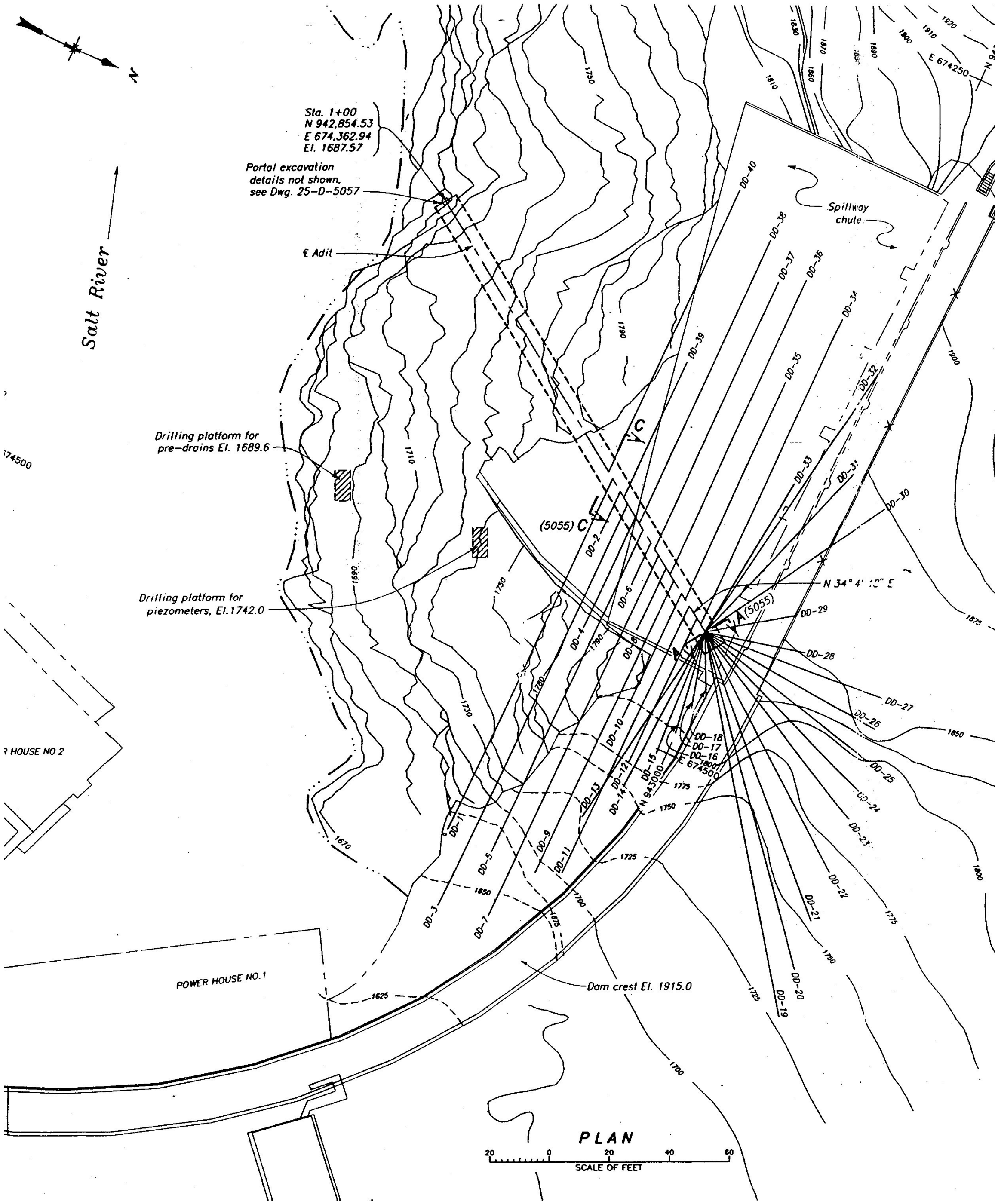


Figure 18.—Plan view of right abutment with deep drains and adit.

Appendix B

Case Histories

B-1—Drain Cleaning at Yellowtail Dam Using High Pressure Water Jetting Equipment.—In 1998 and 1999, drill crew and geology personnel from Reclamation's Great Plains Regional Office cleaned drains in the foundations of Yellowtail Dam and adjacent powerplant using high pressure water jetting equipment. Additional work performed at the time included probing drains in the adjacent left and right foundation drainage tunnels of the dam, and probing drains in the right abutment landslide tunnel. References [50] through [64] list geologic and technical data sources for this work.

a. *Reason for Cleaning the Drains*.—The drains were cleaned to relieve increasing foundation uplift pressures beneath the dam and powerplant area. Uplift pressure gauge readings had been increasing under portions of the dam for several years, and visual inspection by the Regional Geologist of the foundation drains during January of 1998 indicated that both the drains and connecting lateral discharge pipes were partially to completely clogged with deposits of clay, iron bacteria, and calcium carbonate. The main foundation drains were last cleaned in 1986 by a contractor using a modified Roto-Rooter. In 1991, Reclamation drill crews cleaned eight drains that were reported blocked in 1986 using conventional core drilling methods.

b. *Description of the Dam*.—Yellowtail Dam (fig. 1) is a thin-arch concrete structure located on the Bighorn River in south-central Montana, approximately 2 miles upstream from the small town of Fort Smith. It was constructed between 1961 and 1967. The multipurpose facility produces benefits of flood protection, sediment retention, irrigation, power production, enhanced fish and wildlife habitat, and recreational opportunities. The dam has a crest length of 1,480 feet and a structural height of 525 feet. It impounds a long, narrow reservoir extending as far as 72 miles upstream at maximum water surface elevations. At maximum capacity (elevation 3657.0 ft), the reservoir has a surface area of approximately 17,300 acres lying within both Montana and Wyoming. Maximum storage capacity is 1,375,000 acre-feet.

Appurtenant structures include the tunnel spillway, located in the left abutment of the dam, and an irrigation outlet and an evacuation outlet, both of which discharge to the right of the powerplant located near the center of the base of the dam.

c. *Foundation Geology*.—Bedrock in the foundation of Yellowtail Dam and powerplant is Mississippian age Madison Limestone (figs. 2 and 3). The formation is near-horizontally bedded, and mostly consists of hard limestones with interbedded siltstone layers—especially near the top of the unit. Jointing is well developed, but proved to be relatively tight during construction grouting. Several slip planes degrade the rock of the left abutment, and additional grouting was necessary in that area.

Figure 1.—View of the downstream face of the dam from the access road along the river.

d. *Description of the Drainage System*.—Drainage for Yellowtail Dam is accomplished through a series of tunnels, vertical shafts, and galleries, within which 3-inch diameter drain holes have been drilled into the foundation bedrock. The foundation of the dam and the abutments in immediate contact with the lower portion of the dam are drained by a foundation gallery and series of inclined tunnels and vertical shafts or stairwells, from which a total of 153 drain holes radiate out into the bedrock around the perimeter of the structure. These drains are of varying depths with all being less than 200 feet deep. The powerplant foundation is drained by a series of 20 shallow drain holes on the lowest level. All are less than 60 feet deep. The 153 deep drains in the foundation perimeter of the dam and the 20 shallow drains in the downstream portion of the powerplant are the holes that were cleaned with the high pressure water jetting equipment. Total footage cleaned amounted to 24,975.8 feet.

Foundation drainage tunnels (containing 25 drains apiece) lie within each abutment, and another drainage tunnel (with 103 drains) lies just below a landslide high on the right abutment. Drains within these features were probed during the 1998 phase of the cleaning program, but were not considered sufficiently blocked to necessitate cleaning during the 1999 phase of the program.

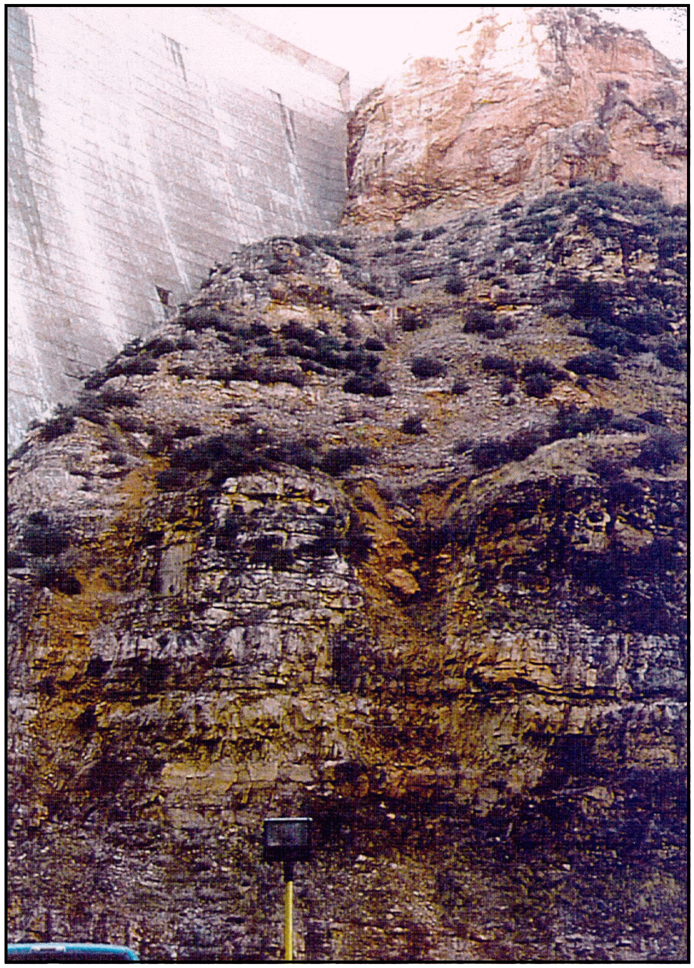


Figure 2.—Madison Limestone outcrop in the left abutment downstream from the dam. Note the weathering in fractured zones. The rock in this abutment is not as sound as in the other abutment.

Grouting and inspection tunnels are located high in each abutment, but those two tunnels contain no drains involved in the recent drain cleaning and probing program.

e. *Individual Drain Construction*.—The floor of the access tunnel or powerplant is flat, or sloping in the inclined sections. A nearby gutter in the concrete floor (fig. 4) provides a means of passing the drain flows to a centralized sump and discharge pumps in the base of the dam. This gutter is located along the upstream side of the access tunnel in the foundation. Three-inch diameter drain holes are drilled in a fan pattern around the perimeter of the dam foundation at a slight angle downstream. Holes in the dam foundation

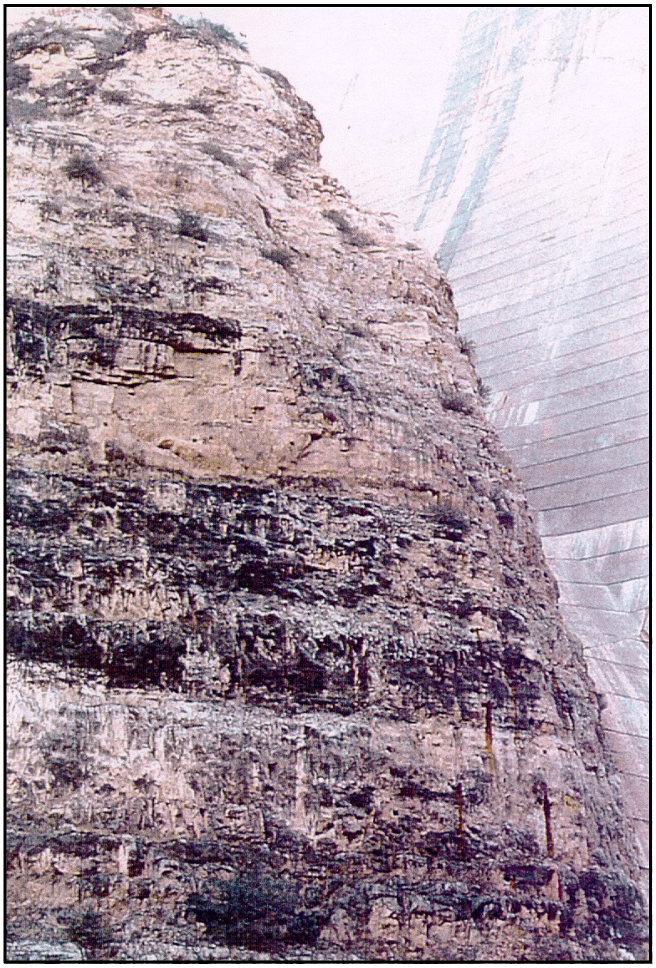


Figure 3.—Sound Madison Limestone in the right abutment downstream from the dam.

vary from near vertical to near horizontal. Powerplant foundation holes are vertical. The top of each hole is completed with a 3-inch diameter steel pipe set in the surrounding concrete, and all except for those in the vertical stairwells are teed to a 2-inch diameter steel lateral discharge pipe in the concrete, which transfers water from the drain to the gutter. Fifty-six drains in the stairwells flow directly into the stairwells and are covered with metal splash shields. They do not have lateral pipes channeling flows to the gutters. All holes except for those in the vertical stairwells are capped with compression-type plumber's packers.

f. *Condition of the Drains Before Cleaning.*—During the January 1998 inspection by the Regional Geologist, it was obvious that the condition of the foundation gallery and

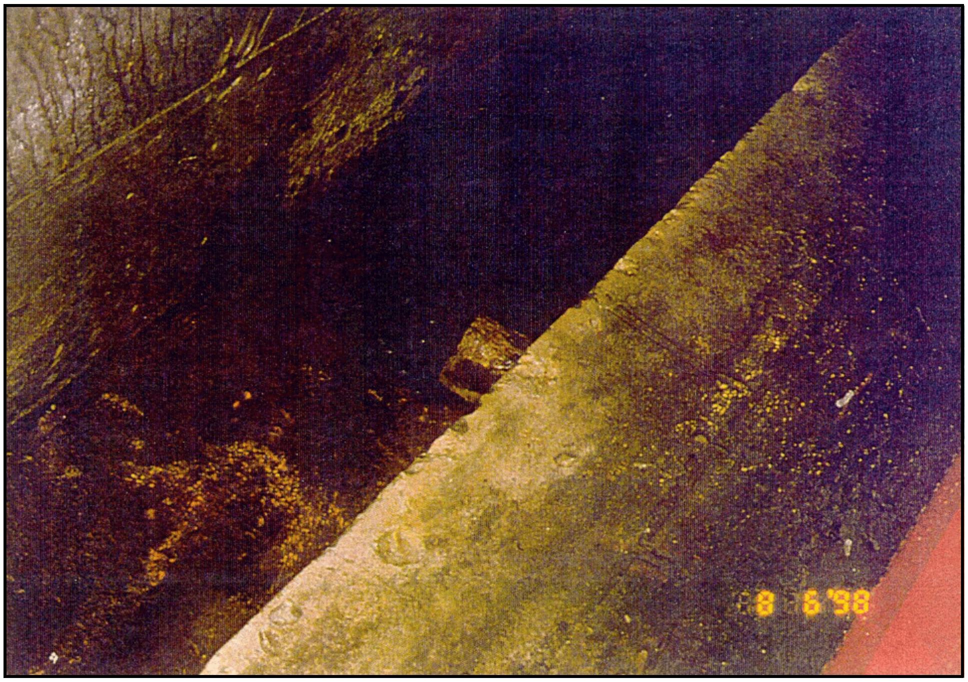


Figure 4.—Gutter along the upstream side of the foundation gallery. The outlet of a lateral discharge pipe is shown near the center of the photograph. It connects to a nearby drain. Most of these pipes were at least partially plugged prior to the drain cleaning operation.

drains dictated remedial actions. The caps to many of the drains were difficult to find because of the accumulation of dirt covering the drains. All of the original screw-in plugs had been replaced (probably in 1986 during the last cleaning) with rubber and metal plumber's packers because the threads in the pipes were rusted out (fig. 5). The metal portions of these packers were rusted and the rubber expandable parts severely rotted in most instances. Most of the lateral discharge pipes between the drains and the gutter were at least partially plugged with iron bacteria or calcium carbonate deposits. Some pipes were so completely plugged that water squirted in the air and then flowed across the floor (fig. 6) when the plumber's packer was removed from the drain. Deposits were visible in the top portions of most drains (fig. 5), and pressurized red oxidized mud had pushed at least one plumber's packer completely out of the drain, and the drain was oozing soft mud (fig. 7). Even the uplift pressure gauges on the walls of the access tunnel were rusted and in suspect working condition (fig. 8).

Drains in the powerplant area appeared to be in much better condition than those in the dam foundation gallery.

Figure 5.—Top of a drain showing the rusted out threads in the 3-inch steel pipe and the deposit of iron bacteria in the upper part of the hole.

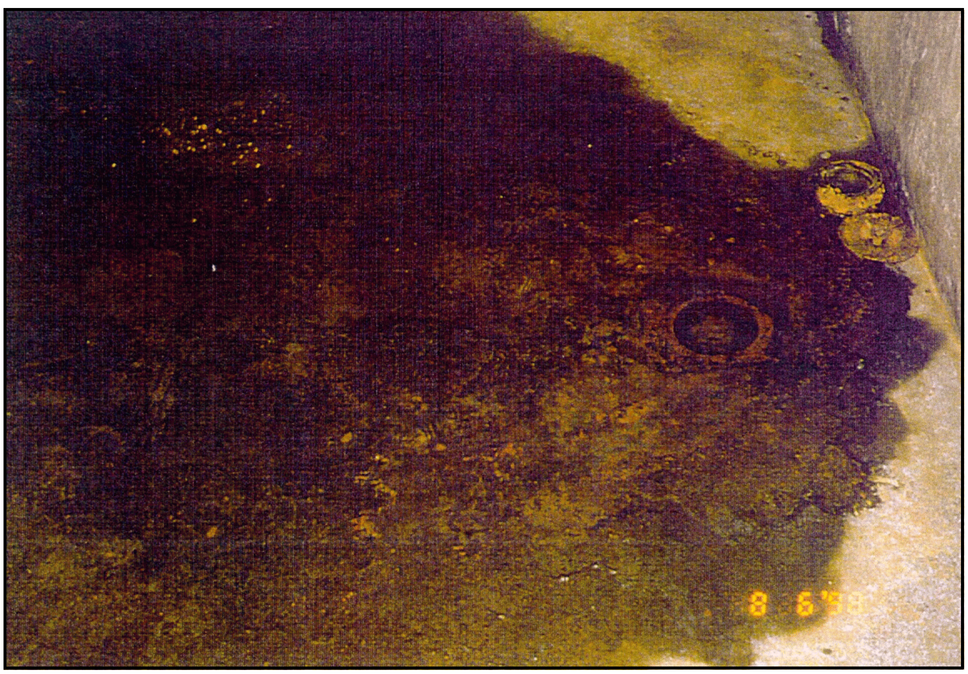


Figure 6.—Water flowing from the top of a drain across the gallery floor because the lateral discharge pipe to the upstream gutter was completely plugged.

Figure 7.—Red clay oozing from a drain under a sloping staircase. The plumber's packer that had capped the hole was pushed out by the pressurized mud.

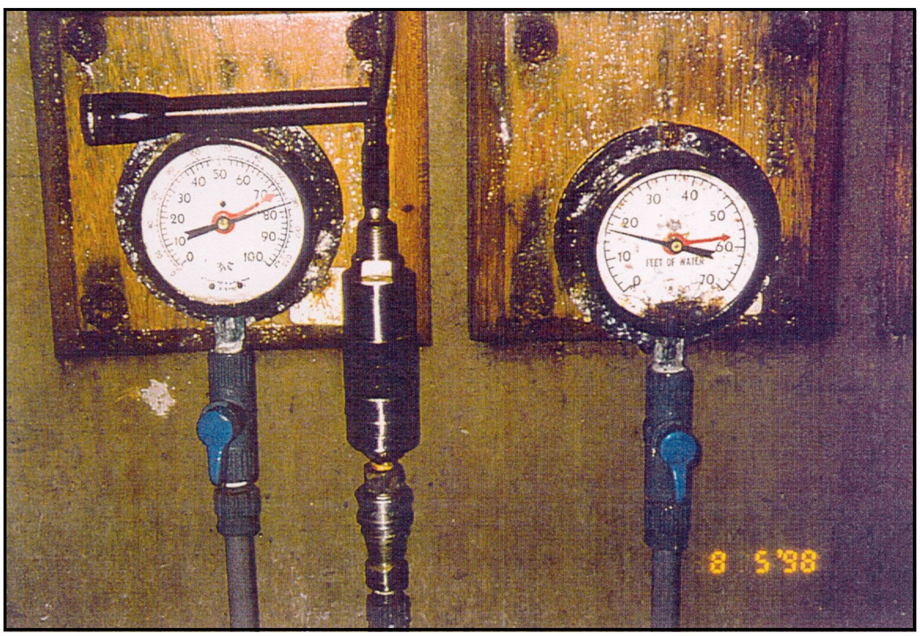


Figure 8.—Gauges on the foundation gallery wall that measure uplift pressures in the underlying bedrock. These old gauges were replaced on November 24, 1998. Unfortunately, many of the new gauge readings do not appear to correlate with the old readings.

g. *Description of the Water Jetting Equipment*.—Near the end of 1998, the Great Plains Region purchased a complete high pressure water jetting system, with the low bidder being Pacific Jetting International Inc. of Placerville, California. The pump is powered by a 215-hp Cummins diesel engine and rated to deliver 15,000 lb/in^2 at 22 gal/min at the pump discharge. Fifteen hundred feet of ½-inch I.D. high pressure hoses in 100- and 50-foot lengths transfer water from the pressure regulator on the pump unit to the control apparatus in the drainage tunnels, which regulates flows going into the drains. A flexible ½-inch I.D. lance hose is connected to a variety of both rotating or nonrotating cleaning heads in the drains. Water is supplied to the water jetting unit by a submersible pump rated at 50 gal/min and 100 lb/in^2 and is double filtered before entering the jetting system. A bypass device diverts excess flows from the delivery pump when they are not needed. Drill crew personnel constructed a portable tripod with electric winch to assist with pulling the flexible lance from high angle deep holes. A separate hand-held wand for cleaning surface materials was also purchased. The cleaning system is mounted on a flatbed trailer along with two large tool boxes of sufficient capacity to contain all of the hoses and other cleaning paraphernalia. Appendix C of this manual provides photographs of the various components of the high pressure water jetting equipment.

h. *Training and Safety Issues*.—As part of the purchase price, the supplier of the high pressure water jetting equipment was required to provide a company representative for 2 days of training involving setup and familiarization with equipment, and safe hands-on operation using Reclamation drill crew personnel. This was all accomplished onsite at Yellowtail Dam after an orientation session in the drill crew warehouse in Billings.

Personal protective equipment had previously been purchased and checked out. It consisted of hard hats with face shields, heavy gloves, and steel-toed boots, with personnel actually running the lance into the drains wearing steel-toed rubber mine boots. Waterproof rain suits were purchased, but generally proved unnecessary.

The high pressure cleaning unit is a new technology for regional drill crews and it has the potential to do serious damage to both personnel and structures. Safety issues are of utmost concern. A job hazard analysis was developed for the particular work site, and the regional industrial hygienist wrote a specific safety procedures booklet for the high pressure water jetting unit. He also was personally present for the prework orientation and the hands-on training at the dam to observe any problems that might develop. Untrained personnel absolutely should not operate this equipment.

i. *Cleaning Methodology*.—Starting with Yellowtail Dam, regional geology and drill crew personnel developed generic methods or procedures to be used with each drain cleaning job. Individual sites are somewhat different, so these general guidelines were meant to be adaptable to each new job and are being further refined as experience is gained with the equipment and how it performs under differing geologic and site conditions.

The first step was to research all available literature to determine geologic conditions in the areas to be cleaned. Special emphasis was placed on available drill logs and geologic sections to determine rock types and special conditions such as weathering, jointing, and the locations of faults and shear zones. At Yellowtail Dam, the rock was determined to be mostly limestone with interbedded siltstone. It had well developed jointing and some slip fracturing in the left abutment.

A site visit was made for determining logistics for moving and setting up equipment, water supply, electrical supply for lights and ventilation (if needed) in the access tunnels, and local safety and operational concerns, and checking for radon gas and hydrogen sulfide (or other noxious gases).

After the initial site visit in 1998, a decision was made to divide the cleaning program at Yellowtail Dam into two separate phases. The first phase was to wash down the foundation tunnel with an electric portable washer capable of producing 2.1 gal/min at 1,200 lb/in^2, flush out the lateral 2-inch discharge pipes connecting the drains with the gutter using the electric washer, and replace all of the rusted and rotted plumber's packers at the tops of the holes with newly purchased rubber and nylon plumber's packers that will not rust. Additional work added to this first phase of cleaning included probing drain holes in the left abutment landslide tunnel, and in the left and right foundation drainage tunnels. The results of the probing are included in section B-1.l, *Probing of Drains*. All work included in the first phase of the program was completed during August of 1998.

The second phase of the cleaning program at Yellowtail Dam was completed between June and September of 1999. A second visit was made to the dam to confirm final logistics before moving in equipment. Access and equipment setup sites were located on each end of the crest with access through manholes, and at the base of the dam through a door near the powerplant. The foundation gallery consisted of a long, flat-floored tunnel at the base of the dam with sloping stairs and vertical stairwells up the abutments. The powerplant drains were on a flat floor with easy access. Clean water was pumped by a submersible pump lowered into the tailrace below the dam or from the reservoir by a pump hung over the upstream side of the dam. All drain areas were lighted, but the electrical supply for other equipment was limited to 110 volts with a 20-ampere breaker, which caused severe problems for the tripod and winch system that the drillers had constructed. There were no radon or noxious gas concerns requiring additional ventilation. Local safety and operational issues were reviewed with project personnel. Since the dam operations and maintenance people only worked 4 days per week, and the drillers worked a compressed work schedule of 8 days on duty and then 6 days off, alternatives for security and unlocking accesses had to be arranged.

The next step in procedures was to videotape several representative drains to determine precleaning conditions such as degree of blockages and types of materials to be removed. The visual condition of the drains, suggestions from the water jetting representative, and knowledge gained from the literature search, determined the pressures, types, and sizes of cleaning heads to use. Adjustments were then made based on videos made after the first few holes were cleaned.

The first attempts at videotaping were learning experiences. Most of the drains were blocked with masses of floating iron bacteria. That material can best be described as slimy reddish brown to nearly black strings and gelatinous clumps. It covered the lens of the camera soon after lowering it into the near vertical drains and prevented further viewing. To overcome this problem, the drains were flushed with clean water using a small electric pump, garden hose, and lengths of flush-joint ½-inch diameter PVC pipe. After flushing, most of the loose iron bacteria were removed, and the harder calcium carbonate deposits were visible on the walls of the drains. A few of the drains contained enough hard blockages that precleaning videos could not be completed. The flatter angled drains were essentially empty of water except for limited flows along the bottom of the holes, so floating material was not a problem. Screwing together and unscrewing the push rods necessary to advance and retrieve the camera in the long flat holes was, however, very time consuming.

The final step in procedures was to monitor as many potential changes in site conditions as possible. In order to judge the effectiveness of the cleaning operation, all uplift pressure gauges and weirs in the dam were read before starting cleaning. Regularly scheduled monthly monitoring was then continued during and after the cleaning operation. In addition, each drain flow was measured before and after cleaning by packering off the gutter outlet pipe and determining the flows with a graduated container and stopwatch from a pipe passing through the top drain packer.

Actual cleaning operations started on June 10, 1999 and continued through September 17, 1999. Three regional drill crew members were utilized in operating the high pressure cleaning unit, with a physical science technician from the regional office being onsite occasionally to do before and after spot checks with the down-hole video camera to monitor results.

Cleaning operations began at the base of the dam near the powerplant with water supplied to the high pressure pump by lines laid from a submersible pump set in the tailrace. After some trial and error adjustments at the beginning of the operation, cleaning pressures and flows were generally kept between 8,000 and 10,000 lb/in^2 and about 14 to 16 gal/min by adjusting the pressure regulator on the high pressure pump and increasing or decreasing the revolutions per minute of the pump motor. Lower pressures usually would not clean out as much hard calcium carbonate scale as desired, and higher pressures tended to enlarge the softer or fractured bedrock zones of the drain holes to an undesirable size. A 2¾-inch outside diameter (O.D.) rotating cleaning head with two backward-thrusting 45° angle jets, two forward-thrusting 45° angle jets, and one forward jet offset in the front of the cleaning head and angled at 10° was selected as doing the best job of cleaning. The size of the backward-thrusting jets had to be adjusted to lessen the forward thrust on the cleaning head, because it was too difficult to pull from the deepest high angle drains as it was first set up from the factory. A feed and retrieval rate of 2 to 4 ft/min seemed to be a satisfactory speed for cleaning the drains. Slower movement tended to erode the walls, and faster movement did not allow the cleaning head enough time to sufficiently remove hard deposits.

Two minor problems soon became apparent. The first was that the ½-inch high pressure hoses delivering water from the pump to the controls at the top of the drains were constantly vibrating while the system was running and wearing through very quickly at any angle or rough contact point such as around corners or over stair treads. This problem was largely alleviated by encasing the delivery hoses inside old fire hoses. Even the fire hoses eventually wore through, so the water delivery system had to be regularly checked for wear and the fire hoses adjusted to new points of contact as they abraded. The second problem was that the 20-ampere breaker on the lighting system did not allow full use of the lights and the winch on the tripod hoist for pulling the flexible lance hose from the near vertical drains. The crew unscrewed as many lights as safety allowed, but surges from starting the winch caused the breaker to open or brownouts. That problem plagued the drillers all across the bottom of the foundation gallery. Once the near horizontal drains in the abutments were reached, the winch was abandoned and the flexible lance hose and cleaning head were pulled from the drains by hand.

The drains were cleaned in the following order: the foundation gallery in the base of the dam, the sloping portions (inclined stairs) of the lower abutments, the powerplant foundation, the upper left abutment, and finally the right abutment. Tear down and setup time for moving between the three main site locations at the bottom and top of the dam took about a half day per move.

Most of the cleaning was completed without incident. The flexible cleaning lance and rotating cleaning head were temporarily stuck in a few drains for times varying from minutes to several hours, but they were quickly shut off in order to prevent hole enlargement. Total hole blockage and subsequent pressure buildup were never experienced. The cleaning tools had to be removed in some instances by washing down from the top of the drain with a ½-inch diameter PVC pipe in addition to pulling.

The near vertical drains in the base of the dam were the easiest to clean, except for pulling the long, flexible cleaning lance from the deepest holes. They generally had the highest flows of any of the drains. When the high pressure rotating cleaning head reduced the hole inclusions to fine sand and silt-sized particles, the flows from the high angle holes helped flush the material up and into the nearby gutter. Only a few minor instances of sticking the cleaning tools in the high angle drains occurred.

Low angle drains higher in the abutments were harder to clean for two reasons. First, they were not filled with water and when the cleaning head pulverized hole inclusions, the material settled to the bottom side of the hole and did not flush out as easily as in the completely water-filled, near vertical holes. Secondly, when the cleaning head tended to parallel the bedding of the near horizontal rock units in the low angle holes, larger pieces of rock tended to fall in behind the cleaning head and could not be flushed out of the drains by the back-pointing jets of the cleaning head. This resulted in the stuck cleaning tools previously mentioned.

After the drain cleaning was completed, the drill crew went through the entire structure and washed down the areas around the drains and flushed any remaining "cuttings" from the gutters. Metal splash guards in the vertical stairwells that had been removed by the drill crew to gain access to drains were not replaced, but were left for project personnel to install, as had been previously agreed to.

j. *Results of the Cleaning Program.*—

1. *Depth Comparisons.*—In most instances, the original drilled depth was known for each drain in addition to the depth to which it had been cleaned in 1986 or 1991 (table 1). The depths to which the drains were cleaned in 1999 were also measured and recorded. Minor discrepancies appear to occur in some of the measurements. Comparison of the data indicates that all drains, except for four, were cleaned to satisfactory depths. Those drains are block 4 - drain 3, block 4 - drain 5, block 8 - drain 5, and block 23 - drain 3. They are all low angle drains occurring high in the abutments. All had low or no flows and could not be cleaned with the high pressure jetting system because of large rock fragments blocking the drains. The overall effect of these holes on structure drainage is negligible, so cleaning them by conventional drilling methods at this time cannot be justified. If at some time in the future other drains need to be cleaned by conventional drilling methods, these drains can be included in that program. They are identified here strictly for easy future reference.

2. *Video Observations.*—Several representative drains were videotaped both before and after cleaning to help adjust cleaning apparatus parameters and determine the effectiveness of the high pressure washer as a cleaning tool. The before-cleaning videos showed that most blockages in the drain holes consisted of floating masses of iron bacteria.

Minor encrustations of calcium carbonate also occurred in nearly all holes—usually along fractures. A few holes contained deposits of red mud. Bedrock in the drains was reasonably sound with the limestone varying from massive to thinly bedded. Occasional thin zones of shale or thinly bedded siltstone were present. Original drilling or subsequent cleaning had sometimes enlarged the weaker bedding zones to possibly as much as 6 to 8 inches in diameter.

After-cleaning videos indicated that pressures of approximately 8,000 to 10,000 lb/in^2 were sufficient to remove all of the clay and iron bacteria plus nearly all of the hard calcium carbonate without doing undue damage to the drain walls. When the pressure was increased above those limits or the cleaning head was allowed to remain too long in one spot, hole enlargement occurred. Softer zones were easily enlarged to an estimated 5 or 6 inches from the original 3-inch diameter. Not all calcium carbonate could be removed without some hole enlargement, so a compromise was made to leave a minor amount in exchange for little or no change in drain diameter.

An especially noteworthy observation was that the high pressure jet cleaner was very effective in removing deposits from cracks and joints. Several blocked fractures showing little or no flow (as determined by particulate movement in the drain) were observed to be flowing substantial amounts of water after the jetting action of the cleaning head had removed the blockages from the fractures.

3. *Weir Measurements.*—Seven weirs measure drain flows in Yellowtail Dam. They are read on a monthly basis. During the drain cleaning phases of 1998 and 1999, the reservoir elevation fluctuated greatly with periods of high inflow, so determination of the effectiveness of the cleaning program had to be interpreted from a combination of past and recent weir flows and past and recent reservoir elevations. Tables 2-A through 2-G present the data used for the interpretations.

The weir readings of July, August, September, October, and November of 1998 and those for August, September, October, and November of 1999 were used for comparisons. Unfortunately during these periods of time, several erratic measurements appear not track the rise and fall of the reservoir elevation. Either the measuring devices were not being accurately read or the cleaning operations during the time affected some of the measurements.

With the difficulties of correlation and limited number of readings for comparison, it appears that the cleaning program caused only a slight long-term increase in overall weir measurements. That will be better confirmed when additional data become available for comparison.

4. *Individual Drain Measurements*.—The flow from each drain was measured shortly before and after cleaning to compare individual flows. This was accomplished by installing a pneumatic packer to block the gutter end of the lateral cross drain pipe, and then measuring the flow from a pipe extending through the center of a packer in the main drain pipe using a graduated container and stopwatch. Those drains in the vertical stairwells do not have lateral cross pipes, so their flows were determined by simply packering off the main drain outlet and measuring the flow from the pipe extending through the packer.

There was some confusion and a learning process involved with the initial methodology for making the flow measurements, so all readings at the start of the program were not entirely comparable. Table 1 shows the individual drain flows before and after cleaning as they can best be interpreted from original field data. Nearly all of the drain flows increased after cleaning. Summing of the measurements shows an increase in flows from the dam foundation drains of approximately 33 percent after cleaning, and approximately 450 percent for the powerplant foundation drains. The greatly increased flow rates for the powerplant drains are probably somewhat exaggerated by the low overall flows and associated likelihood of measurement error influence.

Timing of measurements can dramatically alter the recorded flow rates. If the after-cleaning measurements are completed too soon after cleaning, return flows of water forced into the surrounding bedrock by the cleaning process will significantly increase rates as compared to flow rates taken later. Experience has shown that approximately 2 hours of minimum lag time should expire between the cleaning process and the after-cleaning flow measurement to allow for drain flow stabilization

5. *Uplift Pressures*.—A series of fifteen gauges in the foundation tunnel of Yellowtail Dam monitors uplift pressures exerted by water in the bedrock beneath the structure. A gradual increase in those gauge readings occurred over time and was one of the indications that the drains at the dam needed to be cleaned. The gauges are identified in tables 3-A through 3-E by block, line, and letter, with the letters corresponding to upstream to downstream installations (A—upstream to E—downstream).

Comparison of individual gauge readings before and after cleaning drains in the various blocks where the gauges are placed to monitor is extremely difficult because new gauges were installed on November 24, 1998, to replace old gauges that were showing suspect measurements. The total number of readings since the new gauges were installed is very limited. Readings before and after the date of change are comparable in some instances, but other gauge readings do not seem to correlate. In a few instances, individual readings are obviously incorrect. The following comparisons are based entirely on readings taken after the new gauges were installed and any obvious mistakes were omitted.

The gauge readings show somewhat mixed results produced by the drain cleaning. Most of the gauge readings indicate a significant decrease in feet of head since the drains were cleaned, with the upstream gauges (except A gauges) generally showing the greatest decrease. At similar reservoir elevations, the decreases in head range from a maximum in excess of 50 feet in one upstream gauge to as little as 2 feet or less in some of the downstream gauges. Gauges A in blocks 12, 18, and 22 are exceptions to the general trend, showing little if any decrease in head after the surrounding drains were cleaned. Initial comparisons are based on so little data, because of the recent replacement of gauges and the severe fluctuation of the reservoir within the past year, that more accurate correlations of gauge readings with reservoir elevations should be done in the future after additional measurements have been taken.

k. *Cost Comparisons.*—Total cost for the 1998 and 1999 probing and cleaning program was $119,513, which calculated to $4.86 per foot. Those costs included initial training, videotaping, and the additional costs for probing drains in the left and right foundation drainage tunnels, and the right abutment landslide tunnel. It should be noted, however, that the program was completed under nearly ideal conditions with both competent rock and good access. Experience gained since the Yellowtail Dam program with more difficult geologic conditions at Canyon Ferry Dam in Montana indicates that a realistic cost estimate for future jobs could range from $4 to $8 per foot depending on the circumstances.

Even at greater anticipated costs, the high pressure water jetting system is a bargain compared to the Great Plains Region's most recent drain cleaning costs using conventional drilling. The cheapest drain cleaning project with conventional drilling that the region has completed was the 1997 program at Pueblo Dam in Colorado. That job cost $11.21 per foot for a program that included 9,812.4 feet of 3-inch diameter drains very similar in construction and accessibility to those at Yellowtail Dam. References to other Reclamation and Corps of Engineers drain cleaning costs sometimes are in excess of $20 per foot for conventional drilling projects.

l. *Probing of Drains.*—Drains in the right abutment landslide tunnel and the left and right foundation drainage tunnels were all probed in August 1998 to evaluate the extent of blockages. Regional drill crews completed this probing activity as an addition to their originally scheduled cleaning of the drains beneath the dam and powerplant. Drain holes in the right abutment landslide tunnel had been identified as needing probing evaluation as early as 1995 in the Periodic Facility Review Report for Yellowtail Dam. Since no recent data existed on the condition of drains in the other two tunnels, it was decided to probe them after the probing was completed in the landslide tunnel. All work was completed using multiple 5-foot lengths of ½-inch flush joint PVC pipe.

1. *Right Abutment Landslide Tunnel.*—The right abutment landslide tunnel was constructed during 1964 and 1965, after earlier excavations for the access road to the top of the dam undercut the upper part of the Madison Limestone and triggered landslides in an underlying shaley zone of the Amsden Formation. Stabilization of the abutment has included reshaping and unloading the landslide mass and providing enhanced surface and subsurface drainage. During 1965, 103 3-inch diameter drain holes were drilled at upward angles in the crown of the tunnel. Most of the holes were completed at an attitude near vertical, but a few angle upward near 45 degrees. They extend at varying depths to within about 10 feet of the top of the bedrock surface in the landslide. The holes are lined with 1½-inch diameter PVC perforated pipe with 2 feet of stainless steel pipe grouted into the tunnel crown.

Since at least 1995, recommendations have been made to inspect and probe the depths of the drains in the right abutment landslide tunnel. The Regional Geologist completed a visual inspection in January 1998. At that time, little drainage flowed from any of the holes, and nothing indicated that flows had ever been significant. No records have been located of past drainage from the tunnel, and no records have been located indicating that any of the drains have ever been cleaned since they were originally drilled.

During August of 1998, regional drill crews measured and labeled a total of 103 crown drains in the tunnel. Table 4 shows the resulting depths of the holes. Records have not been located showing the original depths for the holes, so no sure way exists of determining significant blockage. About 25 to 30 percent of the drains appear to now be shallower than surrounding drains, and during probing, the drillers noted red clay present on the probe rods of 4 holes.

Because of the PVC liners, it will be very difficult, if not impossible, to clean hard calcium carbonate from the drains without destroying the liners. If, however, the supposed blockages are composed of red clay filling, as may be indicated by the red clay coating on the probe rods, then the drains can probably be effectively flushed with the region's high pressure water jetting equipment. Methodology would dictate using a small, nonrotating sewer cleaning head and greatly reduced pressures so as not to damage the PVC liners.

However, since no indication exists of any significant past drainage in the right abutment landslide tunnel, such as higher levels of water staining in the outflow ditch, and no indication exists of water pressures causing instability in the overlying landslide, further cleaning is not recommended at this time.

In approximately 5 years, the drains in this tunnel should be probed and the depths compared with those from the 1998 probing to determine if cleaning needs to be considered.

2. *Left Foundation Drainage Tunnel*.—The left foundation drainage tunnel has a rather complicated history regarding drains and cleaning.

During 1970 and 1971, a regional Reclamation drill crew drilled a total of 25 3-inch diameter, unlined, vertical drain holes into the bedrock above the crown of the left foundation drainage tunnel. They were completed approximately on 10-foot centers and had depths varying from 15.5 to 120 feet. Drilling of the drain holes had been delayed after construction of the tunnel because of a possible grouting program in the abutment to close slip fractures. Engineers speculated that grout intrusion would block any drains drilled prior to the grouting.

A video inspection during 1991 showed partial to complete calcium carbonate blockage of all holes. As a result of the video inspection, a regional Reclamation drill crew cleaned the drains during June through August of 1992 using conventional core drilling methods. They encountered hard calcium carbonate in many of the drains.

Nineteen of the 25 holes were probed during 1998. The remaining six were not probed because of unsanitary conditions in the tunnel caused by the buildup of packrat droppings and nest debris. Of the 19 that were probed, 4 showed significant blockages well short of the last drilled depths. Table 5 displays each drain with the drilled, cleaned, and recently probed depths.

Based on the 1998 probing, it appears that the left foundation drainage tunnel needs some remedial work. First, the packrat debris needs to be removed and the tunnel washed out. Then the remaining unprobed holes need to be measured, and those holes exhibiting significant blockages need to be opened.

3. *Right Foundation Drainage Tunnel*.—Drains are also provided in the right foundation drainage tunnel. Twenty-five vertical, unlined, 3-inch diameter drain holes were completed by contract drillers in the crown of the tunnel during 1966. They were drilled approximately on 10-foot centers and had maximum depths of 75 feet from station 0+60 to station 1+60, and 80 feet from station 1+70 to station 2+80 [53].

A records search of regional and project files has resulted in finding no indication that these 25 drains have ever been probed or cleaned since they were drilled in 1966. Probably during 1987, two drains in the tunnel were videotaped with a down-hole camera. The unidentified drains were reviewed during 1990 and determined to show no significant blockages.

Table 6 presents results of the 1998 probing. The drill crew measured and labeled the holes beginning at the entrance to the tunnel and going toward the far end.

Since no record exists of past probing, it is not possible to determine for sure if any of the drains are significantly blocked. However, if the depths and stationing from literature references are correct, it appears that all of the drains except possibly the first two near the tunnel entrance are open to within a few feet of their original drilled depths. Drill crew members reported very little drainage from any of the holes at the time of probing, and there appeared to be no indication of past increased flows.

It did not seem that these right foundation drainage tunnel drains needed to be cleaned after their 1998 probing. In about another 5 years, they should be probed again and the depths compared with the depths derived from the 1998 probing to determine if changes warrant any further action.

m. *Conclusions and Recommendations.*—With good bedrock conditions and reasonable access, high pressure water jetting equipment is a safe and cost effective method of cleaning unlined drains. It is not well suited to all geologic circumstances, and must be operated only by trained and experienced personnel who have the ability to understand and adjust to differing site conditions. It is best utilized at those sites having a competent bedrock foundation devoid of soft layers or fractured zones. At all sites, it should be mandatory that progress of the cleaning program be regularly monitored with down-hole video equipment to both ensure the integrity of the drain holes and obtain an effective cleaning job.

The Great Plains Regional Office has not yet proven the ability of high pressure water jetting equipment to effectively remove extensive solid deposits of hard calcium carbonate from drain holes. In addition, contrary to information from the manufacturer, the region's limited experiments with test cleaning grouted PVC pipes and screens suggests that because of the pressures required, it is doubtful if high pressure equipment can clean hard deposits from PVC lined drains without severely damaging the liners.

Observation of the rate of redeposition of materials in the drains last cleaned in 1986 and 1991 indicate that possibly as long as 10 years may expire before the drains at Yellowtail Dam become clogged enough to require another thorough cleaning. Careful monitoring of uplift pressures and weir flows will prove invaluable in determining the future cleaning schedule. Several representative drains should also be examined with a down hole video camera approximately at 5-year intervals to visually determine their conditions.

It will be much more cost effective to clean the drains on a regular schedule with the high pressure drain cleaner than waiting until they become completely filled with hard calcium carbonate, which could require very time consuming conventional drilling to remove.

The right abutment landslide tunnel drains and the right foundation drainage tunnel drains did not need to be cleaned after the 1998 probing. They should be reprobed in about another 5 years and the depths compared with those obtained during the 1998 probing to determine if cleaning is warranted.

The left foundation drainage tunnel needed remedial work. Rodent droppings and nest debris prevented a complete probing of all 25 drains. Of those that were probed, a few showed significant blockages. This tunnel needs to be cleaned, the remainder of the holes probed, and all, or at least those drains showing blockages, need to be cleaned.

The reason the drains in the left abutment showed more need for maintenance was probably because the holes drain more water than the drains in the right abutment tunnels. Also, some of the grout from the extensive fracture grouting in the left abutment was most likely being dissolved and redeposited in the drain holes by reservoir water slowly seeping through the abutment rock.

TABLE 1

Yellowtail Dam
Drain Hole Cleaning Data
(Recorded Depths and Individual Drain Flow Measurements)

Foundation Gallery

Location Block	Location Hole	Original depth (ft.)	Last cleaned depth 1986 or 1991 (ft.)	Cleaned depth 1999 (ft.)	1999 Date cleaned	Before flow GPM	After flow GPM	Flow increase or decrease GPM
4	1	< 200	165.0	160.5	08/19/99	no flow	no flow	0
4	2	< 200	133.0	150.0	08/19/99	no flow	no flow	0
4	3	< 200	121.0	15.8	08/19/99	no flow	no flow	0
4	4	< 200	143.8	140.0	08/20/99	0.1	0.1	0
4	5	< 200	144.8	31.0	08/20/99	0.3	0.4	0.1
4	6	< 200	145.2	142.5	08/20/99	0.4	no flow	-0.4
5	1	< 200	157.1	154.2	08/20/99	2.5	3.0	0.5
5	2	< 200	172.1	154.5	08/21/99	1.8	2.4	0.6
5	3	< 200	156.7	153.5	08/21/99	1.0	1.2	0.2
6	1	167.0	175.0	110.0	08/23/99	0.4	1.5	1.1
6	2	172.0	176.1	176.0	08/23/99	0.4	1.0	0.6
6	3	175.0	172.3	175.6	08/23/99	0.4	1.1	0.7
6	4	180.0	180.0	172.7	08/23/99	0.5	1.4	0.9
6	5	175.0	174.2	168.5	08/24/99	0.1	0.3	0.2
7	1	100.0	101.3	98.5	08/24/99	< .1	0.1	0.1
7	2	185.0	189.6	186.5	08/24/99	0.7	0.8	0.1
7	3	190.0	196.3	193.5	08/24/99	0.8	1.2	0.4
7	4	191.0	187.5	185.0	08/24/99	0.3	1.4	1.1
8	1	195.0	195.4	194.3	08/25/99	0.3	1.3	1
8	2	182.0	180.3	178.2	08/24/99	0.2	0.5	0.3
8	3	186.0	176.5	172.0	08/25/99	no flow	no flow	0
8	4	183.0	180.5	179.0	08/25/99	no flow	0.3	0.3
8	5	120.0	120.5	33.0	08/25/99	1.8	no reading	-
8	6	181.0	180.8	178.5	08/25/99	10.9	too much flow to measure	-
9	1	125.0	123.2	115.0	08/25/99	0.1	0.2	0.1
9	2	185.0	184.8	162.5	08/25/99	< .1	0.2	0.2
9	3	171.0	171.4	174.0	08/25/99	no reading	no reading	-
9	4	100.0	98.7	99.0	07/14/99	0.1	0.2	0.1
9	5	115.0	109.8	110.7	07/13/99	7.3	7.6	0.3
9	6	192.0	196.3	181.7	07/13/99	< .1	0.1	0.1
9	7	175.0	172.9	173.5	07/12/99	0.4	0.1	-0.3
10	1	187.0	187.0	189.5	07/12/99	no reading	no reading	-
10	2	194.0	186.0	195.6	06/29/99	no flow	no reading	-
10	3	185.0	186.0	188.7	06/29/99	0.3	1.1	0.8
10	4	175.0	168.2	169.0	07/11/99	0.8	1.0	0.2
10	5	165.0	163.8	163.8	07/09/99	0.3	0.3	0
10	6	156.0	156.2	158.0	07/12/99	0.1	0.1	0
10	7	155.0	60.0	158.2	07/11/99	0.1	0.2	0.1
10	8	150.0	153.4	150.2	06/28/99	0.1	0.3	0.2
10	9	148.0	148.0	150.0	07/11/99	0.2	0.2	0
10	10	146.0	150.8	147.0	06/30/99	0.1	no reading	-
10	11	145.0	148.6	147.6	06/30/99	0.9	no reading	-
10	12	144.0	145.8	146.2	06/30/99	no flow	0.1	0.1

TABLE 1

Yellowtail Dam
Drain Hole Cleaning Data
(Recorded Depths and Individual Drain Flow Measurements)

Foundation Gallery

Location Block	Location Hole	Original depth (ft.)	Last cleaned depth 1986 or 1991 (ft.)	Cleaned depth 1999 (ft.)	1999 Date cleaned	Before flow GPM	After flow GPM	Flow increase or decrease GPM
10	13	158.0	158.5	158.9	06/29/99	0.9	0.9	0
10	14	147.0	148.9	150.0	06/29/99	0.5	0.1	-0.4
10	15	150.0	150.0	152.4	06/29/99	0.1	no reading	-
10	16	155.0	151.8	152.5	06/29/99	no flow	0.1	0.1
10	17	156.0	156.0	158.5	06/29/99	0.2	0.4	0.2
10	18	158.0	153.0	158.0	07/09/99	no flow	no flow	0
10	19	156.0	156.8	157.0	07/09/99	0.5	0.8	0.3
10	20	160.0	161.2	154.2	07/10/99	0.1	0.4	0.3
11	1	160.0	162.8	162.7	07/10/99	0.1	no flow	-0.1
11	2	162.0	162.8	163.0	07/11/99	no flow	no flow	0
11	3	160.0	162.5	162.5	07/11/99	1.1	2.7	1.6
11	4	160.0	162.3	162.0	07/10/99	3.1	4.5	1.4
11	5	154.0	155.7	156.5	07/10/99	2.1	3.0	0.9
11	6	155.0	159.9	153.6	06/27/99	3.8	5.0	1.2
12	1	160.0	161.6	161.6	06/27/99	1.9	2.1	0.2
12	2	160.0	160.7	162.2	06/27/99	3.0	3.8	0.8
12	3	160.0	159.6	162.1	06/27/99	2.1	2.5	0.4
12	4	160.0	161.5	161.5	06/26/99	2.1	3.8	1.7
12	5	160.0	161.3	162.2	06/26/99	1.5	1.9	0.4
12	6	160.0	159.6	161.6	06/26/99	2.5	1.2	-1.3
13	1	160.0	161.7	161.6	06/26/99	1.9	3.0	1.1
13	2	160.0	161.7	162.2	06/26/99	0.5	1.5	1
13	3	160.0	161.5	161.5	06/26/99	3.8	5.0	1.2
13	4	160.0	161.6	161.6	06/15/99	3.8	5.0	1.2
13	5	160.0	161.4	161.7	06/25/99	3.8	5.0	1.2
14	1	160.0	161.7	161.7	06/25/99	3.0	3.8	0.8
14	2	160.0	161.7	161.5	06/25/99	1.4	3.8	2.4
14	3	160.0	161.5	161.5	06/25/99	3.0	5.0	2
14	4	160.0	161.5	161.5	06/25/99	1.4	5.0	3.6
14	5	160.0	161.1	161.6	06/24/99	0.4	0.8	0.4
15	1	160.0	161.2	161.4	06/23/99	1.4	3.0	1.6
15	2	160.0	161.3	161.7	06/23/99	0.4	1.9	1.5
15	3	160.0	161.2	161.7	06/23/99	0.8	1.7	0.9
15	4	160.0	160.5	160.5	06/15/99	0.9	1.5	0.6
15	5	160.0	160.7	161.0	06/14/99	no flow	0.8	0.8
16	1	160.0	161.3	161.0	06/14/99	0.5	1.0	0.5
16	2	160.0	161.5	161.0	06/14/99	1.4	1.7	0.3
16	3	160.0	161.2	161.0	06/13/99	2.3	3.8	1.5
16	4	160.0	163.4	163.0	06/13/99	2.3	3.8	1.5
16	5	160.0	162.2	161.0	06/12/99	0.2	0.6	0.4
16	6	160.0	156.3	156.0	06/13/99	2.9	0.8	-2.1

TABLE 1

Yellowtail Dam
Drain Hole Cleaning Data
(Recorded Depths and Individual Drain Flow Measurements)

Foundation Gallery

Location Block	Location Hole	Original depth (ft.)	Last cleaned depth 1986 or 1991 (ft.)	Cleaned depth 1999 (ft.)	1999 Date cleaned	Before flow GPM	After flow GPM	Flow increase or decrease GPM
17	1	160.0	161.1	161.0	06/11/99	11.7	7.5	-4.2
17	2	160.0	164.3	167.0	07/14/99	2.7	5.9	3.2
17	3	160.0	155.3	158.1	07/14/99	1.7	1.9	0.2
17	4	160.0	155.3	158.6	07/14/99	0.5	0.6	0.1
17	5	155.0	153.1	150.5	07/21/99	0.4	0.7	0.3
17	6	155.0	156.2	153.9	07/21/99	0.6	0.9	0.3
17	7	152.0	152.6	150.3	07/21/99	0.1	0.6	0.5
17	8	150.0	152.8	150.7	07/22/99	2.1	2.5	0.4
17	9	152.0	152.9	150.2	07/22/99	0.8	1.3	0.5
17	10	155.0	158.0	154.7	07/22/99	0.3	0.5	0.2
18	1	154.0	155.8	153.0	07/22/99	1.3	1.6	0.3
18	2	155.0	156.8	153.5	07/23/99	0.4	0.7	0.3
18	3	155.0	156.5	153.4	07/23/99	0.6	0.8	0.2
18	4	160.0	161.8	158.6	07/23/99	0.6	1.0	0.4
18	5	164.0	166.7	162.9	07/24/99	0.3	0.7	0.4
18	6	155.0	156.8	150.4	06/10/99	no flow	no reading	-
19	1	145.0	147.8	144.5	07/24/99	0.4	0.6	0.2
19	2	154.0	166.8	153.2	07/24/99	0.6	1.0	0.4
19	3	155.0	161.8	158.4	07/24/99	0.5	1.0	0.5
19	4	155.0	158.8	149.3	07/25/99	0.4	0.8	0.4
19	5	155.0	163.8	155.0	07/25/99	2.4	3.2	0.8
19	6	155.0	163.8	154.8	07/25/99	no flow	no flow	0
19	7	145.0	152.8	142.1	07/26/99	no flow	no flow	0
19	8	152.0	156.9	148.1	07/26/99	0.4	0.9	0.5
19	9	160.0	167.6	158.6	07/26/99	2.9	4.4	1.5
19	10	163.0	168.8	158.3	07/26/99	0.8	1.6	0.8
19	11	167.0	165.8	167.0	07/26/99	0.5	1.3	0.8
19	12	170.0	171.8	161.7	07/27/99	no flow	0.6	0.6
19	13	170.0	171.6	165.7	07/27/99	0.1	0.8	0.7
19	14	170.0	177.8	167.3	07/28/99	0.1	0.2	0.1
20	1	165.0	171.8	163.5	08/04/99	1.2	3.4	2.2
20	2	175.0	180.8	176.7	08/04/99	3.8	4.0	0.2
20	3	182.0	187.8	183.7	08/04/99	1.0	1.1	0.1
20	4	190.0	197.8	193.3	08/04/99	1.7	1.8	0.1
20	5	200.0	202.0	203.3	08/05/99	1.1	1.2	0.1
20	6	185.0	191.5	188.3	08/05/99	4.9	5.0	0.1
20	7	180.0	187.3	184.4	08/05/99	1.7	2.3	0.6
20	8	192.0	187.3	195.4	08/06/99	7.2	8.2	1
21	1	187.0	186.9	185.2	08/06/99	too much flow to measure	too much flow to measure	-
21	2	190.0	191.2	195.2	08/06/99	4.9	5.5	0.6
21	3	187.0	186.3	190.0	08/06/99	1.7	5.5	3.8
21	4	196.0	195.2	210.0	08/07/99	2.2	2.6	0.4

TABLE 1

Yellowtail Dam
Drain Hole Cleaning Data
(Recorded Depths and Individual Drain Flow Measurements)

Foundation Gallery

Location Block	Hole	Original depth (ft.)	Last cleaned depth 1986 or 1991 (ft.)	Cleaned depth 1999 (ft.)	1999 Date cleaned	Before flow GPM	After flow GPM	Flow increase or decrease GPM
21	5	194.0	195.1	194.3	08/07/99	9.8	12.0	2.2
21	6	190.0	191.2	194.2	08/07/99	4.2	4.3	0.1
21	7	195.0	194.2	194.9	08/07/99	0.4	0.5	0.1
22	1	185.0	181.8	185.0	08/08/99	0.1	0.3	0.2
22	2	147.0	145.2	150.0	08/08/99	0.6	0.7	0.1
22	3	176.0	171.2	170.0	08/08/99	2.0	2.6	0.6
22	4	180.0	180.0	185.0	08/08/99	1.1	2.1	1
22	5	175.0	173.1	180.0	09/16/99	< .1	0.7	0.7
23	1	150.0	150.0	152.6	09/16/99	0.4	0.4	0
23	2	155.0	149.5	154.7	09/16/99	no flow	0.1	0.1
23	3	150.0	150.0	32.0	09/16/99	no flow	no flow	0
23	4	160.0	162.1	176.5	09/16/99	no flow	no flow	0
23	5	145.0	144.3	149.5	09/15/99	0.1	0.4	0.3
24	1	145.0	142.7	95.0	09/15/99	0.3	1.3	1
24	2	151.0	151.5	155.2	09/15/99	0.5	3.1	2.6
24	3	136.0	139.8	140.5	09/01/99	0.2	0.9	0.7
24	4	141.0	139.5	142.5	09/01/99	0.2	1.0	0.8
24	5	143.0	141.9	144.1	09/01/99	0.2	1.3	1.1
24	6	144.0	143.8	110.0	09/01/99	0.2	1.0	0.8
24	7	139.0	145.9	149.0	09/01/99	0.3	1.2	0.9
25	1	148.0	150.5	153.7	09/01/99	no flow	0.7	0.7
25	2	145.0	149.9	148.6	09/02/99	0.1	0.4	0.3
25	3	144.0	145.0	138.4	09/02/99	no flow	no flow	0
25	4	139.0	144.0	140.0	09/02/99	0.1	0.2	0.1
25	5	139.0	138.9	114.6	09/02/99	0.1	0.2	0.1
25	6	125.0	120.9	129.0	09/02/99	no flow	no flow	0
26	1	124.0	124.8	127.5	09/02/99	no flow	no flow	0

Power Plant

12	1	50.0	56.0	58.0	08/11/99	< .1	0.1	0.1
13	1	50.0	53.0	51.9	08/11/99	< .1	0.1	0.1
13	2	50.0	51.5	51.0	08/11/99	no flow	< .1	0
13	3	50.0	12.4	12.4	08/11/99	no flow	no flow	0
13	4	50.0	50.5	51.5	08/11/99	0.3	0.9	0.6
13	5	50.0	51.5	48.8	08/11/99	no flow	no flow	0
14	1	53.0	52.8	53.0	08/11/99	< .1	0.8	0.8
14	2	50.0	50.4	51.4	08/10/99	< .1	0.5	0.5
14	3	51.0	52.0	40.0	08/10/99	no flow	0.4	0.4
14	4	52.0	52.0	53.0	08/10/99	0.2	1.0	0.8
15	1	51.0	51.6	53.0	08/10/99	no flow	0.9	0.9
15	2	53.0	52.0	49.8	08/10/99	0.3	1.1	0.8
15	3	45.0	45.0	46.8	08/10/99	0.4	1.2	0.8
15	4	50.0	49.4	51.7	08/10/99	0.2	0.7	0.5
15	5	50.0	50.4	52.3	08/10/99	no flow	no flow	0
16	1	51.0	51.0	52.3	08/09/99	no flow	no flow	0
16	2	52.0	50.8	52.3	08/09/99	no flow	no flow	0
16	3	51.0	51.0	52.0	08/09/99	no flow	no flow	0
16	4	52.0	52.0	52.7	08/09/99	no flow	no flow	0
16	5	52.0	53.0	53.2	08/09/99	no flow	no flow	0

TABLE 2-A

Yellowtail Dam - Weir Readings

Location of Weir: Block 7
Weir Number: WV95B7 (V)

1998				1999		
Date	Reservoir	Flow		Date	Reservoir	Flow
	Elevation	(GPM)			Elevation	(GPM)
				01/25/99	3622.94	10.00
				02/16/99	3615.14	8.49
				03/09/99	3610.55	7.15
04/08/98	3619.91	8.1		04/27/99	3605.10	7.15
05/18/98	3615.53	20.0		05/26/99	3616.89	8.49
06/17/98	3627.23	8.5		06/15/99	3639.17	11.85
07/14/98	3642.12	10.1		07/28/99	3644.57	10.06
08/31/98	3638.58	10.0		08/08/99	3642.01	10.06
09/24/98	3638.60	10.0		09/21/99	3638.93	13.75
10/27/98	3636.47	10.0		10/19/99	3636.75	10.06
11/23/98	3635.20	10.0		11/03/99	3635.13	10.06
12/08/98	3634.82	10.0				

Notes:
Weir is a 90 degree "V" weir

TABLE 2-B

Yellowtail Dam - Weir Readings

Location of Weir: Block 9
Weir Number: WV52B9 (V)

1998				1999		
Date	Reservoir Elevation	Flow (GPM)		Date	Reservoir Elevation	Flow (GPM)
				01/25/99	3622.94	29.20
				02/16/99	3615.14	32.31
				03/09/99	3610.55	26.03
04/08/98	3619.91	29.2		04/27/99	3605.10	26.03
05/18/98	3615.53	26.0		05/26/99	3616.89	26.03
06/17/98	3627.23	29.2		06/15/99	3639.17	32.31
07/14/98	3642.12	32.3		07/28/99	3644.57	32.31
08/31/98	3638.58	29.2		08/08/99	3642.01	29.17
09/24/98	3638.60	26.0		09/21/99	3638.93	32.31
10/27/98	3636.47	26.0		10/19/99	3636.75	35.90
11/23/98	3635.20	32.3		11/03/99	3635.13	29.17
12/08/98	3634.82	29.2				

Notes:
Weir is a 90 degree "V" weir
Weir reads Block 8

TABLE 2-C

Yellowtail Dam - Weir Readings

Location of Weir: Block 12
Weir Number: WV62B12A (V)

1998 Date	Reservoir Elevation	Flow (GPM)		1999 Date	Reservoir Elevation	Flow (GPM)
				01/25/99	3622.94	0.500
				02/16/99	3615.14	0.335
				03/09/99	3610.55	0.335
04/08/98	3619.91	Dry		04/27/99	3605.10	Tr
05/18/98	3615.53	Tr		05/26/99	3616.89	Tr
06/17/98	3627.23	Tr		06/15/99	3639.17	0.335
07/14/98	3642.12	0.18		07/28/99	3644.57	0.223
08/31/98	3638.58	0.22		08/08/99	3642.01	Tr
09/24/98	3638.60	0.22		09/21/99	3638.93	0.335
10/27/98	3636.47	0.34		10/19/99	3636.75	0.670
11/23/98	3635.20	1.60		11/03/99	3635.13	0.223
12/08/98	3634.82	1.60				

Notes:
Weir is a 90 degree "V" weir
Weir is left looking downstream
Tr = Trace

TABLE 2-D

Yellowtail Dam - Weir Readings

Location of Weir: Block 12
Weir Number: WV62B12B (V)

1998				1999		
Date	Reservoir	Flow		Date	Reservoir	Flow
	Elevation	(GPM)			Elevation	(GPM)
				01/25/99	3622.94	0.500
				02/16/99	3615.14	0.335
				03/09/99	3610.55	0.335
04/08/98	3619.91	Dry		04/27/99	3605.10	Dry
05/18/98	3615.53	Tr		05/26/99	3616.89	Tr
06/17/98	3627.23	Tr		06/15/99	3639.17	1.560
07/14/98	3642.12	0.340		07/28/99	3644.57	1.000
08/31/98	3638.58	Tr		08/08/99	3642.01	0.335
09/24/98	3638.60	0.220		09/21/99	3638.93	0.335
10/27/98	3636.47	1.000		10/19/99	3636.75	1.560
11/23/98	3635.20	1.600		11/03/99	3635.13	0.670
12/08/98	3634.82	1.500				

<u>Notes:</u>
Weir is a 90 degree "V" weir
Weir is right looking downstream
Tr = Trace

TABLE 2-E

Yellowtail Dam - Weir Readings

Location of Weir: Block 12
Weir Number: WV55B12A (R)

1998				1999		
Date	Reservoir	Flow		Date	Reservoir	Flow
	Elevation	(GPM)			Elevation	(GPM)
				01/25/99	3622.94	58.3
				02/16/99	3615.14	58.3
				03/09/99	3610.55	58.3
04/08/98	3619.91	58.3		04/27/99	3605.10	52.3
05/18/98	3615.53	58.3		05/26/99	3616.89	52.3
06/17/98	3627.23	58.3		06/15/99	3639.17	64.6
07/14/98	3642.12	71.1		07/28/99	3644.57	71.1
08/31/98	3638.58	58.3		08/08/99	3642.01	64.6
09/24/98	3638.60	58.3		09/21/99	3638.93	71.1
10/27/98	3636.47	58.3		10/19/99	3636.75	71.1
11/23/98	3635.20	64.6		11/03/99	3635.13	64.6
12/08/98	3634.82	71.1				

Notes:
Weir is rectangular weir
Weir is left looking downstream

TABLE 2-F

Yellowtail Dam - Weir Readings

Location of Weir: Block 12
Weir Number: WV55B12B (R)

1998				1999		
Date	Reservoir	Flow		Date	Reservoir	Flow
	Elevation	(GPM)			Elevation	(GPM)
				01/25/99	3622.94	148.0
				02/16/99	3615.14	139.4
				03/09/99	3610.55	148.0
04/08/98	3619.91	174.9		04/27/99	3605.10	139.4
05/18/98	3615.53	156.8		05/26/99	3616.89	148.0
06/17/98	3627.23	184.2		06/15/99	3639.17	243.8
07/14/98	3642.12	223.2		07/28/99	3644.57	233.5
08/31/98	3638.58	174.9		08/08/99	3642.01	223.3
09/24/98	3638.60	174.9		09/21/99	3638.93	174.9
10/27/98	3636.47	156.8		10/19/99	3636.75	174.9
11/23/98	3635.20	174.9		11/03/99	3635.13	174.9
12/08/98	3634.82	165.8				

Notes:
Weir is a rectangular weir
Weir is right looking downstream

TABLE 2-G

Yellowtail Dam - Weir Readings

Location of Weir: Block 20
Weir Number: WV30B20 (V)

1998 Date	Reservoir Elevation	Flow (GPM)		1999 Date	Reservoir Elevation	Flow (GPM)
				01/25/99	3622.94	56.10
				02/16/99	3615.14	47.57
				03/09/99	3610.55	47.57
04/08/98	3619.91	71.40		04/27/99	3605.10	65.97
05/18/98	3615.53	61.04		05/26/99	3616.89	56.10
06/17/98	3627.23	76.70		06/15/99	3639.17	114.89
07/14/98	3642.12	88.41		07/28/99	3644.57	88.41
08/31/98	3638.58	76.70		08/08/99	3642.01	82.58
09/24/98	3638.60	71.40		09/21/99	3638.93	71.36
10/27/98	3636.47	56.10		10/19/99	3636.75	56.10
11/23/98	3635.20	61.00		11/03/99	3635.13	56.10
12/08/98	3634.82	56.10				

Notes:
Weir is a 90 degree "V" weir

TABLE 3-A

Yellowtail Dam
Uplift Pressure Readings

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
4/8/98	3619.91	Line 1 - Block 7	A	3370.9	102
			B	3371.8	60
5/18/98	3615.53	Line 1 - Block 7	A	3370.9	100
			B	3371.8	25
6/17/98	3627.23	Line 1 - Block 7	A	3370.9	103
			B	3371.8	60
7/14/98	3642.12	Line 1 - Block 7	A	3370.9	102
			B	3371.8	55
8/26/98	3639.42	Line 1 - Block 7	A	3370.9	101
			B	3371.8	25
8/31/98	3638.59	Line 1 - Block 7	A	3370.9	101
			B	3371.8	25
9/24/98	3638.60	Line 1 - Block 7	A	3370.9	100
			B	3371.8	60
10/27/98	3636.47	Line 1 - Block 7	A	3370.9	100
			B	3371.8	58
11/24/98	3635.25	Line 1 - Block 7	A	3370.9	0
			B	3371.8	3
12/8/98	3634.82	Line 1 - Block 7	A	3370.9	85
			B	3371.8	52
1/25/99	3622.94	Line 1 - Block 7	A	3370.9	82
			B	3371.8	50
2/16/99	3615.14	Line 1 - Block 7	A	3370.9	81
			B	3371.8	47

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
3/9/99	3610.55	Line 1 - Block 7	A	3370.9	80
			B	3371.8	46
4/27/99	3605.10	Line 1 - Block 7	A	3370.9	70
			B	3371.8	42
5/26/99	3616.89	Line 1 - Block 7	A	3370.9	80
			B	3371.8	40
6/10/99 *	3637.41	Line 1 - Block 7	A	3370.9	92
			B	3371.8	53
6/15/99	3639.79	Line 1 - Block 7	A	3370.9	100
			B	3371.8	55
7/9/99 *	3649.10	Line 1 - Block 7	A	3370.9	99
			B	3371.8	55
7/28/99	3644.57	Line 1 - Block 7	A	3370.9	100
			B	3371.8	55
8/4/99	3642.01	Line 1 - Block 7	A	3370.9	90
			B	3371.8	55
8/18/99 *	3640.22	Line 1 - Block 7	A	3370.9	92
			B	3371.8	53
8/24/99 *	3639.29	Line 1 - Block 7	A	3370.9	90
			B	3371.8	51
9/15/99 *	3639.11	Line 1 - Block 7	A	3370.9	65
			B	3371.8	29
9/21/99	3638.93	Line 1 - Block 7	A	3370.9	57
			B	3371.8	29
10/19/99	3636.70	Line 1 - Block 7	A	3370.9	55
			B	3371.8	30
11/3/99	3635.13	Line 1 - Block 7	A	3370.9	60
			B	3371.8	25

Notes: Pipes are lettered in upstream to downstream direction.
No reading data available from January 1997 to April 1998.
Dates marked with * = readings conducted by Technician or Driller during the cleaning time frame. All other readings conducted by power plant personnel.

TABLE 3-B

Yellowtail Dam
Uplift Pressure Readings

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
4/8/98	3619.91	Line 2 - Block 12	A	3158.5	150
			B	3158.5	210
			C	3158.5	70
			D	3158.5	34
			E	3158.5	50
5/18/98	3615.53	Line 2 - Block 12	A	3158.5	150
			B	3158.5	200
			C	3158.5	60
			D	3158.5	30
			E	3158.5	50
6/17/98	3627.23	Line 2 - Block 12	A	3158.5	150
			B	3158.5	228
			C	3158.5	68
			D	3158.5	34
			E	3158.5	50
7/14/98	3642.12	Line 2 - Block 12	A	3158.5	142
			B	3158.5	250
			C	3158.5	90
			D	3158.5	40
			E	3158.5	55
8/26/98	3639.42	Line 2 - Block 12	A	3158.5	10
			B	3158.5	95
			C	3158.5	30
			D	3158.5	15
			E	3158.5	22

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
8/31/98	3638.59	Line 2 - Block 12	A	3158.5	10
			B	3158.5	95
			C	3158.5	30
			D	3158.5	15
			E	3158.5	22
9/24/98	3638.60	Line 2 - Block 12	A	3158.5	5
			B	3158.5	90
			C	3158.5	30
			D	3158.5	15
			E	3158.5	50
10/27/98	3636.47	Line 2 - Block 12	A	3158.5	60
			B	3158.5	210
			C	3158.5	62
			D	3158.5	32
			E	3158.5	52
11/24/98	3635.25	Line 2 - Block 12	A	3158.5	390
			B	3158.5	215
			C	3158.5	68
			D	3158.5	43
			E	3158.5	42
12/8/98	3634.82	Line 2 - Block 12	A	3158.5	390
			B	3158.5	220
			C	3158.5	70
			D	3158.5	43
			E	3158.5	46
1/25/99	3622.94	Line 2 - Block 12	A	3158.5	380
			B	3158.5	208
			C	3158.5	60
			D	3158.5	42
			E	3158.5	45

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
2/16/99	3615.14	Line 2 - Block 12	A	3158.5	350
			B	3158.5	194
			C	3158.5	55
			D	3158.5	40
			E	3158.5	45
3/9/99	3610.55	Line 2 - Block 12	A	3158.5	370
			B	3158.5	189
			C	3158.5	53
			D	3158.5	40
			E	3158.5	43
4/27/99	3605.10	Line 2 - Block 12	A	3158.5	360
			B	3158.5	180
			C	3158.5	50
			D	3158.5	37
			E	3158.5	43
5/26/99	3616.89	Line 2 - Block 12	A	3158.5	378
			B	3158.5	200
			C	3158.5	56
			D	3158.5	40
			E	3158.5	45
6/10/99 *	3637.41	Line 2 - Block 12	A	3158.5	390
			B	3158.5	259
			C	3158.5	92
			D	3158.5	46
			E	3158.5	51
6/15/99	3639.79	Line 2 - Block 12	A	3158.5	388
			B	3158.5	260
			C	3158.5	94
			D	3158.5	47
			E	3158.5	50

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
6/26/99 *	3646.44	Line 2 - Block 12	A	3158.5	434
			B	3158.5	300
			C	3158.5	99
			D	3158.5	49
			E	3158.5	55
6/27/99 *	3646.92	Line 2 - Block 12	A	3158.5	386
			B	3158.5	240
			C	3158.5	67
			D	3158.5	42
			E	3158.5	55
7/9/99 *	3649.10	Line 2 - Block 12	A	3158.5	388
			B	3158.5	243
			C	3158.5	67
			D	3158.5	40
			E	3158.5	53
7/28/99	3644.57	Line 2 - Block 12	A	3158.5	397
			B	3158.5	225
			C	3158.5	55
			D	3158.5	35
			E	3158.5	45
8/4/99	3642.01	Line 2 - Block 12	A	3158.5	390
			B	3158.5	212
			C	3158.5	47
			D	3158.5	30
			E	3158.5	42
8/18/99 *	3640.22	Line 2 - Block 12	A	3158.5	397
			B	3158.5	201
			C	3158.5	39
			D	3158.5	30
			E	3158.5	39

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
9/15/99 *	3639.11	Line 2 - Block 12	A	3158.5	397
			B	3158.5	194
			C	3158.5	37
			D	3158.5	30
			E	3158.5	39
9/21/99	3638.93	Line 2 - Block 12	A	3158.5	400
			B	3158.5	190
			C	3158.5	30
			D	3158.5	30
			E	3158.5	37
10/19/99	3636.70	Line 2 - Block 12	A	3158.5	395
			B	3158.5	190
			C	3158.5	35
			D	3158.5	30
			E	3158.5	37
11/3/99	3635.13	Line 2 - Block 12	A	3158.5	390
			B	3158.5	190
			C	3158.5	35
			D	3158.5	30
			E	3158.5	35

Notes: Pipes are numbered in upstream to downstream direction.
No reading data available from January 1997 to April 1998.
Dates marked with * = readings conducted by Technician or Driller during the cleaning time frame. All other readings conducted by power plant personnel.

TABLE 3-C

Yellowtail Dam
Uplift Pressure Readings

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
4/8/98	3619.91	Line 3 - Block 15	A	3158.5	400
			B	3158.5	145
5/18/98	3615.53	Line 3 - Block 15	A	3158.5	390
			B	3158.5	135
6/17/98	3627.23	Line 3 - Block 15	A	3158.5	400
			B	3158.5	150
7/14/98	3642.12	Line 3 - Block 15	A	3158.5	400
			B	3158.5	150
8/26/98	3639.42	Line 3 - Block 15	A	3158.5	400
			B	3158.5	145
8/31/98	3638.59	Line 3 - Block 15	A	3158.5	400
			B	3158.5	145
9/24/98	3638.60	Line 3 - Block 15	A	3158.5	400
			B	3158.5	135
10/27/98	3636.47	Line 3 - Block 15	A	3158.5	400
			B	3158.5	132
11/24/98	3635.25	Line 3 - Block 15	A	3158.5	370
			B	3158.5	150
12/8/98	3634.82	Line 3 - Block 15	A	3158.5	370
			B	3158.5	160
1/25/99	3622.94	Line 3 - Block 15	A	3158.5	360
			B	3158.5	160
2/16/99	3615.14	Line 3 - Block 15	A	3158.5	355
			B	3158.5	137
3/9/99	3610.55	Line 3 - Block 15	A	3158.5	350
			B	3158.5	130

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
4/27/99	3605.10	Line 3 - Block 15	A	3158.5	340
			B	3158.5	125
5/26/99	3616.89	Line 3 - Block 15	A	3158.5	350
			B	3158.5	138
6/10/99 *	3637.41	Line 3 - Block 15	A	3158.5	363
			B	3158.5	155
6/15/99	3639.79	Line 3 - Block 15	A	3158.5	0 (see notes)
			B	3158.5	133
7/9/99 *	3649.10	Line 3 - Block 15	A	3158.5	370
			B	3158.5	155
7/28/99	3644.57	Line 3 - Block 15	A	3158.5	360
			B	3158.5	152
8/4/99	3642.01	Line 3 - Block 15	A	3158.5	368
			B	3158.5	150
8/18/99 *	3640.22	Line 3 - Block 15	A	3158.5	370
			B	3158.5	134
9/15/99 *	3639.11	Line 3 - Block 15	A	3158.5	360
			B	3158.5	122
9/21/99	3638.93	Line 3 - Block 15	A	3158.5	360
			B	3158.5	120
10/19/99	3636.70	Line 3 - Block 15	A	3158.5	355
			B	3158.5	120
11/3/99	3635.13	Line 3 - Block 15	A	3158.5	353
			B	3158.5	120

Notes: Pipes are lettered in upstream to downstream direction.
* No reading data available from January 1997 to April 1998.

Reading on pipe A for 6/15/99 is not accurate. PVC was damaged on that day and repaired a few days later.

Dates marked with * = readings conducted by Technician or Driller during the cleaning time frame. All other readings conducted by power plant personnel.

TABLE 3-D

Yellowtail Dam
Uplift Pressure Readings

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
1/22/97	3618.89	Line 4 - Block 18	C	3184.8	55
			D	3184.8	10
2/18/97	3609.66	Line 4 - Block 18	C	3184.8	51
			D	3184.8	10
3/17/97	3604.95	Line 4 - Block 18	C	3184.8	52
			D	3184.8	10
4/10/97	3604.49	Line 4 - Block 18	C	3184.8	52
			D	3184.8	10
5/20/97	3599.20	Line 4 - Block 18	C	3184.8	20.5
			D	3184.8	8
6/25/97	3645.03	Line 4 - Block 18	C	3184.8	66
			D	3184.8	17
7/8/97	3650.43	Line 4 - Block 18	C	3184.8	75
			D	3184.8	16
7/15/97	3651.68	Line 4 - Block 18	C	3184.8	75
			D	3184.8	16
7/24/97	3650.35	Line 4 - Block 18	C	3184.8	76
			D	3184.8	18
8/7/97	3648.04	Line 4 - Block 18	C	3184.8	76
			D	3184.8	16.5
8/14/97	3646.56	Line 4 - Block 18	C	3184.8	70
			D	3184.8	16
9/16/97	3637.01	Line 4 - Block 18	C	3184.8	56
			D	3184.8	11
10/20/97	3637.54	Line 4 - Block 18	C	3184.8	56
			D	3184.8	9

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
11/19/97	3633.93	Line 4 - Block 18	C	3184.8	55
			D	3184.8	10
12/15/97	3632.89	Line 4 - Block 18	C	3184.8	58
			D	3184.8	10
1/15/98	3630.43	Line 4 - Block 18	C	3184.8	60
			D	3184.8	10
2/4/98	3627.23	Line 4 - Block 18	C	3184.8	60
			D	3184.8	10
3/9/98	3618.88	Line 4 - Block 18	C	3184.8	25
			D	3184.8	10
Note: From January 1997 to April 1998 - no reading available for pipes A & B					
4/8/98	3619.91	Line 4 - Block 18	A	3188.5	272
			B	3188.5	110
			C	3184.8	56
			D	3184.8	10
5/18/98	3615.53	Line 4 - Block 18	A	3188.5	262
			B	3188.5	105
			C	3184.8	57
			D	3184.8	10
6/17/98	3627.23	Line 4 - Block 18	A	3188.5	278
			B	3188.5	117
			C	3184.8	57
			D	3184.8	10
7/14/98	3642.12	Line 4 - Block 18	A	3188.5	282
			B	3188.5	55
			C	3184.8	65
			D	3184.8	12

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
8/26/98	3639.42	Line 4 - Block 18	A	3188.5	290
			B	3188.5	120
			C	3184.8	58
			D	3184.8	11
8/31/98	3638.59	Line 4 - Block 18	A	3188.5	290
			B	3188.5	120
			C	3184.8	58
			D	3184.8	11
9/24/98	3638.60	Line 4 - Block 18	A	3188.5	290
			B	3188.5	120
			C	3184.8	50
			D	3184.8	8
10/27/98	3636.47	Line 4 - Block 18	A	3188.5	280
			B	3188.5	120
			C	3184.8	50
			D	3184.8	8
11/24/98	3635.25	Line 4 - Block 18	A	3188.5	258
			B	3188.5	130
			C	3184.8	46
			D	3184.8	10
12/8/98	3634.82	Line 4 - Block 18	A	3188.5	270
			B	3188.5	130
			C	3184.8	50
			D	3184.8	15
1/25/99	3622.94	Line 4 - Block 18	A	3188.5	270
			B	3188.5	125
			C	3184.8	49
			D	3184.8	15

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
2/16/99	3615.14	Line 4 - Block 18	A	3188.5	255
			B	3188.5	120
			C	3184.8	45
			D	3184.8	15
3/9/99	3610.55	Line 4 - Block 18	A	3188.5	250
			B	3188.5	115
			C	3184.8	45
			D	3184.8	14
4/27/99	3605.10	Line 4 - Block 18	A	3188.5	240
			B	3188.5	110
			C	3184.8	42
			D	3184.8	13
5/26/99	3616.89	Line 4 - Block 18	A	3188.5	245
			B	3188.5	112
			C	3184.8	44
			D	3184.8	10
6/10/99 *	3637.41	Line 4 - Block 18	A	3188.5	277
			B	3188.5	132
6/15/99	3639.79	Line 4 - Block 18	A	3188.5	280
			B	3188.5	133
			C	3184.8	52
			D	3184.8	12
7/9/99 *	3649.10	Line 4 - Block 18	A	3188.5	284
			B	3188.5	157
7/28/99	3644.57	Line 4 - Block 18	A	3188.5	275
			B	3188.5	130
			C	3184.8	45
			D	3184.8	10

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
8/4/99	3642.01	Line 4 - Block 18	A	3188.5	272
			B	3188.5	115
			C	3184.8	43
			D	3184.8	7
8/18/99 *	3640.22	Line 4 - Block 18	A	3188.5	266
			B	3188.5	122
9/15/99 *	3639.11	Line 4 - Block 18	A	3188.5	263
			B	3188.5	122
9/21/99	3638.93	Line 4 - Block 18	A	3188.5	260
			B	3188.5	120
			C	3184.8	40
			D	3184.8	8
10/19/99	3636.70	Line 4 - Block 18	A	3188.5	260
			B	3188.5	120
			C	3184.8	37
			D	3184.8	6
11/3/99	3635.13	Line 4 - Block 18	A	3188.5	260
			B	3188.5	120
			C	3184.8	38
			D	3184.8	10

Note: Pipes are numbered in upstream to downstream direction.
No reading available for pipes A & B from January 1997 to April 1998.

Dates marked with * = readings conducted by Technician or Driller during the cleaning time frame. All other readings conducted by power plant personnel. No reading for pipes C & D on these dates.

TABLE 3-E

Yellowtail Dam
Uplift Pressure Readings

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
4/8/98	3619.91	Line 5 - Block 22	A	3346.0	68
			B	3346.0	17
5/18/98	3615.53	Line 5 - Block 22	A	3346.0	150
			B	3346.0	17
6/17/98	3627.23	Line 5 - Block 22	A	3346.0	160
			B	3346.0	17
7/14/98	3642.12	Line 5 - Block 22	A	3346.0	180
			B	3346.0	17
8/26/98	3639.42	Line 5 - Block 22	A	3346.0	160
			B	3346.0	17
8/31/98	3638.59	Line 5 - Block 22	A	3346.0	160
			B	3346.0	17
9/24/98	3638.60	Line 5 - Block 22	A	3346.0	150
			B	3346.0	17
10/27/98	3636.47	Line 5 - Block 22	A	3346.0	140
			B	3346.0	17
11/24/98	3635.25	Line 5 - Block 22	A	3346.0	153
			B	3346.0	0
12/8/98	3634.82	Line 5 - Block 22	A	3346.0	160
			B	3346.0	0
1/25/99	3622.94	Line 5 - Block 22	A	3346.0	160
			B	3346.0	0
2/16/99	3615.14	Line 5 - Block 22	A	3346.0	150
			B	3346.0	0

Date	Reservoir Elevation	Location of Gauge	Pipe	Elevation of Gauge	Pressure Head in Feet
3/9/99	3610.55	Line 5 - Block 22	A	3346.0	140
			B	3346.0	0
4/27/99	3605.10	Line 5 - Block 22	A	3346.0	130
			B	3346.0	0
5/26/99	3616.89	Line 5 - Block 22	A	3346.0	150
			B	3346.0	0
6/10/99 *	3637.41	Line 5 - Block 22	A	3346.0	196
			B	3346.0	5
6/15/99	3639.79	Line 5 - Block 22	A	3346.0	200
			B	3346.0	5
7/9/99 *	3649.10	Line 5 - Block 22	A	3346.0	208
			B	3346.0	9
7/28/99	3644.57	Line 5 - Block 22	A	3346.0	185
			B	3346.0	0
8/4/99	3642.01	Line 5 - Block 22	A	3346.0	170
			B	3346.0	0
8/18/99 *	3640.22	Line 5 - Block 22	A	3346.0	152
			B	3346.0	0
9/15/99 *	3639.11	Line 5 - Block 22	A	3346.0	171
			B	3346.0	0
9/21/99	3638.93	Line 5 - Block 22	A	3346.0	170
			B	3346.0	0
10/19/99	3636.70	Line 5 - Block 22	A	3346.0	170
			B	3346.0	0
11/3/99	3635.13	Line 5 - Block 22	A	3346.0	165
			B	3346.0	0

Notes: Pipes are lettered in upstream to downstream direction.

No reading data available from January 1997 to April 1998.

Dates marked with * = readings conducted by Technician or Driller during the cleaning time frame. All other readings conducted by power plant personnel.

TABLE 4

Yellowtail Dam
Right Abutment Landslide Tunnel
Drain Holes and Probed Depths

Hole Number	1998 Probed Depth (ft.)	Hole Number	1998 Probed Depth (ft.)	Hole Number	1998 Probed Depth (ft.)
D-1	148.2	D-36	5.1	D-71	83.7
D-2	6.9	D-37	100.0	D-72	192.8
D-3	149.6	D-38	173.0	D-73	194.2
D-4	149.5	D-39	7.0	D-74	191.4
D-5	153.5	D-40	152.6	D-75	70.6
D-6	148.9	D-41	98.5	D-76	137.4
D-7	91.3	D-42	22.0	D-77	155.7
D-8	150.2	D-43	32.9	D-78	158.6
D-9	152.3	D-44	17.7	D-79	192.3
D-10	151.2	D-45	179.1	D-80	187.2
D-11	160.7	D-46	183.6	D-81	83.8
D-12	158.3	D-47	184.7	D-82	141.8
D-13	159.3	D-48	188.8	D-83	198.2
D-14	164.5	D-49	187.7	D-84	193.3
D-15	77.5	D-50	188.3	D-85	196.0
D-16	158.3	D-51	187.7	D-86	193.6
D-17	162.6	D-52	188.8	D-87	193.2
D-18	162.9	D-53	189.8	D-88	113.8
D-19	158.1	D-54	141.4	D-89	113.6
D-20	112.2	D-55	185.4	D-90	113.5
D-21	83.2	D-56	185.8	D-91	184.8
D-22	67.7	D-57	113.3	D-92	103.6
D-23	152.2	D-58	27.3	D-93	175.0
D-24	20.2	D-59	180.0	D-94	169.6
D-25	16.8	D-60	169.1	D-95	165.9
D-26	9.0	D-61	85.8	D-96	172.2
D-27	52.2	D-62	183.6	D-97	165.5
D-28	157.8	D-63	188.6	D-98	169.3
D-29	161.2	D-64	183.2	D-99	162.0
D-30	169.4	D-65	139.8	D-100	161.4
D-31	160.0	D-66	197.8	D-101	157.7
D-32	168.6	D-67	121.8	D-102	124.9
D-33	98.6	D-68	107.9	D-103	166.9
D-34	14.3	D-69	100.5		
D-35	155.4	D-70	125.9		

Notes:
References indicate that original depths range from 147 to 197 feet. There are no confirmed individual hole depths.

Holes were renumbered at time of 1998 probing.

TABLE 5

Yellowtail Dam
Left Abutment Foundation Drainage Tunnel
Drain Holes and Probed Depths

Hole Number	Station Number	1970-71 Logs Depth (ft.)	1992 After Drilling Depth (ft.)	1998 Probed Depth (ft.)
98-1	3+12.5	118.6	119.0	118.2
98-2	3+02.5	117.7	116.9	117.2
98-3	2+92.5	115.2	114.3	1.6
98-4	2+82.5	119.9	119.2	119.6
98-5	2+72.5	120.0	120.0	119.9
98-6	2+62.5	118.8	117.3	117.5
98-7	2+52.5	119.6	118.9	119.1
98-8	2+42.5	120.0	119.3	119.5
98-9	2+30	120.0	118.2	34.4
98-10	2+20	119.6	119.5	119.5
98-11	2+11	119.3	118.6	118.8
98-12	2+01	119.0	118.6	118.2
98-13	1+91	120.0	119.5	32.1
98-14	1+81	120.0	119.3	32.0
98-15	1+71	120.0	119.3	119.6
98-16	1+60	120.0	119.2	119.5
98-17	1+50	120.0	119.7	*
98-18	1+40	119.0	118.6	*
98-19	1+30	120.0	120.0	*
98-20	1+20	119.6	119.1	*
98-21	1+11	115.0	95.0	*
98-22	1+00	94.5	95.0	*
98-23	0+90	65.0	66.0	65.6
98-24	0+80	40.3	40.8	40.8
98-25	0+70	15.5	17.2	17.5

Notes:
Holes were renumbered at time of 1998 probing. Hole 98-25 is first hole entering tunnel.

* Holes 98-17 through 98-22 were not probed due to unsanitary conditions caused by rodent infestation in this area of tunnel.

TABLE 6

Yellowtail Dam
Right Abutment Foundation Drainage Tunnel
Drain Holes and Probed Depths

Hole Number	1998 Probed Depth (ft.)
D-1	45.7
D-2	67.0
D-3	74.3
D-4	75.0
D-5	73.9
D-6	74.7
D-7	70.3
D-8	74.7
D-9	72.3
D-10	74.2
D-11	79.5
D-12	77.6
D-13	78.5
D-14	79.3
D-15	79.2
D-16	79.0
D-17	79.0
D-18	79.3
D-19	80.1
D-20	79.7
D-21	79.8
D-22	79.2
D-23	79.6
D-24	77.6
D-25	78.5

Notes:
Holes were renumbered at time of 1998 probing. Hole D-1 is first hole entering tunnel.

Records indicate that the drainage holes were drilled to varying depths ranging from a maximum of 75 feet between stations 0+60 and 1+60, and a maximum depth of 80 feet between stations 1+70 and 2+80. No record was found of the original individual drilled hole depths.

B-2—Drain Cleaning at Canyon Ferry Dam Using High Pressure Water Jetting Equipment.—Regional Reclamation drill crew and geology personnel used high pressure water jetting equipment to clean the foundation drains in Canyon Ferry Dam during 1998 and 1999. References [65] through [69] list geologic and technical data sources for this work.

a. *Reason for Cleaning the Drains.*—Foundation drains at Canyon Ferry Dam were last cleaned in 1972 and 1973. At that time, several drains were partially or solidly plugged. Those drains were cleaned by conventionally drilling the holes to 3 inches in diameter and reaming the lateral discharge or cross pipes to 1 inch in diameter. It was suspected that the drains became plugged because of the 1965 grouting of the contraction joints in the dam.

A *Report of Findings Comprehensive Facility Review for Canyon Ferry Dam*, dated December 1998, recommended that the foundation drains be cleaned every 6 years. The report indicated that there was some evidence that the drains may be plugging. During a precleaning site visit by the Regional Geologist in 1998, it was evident that some of the lateral discharge pipes or cross pipes were partially plugged. In addition, black algae or iron bacteria covered the gallery floor in several areas in the foundation drainage tunnels. Calcium carbonate deposits and other loose debris were present in the foundation gallery drainage ditch.

b. *Description of the Dam* .—Canyon Ferry Dam and Powerplant (fig. 9) along with other appurtenant works, comprise the Canyon Ferry Unit of the Pick-Sloan Missouri Basin Program. The dam is a concrete gravity structure located on the Missouri River in west-central Montana about 17 miles northeast of Helena. It was constructed between 1949 and 1954. The multipurpose Canyon Ferry Unit provides benefits of flood protection, irrigation, power production, municipal and industrial water supply, and recreational opportunities. The dam has a crest length of 1,000 feet and a structural height of 225 feet. It creates the 2,051,000-acre-foot (at elevation 3800.0) Canyon Ferry reservoir, which is approximately 25 miles long and 4 miles wide at the widest point. The spillway is an overflow section in the central portion of the dam. The powerplant, constructed of reinforced concrete, is located on the right downstream toe of the dam adjacent to the spillway apron.

c. *Foundation Geology.*—Bedrock in the foundation of Canyon Ferry Dam consists of Empire Shale in the Precambrian sedimentary unit known as the Belt Series (fig. 10). The shale is a massive, exceedingly fine-grained, dense, hard, and brittle rock which has been classified as hornfels. Because of the brittle nature of the rock, structural deformations have caused it to have a very complex, closely spaced system of joints. Superimposed on these joints are faults and shears with crushed zones, clay-gouge seams, and shattered zones. Two faults crossing the foundation in the areas of blocks 8 and 15 have associated with them

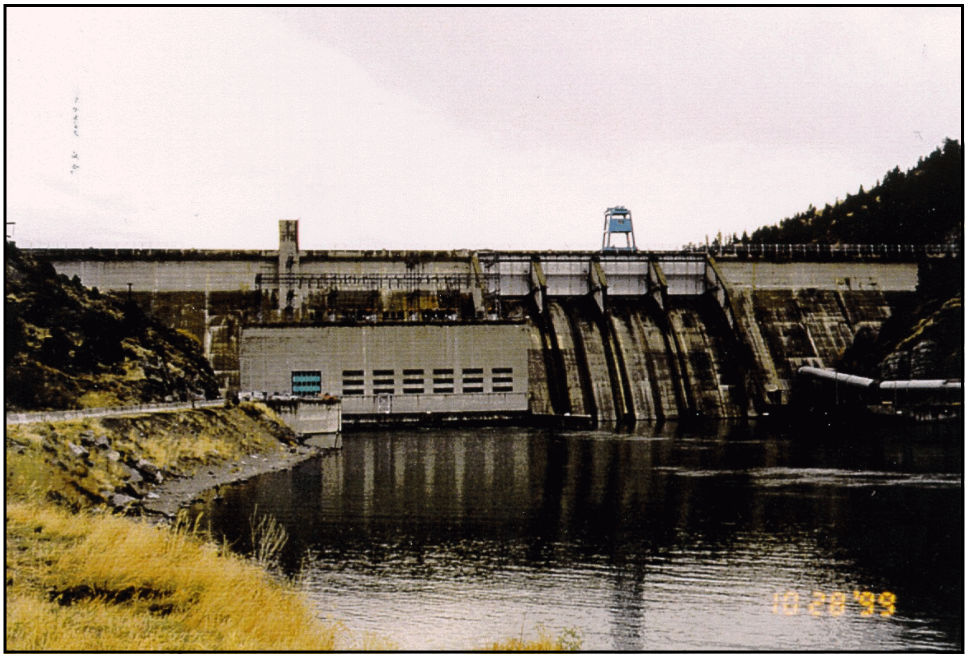


Figure 9.—View of the downstream side of the dam from the access road along the river.

crushed and brecciated zones. Figure 11 shows the bedrock in the left abutment, and Reclamation drawings 296-613-619 and 296-613-620 show the geologic section along the foundation gallery.

d. *Description of the Drainage System*.—Drainage of the foundation beneath the dam consists of a row of holes drilled from the foundation gallery and tunnels into the underlying rock. These holes were drilled through 4-inch diameter pipes embedded in the concrete and are angled downstream at approximately 10 from the vertical. The embedded pipes extend 6 to 12 inches into the underlying rock. The drain holes are spaced on 10-foot centers and were supposed to have a minimum diameter of 3 inches. During the cleaning process, it was discovered that a few holes had been reduced to a smaller size. There is a total of 118 drain holes (90 across the bottom and up each abutment side, 11 in the left abutment foundation tunnel, and 17 in the right abutment foundation tunnel). All hole depths are less than 100 feet. During the cleaning process, it was discovered that one hole was completely plugged with a wooden plug (block 19, hole 68); therefore, this hole was not cleaned. No documentation could be located indicating the reason for this plugged hole.

A nearby drainage ditch in the concrete floor provides a means of passing the drain flows to a centralized sump and discharge pump in the base of the dam (block 12). The drainage

Figure 10.—Empire Shale, classified as hornfels, right abutment downstream from the dam.

ditch (fig. 12) is located along the upstream side of the access tunnel in the foundation. Water is transferred from the drain hole into the drainage ditch through a 2-inch diameter steel lateral discharge pipe or cross pipe embedded in the concrete floor. All drain holes are capped with 4-inch threaded caps (fig. 13). Table 1 lists individual hole numbers and original or probed depths.

e. *Condition of the Drains Before Cleaning.*—Cross pipes were plugged with bacterial sludge. Some pipes had calcium carbonate buildup mostly on the ends of the pipes. A few were completely plugged. Floors and ditches located in the tunnels were stained with a black algae or iron bacteria that gave off an unpleasant odor (fig. 14).

Figure 11.—Empire Shale in the left abutment downstream from the dam.

f. *Description of Cleaning Unit*.—The drains were cleaned using the Great Plains Region's high pressure water jetting equipment. The unit consists of a Butterworth Liquiblaster pump powered by a 215-hp Cummins diesel engine rated to deliver a maximum of 15,000 lb/in^2 at 22 gal/min at the pump discharge. The pressure is regulated using a nitrogen pressure regulator located close to the high pressure pump. Water is delivered, using ½-inch I.D. high pressure supply hose, from the high pressure pump and nitrogen pressure regulator to a foot-operated dump valve and flow control pressure cart. The foot-operated dump valve and flow control pressure cart are located close to the top of the drain hole in the drainage tunnel and regulate flows going into the hole. With the dump valve engaged, the pressurized water is supplied to a ½-inch I.D. flexible lance hose with the cleaning head attached, which is advanced down the drain hole. (Photographs of the high pressure water jetting equipment are provided in app. C of this manual.)

g. *Cleaning Methodology and Procedures*.—The drain cleaning was accomplished in two phases. The first phase, which was initiated and completed in September 1998, consisted of preliminary cleaning of the drain lateral discharge pipes or cross pipes, the tunnel gallery floors, and the drainage ditches. This procedure was accomplished using a small portable electric-powered low pressure washer unit rated at 1,200 lb/in^2 at 2.1 gal/min. Several buckets of debris were shoveled out of the drainage ditch and carried out of the dam

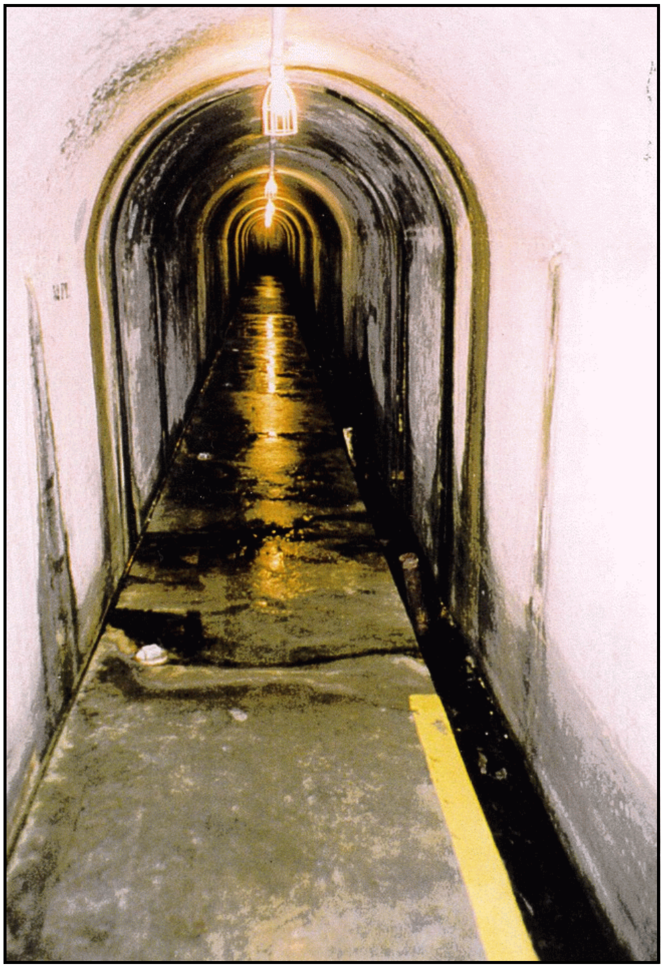


Figure 12.—Drainage ditch located along the upstream side of the access tunnel or gallery. Water is passed from the drain holes into the drainage ditch through a 2-inch steel pipe embedded in the concrete floor. This view looks along the right abutment foundation drainage tunnel.

into a trash dumpster. All drain hole caps were inspected, and approximately 20 of the old caps were replaced.

The second phase of cleaning the drains was initiated and completed in October 1999 using the high pressure water jetting unit. A site visit was made for determining logistics for moving and setting up equipment, water supply, electrical supply, and access to the drainage gallery and tunnels. In addition, all safety issues were discussed.

Figure 13.—Top of a drain hole with a 4-inch threaded cap. The pipe is 4 inches in diameter, and the hole becomes 3 inches upon entering bedrock.

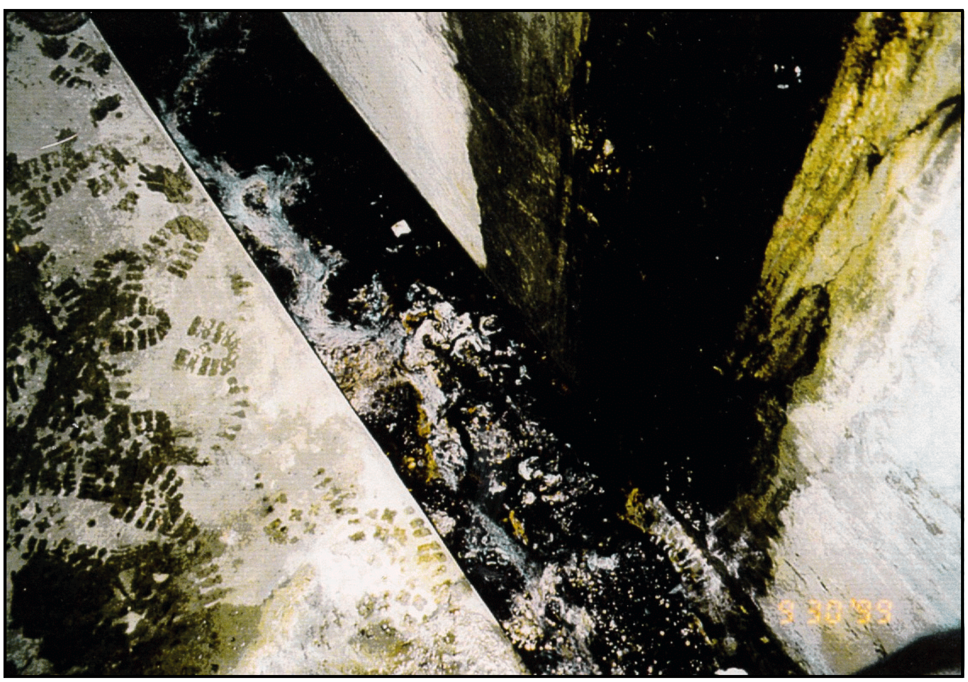


Figure 14.—Drainage ditch and wall stained with black algae or iron bacteria.

Figure 15.—Weir readings are recorded before and after cleaning.

Figure 16.—Uplift pressure gauge readings are recorded before and after cleaning.

It was decided to set up the trailer-mounted high pressure pump unit behind the powerplant at the base of the dam. The water supply to the pump unit was attained from a fire hose water hook-up located in the powerplant. The high pressure supply hoses were laid out from the pump unit through the back door of the powerplant, down to the first level, and into the foundation gallery. The maximum amount of high pressure supply hose used was approximately 900 feet, which reached to the top of the left abutment.

Before the cleaning process began, several drain holes were videotaped, using the Region's down-hole video camera, to determine the condition of the rock and the cleaning pressures to use. To get a good view of the drain hole wall, the holes needed to be flushed. This was accomplished using a regular garden hose and ½-inch PVC pipe.

Individual flow measurements were recorded before cleaning and after cleaning. Table 1 indicates these flow measurements. Weir readings are measured and recorded monthly by the powerplant personnel. Two weirs are located in the foundation gallery drainage gutter on either side of a collection and pumping sump in block 12 (fig. 15). One of the these weirs measures total drain flow from blocks 5 to 12, and the other measures total drain flow from blocks 12 to 22. Table 2 indicates weir readings from January 1997 through January 2000.

Uplift pressure gauge readings (fig. 16) were recorded prior to the cleaning program, and powerplant personnel monitor them monthly. The measuring system consists of four lines of measuring points running upstream-downstream in blocks 8, 11, 15, and 18. On all four lines, the gauge farthest upstream, gauge A, reads uplift pressures on the upstream side of the main grout cutoff curtain and foundation drains; gauge B reads uplift pressures existing downstream from the grout curtain but upstream from the foundation drains; and the remainder of the gauges measure uplift pressures at points downstream from both the grout curtain and foundation drains. Tables 3-A through 3-D indicate uplift pressure readings from January 1997 through January 2000.

After reviewing the foundation geology, it was determined to start the cleaning in block 12, in an area where the rock was more stable. In the more stable rock, the cleaning pressures varied from 5,500 to 7,000 lb/in^2, 18 to 20 gal/min, with a feed rate of 3 to 4 ft/min. In areas of highly sheared and shattered rock, the holes were flushed using 1,000 to 1,500 lb/in^2. Two different types of cleaning heads were used. With the higher pressures, a 2¾-inch O.D. rotating cleaning head with 5 jets was used. Where lower pressures were needed, a smaller ¾-inch nonrotating cleaning head similar to a sewer flushing nozzle was used. This smaller nozzle is capable of providing a maximum of about 2,000 lb/in^2. The drains were cleaned in the following order: block 12 through block 17, right abutment tunnel, left abutment tunnel, blocks 11 through 4 in the left abutment, and finally blocks 19 through 23 in the right abutment. Cleaning pressures were adjusted depending on the condition of rock encountered in each drain hole. The video results indicated that using the lower pressures did not effectively remove all of the calcium carbonate from the holes but did flush the loose debris out. Even the higher pressures of 5,500 to 6,500 lb/in^2 were not able to clean all the calcium carbonate from the cracks but were able to break through any large deposits. To completely remove all hard calcium carbonate, experience indicates that pressures of approximately 9,000 lb/in^2 are necessary. The rock at Canyon Ferry is too fragile to withstand that amount of pressure.

All but 19 drain holes were cleaned or flushed to within 5 feet of the original or probed depth. Table 1 indicates the cleaned depths. Some problems were encountered with those 19 holes. The majority of the problems occurred in various holes toward the right abutment in blocks 15, 16, 17, 19, 20, 21, and 22. Due to the fault zone located in the area of block 15 and 16, the rock is highly sheared and shattered. The shattered rock is cut by two or more very closely spaced sets of joints, and the resulting interlocking rock fragments are often loose in place. When the 2¾-inch cleaning tool entered drain holes in this area, or as it was being pulled out of the holes, some of the loose rocks caved in or sloughed, causing the tool to get stuck. At this point, it was decided to use the smaller nonrotating cleaning head with low pressure, thus mainly flushing the holes. Even with the lower pressures, some caving was encountered in the fault breccia areas. The holes in these areas may need to be redrilled at some future date and cased with slotted PVC to keep them open. Blocks 19 through 22 also produced some problem holes. The cleaning tool would get stuck in areas where the bedrock was shattered and loose rock fragments caved into the holes. The 19 problem holes are indicated on table 1 with the suspected problem noted in the remarks section.

After the cleaning was completed, all gallery floors, stairs, areas around the drain holes, and drainage gutters were washed down using the small portable, electric-powered pressure washer.

h. *Results of the Cleaning.*—

1. *Flow Measurements.*—The individual flow measurements were recorded about 2 hours after each drain was cleaned. It was decided to wait at least 2 hours in order to allow time for the water introduced into the hole, while cleaning, to exit. Because of the low flow rates and the associated difficulty in taking the measurements, the individual flows did not appear to increase in most of the holes. However, the overall flows did increase approximately 500 percent. The weir readings used for this comparison were taken 2 days after the cleaning was initiated (October 5, 1999), and approximately 1 month after the cleaning was completed (December 2, 1999), when the reservoir elevation varied by only 0.01 foot. Table 2 indicates this overall flow increase. As additional readings become available for comparison, the effectiveness of the cleaning program can be better evaluated.

2. *Uplift Pressures*.—Uplift pressure reading comparisons of before cleaning and after cleaning were made using comparable reservoir elevations. For example, pressure readings taken on October 5, 1999 (2 days after the cleaning procedure was initiated) were compared with readings taken on December 2, 1999 (approximately 1 month after completion of the cleaning program). The reservoir elevation was 0.01 foot higher, and yet the pressures in all of the A gauges decreased. However, results were mixed for the downstream gauges. Some decreased, some increased, and some remained the same.

A second comparison was made using the dates of April 9, 1998 (before cleaning) and January 4, 2000 (after cleaning) when the reservoir elevation was 0.06 feet higher. All of the A gauges showed a decrease in pressures ranging from 2 to 10 feet of head, while the downstream gauges decreased in a range from 1 to 8 feet of head.

A third comparison was made using the dates of March 10, 1998 (before cleaning) and October 31, 1999 (after cleaning) when the reservoir was 0.03 foot lower. All of the A gauges decreased in pressures ranging from 4 feet of head in Line 3, block 15 to 15 feet of head in Line 2, block 11. All downstream gauges showed decreases ranging from 1 to 12 feet of head pressure with the exception of gauge B in Line 4, which increased by 1 feet of head. Table 4 shows the dates, reservoir elevations, and uplift pressures used for the above comparisons, and tables 3-A through 3-D list all monthly uplift pressure readings over a 3-year period.

i. *Cost Comparisons*.—Total cost for the 1998 and 1999 cleaning program was approximately $59,000, which calculated to about $9.62 per foot. There were 117 drain holes cleaned totaling 6,134.3 feet. In addition, 117 cross pipes were cleaned. Because of the highly sheared and shattered rock (particularly in the fault zones), the pressures had to be adjusted several times. The cost per foot of this job ended up being significantly higher than the recent Yellowtail Dam cleaning program. This higher cost is attributed to problems encountered with the rock conditions, which required numerous procedural adjustments and more videotaping. However, it was still only about 50 percent of what the cost would have been if completed totally by conventional drilling.

j. *Summary and Recommendations*.—Most of the material blocking the drains at Canyon Ferry Dam appeared to be some type of bacterial deposit that was reasonably easy to remove at lower water pressures. The harder calcium carbonate deposits could not be completely removed by the water pressures allowable because of the poor foundation rock conditions. Therefore, the final results of the drain cleaning operation were somewhat in doubt.

Based on the limited amount of data now available, it appears that the drain cleaning at Canyon Ferry Dam was more successful than first thought. The initial before-and-after individual drain flow measurements did not indicate much difference in after-cleaning readings. This, however, is probably attributable to low reservoir elevations and correspondingly low flows at the time of cleaning. A limited comparison of flows at higher elevations has now been possible. Most of the uplift pressure gauge readings have shown a decrease in pressures, and the weirs measuring collective drain flows have shown a very significant increase in overall quantities.

The cleaning program further confirmed that the high pressure jetting method is not a complete answer to all drain cleaning situations. It can, however, be adjusted and adapted to obtain acceptable results under most circumstances. The overall highly jointed foundation rock and even worse gouge-filled fault zones at Canyon Ferry Dam proved to be a real test for the cleaner.

Approximately 84 percent of the drains were cleaned or flushed in a successful manner to their original or probed depths. Nineteen of the holes produced sloughing or caving problems. They were located in zones containing shattered rock and had very low to no flows both before and after cleaning. These holes are specifically noted on table 1, along with the suspected problem depths. In the future, when it becomes necessary to again clean the other holes, these problem holes in the areas of bad rock should be redrilled using conventional drilling methods and continuously cased with slotted PVC well screen. That will hold the holes open even if the surrounding rock collapses and will make it possible to flush mud or bacterial deposits from the drains using low pressures without damaging the screens. If they become clogged with hard calcium carbonate, it is doubtful if they can be cleaned without destroying the PVC, but at least the useful life of the drain holes can be extended to a maximum limit.

An alternative solution would be to drill new holes between the old drains to replace the problem holes. It is likely, however, that the same bad rock conditions that caused the problems with the original drains, would again cause the new holes to collapse during drilling or the first cleaning. Also, these replacement drains would require the construction of new lateral discharge pipes in the concrete floor of the gallery, which would be quite expensive.

The timing of the next round of drain cleaning should be based mainly on observation of the uplift pressure gauge readings and performance of the weirs. About every 5 years, even if the pressures and weir readings do not indicate a problem, a few representative drains should be inspected with a down-hole video camera to confirm the status of drain deposits. It is much more cost effective to remove sediments while they are soft scale on the walls of holes rather than have to remove hard deposits from completely plugged drains.

TABLE 1

Canyon Ferry Dam
Drain Hole Cleaning Data
(Recorded Depths and Individual Drain Flow Measurements)

Foundation Gallery

Location Block	Location Hole	Original or Probed depth (ft.)	Cleaned depth (ft.)	Date cleaned	Before flow GPM	After flow GPM	Flow increase or decrease GPM	Cleaning Pressure (PSI)	Remarks
4	1	72.0	72.0	10/19/99	no flow	no flow	0	6600	
4	2	72.0	71.0	10/19/99	no flow	no flow	0	6600	
4	3	66.0	66.0	10/19/99	no flow	no flow	0	6600	
4	4	70.0	68.0	10/19/99	no flow	no flow	0	6600	
5	5	73.0	18.5	10/19/99	no flow	no flow	0	6600	rocks fell in at 16 - 18' (strong shear plane)
5	6	72.0	72.0	10/19/99	no flow	no flow	0	6600	
5	7	70.0	70.0	10/19/99	no flow	no flow	0	6600	
5	8	76.0	77.0	10/19/99	no flow	no flow	0	6600	
6	9	86.0	86.0	10/19/99	no flow	no flow	0	6600	
6	10	78.0	80.0	10/18/99	no flow	no flow	0	7000	
6	11	73.0	73.0	10/18/99	no flow	no flow	0	7000	
6	12	76.0	76.0	10/18/99	no flow	no flow	0	7000	
6	13	78.0	78.0	10/18/99	no flow	no flow	0	7000	
7	14	80.0	80.0	10/18/99	no flow	no flow	0	7000	
7	15	76.0	76.0	10/18/99	no flow	no flow	0	7000	
7	16	82.0	82.0	10/18/99	no flow	no flow	0	7000	
7	17	77.0	77.0	10/18/99	no flow	no flow	0	1400	
8	18	70.0	70.0	10/18/99	no flow	no flow	0	1400	
8	19	60.0	60.0	10/18/99	< .1	no flow	0	1400	
8	20	54.0	54.0	10/18/99	no flow	< .1	0	1400	
8	21	52.0	52.0	10/18/99	< .1	< .1	0	1400	
8	22	40.0	40.0	10/18/99	< .1	no flow	0	1400	
9	23	37.0	37.0	10/18/99	0.4	0.3	-0.1	1400	
9	24	33.0	33.0	10/18/99	0.2	0.2	0	1400	
9	25	33.0	33.0	10/18/99	< .1	0.1	0.1	1400	
10	26	35.0	35.0	10/17/99	0.1	0.1	0	5500	
10	27	61.0	61.0	10/17/99	0.2	0.3	0.1	5500	
10	28	66.0	66.0	10/17/99	no flow	no flow	0	5500	
10	29	67.0	67.0	10/17/99	no flow	no flow	0	5500	
10	30	63.0	26.5	10/17/99	2.0	1.9	-0.1	5500	rocks fell in at 23' (strong shear plane)
10	31	64.0	64.0	10/17/99	0.2	0.4	0.2	5500	
11	32	62.0	62.0	10/17/99	0.2	0.3	0.1	5500	
11	33	59.0	59.0	10/17/99	0.1	0.2	0.1	5500	
11	34	60.0	60.0	10/17/99	0.1	0.1	0	5500	
11	35	57.0	57.0	10/17/99	**	**	-	5500	
11	36	61.0	61.0	10/17/99	0.2	0.2	0	5500	
11	37	57.0	57.0	10/17/99	no flow	0.1	0.1	5500	
12	38	57.0	57.0	10/17/99	0.2	0.2	0	5500	
12	39	57.0	57.0	10/03/99	< .1	< .1	0	8000	
12	40	60.0	63.3	10/03/99	no flow	< .1	0	6500	
12	41	61.0	60.2	10/04/99	< .1	< .1	0	6500	
12	42	60.0	61.2	10/04/99	< .1	< .1	0	6500	
12	43	59.0	58.8	10/04/99	no flow	< .1	0	6500	
13	44	57.0	56.4	10/04/99	< .1	< .1	0	6500	
13	45	58.0	58.0	10/04/99	< .1	< .1	0	6500	gas bubbles coming up from bottom of hole
13	46	56.0	55.5	10/05/99	< .1	< .1	0	6500	

TABLE 1

Canyon Ferry Dam
Drain Hole Cleaning Data
(Recorded Depths and Individual Drain Flow Measurements)

Foundation Gallery

Location Block	Location Hole	Original or Probed depth (ft.)	Cleaned depth (ft.)	Date cleaned	Before flow GPM	After flow GPM	Flow increase or decrease GPM	Cleaning Pressure (PSI)	Remarks
13	47	56.0	56	10/05/99	< .1	no flow	0	6500	
13	48	57.0	52.9	10/05/99	no flow	< .1	0	6500	
13	49	57.0	56.2	10/05/99	no flow	< .1	0	6500	
14	50	60.0	60.2	10/05/99	< .1	< .1	0	6500	
14	51	57.0	55.2	10/05/99	< .1	< .1	0	6500	
14	52	63.0	61.7	10/06/99	< .1	no flow	0	6500	
14	53	58.0	52.2	10/06/99	< .1	no flow	0	6500	
14	54	63.0	62.1	10/06/99	< .1	no flow	0	6500	
14	55	85.0	80.2	10/06/99	no flow	< .1	0	6500	
15	56	85.0	80.4	10/06/99	< .1	< .1	0	5500	
15	57	83.0	72.8	10/06/99	no flow	< .1	0	5500	fault gouge and breccia - clay
15	58	83.0	32	10/14/99	< .1	no flow	0	1000	rocks fell in at approx 32' (fault breccia)
15	59	33.0	31.5	10/06/99	0.1	0.1	0	5500	
15	60	66.0	51.5	10/06/99	< .1	no flow	0	5500	rocks fell in at approx 51' (fault breccia)
16	61	63.0	45.8	10/06/99	no flow	no flow	0	5500	rocks fell in at approx 45.8' (loose rock fragments)
16	62	57.0	23.5	10/06/99	0.1	0.1	0	5500	rocks fell in at approx 23.5' - gas bubbles
16	63	58.0	62	10/14/99	*	*	-	1000	
16	64	62.0	14.0	10/14/99	no flow	no flow	0	1000	rocks fell in at 14' (strong shear plane)
16	65	38.0	25.0	10/14/99	no flow	no flow	0	1000	hole caved at 25' - camera could only go to 13'
17	66	72.0	24.0	10/14/99	no flow	no flow	0	5500	rocks fell in at approx. 21'
17	67	29.0	30.0	10/14/99	no flow	0.1	0.1	5500	
17	68	70.0	76.8	10/14/99	< .1	< .1	0	5500	
17	69	52.0	51.5	10/15/99	0.1	< .1	-0.1	7000	
17	70	62.0	61.2	10/15/99	0.1	0.2	0.1	7000	
17	71	60.0	58.9	10/15/99	< .1	0.1	0.1	7000	
19	68	0.0	0.0	-	***	***	-		
19	69	27.0	27.0	10/27/99	no flow	< .1	0	6900	
19	70	27.0	27.0	10/27/99	no flow	no flow	0	6900	
19	71	27.0	28.0	10/27/99	no flow	no flow	0	6900	
19	72	30.0	28.0	10/27/99	no flow	no flow	0	6900	rocks fell in at 28' where concrete meets bedrock
20	73	47.0	46.0	10/27/99	no flow	no flow	0	6900	rocks fell in at 42'
20	74	47.0	25.5	10/27/99	0.1	0.1	0	1500	rocks fell in at 25.5'
20	75	55.0	31.5	10/27/99	< .1	< .1	0	1500	rocks fell in at 31.5'
20	76	61.0	59.5	10/27/99	< .1	no flow	0	6900	
21	77	69.0	67.0	10/27/99	no flow	0.1	0.1	6900	rocks fell in at 67'
21	78	72.0	52.0	10/27/99	< .1	0.1	0.1	6900	rocks fell in at 48'
21	79	56.0	51.0	10/27/99	no flow	no flow	0	1500	
21	80	85.0	65.0	10/27/99	no flow	no flow	0	1500	could not get camera past 27'
21	81	92.0	72.5	10/28/99	no flow	no flow	0	1500	
22	82	89.0	82.0	10/28/99	no flow	no flow	0	1500	rocks falling in on way out of hole
22	83	86.0	76.5	10/28/99	no flow	no flow	0	1500	
22	84	78.0	74.0	10/28/99	no flow	no flow	0	1500	
22	85	83.0	83.0	10/28/99	no flow	no flow	0	1500	
23	86	76.0	76.0	10/28/99	no flow	no flow	0	1500	

TABLE 1

Canyon Ferry Dam
Drain Hole Cleaning Data
(Recorded Depths and Individual Drain Flow Measurements)

Foundation Tunnel - Left Abutment

Location Block	Location Hole	Original or Probed depth (ft.)	Cleaned depth (ft.)	Date cleaned	Before flow GPM	After flow GPM	Flow increase or decrease GPM	Cleaning Pressure (PSI)	Remarks
1	A	22.0	22.0	10/16/99	< .1	no flow	0	5000 *	
1	B	25.0	25.0	10/16/99	< .1	< .1	0	5000	
2	C	28.0	28.0	10/16/99	< .1	< .1	0	5000	
2	D	31.0	31.0	10/16/99	0.1	0.1	0	5000	
3	E	34.0	34.0	10/17/99	< .1	no flow	0	5500	
3	F	37.0	37.0	10/17/99	< .1	< .1	0	5500	
4	G	41.0	41.0	10/17/99	0.1	0.1	0	5500	
4	H	42.0	42.0	10/17/99	0.1	0.1	0	5500	
5	I	44.0	44.0	10/17/99	< .1	< .1	0	5500	
5	J	5.0	5.0	10/17/99	0.2	0.2	0	5500	
6	K	53.0	53.0	10/17/99	0.1	no flow	-0.1	5500	

Foundation Tunnel - Right Abutment

Location Block	Location Hole	Original or Probed depth (ft.)	Cleaned depth (ft.)	Date cleaned	Before flow GPM	After flow GPM	Flow increase or decrease GPM	Cleaning Pressure (PSI)	Remarks
7	L	66.0	65.8	10/15/99	< .1	< .1	0	7000	
7	M	64.0	64.0	10/15/99	< .1	< .1	0	7000	
7	N	65.0	65.0	10/15/99	no flow	no flow	0	7000	
8	O	52.0	52.0	10/15/99	no flow	no flow	0	1600	
8	P	52.0	52.0	10/15/99	no flow	no flow	0	1600	
9	Q	50.0	50.0	10/16/99	**	**	-	1500	
9	R	46.0	46.0	10/16/99	no flow	no flow	0	1500	
10	S	43.0	43.0	10/16/99	< .1	< .1	0	1500	
10	T	41.0	41.0	10/16/99	< .1	< .1	0	1500	
11	U	37.0	36.5	10/16/99	< .1	< .1	0	1500	
11	V	37.0	37.0	10/16/99	< .1	< .1	0	1500	
12	W	34.0	33.5	10/16/99	< .1	no flow	0	1500	
12	X	32.0	32.0	10/16/99	no flow	no flow	0	1500	
13	Y	29.0	29.0	10/16/99	< .1	< .1	0	1500	
13	Z	27.0	27.0	10/16/99	< .1	no flow	0	1500	
14	AA	25.0	25.0	10/16/99	< .1	no flow	0	1500	
14	BB	20.0	20.0	10/16/99	0.1	0.1	0	1500	

* Unable to take reading - could not get packer into hole because of stairs.
** Unable to take reading - packer would not seal in this hole.
*** Hole has been plugged with wood.

TABLE 2

Canyon Ferry Dam
Weir Readings
JANUARY 1997 TO JANUARY 2000

1997		Blocks 5 - 12		Blocks 12 - 22		Total Discharge
Date	Reservoir El.	Flow (GPM)		Flow (GPM)		Flow (GPM)
01/02/97	3788.24	88.414		88.414		176.828
02/04/97	3783.47	23.338		26.030		49.368
03/04/97	3777.01	11.871		10.115		21.986
04/01/97	3771.76	8.524		2.128		10.652
05/20/97	3777.31	3.700		0.012		3.712
06/17/97	3797.99	5.816		0.187		6.003
07/02/97	3797.30	5.816		0.068		5.884
07/31/97	3796.61	7.093		5.816		12.707
09/16/97	3794.74	4.687		3.700		8.387
09/30/97	3794.40	7.093		7.093		14.186
11/13/97	3793.90	18.179		51.612		69.791
12/04/97	3793.74	23.338		61.037		84.375

1998		Blocks 5 - 12		Blocks 12 - 22		Total Discharge
Date	Reservoir El.	Flow (GPM)		Flow (GPM)		Flow (GPM)
01/08/98	3791.61	47.573		107.712		155.285
02/12/98	3789.44	29.172		71.359		100.531
03/10/98	3787.67	20.644		56.100		76.744
04/09/98	3787.04	11.871		20.644		32.515
05/05/98	3785.99	7.093		4.687		11.780
06/02/98	3791.98	5.816		3.700		9.516
07/09/98	3798.88	4.687		5.816		10.503
08/17/98	3794.31	4.687		5.816		10.503
09/10/98	3790.72	3.700		2.128		5.828
10/23/98	3789.45	3.700		7.093		10.793
10/28/98	3790.20	13.796		23.335		37.134
12/07/98	3792.52	56.100		65.974		122.074

1999		Blocks 5 - 12		Blocks 12 - 22		Total Discharge
Date	Reservoir El.	Flow (GPM)		Flow (GPM)		Flow (GPM)
01/05/99	3790.64	82.579		94.697		177.276
02/01/99	3789.79	61.037		82.579		143.616
02/11/99	3789.27	56.100		77.359		133.459
03/03/99	3787.48	39.494		43.085		82.579
04/06/99	3781.86	11.871		15.897		27.768
05/03/99	3780.20	7.093		5.816		12.909
06/01/99	3784.82	5.816		2.850		8.666
07/01/99	3795.82	8.524		2.128		10.652
08/03/99	3793.77	7.093		1.528		8.621
09/01/99	3790.79	5.816		1.528		7.344
10/05/99	3788.10	2.128		1.528		3.656
11/04/99	3787.56	3.700		7.093		10.793
12/02/99	3788.11	7.093		13.796		20.889

2000		Blocks 5 - 12		Blocks 12 - 22		Total Discharge
Date	Reservoir El.	Flow (GPM)		Flow (GPM)		Flow (GPM)
01/04/00	3787.10	32.314		51.612		83.926

Notes: Readings recorded on October 5, 1999 and December 2, 1999 were used for comparison of overall flow increase.

TABLE 3-A

Canyon Ferry Dam
Uplift Pressure Gauge - Line 1, Block 8, Sta. 3+84

1997						
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**
01/02/97	3788.24	78	10	0	0	0
02/04/97	3783.47	76	10	0	0	0
03/04/97	3777.01	70	10	0	0	0
04/01/97	3771.76	64	8	0	0	0
05/20/97	3777.31	64	8	0	0	0
06/17/97	3797.99	82	10	0	0	0
07/02/97	3797.30	84	10	0	0	0
07/31/97	3796.61	84	10	0	0	0
09/16/97	3794.74	84	10	0	0	0
09/30/97	3794.40	83	10	0	0	0
11/13/97	3793.90	81	10	0	0	0
12/04/97	3793.74	82	10	0	0	0

1998						
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**
01/05/98	3790.64	78	8	0	0	0
01/08/98	3791.61	82	9	0	0	0
02/12/98	3789.44	80	10	0	0	0
03/10/98	3787.67	80	10	0	0	0
04/09/98	3787.04	78	10	0	0	0
05/05/98	3785.99	76	10	0	0	0
06/02/98	3791.98	79	10	0	0	0
07/09/98	3798.88	84	10	0	0	0
08/17/98	3794.31	82	10	0	0	0
09/10/98	3790.72	78	8	0	0	0
10/23/98	3789.45	77	8	0	0	0
10/28/98	3790.20	76	8	0	0	0
12/07/98	3792.52	78	8	0	0	0

1999						
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**
02/01/99	3789.79	78	8	0	0	0
02/11/99	3789.27	78	8	0	0	0
03/03/99	3787.48	78	8	0	0	0
04/06/99	3781.86	72	8	0	0	0
05/03/99	3780.20	70	8	0	0	0
06/01/99	3784.82	70	8	0	0	0
07/01/99	3795.82	80	10	0	0	0
08/03/99	3793.77	80	9	0	0	0
09/01/99	3790.79	78	8	0	0	0
* 09/29/99	3788.76	78	8	0	0	0
10/05/99	3788.10	78	4	0	0	0
* 10/29/99	3787.57	75	7	0	0	0
* 10/31/99	3787.64	75	4	0	0	0
11/04/99	3787.56	76	8	0	0	0
12/02/99	3788.11	76	7	0	0	0

2000						
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**
01/04/00	3787.10	76	7	0	0	0

Notes: Pipes are lettered in upstream to downstream direction (See Drawing 296-D-347).
* Denotes uplift pressure gauge reading done by drill crew or technician during cleaning time frame.
All other readings performed by powerplant personnel.

TABLE 3-B

Canyon Ferry Dam
Uplift Pressure Gauge - Line 2, Block 11, Sta. 5+70

1997									
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**	**Pipe F**	**Pipe G**
01/02/97	3788.24	124	32	12	14	14	30	32
02/04/97	3783.47	124	32	14	14	15	30	32
03/04/97	3777.01	118	30	12	14	14	30	32
04/01/97	3771.76	112	28	12	14	14	30	32
05/20/97	3777.31	112	26	12	14	14	30	32
06/17/97	3797.99	128	30	14	14	14	26	30
07/02/97	3797.30	128	30	14	14	14	28	29
07/31/97	3796.61	124	28	14	14	14	28	30
09/16/97	3794.74	118	28	14	14	14	28	30
09/30/97	3794.40	118	28	13	14	14	28	30
11/13/97	3793.90	118	28	12	14	14	30	32
12/04/97	3793.74	118	28	13	12	14	30	32

1998								
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**	**Pipe F**	**Pipe G**
01/05/98	3790.64	122	28	12	10	12	28	31
01/08/98	3791.61	125	30	13	11	14	30	32
02/12/98	3789.44	127	32	13	11	14	30	32
03/10/98	3787.67	127	32	13	11	14	40	32
04/09/98	3787.04	128	32	13	11	13	30	32
05/05/98	3785.99	125	30	13	11	13	30	32
06/02/98	3791.98	127	30	14	12	14	30	32
07/09/98	3798.88	128	30	14	12	14	28	30
08/17/98	3794.31	121	28	14	10	14	29	30
09/10/98	3790.72	116	27	13	11	14	29	31
10/23/98	3789.45	113	27	13	11	14	29	31
10/28/98	3790.20	114	26	12	10	14	28	30
12/07/98	3792.52	118	28	12	10	14	28	31

1999								
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**	**Pipe F**	**Pipe G**
02/01/99	3789.79	126	30	12	10	14	29	31
02/11/99	3789.27	126	31	12	10	14	30	31
03/03/99	3787.48	126	32	12	10	14	30	32
04/06/99	3781.86	124	32	14	12	14	30	32
05/03/99	3780.20	120	30	12	10	14	30	32
06/01/99	3784.82	118	29	13	11	14	30	32
07/01/99	3795.82	126	30	13	11	14	30	31
08/03/99	3793.77	123	30	14	11	14	29	31
09/01/99	3790.79	118	29	13	11	14	30	31
* 09/29/99	3788.76	116	28	12	10	14	28	30
10/05/99	3788.10	114	28	13	11	14	29	30
* 10/18/99	3787.72	112	26	10	9.5	11	27	31
* 10/29/99	3787.57	112	27	11	9	12	28	31
* 10/31/99	3787.64	112	27	11	9	12	28	31
11/04/99	3787.56	112	28	12	11	13	30	31
12/02/99	3788.11	112	28	12	11	13	30	32

2000								
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**	**Pipe F**	**Pipe G**
01/04/00	3787.10	118	30	11	10	13	30	31

Notes: Pipes are lettered in upstream to downstream direction (See Drawing 296-D-347).

* Denotes uplift pressure gauge reading done by drill crew or technician during cleaning time frame. All other readings performed by powerplant personnel.

TABLE 3-C

Canyon Ferry Dam
Uplift Pressure Gauge - Line 3, Block 15, Sta. 7+80

1997 Date	Reservoir El.	Pipe A	Pipe B	Pipe C	Pipe D	Pipe E	Pipe F	Pipe G
01/02/97	3788.24	148	62	16	28	34	38	36
02/04/97	3783.47	146	60	14	20	32	38	36
03/04/97	3777.01	140	60	14	30	34	36	36
04/01/97	3771.76	136	58	14	30	34	38	36
05/20/97	3777.31	135	56	15	30	34	38	36
06/17/97	3797.99	154	64	14	30	34	38	36
07/02/97	3797.30	153	63	15	29	34	38	36
07/31/97	3796.61	154	64	15	30	32	38	35
09/16/97	3794.74	152	62	14	28	32	36	35
09/30/97	3794.40	152	62	14	29	33	36	35
11/13/97	3793.90	151	60	14	29	31	33	32
12/04/97	3793.74	150	62	17	29	33	37	35

1998 Date	Reservoir El.	Pipe A	Pipe B	Pipe C	Pipe D	Pipe E	Pipe F	Pipe G
01/05/98	3790.64	148	60	16	28	32	36	34
01/08/98	3791.61	150	62	18	29	32	36	34
02/12/98	3789.44	150	62	18	29	32	37	35
03/10/98	3787.67	150	62	18	29	33	38	35
04/09/98	3787.04	150	62	18	29	33	38	35
05/05/98	3785.99	148	62	18	29	32	37	35
06/02/98	3791.98	150	62	17	29	32	37	35
07/09/98	3798.88	155	64	18	29	32	37	35
08/17/98	3794.31	153	63	17	29	32	37	34
09/10/98	3790.72	149	60	17	28	32	36	34
10/23/98	3789.45	147	60	16	28	32	36	35
10/28/98	3790.20	148	60	16	28	32	36	34
12/07/98	3792.52	148	60	16	28	32	36	34

1999 Date	Reservoir El.	Pipe A	Pipe B	Pipe C	Pipe D	Pipe E	Pipe F	Pipe G
02/01/99	3789.79	148	60	16	28	32	37	34
02/11/99	3789.27	148	60	16	28	32	37	34
03/03/99	3787.48	148	60	16	28	32	36	34
04/06/99	3781.86	143	60	18	28	33	38	34
05/03/99	3780.20	142	60	16	28	32	36	34
06/01/99	3784.82	142	59	17	28	32	37	34
07/01/99	3795.82	151	62	17	28	32	36	34
08/03/99	3793.77	152	62	17	28	32	37	34
09/01/99	3790.79	149	62	18	29	33	37	34
* 09/29/99	3788.76	149	62	16	27	32	35	32
10/05/99	3788.10	146	60	16	28	32	36	34
* 10/18/99	3787.72	144	54	12	27	30	35	32
* 10/29/99	3787.57	144	56	12	28	32	37	34
* 10/31/99	3787.64	146	56	12	27	30	36	34
11/04/99	3787.56	144	58	13	28	32	37	34
12/02/99	3788.11	144	58	13	28	32	37	34

2000 Date	Reservoir El.	Pipe A	Pipe B	Pipe C	Pipe D	Pipe E	Pipe F	Pipe G
01/04/00	3787.10	144	58	12	28	32	36	34

Notes: Pipes are lettered in upstream to downstream direction (See Drawing 296-D-347).
* Denotes uplift pressure gauge reading done by drill crew or technician during cleaning time frame. All other readings performed by powerplant personnel.

TABLE 3-D

Canyon Ferry Dam
Uplift Pressure Gauge - Line 4, Block 18, Sta. 9+35

1997						
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**
01/02/97	3788.24	30	0	0	8	0
02/04/97	3783.47	32	0	0	8	0
03/04/97	3777.01	32	0	0	8	0
04/01/97	3771.76	28	0	0	6	0
05/20/97	3777.31	20	0	0	6	4
06/17/97	3797.99	38	0	0	8	6
07/02/97	3797.30	38	0	0	8	6
07/31/97	3796.61	34	0	0	6	5
09/16/97	3794.74	32	0	0	6	6
09/30/97	3794.40	32	0	0	6	6
11/13/97	3793.90	32	0	0	6	6
12/04/97	3793.74	34	0	0	7	8

1998						
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**
01/05/98	3790.64	84	10	4	4	0
01/08/98	3791.61	61	0	4	7	8
02/12/98	3789.44	79	4	5	8	0
03/10/98	3787.67	83	9	7	8	0
04/09/98	3787.04	82	12	8	8	0
05/05/98	3785.99	79	6	9	9	0
06/02/98	3791.98	78	10	8	8	0
07/09/98	3798.88	82	0	6	8	0
08/17/98	3794.31	84	0	5	4	0
09/10/98	3790.72	84	6	4	5	0
10/23/98	3789.45	83	8	5	5	0
10/28/98	3790.20	82	10	4	4	0
12/07/98	3792.52	84	12	5	5	0

1999						
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**
02/01/99	3789.79	86	11	0	4	0
02/11/99	3789.27	88	12	0	7	0
03/03/99	3787.48	86	12	0	5	0
04/06/99	3781.86	81	12	0	4	0
05/03/99	3780.20	76	10	0	4	4
06/01/99	3784.82	78	11	0	5	5
07/01/99	3795.82	85	11	0	5	0
08/03/99	3793.77	85	10	0	5	0
09/01/99	3790.79	82	11	0	6	0
* 09/29/99	3788.76	80	12	0	5	0
10/05/99	3788.10	78	11	0	7	8
* 10/29/99	3787.57	76	10	0	0	0
* 10/31/99	3787.64	76	10	0	0	0
11/04/99	3787.56	77	10	0	6	6
12/02/99	3788.11	77	11	0	6	11

2000						
Date	**Reservoir El.**	**Pipe A**	**Pipe B**	**Pipe C**	**Pipe D**	**Pipe E**
01/04/00	3787.10	76	10	0	6	0

Notes: Pipes are lettered in upstream to downstream direction (See Drawing 296-D-347).

* Denotes uplift pressure gauge reading done by drill crew or technician during cleaning time frame. All other readings performed by powerplant personnel.

TABLE 4

Canyon Ferry Dam
Uplift Pressure Reading - Comparison

Line 1 Block 8

Date	Reservoir El. (ft.)	Pipe A	Pipe B	Pipe C	Pipe D	Pipe E		
03/10/98	3787.67	80	10	0	0	0		
10/31/99	3787.64	75	4	0	0	0		
Difference	-0.03	-5	-6	0	0	0		
04/09/98	3787.04	78	10	0	0	0		
01/04/00	3787.10	76	7	0	0	0		
Difference	0.06	-2	-3	0	0	0		
10/05/99	3788.10	78	4	0	0	0		
12/02/99	3788.11	76	7	0	0	0		
Difference	0.01	-2	3	0	0	0		

Line 2 Block 11

Date	Reservoir El. (ft.)	Pipe A	Pipe B	Pipe C	Pipe D	Pipe E	Pipe F	Pipe G
03/10/98	3787.67	127	32	13	11	14	40	32
10/31/99	3787.64	112	27	11	9	12	28	31
Difference	-0.03	-15	-5	-2	-2	-2	-12	-1
04/09/98	3787.04	128	32	13	11	13	30	32
01/04/00	3787.10	118	30	11	10	13	30	31
Difference	0.06	-10	-2	-2	-1	0	0	-1
10/05/99	3788.10	114	28	13	11	14	29	30
12/02/99	3788.11	112	28	12	11	13	30	32
Difference	0.01	-2	0	-1	0	-1	1	2

Line 3 Block 15

Date	Reservoir El. (ft.)	Pipe A	Pipe B	Pipe C	Pipe D	Pipe E	Pipe F	Pipe G
03/10/98	3787.67	150	62	18	29	33	38	35
10/31/99	3787.64	146	56	12	27	30	36	34
Difference	-0.03	-4	-6	-6	-2	-3	-2	-1
04/09/98	3787.04	150	62	18	29	33	38	35
01/04/00	3787.10	144	58	12	28	32	36	34
Difference	0.06	-6	-4	-6	-1	-1	-2	-1
10/05/99	3788.10	146	60	16	28	32	36	34
12/02/99	3788.11	144	58	13	28	32	37	34
Difference	0.01	-2	-2	-3	0	0	1	0

Line 4 Block 18

Date	Reservoir El. (ft.)	Pipe A	Pipe B	Pipe C	Pipe D	Pipe E		
03/10/98	3787.67	83	9	7	8	0		
10/31/99	3787.64	76	10	0	0	0		
Difference	-0.03	-7	1	-7	-8	0		
04/09/98	3787.04	82	12	8	8	0		
01/04/00	3787.10	76	10	0	6	0		
Difference	0.06	-6	-2	-8	-2	0		
10/05/99	3788.10	78	11	0	7	8		
12/02/99	3788.11	77	11	0	6	11		
Difference	0.01	-1	0	0	-1	3		

Appendix C

Drain Cleaning Equipment

page

C-1—Remote Controlled Video Inspection of Drains.—Reclamation's use of remotely controlled video inspection (RCVI) is becoming an increasingly useful method for examining small or inaccessible drains, as well as conduits and pipelines. Drains are used to reduce pore water pressures and uplift pressures, and safely convey and discharge seepage water. The ability of drains to perform this function may become compromised by age, mechanical deterioration, loads on the drain pipe that exceed the capacity of the pipe or chemical attack. When a drain can no longer safely perform its intended function, the safety of the dam or water project may become jeopardized. Earthfill dams are vulnerable to toe drain deterioration (see the *Case History of Toe Drain Inspections* in ch. 4 for examples), and typical failure modes involving drains include:

- The potential for a partial or a complete failure of the dam through a mechanism whereby embankment material is eroded into the toe drain by seepage resulting from high pore pressures within the portion of the embankment that immediately surrounds the toe drain. Such an erosion mechanism could progress into the embankment, toward the reservoir, and could possibly result in a sudden release of the reservoir. The erosion of embankment material could occur through open/separated joints, deterioration of rivets, cracks, deterioration of flow surfaces, collapsed toe drain, etc.

- The potential for a partial or complete failure of the dam as a result of continued long term structural deterioration and collapse of a toe drain. The void caused by the collapse of the toe drain provides an open "pipe," allowing the uncontrolled flow of reservoir water and the resulting erosion of the surrounding embankment materials.
- The potential for a partial or complete failure of a spillway or outlet works chute or stilling basin floor slab as a result of piping of foundation materials into the drain and undermining of the foundation for the structure.

- The potential for a partial or complete failure of a concrete dam or a spillway or outlet works structure due to a blockage in the drainage system. This blockage could result in ineffective drains, the buildup of uplift pressures, and a sliding failure within the structure foundation or at the structure/foundation contact.

Numerous case histories provide documented evidence that many dams have failed as a result of these potential failure modes. These dam failures, along with the increasing age of Reclamation's inventory of dams and recent improvements in inspection technology, have prompted Reclamation to institute a program of remote video inspections of drains.

RCVI can provide significant improvements over other methods of inspection, such as physical inspection, where an inspector crawls through the drain or conduit (30-in or larger) and documents the conditions, manual inspection, where a sled with a camera is pushed through the drain or conduit using long push rods, and mechanical inspection, where a camera tethered to a wire rope and is pulled through the drain or conduit. RCVI has the advantages of being able to examine drains or conduits regardless of size limitations, has complete mobility, and provides real time video images. Generally, an RCVI consists of a video camera attached to a self-propelled transport vehicle. An operator remotely controls the transport vehicle and camera. The camera can provide both longitudinal and circumferential views of the interior of the drain or conduit surfaces. Video images are transmitted from the camera to a television monitor, from which the operator can view the conditions within the drain or conduit.

a. *Guidelines for Initiating a Remotely Controlled Video Inspection*.—The water and soil or rock environment can greatly affect the condition and service life of a drain. RCVI can be used to effectively assess and monitor the conditions within the drain. Some general guidelines for when to perform an RCVI are:

- When obvious seepage areas or depressions along the drain alignment or at the exit portal area exist
- When the clarity or volume of water being discharged from the drain changes
- After any significant seismic activity near the dam. Deformation of an embankment dam may cause spreading of the drain joints.
- When drains are subjected to corrosive environments caused by certain water and soil conditions and the drains are constructed of a material vulnerable to corrosion
- When drains approach their life expectancy. For instance, CMP can be expected to deteriorate significantly after about 25 to 50 years of operation.
- If the drain has not been inspected since its original construction

Over the years, many types of materials have been used for drains, including reinforced concrete pipe, CMP, and clay tile. More recently, plastic pipe such as PVC and HDPE are also being used. Some drain materials are more prone to the development of problems and may require a more frequent inspection program. For example, CMP is subject to chemical and galvanic corrosion, joint leakage, and live load distortion. RCVI can be used to develop a baseline from which future inspections can be compared and the degree of continuing deterioration determined.

b. *Remotely Controlled Video Inspection Equipment*.—RCVI was initially used for gas/oil and sewer pipelines. Over the last 10 years, RCVI has expanded into many applications, such as toe drain and conduit inspection. In that time period, the robotic equipment used for video inspection has changed significantly. The latest trend in RCVI equipment is for modular efficiency, so more versatility and a wider range of applications can be provided. The benefit of modular design is the reduction of added costs required for "application specific" and "custom designs."

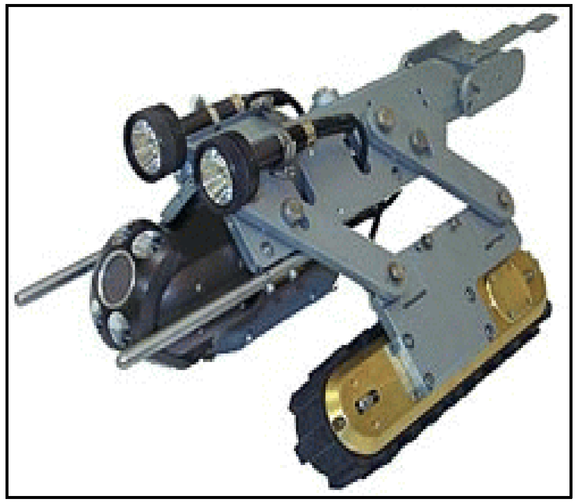


Figure 1.—RCVI equipment (VersoTrac System) used by Reclamation.

Typical RCVI equipment and features:

- *Camera*.—Underwater viewing, solid state circuitry, color, wide angle zoom lens, remote controlled optical focus, automatic iris with manual override, variable speed pan and tilt, operating temperature range of 32 to 122 °F, underwater lights, shock and vibration resistant, and easy transportation and storage

Figure 2.—RCVI instrumentation and control equipment.

- *Transport vehicle*.—Depth rating of 100 feet, forward, neutral and reverse clutch, speed control (0 to 30 ft/min), operating temperature range of 32 to 122 °F, direct current motor, varying operating voltage on models (24 V to 110 V), rubber tracked belts or tires, adjustable tracks or tires for larger diameter drains, lowering harness, and camera adaptor plate. Drains with diameters from 4 inches to 240 inches can be inspected.

- *Cable reel*.—Self-laying, direct current motor, motorized rewind, cable comes in variety of lengths (up to 1,500 ft, depending on cable diameter), footage meter, and manual operator override braking system

- *Control unit*.—Color monitor, VCR VHS with built-in microphone and push talk button, transport vehicle controller, pan and tilt camera controller, and data logger system (electronic footage display, date/time, alpha/numeric input, data card hardware for data transfer to PC). For the purposes of a longer storage of media, the use of DVD is becoming standard practice. The control unit is normally transported to the site in a truck or van. Customized trucks and vans are available. In difficult access locations, a portable system can be installed on an all-terrain vehicle.

RCVI services are available in many areas. However, the quality of video inspection equipment can vary greatly. Any company selected to perform RCVI services should be experienced and have a wide range of available equipment for various site conditions. Costs for RCVI services vary, depending upon the location of the job, remoteness of the site, access difficulties at the site, etc. If multiple drain inspections are done for a number of dams during the same trip, the average cost per video inspection will be usually be lower. Some RCVI companies also provide cleaning services.

Dam owners and dam safety organizations may want to purchase their own video inspection equipment. This becomes especially economical, if annual video inspections are needed at numerous dams. Costs for purchasing video inspection equipment depend on the quality and options selected.

c. *Guidelines for Performing a Remotely Controlled Video Inspection*.—The success of an RCVI depends upon the quality of the equipment and the experience of the operator. An RCVI usually requires a two-person crew, consisting of an operator and cable reel handler. Additional crew members may be required in difficult access locations.

An important part of any RCVI is the technical evaluation of the conditions observed during the videotaped inspection. This evaluation should be performed by a qualified and experienced professional engineer. The engineer should prepare a Report of Findings which documents all problem areas observed and provides recommendations for future actions.

Tips for a successful video inspection:

- *Light*.—The amount of light provided is critical to the success of the inspection. Without the proper amount of light, areas of concern cannot be observed clearly. Lack of clarity hinders making definitive conclusions about the integrity of the conduit. Also, the larger the diameter of the conduit, the more light that is needed. A trial-and-error procedure maybe required to obtain the sufficient amount of light. The ability to vary light intensity and control glare is an important feature to consider for the RCVI equipment.

- *Camera*.—The video camera should be able to pan and tilt and also look straight ahead. Not all inspections involve horizontal conduits. Inspections of vertical drops are sometimes required. The video camera should be able to accommodate different orientations.

- *Footage meter*.—A footage meter should be superimposed on the videotape. This meter makes identifying specific locations within the conduit much easier.

- *Narration.*—All inspection videotapes should include a narration by the operator. The operator should describe in detail what is being seen. Narration should note mineral deposition, changes in the slope of the invert, changes in the depth or velocity of water (inflow), condition of drain joints, etc.

- *Drawings and photographs.*—Copies of all available design and/or as-built drawings of the dam and drains should be on-site during the video inspection for immediate reference and confirmation of details and features seen in during the inspection.

- *Measurements and data collection.*—The inspection and the technical evaluation will be greatly enhanced if the following data are collected at the time of the video inspection: reservoir water level, outflow from the drains, any relevant data on nearby piezometer levels or uplift levels, history of operations, and time/date.

- *Videotape library.*—The inspectors should review all previous inspection videotapes (if available) prior to doing the video inspection. This will provide a baseline reference, so the rate of any continuing deterioration can be noted.

d. *Evaluating the Results of a Remotely Controlled Video Inspection.*—Conditions observed during an RCVI vary depending on the drain materials used. Typical inspection observations are summarized below:

- *Leaking joints.*—Watertight joints are not always obtained during construction. If a joint is leaking, mineral deposition may be observed inside the drain. As water enters the drain and evaporates, minor precipitates are deposited. Deposition will usually occur in drains with little or intermittent flow. Joints may also become separated after installation as a result of settlement of embankment or fill material placed above the drain.

- *Corrosion.*—Metal or steel pipes can deteriorate due to chemical or electrochemical reactions to the environment. Corrosion can occur on the flow surface or on the outside of the drain.

- *Shape distortion.*—Due to poor compaction, the surrounding backfill does not provide the required support. This can result in loss of cross section. Shape distortion is most common with flexible pipes such as PVC, HDPE, and corrugated metal pipes.

- *Misalignment.*—A sag in the invert or a deflection of a joint may indicate settlement or poor installation.

- *Cracking.*—Longitudinal or transverse cracking is usually a sign of distress caused by overloading, subgrade problems, or differential movement.

- *Concrete deterioration*.—Spalling is usually a sign of overloading or caused by water corroding the reinforcing steel. The oxidized steel expands, resulting in a separation of the concrete covering the reinforcement.

e. *Summary*.—Remotely controlled video inspection is a useful tool for assessing the condition of drains. As the existing inventory of dams and water projects continues to age, more drain-related safety concerns will require inspection and evaluation. After viewing the conditions within the drain, the dam safety official can better determine if corrective actions are warranted. Depending on the severity of the conditions, more frequent video inspections may be required.

C-2—Evaluating the Performance of Remotely Controlled Video Inspection Equipment in Double Walled, High Density Polyethylene Pipe.—

a. *Purpose*.—In 2000, Reclamation's Technical Service Center (TSC) implemented a program of remotely controlled video inspection as part of their dam safety program. This program provides for the inspection of toe drains, wall drains, structural underdrains, pressure relief wells, siphons, pipelines, and outlet works/spillway conduits. Generally, a complete video inspection includes (1) performance of the video inspection, (2) a technical evaluation of observed conditions (including recommendations for future actions), and (3) documentation of the inspection and evaluation (including photographs captured from videotape and a detailed inspection log). Experience has shown that the pipe configuration (diameter, bends, invert slopes, and existing invert conditions such as gravel, sediments, and bacterial growths) greatly affect the success of a video inspection. This section is intended to provide guidance to geotechnical engineers in laying out toe drain systems for embankment dams, so that remotely controlled video inspection equipment will be able to navigate within new toe drain installations.

b. *Introduction*.—The use of remotely controlled video inspection (RCVI) has greatly advanced the capability to evaluate existing conditions within small and inaccessible pipes and conduits, such as toe drains, wall drains, structural underdrains, pressure relief wells, siphons, pipelines, and outlet works/spillway conduits. Pipes and conduits may be inaccessible due to diameter size or confined space entry. RCVI differs from other methods of inspection, such as (1) physical inspection, where an inspector crawls through the conduit (30 in. or larger) and documents the conditions; (2) manual inspection, where a camera attached to a sled is pushed through the conduit using long push rods; and (3) mechanical inspection, where a camera tethered to a wire rope is pulled through the conduit. RCVI has the advantages of being able to examine pipes and conduits regardless of size limitations, having complete mobility, and providing real time video images. Generally, an RCVI consists of a video camera attached to a self-propelled transport vehicle (crawler). The transport vehicle and camera are commonly referred to as a camera-crawler. An operator remotely controls the camera-crawler. The camera can provide both longitudinal and circumferential views of the interior of the pipe or conduit. Video images are transmitted from the camera to a television monitor, from which the operator can view the conditions within the pipe or conduit. The video images are recorded onto videotape, compact disc, or DVD for future evaluation and documentation.

Over the years, many types of materials have been used for pipes and conduits. Commonly used materials have consisted of cast-in-place reinforced concrete, reinforced concrete pipe, corrugated metal pipe (CMP), welded steel pipe, clay tile, asbestos-cement, and plastics such as high density polyethylene (HDPE) and polyvinyl chloride (PVC).

The majority of Reclamation's spillways and outlet works are large enough for physical inspection of the conduit. However, toe drains used at embankment dams are usually too small to be physically inspected. Most toe drains for embankment dams have never been inspected. The Technical Service Center has been performing video inspection of toe drain systems over the past few years using RCVI. Experience with recent video inspections has shown that past design practices for toe drains do not always allow for the accommodation of RCVI equipment. This report will focus on evaluating a variety of toe drain pipe configurations for diameter, bend, invert slope, and invert conditions to accommodate RCVI equipment.

The Technical Service Center most frequently specfies double walled, high density polyethylene (known as HDPE) for new toe drains. An RCVI equipment testing program for HDPE was developed and conducted in 2002. Tests were conducted for a wide variety of pipe diameters, bends, invert slopes, and invert conditions. This section summarizes the results of those tests and provides recommendations concerning the layout of toe drain systems to accommodate inspection using RCVI equipment.

c. *Conclusions.*—The following general guidelines are based on the results of the performance testing of RCVI equipment within double walled HDPE pipe. All performance testing in this research program was based on the assumption that the camera-crawler would travel up the pipe from a downstream location. Toe drain designs that provide an upstream access location from which the camera-crawler can gain entry allow for improved cable tether pulling capacity, since the camera-crawler can move downward on a sloping decline. Sloping declines generally do not result in camera-crawler traction issues. For the camera-crawler backout process, the transport vehicle has a free-wheeling clutch mechanism on the track unit that allows for high speed retrieval either manually or by a cable take up reel. Although not tested in this research program, the upstream access location would also benefit camera-crawler navigation around pipe bends and allow for the use of steeper invert slopes, since the effect of cable drag would be lessened. Providing upstream access locations would be especially important where steeper invert slopes may be required, such as on abutments.

- *Pipe diameters.*—The minimum recommended pipe diameter to successfully accommodate RCVI equipment is 8 inches. Although camera-crawlers are available for pipes smaller than 8 inches, they are very limited in cable tether pulling capacity and generally do not have sufficient traction for use in toe drain inspection. In addition, the cameras typically only have a fixed lens and the transport vehicle is not steerable. Camera-crawlers used in pipes with diameters between 8 and 12 inches generally have cameras with some pan, tilt, and zoom capabilities, but generally are not steerable. Camera-crawlers used in pipes with diameters of 15 inches or larger are steerable, have a greater cable tether pulling capacity, and have cameras which can provide a wider array of optical capabilities including pan, tilt, and zoom.

- *Pipe bends*—The maximum recommended bend angle to successfully accommodate RCVI equipment is 22.5 degrees. In pipes with diameters of 8 and 10 inches, the camera-crawler cannot be navigated around bends greater than 45 degrees, since the camera cannot clear the pipe crown as it travels through the bend. If sharper bends are required in pipes with diameters of 8 and 10 inches, a series of 22.5-degree bends are recommended. Each bend should be connected to a minimum 5-foot length of pipe to allow the camera-crawler to navigate around the bend segment and provide adequate crown clearance. Pipes with diameters of 12 inches or larger can have bends that exceed 22.5 degrees, but drag friction then reduces the cable tether pulling capacity by as much as 75 percent.

- *Invert slope inclination*.—The maximum recommended invert slope inclination to successfully accommodate RCVI equipment is 5 degrees. The difference in invert slope inclination between flat and 10 degrees can reduce cable tether pulling capacity by as much as 70 percent depending upon the pipe diameter, degree of pipe bend, and the invert condition. Invert slopes from 0.01 to about 0.09 (5 degrees) would appear to be the most reasonable inclination. Slopes greater than 10 degrees are not recommended, due to the significant loss of traction that occurs when camera-crawlers are pulling long cable tethers.

- *Distance between manholes or access entry locations*.—The maximum distance between manholes or access entry locations can range between 500 and 2,000 feet, but highly depends upon the pipe diameter, bends, invert slopes, and invert conditions. The designer will need to take these limitations in account when selecting the appropriate distance between manholes or access entry locations. In pipes with diameters of 8, 10, and 12 inches, the maximum distance should not exceed about 1,000 feet. This assumes that access is available on both ends of the pipe. If access will be only be available on the downstream end of the pipe, then the maximum distance should be limited to about 500 feet. The graphs at the end of this section provide more information on how pipe diameter, bends, invert slopes, and invert conditions affect the cable pulling capacity of camera-crawlers and the maximum distance between manholes or access entry locations. In pipes with diameters of 15 and 18 inches, the maximum distance should not exceed about 2,000 feet. This assumes that access is available on both ends of the pipe. If access will be only be available on the downstream end of the pipe, then the maximum distance should be limited to about 1,000 feet. The graphs provide more information on how pipe diameter, bends, invert slopes, and invert conditions affect the cable pulling capacity of camera-crawlers and the maximum distance between manholes or access entry locations.

Figure 3.—Apparatus used for performance testing of remotely controlled video inspection equipment within HDPE pipe.

d. *Investigation*.—The Technical Service Center has a wide variety of remotely controlled video inspection equipment. A research project was initiated to test the performance capabilities of the most commonly used equipment.

1. *Testing Apparatus*.—In June 2002, the TSC's laboratory construction shop constructed a testing apparatus. The apparatus was designed to be multifunctional, so it could be configured to accommodate a variety of pipe diameters, bends, invert slope inclinations, and invert conditions. Some of the tests required supplemental support assistance supplied by a small, 1,500-lb capacity boom crane mounted on a truck. Figure 3 shows the testing apparatus.

2. *HDPE Pipe*.—The HDPE pipe used in the performance testing was N12 solid double walled, manufactured by Advanced Drainage System. This pipe has corrugations on the exterior and is smooth walled on the interior. It meets the requirements of AAHSHTO M 294.

Figure 4 shows the pipe and bends for the 8-inch diameter HDPE pipe. The 90-degree bend was made by joining two 45-degree bends. This joining method is preferred for RCVI equipment, as it reduces the sharpness of the bend and generally improves the navigation

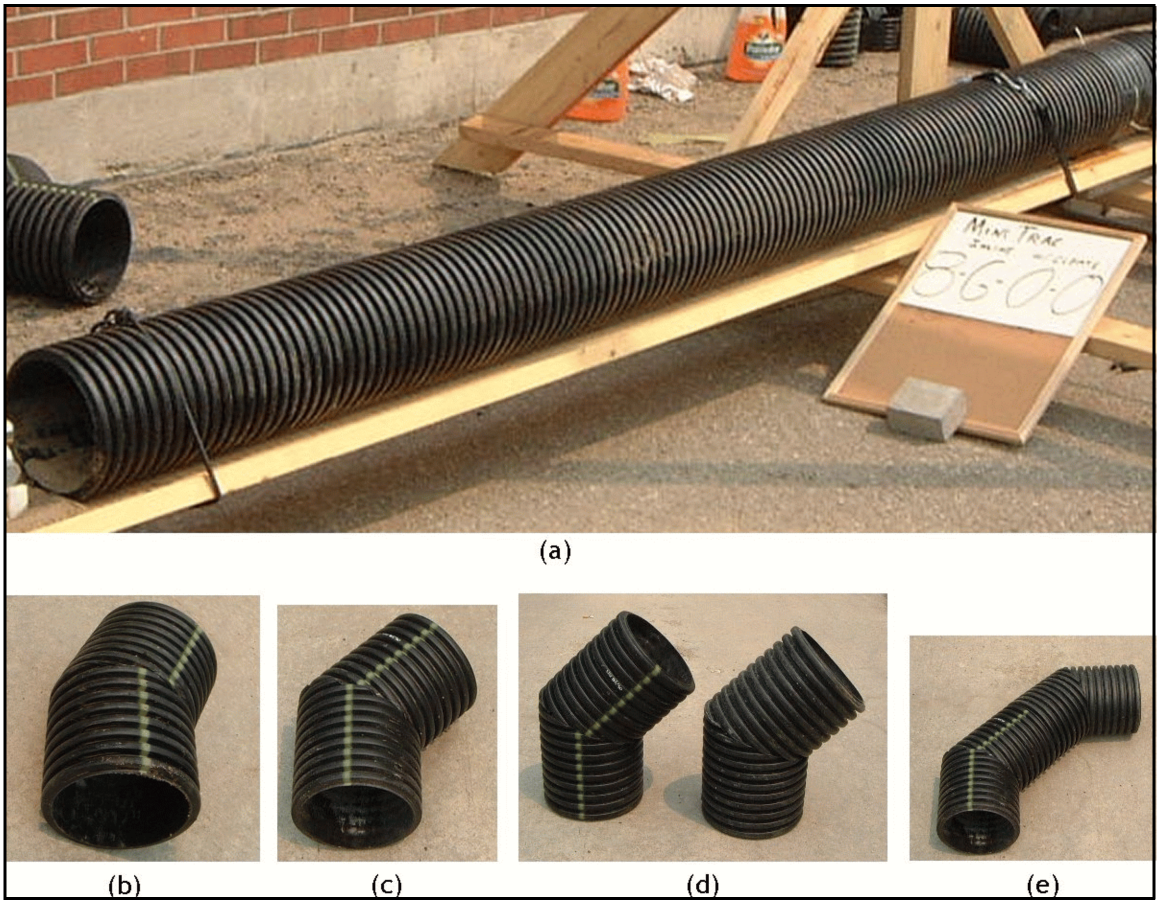


Figure 4.—8-in. diameter HDPE pipe and the bends used: (a) a 10-ft length of 8-in. diameter HDPE pipe; (b) a 22.5° bend; (c) a 45° bend; (d) two 45° bends used to form a 90° bend; (e) a 90° bend made from joining two 45° bends.

capability of the camera-crawler through the pipe. Although not tested, the 10-inch diameter pipe would be similar.

Figure 5 shows the pipe and bends for the 15-inch diameter HDPE pipe. The manufacturer fabricates the 45- and 90-degree bends as a one piece unit. The pipe and bends for the 12- and 18-inch diameter pipes are similar. The fabricated bend is preferred for RCVI equipment, as it reduces the sharpness of the bend and makes navigation of the camera-crawler easier through the pipe.

All pipe bends were connected using the manufacturers' supplied couplers and secured together using plastic cable ties (figs. 6 and 7).

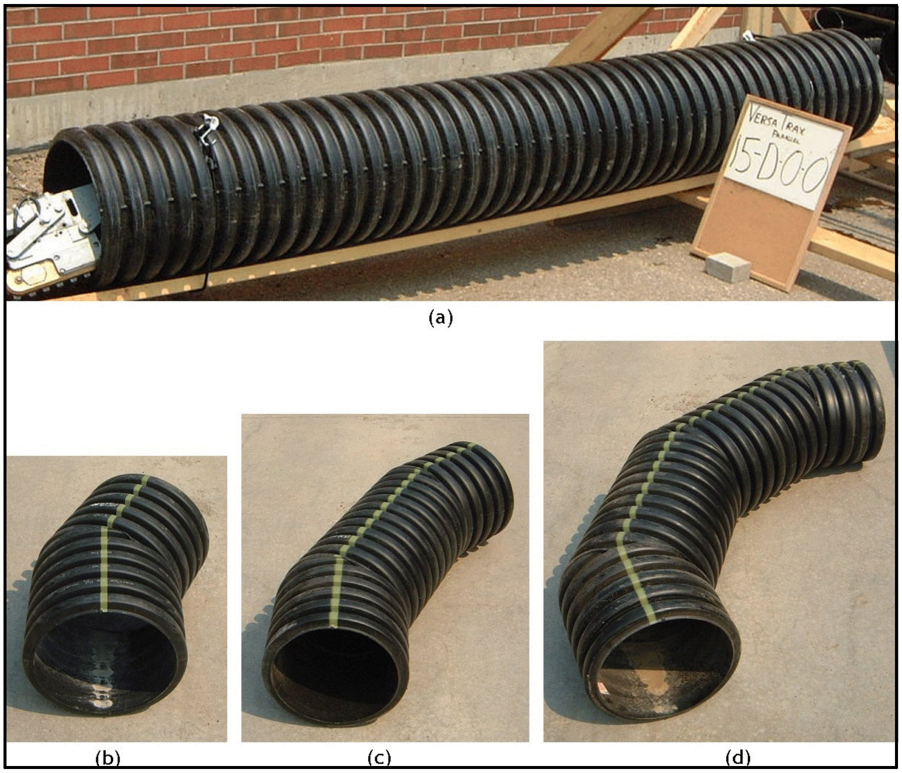


Figure 5.—15-in. diameter HDPE pipe and the bends used: (a) a 10-ft length of 15-in. diameter HDPE pipe; (b) a 22.5° bend; (c) a fabricated 45° bend; (d) a fabricated 90° bend.

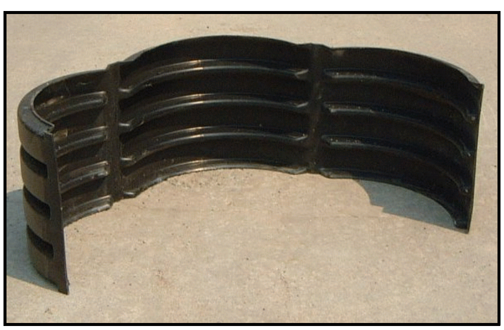


Figure 6.—Manufacturer supplied coupler.

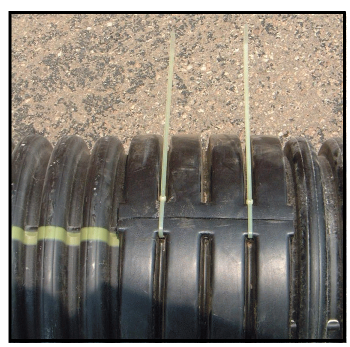


Figure 7.—Plastic cable ties used for securing the pipe bends.

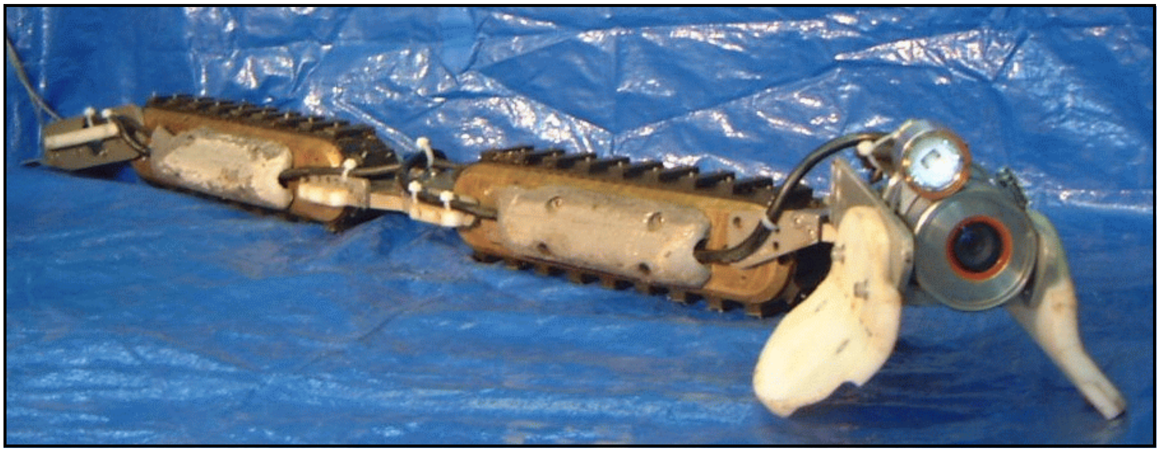


Figure 8.—The MiniTrac remotely controlled inspection system.

3. *RCVI Equipment*.—Performance tests were conducted using two types of camera-crawlers, the MiniTrac and the VersaTrax 150 systems. Each camera-crawler system consisted of a transport vehicle and a camera. Inuktun Services Ltd. manufactured the transport vehicles and R.J. Electronics manufactured the cameras.

The MiniTrac system (fig. 8) uses a transport vehicle with two motorized brass track units in an inline configuration.

The VersaTrax 150 system (fig. 9) uses a transport vehicle with two motorized brass track units in a parallel configuration.

Individual control units, plugged into a standard 120VAC 60-Hz power source, power the track units, lights, and cameras. The camera-crawler is connected to the track unit control by use of the cable tether. The cable tether consists of two 500-foot-long polyurethane jacketed coaxial cables joined together by a connector assembly. The cable tether feeds the power and control signals to the transport vehicle, camera, and lights and also returns data from the sensors to the controllers. The control units for the lights and cameras are connected directly to the track control unit. Table 1 summarizes the camera-crawler specifications.

An operator (fig. 10) controls the movement of the camera-crawler's track units, lights, and camera.

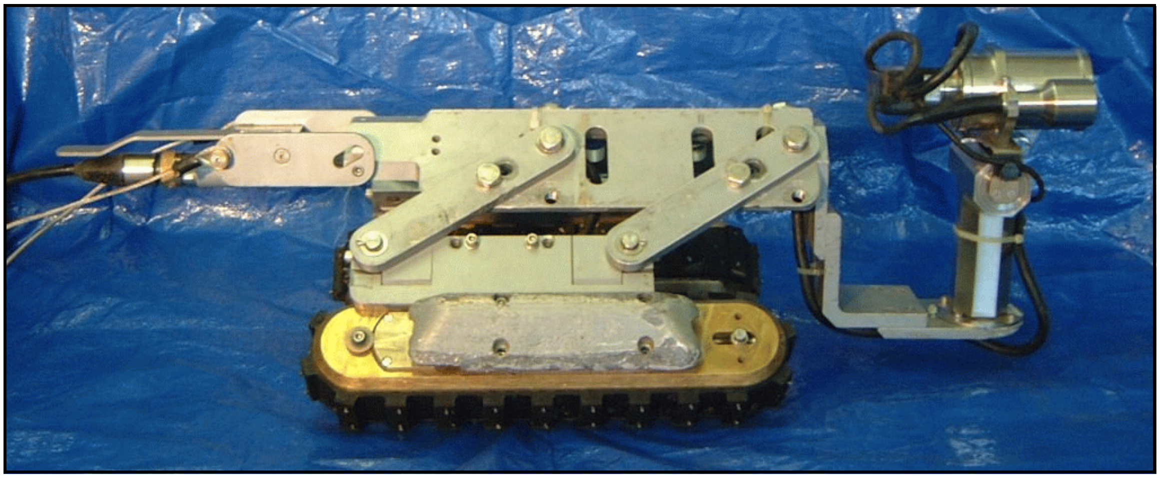


Figure 9.—The VersaTrax 150 remotely controlled inspection system.

Figure 10.—Operator controlling the remote inspection equipment.

Table 12.—Camera-crawler system specifications

Specification	MiniTrac—inline	VersaTrax 150—parallel
Depth rating (ft)	100	100
Maximum speed (ft/min)	32	32
Operating temperature (°F)	0-120	0-120
Height (in.)	7	15
Length (in.)	62	35
Width (in.)	9	14
Steerable	No	Yes

4. *Performance Tests.*—The performance tests were designed to determine how many linear feet of cable tether the transport vehicle can pull and to determine the navigability of the camera-crawlers in various pipe configurations.

To simulate conditions normally encountered in performing a video inspection, the cable tether was laid flat on the ground (which in this case was the asphalt parking lot). While this method does not exactly match possible coefficients of friction experienced in toe drains, it was felt that due to the wide variation of invert conditions that typically exist in pipes, this would be conservative.

The HDPE pipe was positioned on the testing apparatus and secured in place using rubber bungee cords (fig. 11).

Figure 11.—Bungee cords used to secure the pipe to the testing apparatus.

Table 2 summarizes the testing variables studied in the performance tests.

Table 13.—Testing variables

Camera-crawler system	Material type	Diameters (in.)	Invert slopes (degrees)	Bend radius (degrees)	Invert conditions
Mini Trac—inline	ADS N-12	8 and 12	Flat, 10, and 30	0, 22.5, 45, 90	Dry, slippery, and gravel
Versa Trax—parallel	ADS N-12	15 and 18	Flat,10, and 30	0, 22.5, 45, 90	Dry, slippery, and gravel

Dish soap (fig. 12) was used on the pipe invert to simulate the slippery conditions that can develop in toe drain pipes due to bacterial growth and sediment deposition.

A random mixture of fine and coarse aggregates (fig. 13) was used to simulate deposits that occasionally collect on the inverts of toe drain pipes.

e. *Discussion and Results.*—

1. *Use of Cleats.*—Preliminary performance tests were conducted with track belts supplied by the manufacturer. These track belts have deep rubber lugs. The traction performance of the transport vehicles using the lug track belts was disappointing. To

Figure 12.—Dish soap used to simulate slippery invert conditions.

Figure 13.—Random mixture of fine and coarse aggregates used to simulate sediment deposition on the pipe invert.

improve traction, cleats (fig. 14) were installed into the track belts. These cleats consisted of two No. 6 stainless steel screws (¾ in. long) drilled through each lug on the track belt. The pointed tip of each screw was cut off using a bolt cutter.

Comparative traction performance tests were conducted using the MiniTrac system. Graphs 1 and 2 show the cable tether pulling capacity of the MiniTrac system using track belts with and without cleats for a dry invert with no slope and a 10-degree slope inclination. For the dry invert, no slope test, the use of cleated track belts resulted in a cable tether pulling capacity that was almost twice as much as the noncleated track belt. However, for the dry invert, 10-degree slope test, the cleated track belts provided added traction that resulted in almost four times the cable tether pulling capacity as the noncleated track belt. No tests were conducted with the VersaTrax 150 system, since similar results were expected.

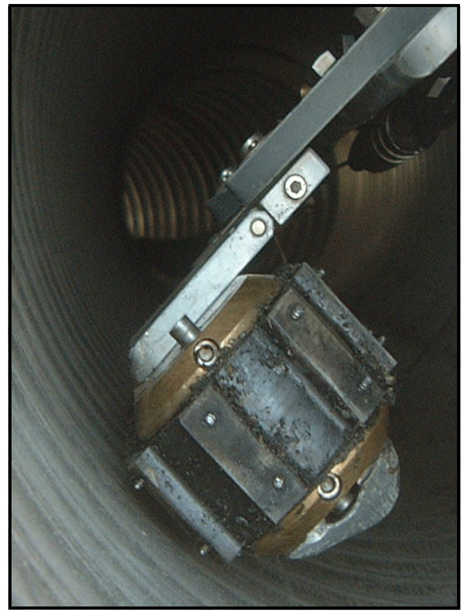


Figure 14.—Cleated track belt on the VersaTrax 150 system.

Conclusion: The cleated track belts provided significant added traction over the noncleated track belts. The added traction using cleated track belts improved cable tether pulling capacity by as much as four times that of the noncleated track belts. Due to the improved traction, all subsequent performance tests were conducted using cleated track belts.

2. *Pipe Diameter*.—These performance tests evaluated the effects of pipe diameter on RCVI equipment. The size of the pipe diameter determines the type of RCVI equipment to be used for inspection. Camera-crawlers can be easily configured in the field for many sizes of pipe diameters.

Performance tests were conducted using the MiniTrac and VersaTrax 150 systems. A variety of pipe diameters were tested to see what entrance limitations are encountered with the camera-crawlers. Successful entry into pipes can be limited by the positioning of the lights and camera on the transport vehicle. Improper positioning of the lights and camera can result in elimination of the required crown clearance within the pipe. Although camera-crawlers are available for pipes smaller than 8 inches, they are very limited in cable tether pulling capacity and generally do not have sufficient traction for use in toe drain inspection. The MiniTrac system could easily enter all pipes 8 inches or larger (fig. 15). The VersaTrax system could enter all pipes 15 inches or larger (fig. 16). One caveat to note:

Figure 15.—The MiniTrac system inside an 8-in. diameter pipe.

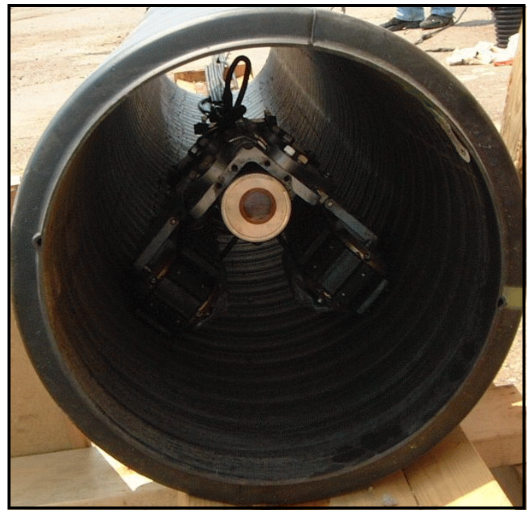


Figure 16.—The VersaTrax 150 system inside an 15-in. diameter pipe.

even though a camera-crawler can enter a pipe, other issues such as pipe bends may present navigation problems. The effects of pipe bends on camera-crawler navigation were evaluated in other performance tests discussed later in this report.

Conclusion: To allow for more clearance at the pipe crown, the lights can be moved slightly to other more favorable orientations on the camera. The lights can be held in place by using hose clamps. Although camera-crawlers are available for pipes smaller than 8 inches, they are very limited in cable tether pulling capacity and generally do not have sufficient traction for use in toe drain inspection. In addition, the cameras typically only have a fixed lens and the transport vehicle is not steerable. Camera-crawlers used in pipes with diameters between 8 and 12 inches generally have cameras with some pan, tilt, and zoom capabilities, but generally are not steerable. Camera-crawlers used in pipes with diameters of 15 inches or larger are steerable, have a greater cable tether pulling capacity, and have cameras which can provide a wider array of optical capabilities including pan, tilt, and zoom.

3. *Pipe Bends.*—These performance tests evaluated the effects of pipe bends on RCVI equipment. The following issues were investigated:

- *Frictional drag.*—Frictional drag on the cable tether is caused when the cable tether is pulled around the bend by the camera-crawler. The sharper the pipe bend, the more frictional drag on the cable tether.

- *Clearance*.—Stabilization wings (fig. 18) are used on camera-crawlers to provide stability as it travels through the pipe. These wings are not required in pipes with diameters of 8 inches. However, in pipes with diameters larger than 8 inches, the stabilization wings are normally used to prevent the camera-crawler from trying to run up the side of the pipe and turning over onto its side. Pipe bends present navigation problems for camera-crawlers since the lights and stabilization wings reduce required clearance and interfere with advancement through the bend.

Performance tests were conducted using the MiniTrac and VersaTrax 150 systems to evaluate the effects of frictional drag at pipe bends on the cable tether pulling capacity.

For the MiniTrac system, tests were conducted in 8- and 12-inch diameter pipes with pipe bends of 0, 22.5, 45, and 90 degrees, with invert inclinations of flat and 10-degrees, and with invert conditions of dry, slippery, and gravel.

Graphs 2, 3, and 4 summarize the results of tests using an 8-inch diameter pipe. The MiniTrac system could be navigated through 0- and 22.5-degree pipe bends, but cable tether was significantly reduced by about 15 to 40 percent depending upon the invert slope inclination and the invert condition. The camera-crawler could not be navigated through a 45-degree pipe bend (fig. 17), since the camera did not clear the pipe crown as it traveled through the bend. The use of stabilization wings (fig. 18) affects travel through bends

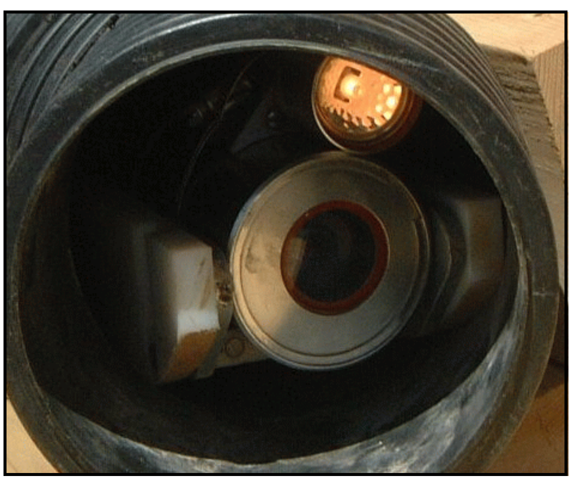


Figure 17.—The MiniTrac system could not be navigated through a 45° bend in an 8-in. diameter pipe due to the lack of crown clearance.

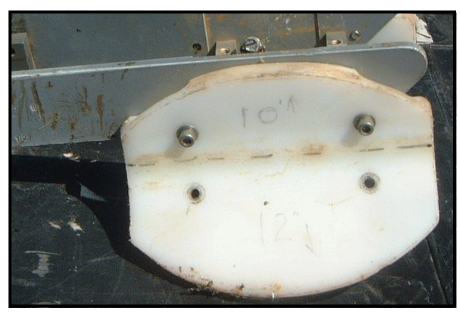


Figure 18.—Stabilization wings used on the MiniTrac system.

sharper than 22.5-degrees. No bend performance tests were conducted on 10-inch diameter pipe, since the results were expected to closely match those from the 8-inch diameter pipe.

Graphs 5 and 6 summarize the results of tests using a 12-inch diameter pipe. The MiniTrac system could be navigated through 0-, 22.5-, 45-, and 90-degree pipe bends, but cable tether was significantly reduced by 50 to 75 percent depending upon the invert slope inclination and the invert condition.

For the VersaTrax 150 system, tests were conducted using 15- and 18-inch diameter pipes, with pipe bends of 0, 22.5, 45, and 90 degrees, invert inclinations of flat and 10-degrees, and invert conditions of dry, slippery, and gravel.

Graph 7 summarizes the results of tests using a 15-inch diameter pipe. The VersaTrax 150 system could be navigated through 0-, 22.5-, 45-, and 90-degree pipe bends, but cable tether was significantly reduced by 30 to 40 percent depending upon the invert slope inclination and the invert condition. The slippery and gravel inverts were not tested for the 15-inch diameter pipe, since the results were expected to closely match those of the 18-inch diameter pipe.

Graphs 8, 9, and 10 summarize the results of tests using an 18-inch diameter pipe. The VersaTrax 150 system could be navigated through 0-, 22.5-, 45-, and 90-degree pipe bends, but cable tether was significantly reduced by 30 to 60 percent depending upon the invert slope inclination and the invert condition.

Conclusion: In pipes with diameters of 8 inches, the camera-crawler cannot be navigated around bends greater than 22.5 degrees, since the camera does not clear the pipe crown as it travels through the bend. Pipes with diameters 12 inches or larger can have bends that exceed 22.5 degrees, but the cable tether pulling capacity is reduced by as much as 30 to 75 percent due to drag friction.

4. *Invert Slope Inclination*.—These performance tests evaluated the effects of invert slope inclination on RCVI equipment. The degree of invert slope inclination reduces the amount of cable tether a camera-crawler can pull due to a loss of traction. Performance tests were conducted for flat, 10-, and 30-degree invert slopes. Additional traction can be provided with the addition of lead weights mounted to the frame or track units of the transport vehicle (fig. 19).

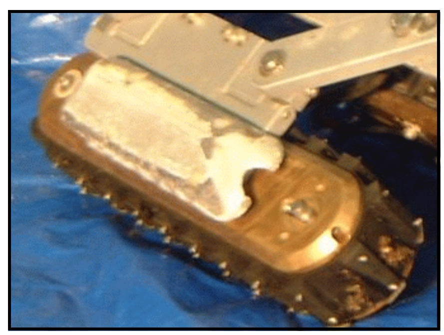


Figure 19.—Lead weight attached to the VersaTrax 150 system to provide added traction.

Figure 20.—VersaTrax 150 system traveling up a 30-degree dry invert slope.

Graphs 2 through 10 show the effects of flat and 10-degree invert slope inclination for 8-, 12-, 15-, and 18-inch diameter pipes. In general, invert slope inclination reduces the amount of cable tether a transport vehicle can pull by 10 to 70 percent depending upon the pipe diameter, degree of pipe bend, and the invert condition. Only the VersaTrax 150 system was able to travel up an invert slope inclination of 30 degrees. However, due to a significant loss of traction, only a very short length of cable tether could be successfully pulled up the slope. Also, for the 30-degree inclined slope, the VersaTraz 150 system could only travel up a dry invert (fig. 20) and could not navigate any pipe bends.

Conclusion: The difference in invert slope inclination between flat and 10 degrees can reduce cable tether by 10 to 70 percent depending upon the pipe diameter, degree of pipe bend, and the invert condition. The use of invert slopes in the range of 0.01 to about 0.09 (5 degrees) would appear to be the most reasonable maximum invert slope inclination. Slopes greater than 10 degrees are not recommended due to the significant loss of traction that occurs when camera-crawlers are pulling long cable tethers. These performance tests were conducted for inclined invert slopes. Sloping declines generally do not result in camera-crawler traction issues. Camera-crawlers have a much greater cable tether pulling capacity on a sloping decline. The transport vehicle has a free wheeling clutch mechanism on the track unit that allows for high speed retrieval either manually or by a cable takeup reel. Therefore, no performance tests were conducted for sloping declines. A designer should consider both downstream and upstream access entry locations.

5. *Invert Conditions.*—Invert conditions greatly affect the success of a video inspection. Invert conditions that reduce traction of the transport vehicle also reduce the amount of available cable tether. Bacterial growth and sediment deposition are the most common types of invert conditions. Since slippery invert conditions are more common than gravel, the performance testing focused on comparing cable tether pulling capabilities on dry and slippery invert conditions.

Figure 21.—Example of sediment deposition encountered during a video inspection of a toe drain.

Figures 21, 22, and 23 show examples of invert conditions that have been encountered during recent remotely controlled video inspections of toe drains by the Technical Service Center.

Graphs 2 through 10 show the effects of dry, slippery, and gravel invert conditions for 8-, 12-, 15-, and 18-inch diameter pipes, pipe bends of 0-, 22.5-, 45-, and 90-degree bends, and

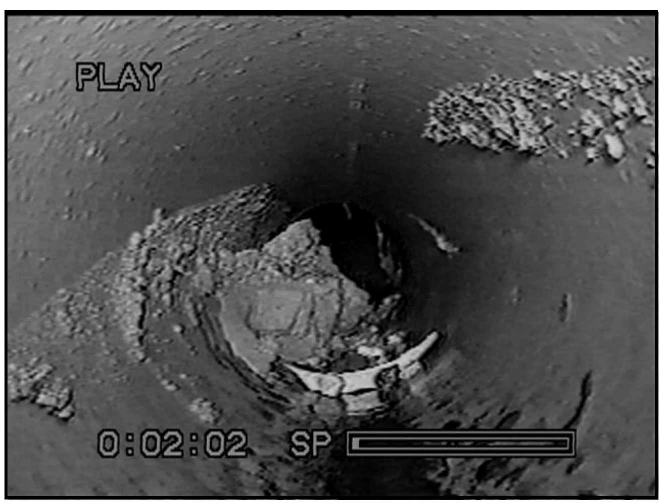


Figure 22.—Example of gravel and fines deposition encountered during a video inspection of a toe drain.

Figure 23.—Example of bacterial growth encountered during a video inspection of a toe drain.

Figure 24.—Example of the dry invert condition.

Figure 25.—Example of the slippery invert condition.

invert slope inclinations of flat and 10 degrees. In general, invert conditions reduced the amount of cable tether a transport vehicle can pull by 10 to 60 percent depending upon the pipe diameter, degree of pipe bend, and the invert slope inclination. Figures 24, 25, and 26 show dry, slippery, and gravel invert conditions, respectively.

Figure 26.—Example of the gravel invert condition.

Conclusion: Slippery and gravel invert conditions can reduce cable tether pulling capacity by 10 to 60 percent compared to dry invert conditions depending upon the pipe diameter, degree of pipe bend, and the invert slope inclination. Adverse invert conditions generally occur gradually over time. Periodic cleaning of the pipe can reduce the impacts on cable tether due to invert conditions.

The effects of flowing water were not modeled as part of this research program due to the physical limitations of trying to put flowing water through the pipe. However, based on actual experiences encountered during video inspections of toe drain pipes with flowing water in them, its presence does not greatly affect tether pulling capacity.

Bibliography

Inuktun Services Ltd, *VersaTrax Pipe Inspection System Manual #839*, September 2000.

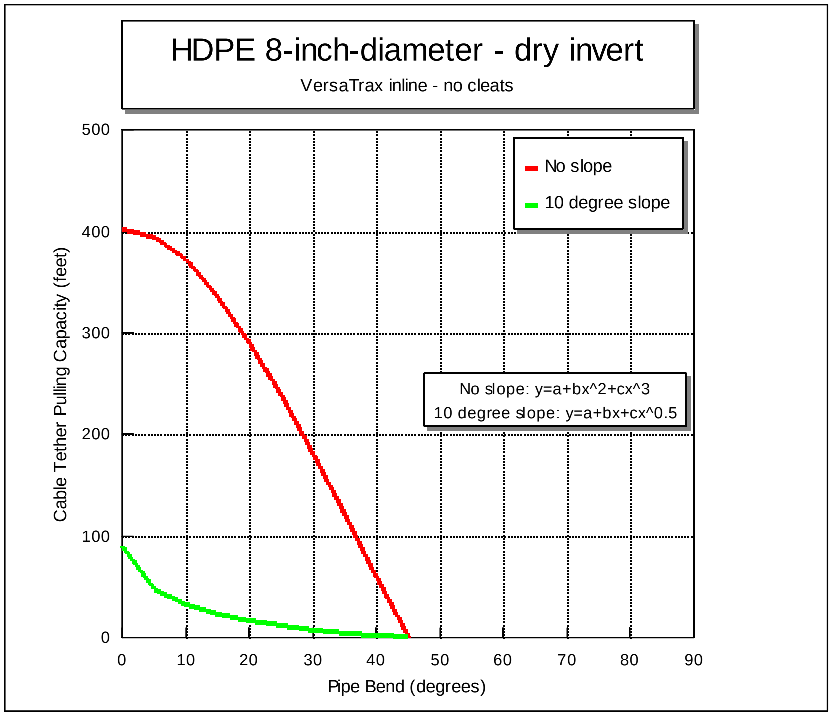
Graphs of Camera-Crawler Performance

Graph 1

Curve equation coefficients

No slope:
a = 405.31013000000
b = -0.88572100000
c = 0.00878011470

10° slope:
a = 90.10236200000
b = 1.36118170000
c = -22.43697600000

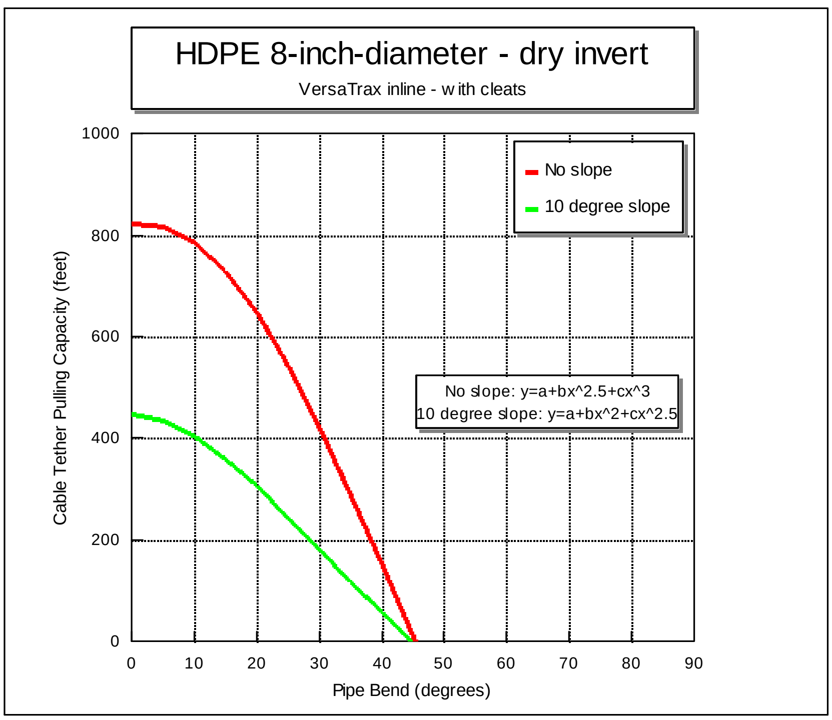


Graph 2

Curve equation coefficients

No slope:
a = 824.03888000000
b = -0.17866936000
c = 0.01770202400

10° slope:
a = 446.90842000000
b = -0.62396908000
c = 0.05995915900

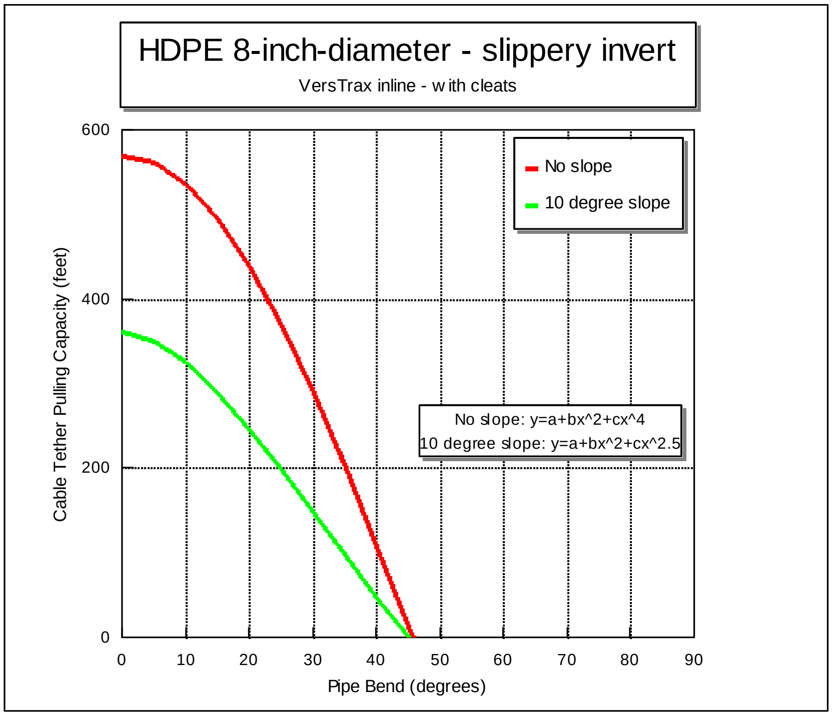


Graph 3

Curve equation coefficients

No slope:
a = 571.04876000000
b = -0.34338478000
c = 0.00003367892

10° slope:
a = 361.07671000000
b = -0.49988957000
c = 0.04799311700

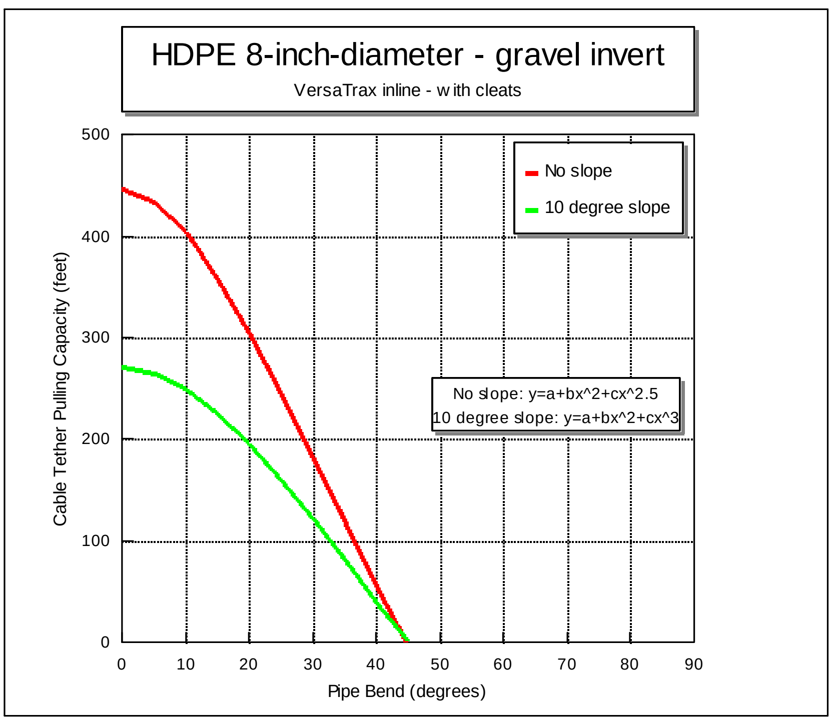


Graph 4

Curve equation coefficients

No slope:
a = 446.90842000000
b = -0.62396908000
c = 0.05995915900

10° slope:
a = 271.17391000000
b = -0.23367840000
c = 0.00222436930

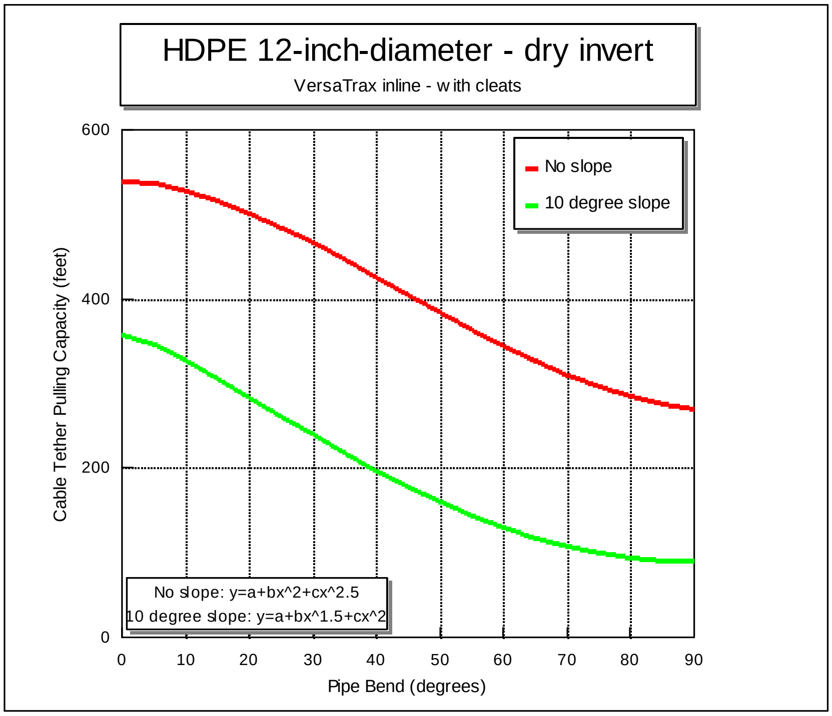


Graph 5

Curve equation coefficients

No slope:
a = 540.26918000000
b = -0.14719461000
c = 0.01199827900

10° slope:
a = 358.42369000000
b = -1.28203750000
c = 0.10203537000

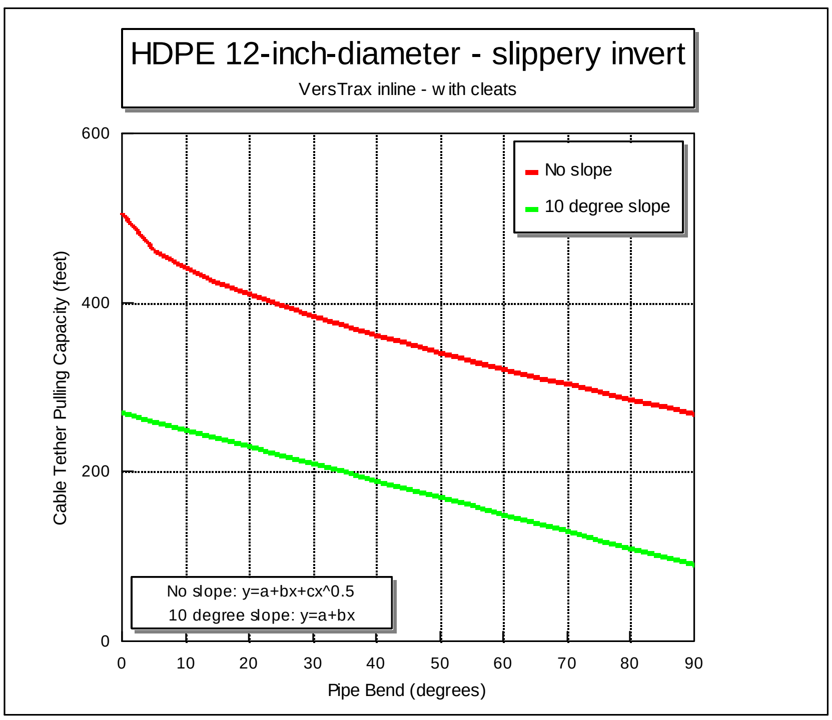


Graph 6

Curve equation coefficients

No slope:
a = 540.11567000000
b = -0.01254663800
c = -17.76999600000

10° slope:
a = 269.21184000000
b = -0.64101875000
c = 0.05101768500

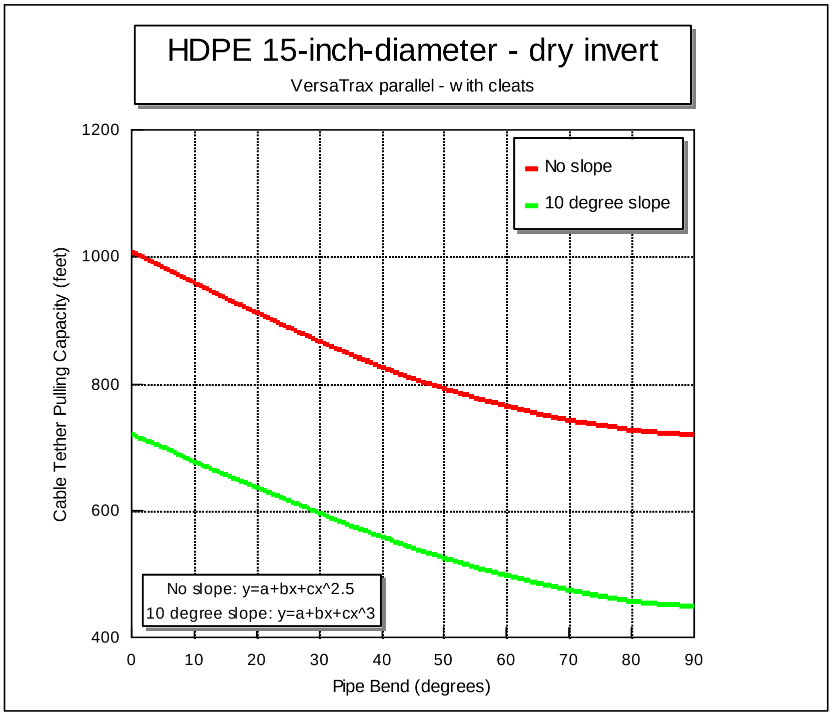


Graph 7

Curve equation coefficients

No slope:
a = 1009.93020000000
b = -5.11496080000
c = 0.00221790510

10° slope:
a = 721.55172000000
b = -4.30706080000
c = 0.00015893288

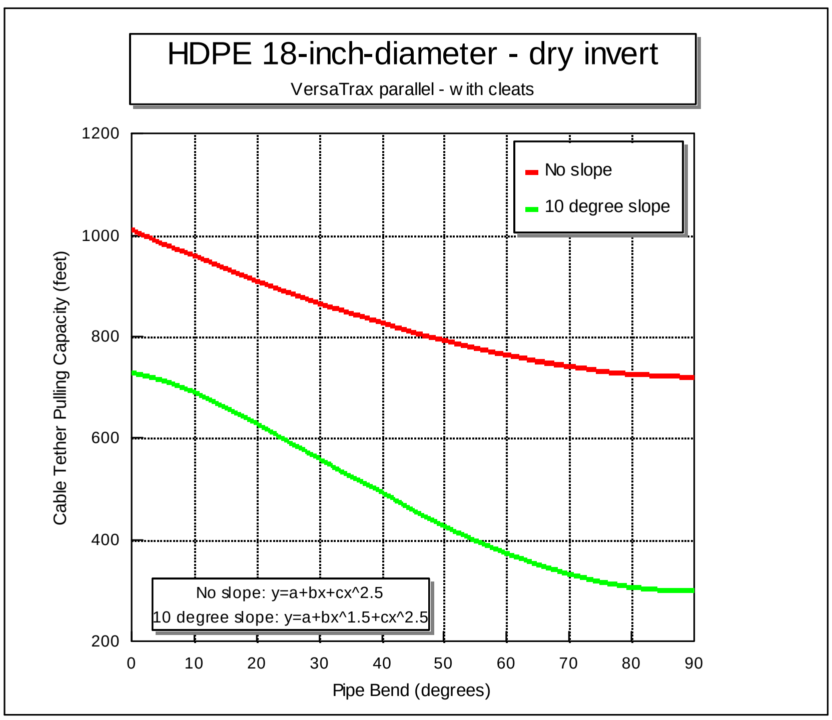


Graph 8

Curve equation coefficients

No slope:
a = 1009.93020000000
b = -5.11496080000
c = 0.00221790510

10° slope:
a = 729.14087000000
b = -1.28519060000
c = 0.00868130570

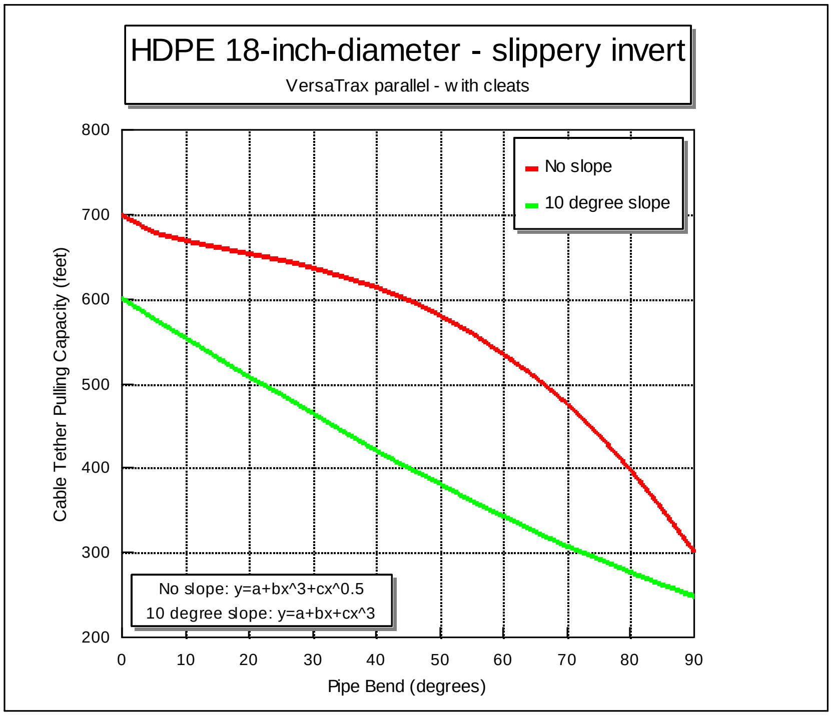


Graph 9

Curve equation coefficients

No slope:
a = 699.68875000000
b = -0.00042852570
c = -9.19022890000

10° slope:
a = 598.24854000000
b = -1.42448610000
c = 0.11337263000

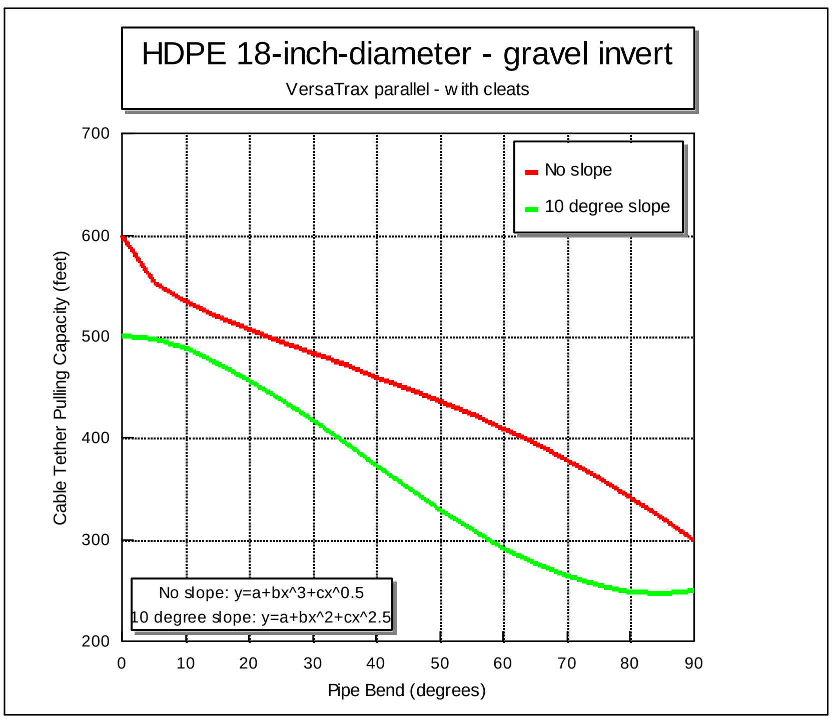


Graph 10

Curve equation coefficients

No slope:
a = 598.73047000000
b = -6.22651110000
c = 0.42438920000

10° slope:
a = 500.00000000000
b = -2.22222220000
c = 0.00000000000



C-3—High Pressure Water Jetting Equipment.—The Great Plains Regional Office of the Bureau of Reclamation purchased a complete high pressure water jetting system in 1998. The system provides the capability of adjusting cleaning pressures and nozzle configurations to best suit the site conditions. The major pieces of equipment (diesel engine and pump) can be located remotely from the drains being cleaned. The pump is powered by a 215-hp diesel engine (figs. 27 and 28) and rated to deliver 15,000 lb/in^2 at 22 gal/min at the pump discharge. Fifteen hundred feet of ½-inch I.D. high pressure hoses in 100- and 50-foot lengths transfer water from the pressure regulator on the pump unit (fig. 29) to the control apparatus in the drainage tunnels (fig. 30), which regulates flows going into the drains. A flexible ½-inch I.D. lance hose (fig. 31) is connected to a variety of both rotating or nonrotating cleaning heads (figs. 32 through 34) in the drains.

A submersible pump, rated at 50 gal/min and 100 lb/in^2, supplies water to the water jetting unit and is double filtered before entering the jetting system (fig. 35). A bypass device (fig. 36) diverts excess flows from the delivery pump when they are not needed. A portable electric car wash unit (fig. 37) was also purchased for cleaning the lateral discharge pipes from concrete dam foundation drains as well as for performing general cleaning of gallery walls and floors. Figure 38 shows equipment for flushing drains before and after cleaning. Individual drains are usually isolated and measurements taken before and after drain cleaning

Figure 27.—High pressure water jetting unit used to clean drains. Pump powered by a 215-hp diesel engine.

Figure 28.—Trailer supporting pump, tool boxes and diesel engine. The black hoses are high pressure ½-inch delivery lines. The green coiled hose is the ½-inch flexible lance that is used with the cleaning head down the drain.

Figure 29.—Driller adjusting the pressure regulator between the high pressure pump and the delivery lines supplying water to the control.

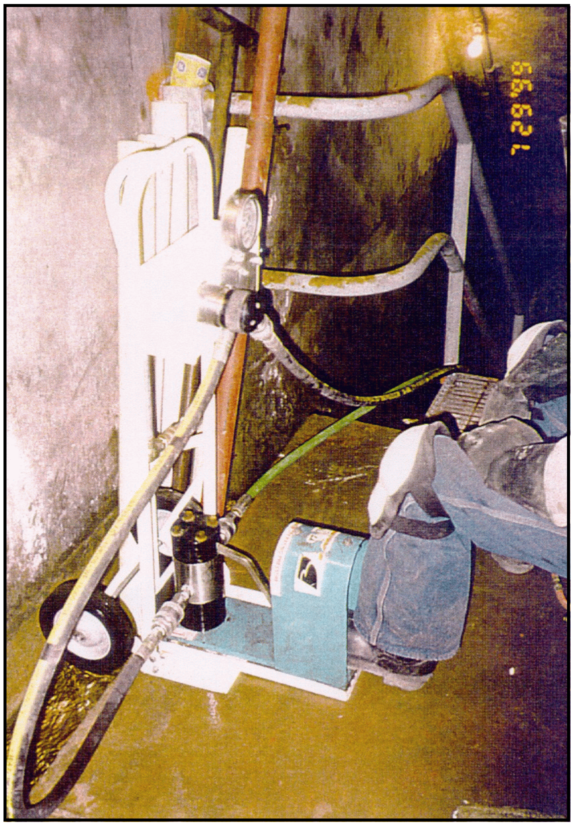


Figure 30.—Control unit near the drains that serves as a foot-operated dump valve for releasing pressurized water to the drain cleaning head. It is equipped with a gauge for monitoring pressure at the end of the ½-inch high pressure supply line coming from the main pump.

Figure 31.—Green flexible lance hose being lowered into a drain using the tripod with electric winch that is being used to help retract the cleaning unit from deep, near vertical holes.

(figs. 39 and 40). Videotaping of drain holes before and after cleaning is also important for evaluating the effectiveness of drain cleaning (fig. 41). The cleaning system is mounted on a flatbed trailer along with two large tool boxes of sufficient capacity to contain all of the hoses and other cleaning paraphernalia.

a. *Drain Cleaning.*—Pump pressures of about 11,000 lb/in^2 to 12,000 lb/in^2 and a flow rate of around 20 gal/min are typically used with the equipment. With those settings and the appropriate cutting head configurations, a good cleaning rate is about 2 to 4 ft/min. At that rate, nearly all calcium carbonate deposits are removed with only minimal damage to soft foundation materials, such as limestone. At higher pressures, greater flow rates, and slower feed rates, softer zones of the foundation are removed in addition to the calcium carbonate deposits. This can enlarge the diameter of a drain hole by several inches.

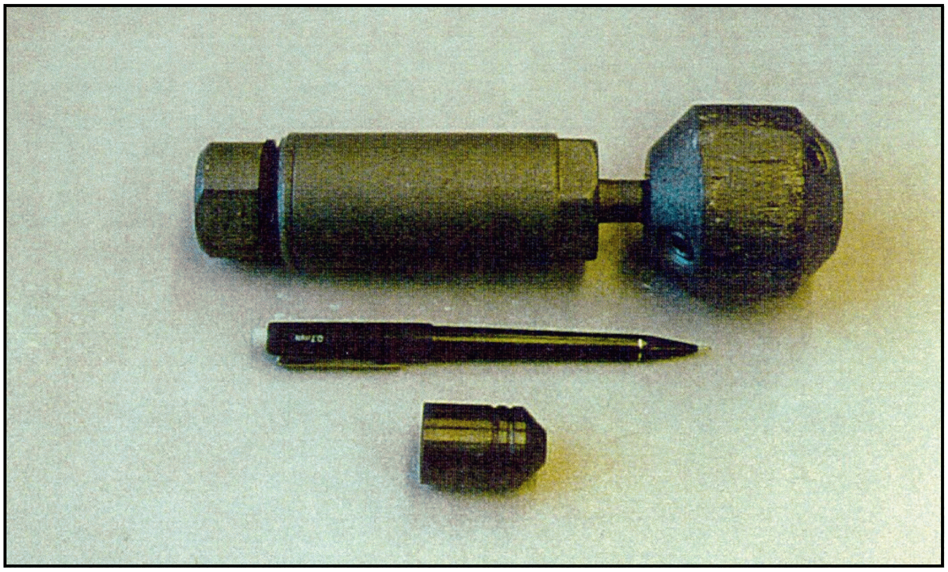


Figure 32.—2¾-inch O.D. rotating cleaning head shown at top, and ¾-inch O.D. nonrotating sewer cleaning nozzle at the bottom. The top unit has five replaceable jets of varying sizes to adust flow rates and pressures. Two point back at 45°. Two point forward at 45°. One is offset from the front at 10°. The bottom nozzle has 10 predrilled holes. Six are around the outer perimeter and point straight out. Four point straight out from the front.

The drain cleaning equipment typically takes about a half day to set up, and moving from hole to hole takes 30 minutes or less if the holes are reasonably close together. There are no heavy parts to move as with a conventional drill, only a tripod and winch assembly. The main pump is very noisy, but is located remotely from the drains being cleaned (outside of the dam for concrete dam foundation drains). Inside a confined space, such as a drainage gallery, the noise is minimal and earplugs are not required.

The long stiff hoses used to deliver water from the pump to the drain holes need to be protected from excessive abrasion caused by constant vibration and need to be manipulated to avoid kinking. Initial selection of the appropriate settings and video observation to avoid hole damage at the start of each job or after significant setting changes are time consuming. Spot checking during drain cleaning with a video camera should be performed to make sure no unacceptable changes occur because of changes in the rock being cleaned.

The total cost for cleaning 3-inch diameter concrete dam foundation drains with this equipment at Reclamation dams has ranged from about $6 to $12 a foot.

b. *Cost of the Water Jetting Equipment.*—Total cost of all equipment purchased for cleaning and monitoring was approximately $129,722 (1998 costs), itemized as follows:

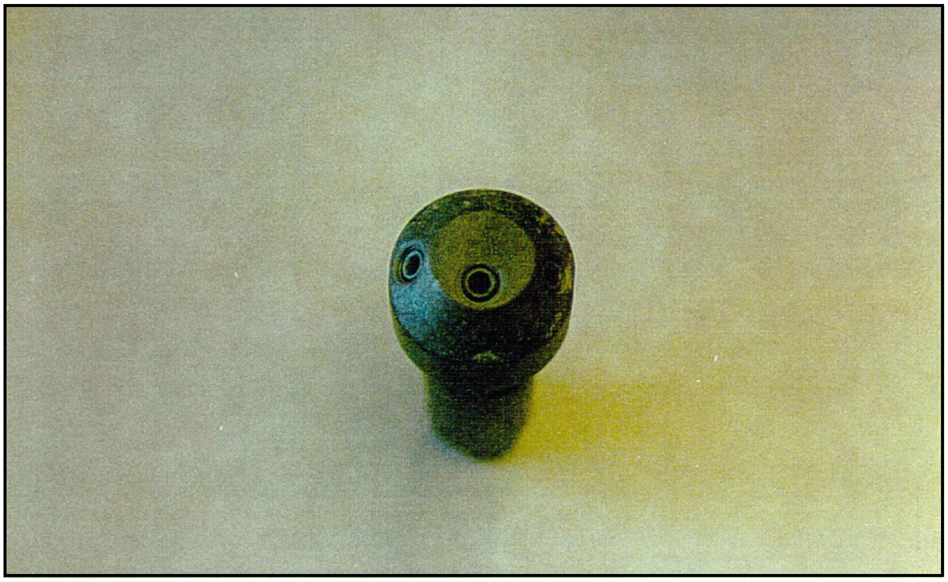


Figure 33.—Front view of the rotating cleaning head, showing the opposing side jets pointing forward at 45° and the offset front jet pointing forward at 10°.

Jet washer unit	$92,128
Tripod and winch	6,530
Electric car washer	1,005
Submersible pump	805
Generator	2,432
Flatbed trailer	4,430
Tool boxes	798
Packers	700
Safety items	1,009
Down-hole camera equipment	19,885
Total	$129,722

Most of the cost was included in the initial purchase of the jet washer unit and the down-hole camera equipment. The remainder of the items were purchased as needed.

As part of the purchase price, the supplier of the high pressure water jetting equipment was required to provide a company representative for 2 days of training involving setup and familiarization with equipment, and safe hands-on operation using Reclamation drill crew personnel.

c. *Safety*.—Personal protective equipment had previously been purchased and checked out. It consisted of hard hats with face shields, heavy gloves, and steel-toed boots, with personnel actually running the lance into the drains wearing steel-toed rubber mine boots. Waterproof rain suits were purchased, but generally proved unnecessary.

The high pressure cleaning unit has the potential to do serious damage to both personnel and structures. Safety issues are of utmost concern. A job hazard analysis should be developed for the particular work site. A Reclamation Industrial Hygienist has written a specific safety procedures booklet for the high pressure water jetting unit. Because of the significant safety issues, untrained personnel should not be allowed to operate this equipment.

Figure 34.—Front view of nonrotating sewer cleaning nozzle. One predrilled hole points straight forward from the center. Three other predrilled evenly spaced holes point straight forward from the beveled portion. This cleaning head is used for flushing with pressures of less than 2,000 lb/in^2.

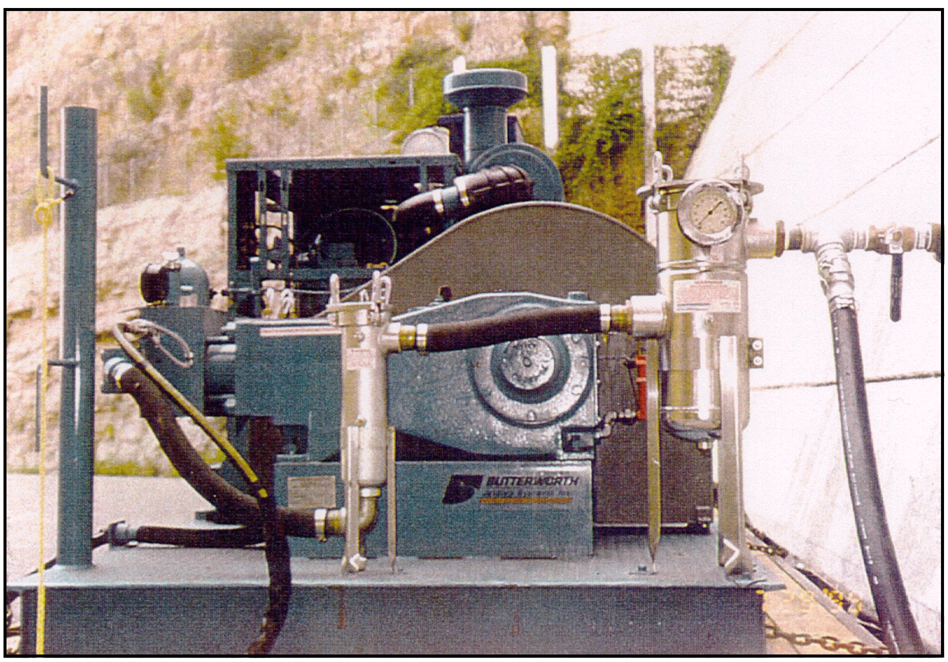


Figure 35.—Double filter system for cleaning the water entering the high pressure pump. Clean water prolongs the life of the pump, cleaning jets, and nozzles. The filters are changed at least daily—more often if the water source is dirty.

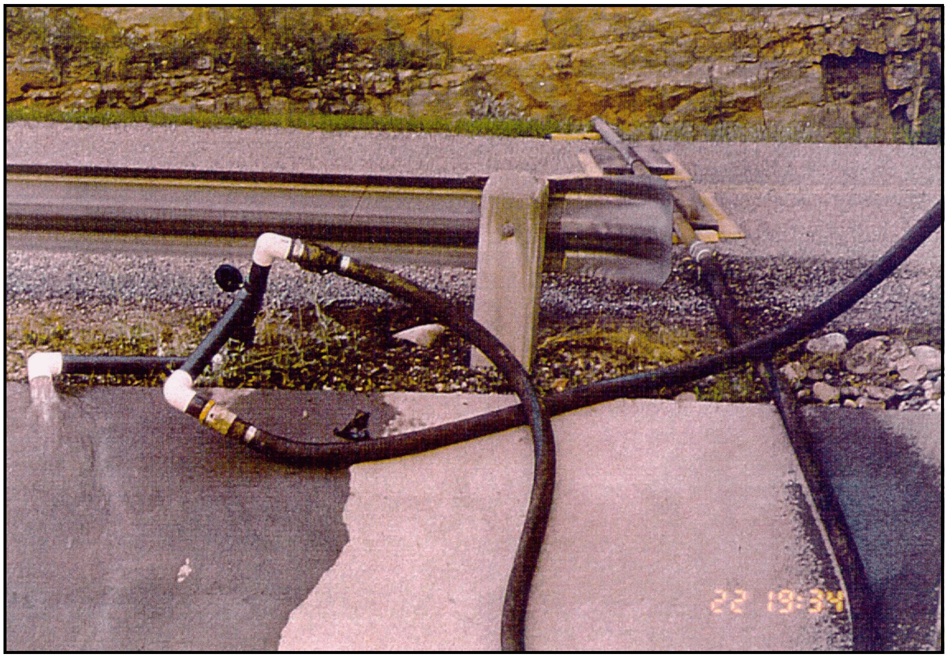


Figure 36.—Bypass device used to divert all or part of the flow from the supply pump when the flow is not being used.

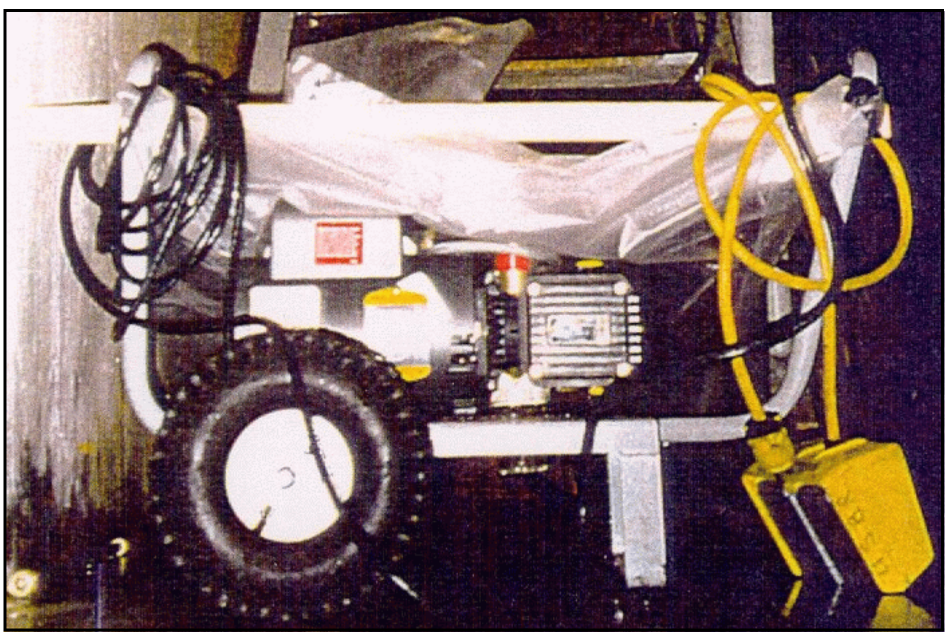


Figure 37.—Portable electric car wash unit used for cleaning the lateral discharge pipes from the drains. It is capable of producing 2.1 gal/min at 1,200 lb/in^2.

Figure 38.—Electric pump, garden hose, and flush-jointed PVC used to flush drains before making an initial down-hole video examination. Water was obtained by placing a temporary dam in the nearby drainage gutter.

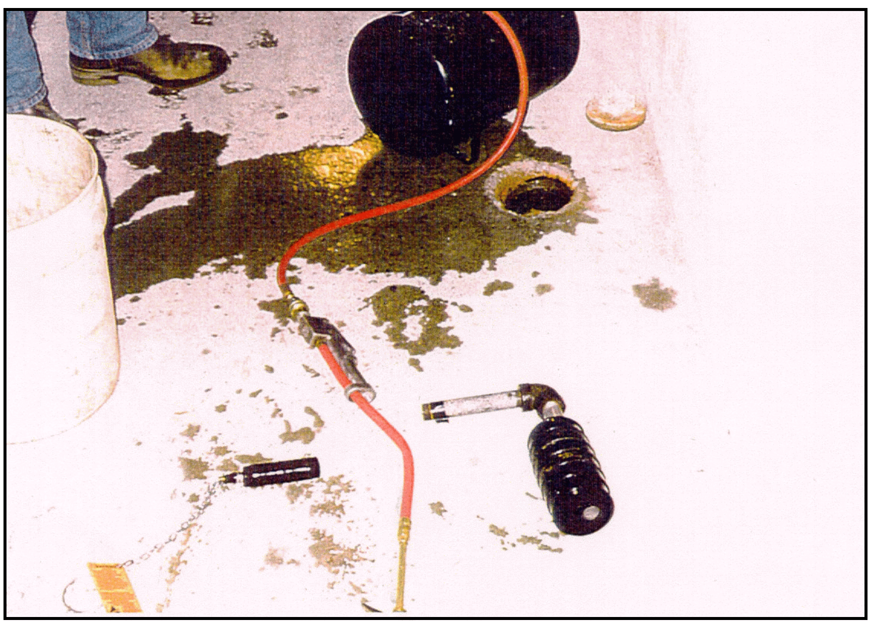


Figure 39.—Portable air tank and pneumatic packers to take before and after cleanout drain flow measurements. The small packer closes the end of the lateral discharge pipe, and the larger packer with the elbow pipe extension goes in at the top of the drain.

Figure 40.—Individual drain flow measurement being recorded.

Figure 41.—Down-hole video equipment used to determine the condition of the drains before and after cleaning.

C-4—Drilling Equipment for Drains in Limited Access Areas.—For many years, Reclamation drill crews have used small portable drills to drill holes and clean drains in tunnels, drainage galleries, outlet works, spillways, and other areas having restricted or confined spaces. The drills are of necessity small and lightweight, use short rods and barrels and a variety of bits, depending on the individual job, and are usually powered by air or electricity. The power supply often must be located at a considerable distance from the drill site to meet acceptable air quality standards. Most of the drilling has been limited to hole diameters of less than 4 inches and maximum depths of less than 150 feet.

Success rates of the drilling operations using these types of equipment have been quite erratic. Most of the drills are slow and of limited power and capabilities. Historically, very poor documentation has been made of drilling rates and the costs for operations using these types of equipment.

Five different types of portable drills are presented here. No endorsement of specific drills or drill manufacturers is intended. These drills are presented because of the familiarity that Reclamation drill crews have gained in using these drills. Each drill is described separately. Drills may be available or may become available in the future which are better suited for a given application than those presented.

Drilling rates and costs are presented based on recent experience at Pueblo Dam, where two of the drills were used.

a. *Drill Types.*—

- *Longyear 65 Diamond Core Drill.*—This drill has long been a standard for drilling in confined or poorly accessible spaces. It is a single-column-mounted, screw feed drill with an "E" size quill rod powered by an attached 20-hp rotary air motor. The motor requires at least 90 lb/in^2 of air at 250 ft^3/min from a 2-inch diameter supply line. The drill has a pneumatic rod puller and is light enough (about 200 lb) to be carried by two people with difficulty, or it can be disassembled into even lighter pieces. It mounts on a 3-inch diameter single column with a swivel-knuckle that allows for angle drilling in a 360° radius. Drill water is supplied through a water swivel at the top of the quill rod. The drill is rated to a maximum depth of 300 feet, using NX size tools. Larger sizes are not recommended. It has a stroke of 2 feet and a manual chuck.

  The Longyear 65 drill is a very common drill that is highly adaptable to many drilling situations, and because of the good torque and down-hole pressure capability, it is a good choice for drilling new holes. The main disadvantage is that the screw feed does not allow for varying the feed rate without changing the speed of the bit rotation. It is a very dependable but slow drill.

- *Hilti DD 250E Diamond Core Drill*.—This is an electrically powered, column-mounted drill on a portable stand. The power unit is a 115-volt, 20-ampere electric motor that is designed to use a masonry bit or hole saw. This drill may be modified to use standard core bits up to 6 inches in diameter. Drill water is supplied through a water swivel between the motor and bit. The motor mounting bracket slides over a square column that is bolted to a base plate, which in turn is bolted with cinch anchors to the surface to be drilled. The driller applies up or down pressure through the use of a rack and pinion manual feed. The drill also has a swivel mount for drilling angle holes up to horizontal.

  The Great Plains and Pacific Northwest Regional drill crews use this drill regularly. It is an excellent drill if used for the proper purposes. It is light in weight (approximately 125 lb fully assembled) and can be moved, set up, and operated by one person. One three-man crew can easily operate two of these drills simultaneously. Without modification, they can drill large diameter concrete cores by using a thin kerf hole saw. Retrieval of core is accomplished by drilling to the depth desired or limit of the bit, removing the bit from the hole, breaking the core with a wedge, and then going back over the core with a core lifter and pulling it from the hole. Drilling core sizes over 6 inches in diameter is not recommended, although concrete cores of up to 10 inches in diameter have been drilled. The manufacturer supplies accessories that allow for drilling above horizontal angles. This drill is slow and of limited power.

- *Jet Rock Drill*.—This model JRD-40R-1 drill is a very small (41 lb) air-powered unit designed to be hand held. It was modified to be swivel mounted on one of masonry drill columns described above. A water swivel was fitted between the motor and the bit. The drill was recently used in a spiral staircase area at Pueblo Dam where access for a larger drill was prohibited. It was used to clean "N" size drill holes that were up to 65.2 feet deep. The use of the drill at Pueblo Dam was far past its design capacity. As a consequence, numerous problems developed with the drill shaft twisting off, the water swivel breaking, and the air vanes in the motor shearing. A drill of this type should be limited to use in cleaning small, shallow holes. It is not designed for coring operations.

- *Sprague and Henwood Core Drill*.—The Mid-Pacific Regional drill crew owns the Sprague and Henwood core drill. It is an air powered, double-column-mounted, hydraulic feed core drill with a 2-foot stroke. Water is supplied through a swivel at the top of the quill, which can be any size drill rod from "E" through "B". Rods are added or removed from the drill string either by removing the water swivel each time or by breaking the drill rods under the manual chuck. The air motor requires a 2-inch air supply line with 450 ft^3/min at 90 lb/in^2 of air. The feed rates are controlled by a hydraulic control system that is separate from the air motor used to turn the drill rods. This arrangement allows the driller to vary the feed rate or the rotation rate independent of each other to better adapt to changing drilling conditions. The drill is capable of drilling holes ranging in size from "E" to at least 6 inches in diameter. It is rated to a depth of 200 feet, using "N" size equipment.

A modified skid was constructed that attached to the double columns and enabled the driller to move the whole drill short distances without disassembling it. The skid mount still allowed the rig to be used for angle hole drilling, and be either bolted down at hole sites or held in place by screw jacks on top of the columns. The drill was moved from site to site on pipe rollers placed under the skids.

This drill is not really powerful enough to quickly drill new holes of "N" size or larger. Major disadvantages are its weight of over 500 lb fully assembled (or broken down into approximately 200-lb maximum size parts), the need for having two or more people to move and set up the equipment, and the requirement of a large air compressor for the air supply. This drill is no longer available from the manufacturer.

- *Diamec 232 E Drill.*—The Great Plains Regional drill crew purchased the Atlas Copco Diamec 232 E core drill in early 2001. The drill was put into immediate service coring new holes to inspect and sample lift lines in Pueblo Dam. The Diamec 232 E is an electrically driven, all hydraulic drill, which may be set up as three separate components. The drill components include the core drill, the operator's control panel, and the power unit. A skid-mounted electric water pump is included with the system, but other water supplies could be used.

  The drill component features an A-size hydraulic chuck with a maximum axial holding torque of 6,600 lbf, and a rod holder with a maximum holding force of 2,700 lbf. The maximum feed beam, down-hole pressure is 3,400 lbf pulling and 4,500 lbf pushing. The rod running speed is 2.6 ft/s traveling in and 3.3 ft/s traveling out. The rotation unit is variable speed, ranging from 550 to 2,200 r/min under load, and has a maximum torque of 6,600 lbf. The drill may be mounted on either a single column mount or an adjustable, skid-type, 180° mounting frame. Between the two mounting types, and the hydraulic operation, the drill can operate oriented in virtually any direction.

  The operator's control panel includes a system pressure gauge, a feed pressure gauge, a flush water pressure gauge, a feed force control valve, and a rod holder control valve. In typical usage, the control panel may be positioned between the other two components and separated, by use of hydraulic lines, from the other components by a distance of up to 20 feet. This separation allows the operator a clear view of the drill and hole collar while remaining a safe distance from both the core drill—important when drilling up and collared in fractured material—and the power unit, allowing some noise buffer between the operator and the equipment.

The power unit is the heaviest component, weighting 507 lb. The effects of moving this weight are greatly reduced by mounting on a manufacturer-supplied, skid-type frame that incorporates three wheels. The power unit consists of a 20-hp motor that operates on 60 Hz at 440-volt, 3-phase electric power. By use of compatible #6 AWG wiring rated at 72 amps, the power unit may be tethered over 750 feet from an acceptable power source. The power unit operates at a 71 dB sound power emission while an equally powered diesel unit operates at 110.8 dB.

The advantages of this drill are speed, power, noise level, and versatility. Disadvantages include slow set up and difficulty in initial insertion into a gallery or other restricted or confined space.

b. *Drill Rods, Core Barrels, and Bits.*—Drill rods, core barrels, and bits are made in a variety of sizes. Three basic sizes are used for the drills discussed here: "E," which has a diameter of about 1.47 inches; "A," which has a diameter of about 1.89 inches; and "N," which has a diameter of about 2.97 inches. These sizes are approximate and vary slightly from one manufacturer to another. The other designations for drill rods, such as W or WJ, indicate the type of threads. The following drill rods, core barrels, and bits were used with the four types of portable drills discussed above.

- *Longyear 65 Diamond Core Drill.*—This drill uses an "E" quill rod with three threads per inch. From that rod, Great Plains Regional drill crews have substituted "AW" rods, "AWJ" rods, "NW" rods, or "NWJ" rods, depending on the job. The Great Plains Region presently has about 165 feet of "AWJ" rods in 1-, 2-, and 5-foot lengths for use with these small drills.

  These various rod sizes can be used to substitute a likewise variable group of barrels or solid "plug" bits, depending on the objective of the individual job. For most of the core drilling, 5-foot or shorter "N" series barrels have been used. For drain cleaning, where no core is required, some type of "N" size plug bit to fit the existing holes has generally been used.

- *Hilti DD 250E Diamond Core Drill.*—This drill uses 1¼-inch masonry drill rods with 7 threads per inch that screw into a variety of masonry bits and hole saws ranging in size from about 1½ to 10 inches O.D. The drill can also use other types of rods and core barrels, but is too underpowered to use them effectively.

- *Jet Rock Drill.*—This drill is restricted to substituting "AWJ" rods attached to either an "N" size core barrel for coring (with very limited success), or an "N" size plug bit or a 215/16-inch rock bit for drain cleaning.

- *Sprague and Henwood Core Drill.*—This drill can use any of the previously mentioned rods, barrels, and bits but is not recommended for use with masonry bits.

- *Diamec 232 E Drill.*—Reclamation's experience with this drill has been recent. To date, the drill has been used with "AWJ" rods, and "N" series core barrels. Any "N" series bit can be used with this drill. The Great Plains Region plans to use this drill with a 6-inch diameter core barrel.

c. *Drilling Rates and per Foot Costs.*—No detailed records are available documenting former drilling programs using small, portable drills. Drilling rate data were obtained from the recent drain cleaning at Pueblo Dam, where detailed records were kept of the activities of the jet rock drill and the Sprague and Henwood core drill. The two drills were utilized in areas having vastly different access problems, but similar drilling conditions.

The jet rock drill was used exclusively for cleaning shallow "N" size holes in a vertical spiral staircase area that was too small for almost any other type of drill. A crew of 2 people cleaned 29 angle holes averaging 54 feet deep (a total of 1,552 ft) over a period of 21 shifts of 10 hours each. Part of this time involved removing and replacing stairs and repairing the drill, which was sadly mismatched for the job. The drilling progress rate amounted to approximately 74 feet per shift, or 7.4 feet per hour. Based on the proportion of total labor and total job cost, the jet rock drill costs about $24.81 per foot to operate (not including the cost of numerous repairs).

The Sprague and Henwood core drill was used to clean 83 "N" size vertical holes averaging 99.5 feet deep (a total of 8,260.7 ft) in the main gallery at Pueblo Dam. Access was good with no tunnel invert obstructions, and the fully assembled drill was moved from hole to hole on small rollers made of pipe placed under the skids. Two crew members ran the rig and worked 10 hour shifts. The drill ran mostly trouble-free. The progress rate for this drill over a period of 39 shifts was 212 feet per shift or about 21 feet per hour. This amounted to a cost of approximately $8.66 per foot.

The total job cost was $110,000 for 9,812 feet of drain cleaning of "N" size holes that were partially encrusted with mostly calcium carbonate. This produced an overall cost of approximately $11.21 per foot using the two drills.

d. *Conclusions and Recommendations.*—None of the five types of small portable drills used by the Bureau of Reclamation drill crews and discussed here has proven to be the "perfect" drill for every job. Probably no such drill exists, because each job seems to be very site specific with its own individual problems. For those places where it can be used, the Diamec 232 E rig is by far the best choice, especially when it comes to the speed of drilling. The Longyear 65 is probably the most versatile with regards to accessability and adaptability but is very slow, because it does not have a separate feed control. Both the Hilti DD 250E Diamond core drill and the jet rock drill have very limited applications. The Longyear and Sprague and Henwood drills are no longer being manufactured, and repairs may soon become a problem.

A good drill for drilling or cleaning drains would be a small, electrically powered, skid- or column-mounted drill capable of drilling at any angle. For use in tunnels with remote access and no electric power source, the drill should have a separate skid- or trailer-mounted, diesel generator power unit complete with scrubber. The generator should have enough capacity to run not only the drill, but also lights and circulation fans. The requirement could cause the generator to be too large for reasonable use, so other options may have to be considered. The drill should have a hydraulic feed control separate from the power unit so quick advancement can be made through those portions of drain holes where little or no blockage is encountered. An electrically powered drill is preferred, because it is much quieter than an air-powered one, and does not require the double hearing protection now necessary with an air drill like the Longyear 65.

For drill rods to be used with the small portable drills, "AWJ" rods in 1-, 2-, and 5-foot lengths work very well. They are light in weight yet sufficiently strong, short enough to work in small tunnels, and screw together easily because of their tapered ends.

C-5—Guidance on Sampling, Transportation, and Analysis of Materials in Drains.—The topic of monitoring seepage sediments in dams is a reoccurring one. Examples of sediments that may be present in a drain are soils, clay to sand size sediments, biological growths and films, and precipitates.

Investigators often need guidance to estimate costs and specify sampling procedures for dam safety monitoring programs. Without guidance, investigators may sample critical material and handle it inappropriately, which requires resampling and loss of time and resources. The brief guidance in this appendix provides advice on how to effectively determine what is fouling a drain. This appendix includes a discussion of drain inspections, materials, sampling, transportation, and testing.

This appendix should be used as a practical guide. It should not be considered complete or a definitive dissertation on microbiology and sampling.

a. *Inspections*.—It is likely that deposited materials will be discovered in drains during operation and maintenance (O&M) inspections and activities. The Dam Safety Office, the Technical Service Center (TSC), and regional and area offices conduct dam inspections within the Bureau of Reclamation as part of Comprehensive Facility Reviews (CFRs) and Periodic Facility Reviews (PFRs). Annual Facility Inspections are performed in years in which a CFR or PFRs are not scheduled. Drain inspections are typically performed using closed circuit television (CCTV) equipment.

If deposited materials are present, they will be revealed during inspections or monitoring of the drain. The sampling will typically be scheduled for a later date or routine O&M activities. Instructions to O&M personnel should be clear and concise. Figures 42 to 53 contain photographs that show examples of biofouling, biofilms, bacterial growths, mineralization, sediments, and vegetation found in drains during inspections.

b. *Material typically found in drains*.—

1. *Sediments*.—Sediments may be transported in seepage and collected at outfalls and drain outlets. Sediments may be evidence of piping or internal erosion of a structure, which may have serious structural consequences. Evidence of erosion of dam or foundation materials surrounding a drain requires immediate attention. Continued erosion could result in partial or complete dam failure. If piping is suspected, a sample of the suspected eroding material should be collected by drilling or excavation. This is to allow the potential parent material to be compared to the drain sediment.

Samples of any significant or unusual buildup of sediments in a drain should be petrographically examined to provide evidence for piping with in the dam or foundation.

Figure 42.—The manhole shows an example of biofouling

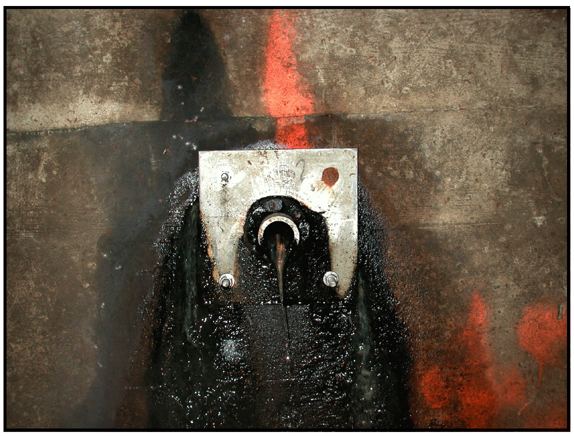


Figure 43.—The weep shows an example of a sulfate (black) related biofilm

Figure 44.—The outlet shows an example of biofouling.

Figure 45.—The weep hole shows an example of sulfur (yellow) and phosphorous (white) related biofilm.

Figure 46.—The weir shows an example of biofouling.

Figure 47.—The bacterial growth is partially covering the inside of 8-inch diameter HDPE pipe.

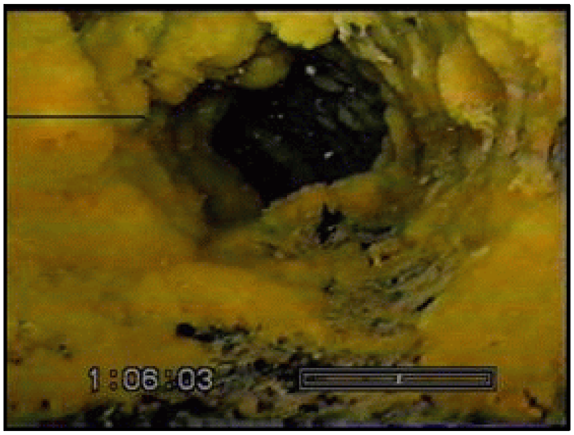


Figure 48.—Bacterial grow is shown completely covering the 8-inch diameter HDPE pipe.

Figure 49.— Bacterial grow is shown completely covering the 8-inch diameter HDPE pipe.

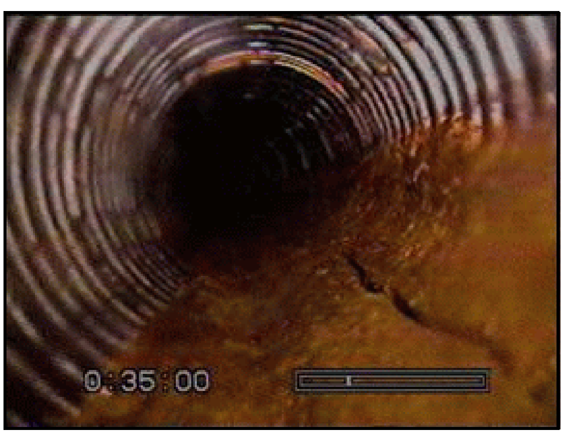


Figure 50.—Bacterial growth is shown covering the 8-inch diameter HDPE pipe invert.

Figure 51.—Calcium carbonate precipitate was observed covering the 18-inch pipe interior about 60 ft upstream of the outfall exit portal of a toe drain.

Figure 52.—Sediments were observed fouling the invert portion of a 12-inch diameter HDPE pipe.

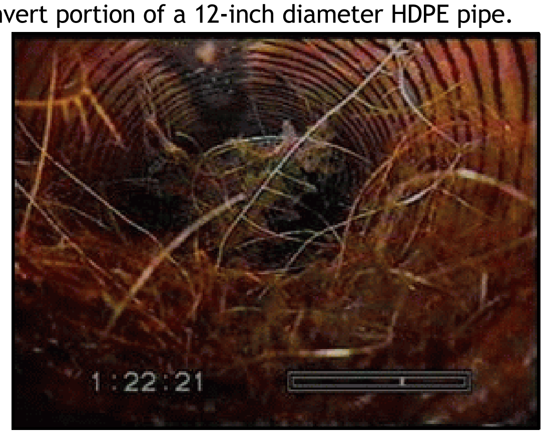


Figure 53.—Vegetative growth shown fouling 8-inch-diameter HDPE pipe.

This type of examination is most effective if accompanied by a sample of material from the suspected source or sources of the parent material. This may require that samples of several additional materials be sampled.

2. *Mineralization and encrustations.*—The accumulation of minerals deposited in a drain or the material surrounding a drain hinders water from exiting or being removed from a structure. An excellent discussion of mineralization and encrustation and groundwater constituents can be found in the *Ground Water Manual* [1] and Driscoll [2].

3. *Biofilms and drains.*—Biofilms are composed of populations or communities of microorganisms adhering to surfaces. These microorganisms form slimelike mats, which bacteria adhere to. The fouling caused by biofilms is called biofouling. Biofilms may be found on essentially any surface in which sufficient moisture is present. Their development is most rapid in low flowing systems, where adequate nutrients are available, for example, drains. The following is a discussion of some biofilms that affect drains:

- *Iron-related bacteria.*—Bacterially rich organic slimes commonly observed in Reclamation structures are composed of iron-related bacteria. Iron-related bacteria films can be sticky and cause drain blockage. Any of the following symptoms suggests the presence of iron-related bacteria or other microflora:

    - Orange, red, brown, and black colored slime
    - Slimy deposits blocking main lines and laterals
    - Unpleasant odor in water
    - Slimy, rusty deposits in water collection systems
    - Severe staining on concrete surfaces
    - Oil like films on surface water
    - White flocking, like finely shredded tissue paper, floating in water

  Iron-related bacteria are a diverse group of microorganisms widely distributed in nature. They are found naturally in fresh and salt waters and in soils. Iron bacteria are a nuisance microorganism capable of transforming dissolved iron and manganese to an insoluble form that can cause severe fouling and plugging in pipes, plumbing, well pumps, treatment plants, and distribution systems.

They tend to grow much faster and in greater quantities when the temperature rises in a drain or when exposed to air. The result of the iron bacteria converting soluble iron, from a soluble state (Fe^{2+}), to the insoluble form (Fe^{3+}), is referred to as "red water." It is in this stage that iron and manganese become deposited on the outside of the bacteria cell sheaths and the slimes they produce. The bacteria cell sheaths and slimes become encrusted with iron and manganese.

• *Other microflora that may be found in drains and wells.*—

N *Sulfate-reducing bacteria.*—Sulfate-reducing bacteria live in oxygen-deficient water. They reduce sulfur compounds, producing hydrogen sulfide gas in the process. Hydrogen sulfide gas is foul-smelling and highly corrosive. Sulfate-reducing bacteria are more common than iron bacteria. The most obvious sign of a sulfur bacteria problem is the distinctive "rotten egg" odor of hydrogen sulfide gas. The bacteria respire oxygen in sulfate ion and create hydrogen sulfide gas. Sulfate-reducing bacteria occur in waters where oxygen is absent and sufficient dissolved organic materials are present [3].

Iron bacteria may coexist with sulfate-reducing bacteria. Iron bacteria and sulfate-reducing bacteria contamination are often difficult to tell apart, because the symptoms are similar. Sulfate-reducing bacteria often live in complex symbiotic relationships with iron bacteria, so both types may be present.

N *Sulfur-oxidizing bacteria.*—Sulfur-oxidizing bacteria require oxygen to grow and convert sulfides to sulfuric acid or hydrogen sulfide to sulfates. They can be colorless, purple, or green.

N *Algae.*—Algae are small single or simple multicellular plantlike organisms that grow in the presence of light by photosynthesis. Algae occur in shallow wells or drains where there are adequate nutrients.

N *Heterotrophic bacteria.*—Heterotrophic bacteria are able to utilize organic materials as their principle source of energy and carbon for survival, growth, and synthesis.

c. *Sampling.*—During the course of an inspection, it may be determined that drains are plugged, or deposition of material in outfall or seeps is reducing the effectiveness of drains or otherwise causing a problem. Good digital photographs can help the field office and laboratory personnel communicate.

A sampling plan should be developed by those who conduct drain maintenance in consultation with project engineers and field personnel. A discussion of the problem and how to implement the plan will help define the critical issues before sampling, so appropriate analyses can be made by the proper personnel. Often, the issue is how much sample needs to be collected or under what conditions should the sample be shipped to provide the analyst with what he or she needs to conduct an effective analysis.

Information provided with submitted samples should include a clear statement defining:

- The problem and/or what information is sought
- Names of Project, Area, Region, and/or Technical Service Center personnel familiar with the problem
- Sample location information including amount and location of the deposit
- Knowledge of the type of material previously taken from drains, if known
- Any other relevant data
- The required deadline for results.

Appropriate personnel in the Technical Service Center or your contract laboratory should be contacted with any questions concerning type, quantity, selection, preparation, and shipment of representative samples. Submitted samples should be representative of the material intended for analyses. The analyst should be able to provide a complete cost estimate for the recommended work to be performed.

Upon arrival of samples in the laboratory, the analyst will determine which tests are to be performed based on the purpose of the examination and previous communication with project personnel. Photographs of submitted samples will be provided upon request. Because more than one analysis may be performed on a sample, enough material for each procedure should be submitted.

- *Inorganic material*.—If the material contaminating the drain appears to be sediments (mineral and soil material), then no special precautions are usually needed regarding holding times.

  Every effort should be made to obtain a representative sample, that is, a sample or group of samples selected to typify the larger population.

If laboratory identification of a precipitate is required, a representative sample, at least 1 teaspoon or 50 grams, should be sent to a qualified laboratory for examination. Typically, calcium carbonate deposits can be easily identified by application of a mixture of 3:1 distilled water to hydrochloric acid as described in Reclamation's *Engineering Geology Field Manual* [4, p. 43]. Calcium carbonate is also easily and inexpensively determined by microscopic examination.

Water carrying suspended sediments can be sampled by taking a water sample. The sample should be a sufficient volume to allow at least 1 teaspoon or 50 grams of sediment to settle.

Excessive amounts or unusual materials in a drain may require a sample. Ensure that all particle sizes are represented by taking a sample large enough to ensure an adequate population of all particle sizes. If only a limited amount of material is available, take everything. If abundant material is available, an average sample can be assembled by taking a scoop from 30 different parts of the sample to yield a representative sample free of grouping and segregation error.

Reclamation's *Concrete Manual*, designation 7 [5, p. 511], offers guidance on the amount of material required with respect to particle size. A typical sand sample, with particle sizes ranging from 0.074 to 4.75 mm, should weigh about 500 grams (1 lb) or equal about a pint of material. The weight or volume requirement increases with increasing particle size.

- *Organic material*.—If the material contaminating the drain is suspected to be organic, an analyst should be contacted in advance of field sampling to ensure the sample is properly handled and preserved so that it survives the trip to the laboratory undamaged. Usually, the analyst recommends that the organic material sample is placed in a clean container, transported in an insulated picnic cooler with sealed "blue ice" containers, and shipped to the laboratory as soon as possible to reduce the holding time. Planning should include making sure a qualified analyst is on duty to accept the shipment. About 500 mL (1 pint) of fresh material is required for a positive identification. The TSC's Ecological Research and Investigations Group analyzes and identifies organic material.

Water samples should be collected in clean, 500-mL nalgene bottles using aseptic techniques, placed in an iced cooler, and shipped immediately to the laboratory. Collect the samples early in the work week so the water can be cultured upon arrival.

Biofilms and slimes should also be collected in clean, nalgene bottles or stout plastic bags using aseptic techniques and placed in an iced cooler and shipped immediately to the laboratory. Collect the samples early in the work week so the water can be microscopically examined upon arrival.

Description of aseptic technique for sampling water.—Have latex gloves and isopropyl alcohol on hand. After putting on gloves, wash hands and sample bottle and top with some alcohol. Open the sample bottle as close to the sampling location as possible, taking care not to contaminate the bottle top by facing the cap in the bottom-up position or leaving the cap off for excessive time, to reduce the chance of airborne bacterial contamination. Triple rinse the bottle with sample water then fill the bottle with sample water and cap. Label each bottle with the sample location and place in cooler. If requested in advance, the Ecological Research and Investigations Group will prepare a cooler with sample bottles and send it to the collection site.

d. *Transportation*.—

1. *Transporting inorganic materials to the Laboratory*.—Ship the samples by any reasonable means in a competent container directly to your contract laboratory or the Bureau of Reclamation Earth Sciences and Research Laboratory Group (call for current laboratory location):

Earth Sciences and Research Laboratory Group
Mail code D-8340
Denver Federal Center
6th and Kipling
Denver CO 80225
303-445-2329

2. *Transporting organic materials to the Laboratory*.—As soon as possible, store the labeled samples in a picnic cooler. Use sealed "blue ice" cartridges to chill the cooler and samples. Ship the samples OVERNIGHT EXPRESS directly to the Bureau of Reclamation Ecological Research and Investigations Group (ERIG) (call for current laboratory location) or your contract laboratory. It is necessary to contact the analyst prior to sampling and shipment to insure the sample is properly received in the laboratory.

Ecological Research and Investigations Group
Mail code D-8220
Denver Federal Center
6th and Kipling
Denver CO 80225
303-445-2200

e. *Testing of materials.*—

1. *Inorganic materials.*—Soil and soil-like materials from drains are petrographically examined to determine mineralogical composition, organic fraction, and origin usually for documentation purposes. Soil and soil-like material in a drain is analyzed to identify the mineralogical composition and to detect the presence of minerals and rock types that determine origin, occurrence, and history of the sample. If a sample of the construction or foundation material surrounding the drain or suspected to be the source material is submitted for examination, the samples can be compared for common mineralogical composition. Note that the suspected source material may not be adjacent to the drain, but may be located farther upstream.

The petrographic examination of soils generally includes a description of the submitted sample and a determination of the mineralogical composition and estimated volume percentages.

The soil and soil-like material analysis results can be applied to the material in the field only to the extent that the submitted sample represents that material.

2. *Organic materials.*—Two approaches are generally considered. One is to sample the water that has passed over the biofilm using aseptic techniques to reduce sample contamination. The second technique is to remove some of the slime or biofilm from the original site for microscopic examination.

The ERIG laboratory performs bacterial activity reaction tests and light microscope examinations. The ERIG laboratory performs analytical testing for water, solid samples, and hazardous wastes, research and special studies to solve environmental, operation and maintenance, and engineering problems.

f. *Bibliography.*—

[1] Bureau of Reclamation, *Ground Water Manual*, second edition, U.S. Government Printing Office, Washington DC, 1995.

[2] Driscoll, F.G, *Groundwater and Wells*, second edition, Johnson Division, St. Paul, Minnesota, 1986.

[3] Cullimore, D.R., *Practical Manual of Groundwater Microbiology*, Lewis Publishers, Chelsea, Michigan, 1992.

[4] Bureau of Reclamation, *Engineering Geology Field Manual*, second edition, Denver, Colorado, 1991.

[5] Bureau of Reclamation, *Concrete Manual*, eighth edition, U.S. Government Printing Office, Washington DC, 1981.

g. *Further references.*—

Ford, H.W., *Iron Ochre and Related Sludge Deposits in Subsurface Drain Lines, Florida Cooperative Extension Service*, Circular 671, 1993. http://edis.ifas.ufl.edu/BODY_AE026

Hem, J.D., *Study and Interpretation of the Chemical Characteristics of Natural Water*, third edition, U.S. Geological Survey, Water Supply Paper 2254, 1985.

Lennox, J.E., *A Manual of Biofilm Related Exercises, A Biofilm Primer* http://www.personal.psu.edu/faculty/j/e/jel5/biofilms/primer.html

McCook, D.K., *White Paper on the Impacts of Aging of Seepage Control/Collection System Components on Seepage Performance*, ASDSO/FEMA, 2000.

University of Alaska Fairbanks, Water Treatment Plant, Iron Bacteria http://www.uaf.edu/fs/water/ironbact.html