WATER OPERATION AND MAINTENANCE

BULLETIN NO. 153

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IN THIS ISSUE

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The Water Operation and Maintenance Bulletin is published quarterly for the benefit of those operating water supply systems. Its principal purpose is to serve as a medium of exchanging information for use by Bureau personnel and water user groups for operating and maintaining project facilities.

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Riverside Diversion Dam,
Rio Grande Project,
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UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION

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ELECTRONIC DATALOGGERS FAIL IN STEEL RECORDING WELLS[1]

During the summer of 1989, the staff of the irrigation branch of Alberta Agriculture noticed the repeated failure of their electronic dataloggers installed in corrugated metal pipe (CMP) and steel recording wells. Replacement instruments would record data for about 2 weeks and then fail again.

Brian Cook, an electronics technologist with the branch, believes "the problem occurred when the CMP or any other metal in contact with moist soil becomes a primitive battery or galvanic cell generating between 0.7 to 1.1 volts." This voltage, he says, along with the high humidity found in recording wells and manholes is enough to destroy most micro chip based instruments and computers. Where possible, he suggests electronic equipment should be installed in PVC, fiberglass, concrete, or other nonconductive recording wells and manholes.

If, however, electronic equipment must be installed in a buried metal structure, he recommends using a desiccant product such as Silica-Gel in a tightly closed instrument box which is electrically insulated from all other metal.

For further information, please contact Brian Cook, Electronics Technologist, Irrigation Branch, Alberta Agriculture, Agriculture Centre, Lethbridge, Alberta T1J 4C7; telephone (403) 381-5879.

Alberta Agriculture has discontinued use of corrugated metal pipe and steel recording wells to house their electronic dataloggers.

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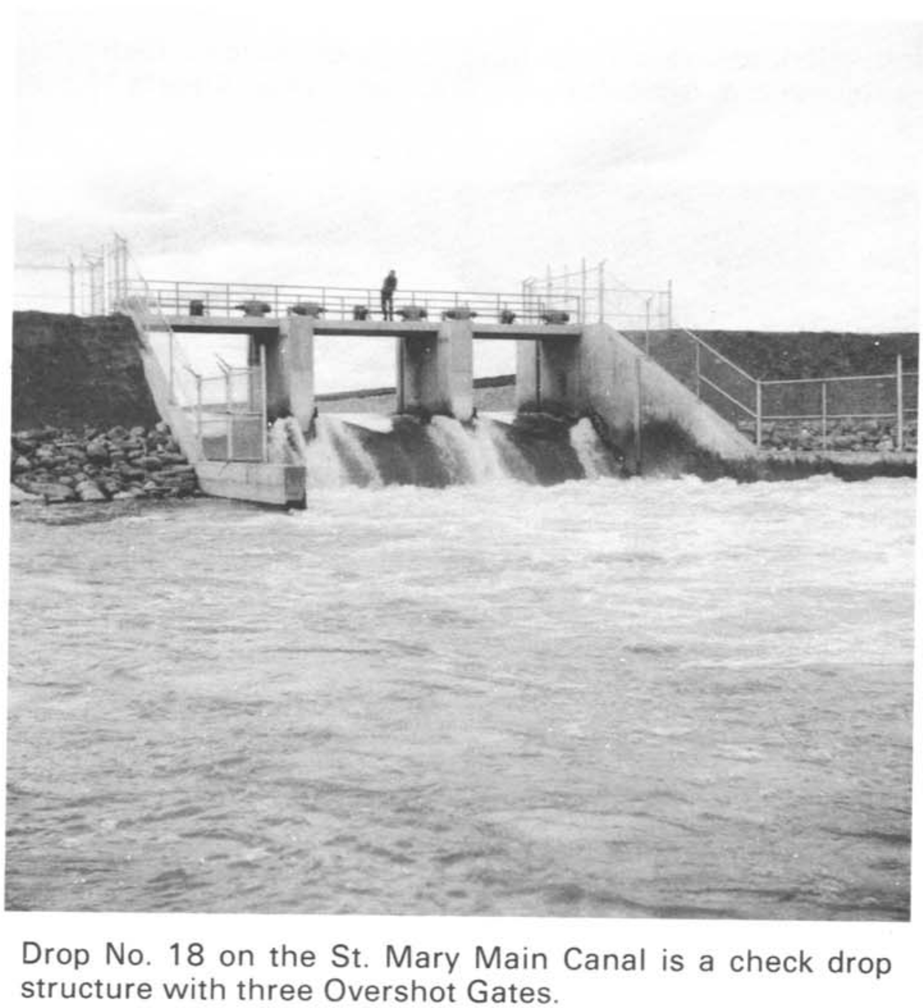
A 1st FOR AN IRRIGATION STRUCTURE[1]

Automated Overshot Gate Wins "Award of Excellence"

UMA Engineering Ltd. (UMA) is changing the thinking of design engineers on how to control water in open channels (traditionally done with undershot gates) with the development of its award winning "Automated Overshot Gate." The automated overshot gate has not only won the "Award of Excellence," jointly sponsored by the Canadian Consulting Engineer Magazine and the Association of Consulting Engineers of Canada, but has been widely accepted by the "people who run the water."

UMA countered the trend of half a century of hydraulic design work in arriving at this innovative design. The overshot gate is simple! It can best be described as: a rectangular panel, hinged across the bottom, that is raised and lowered by two cables attached to the upper corners. Stationary sidewalls guide the flow of water up and over the gate panel. Liken it to a drawbridge placed in a canal.



Drop No. 18 on the St. Mary Main Canal is a check drop structure with three Overshot Gates.

[1] Reprinted with permission from the Editor, Water Hauler's Bulletin, Alberta Agriculture Centre, Alberta, Canada T1J 4C7, Winter 1990 issue.

Hydraulically, the overshot gate is a moveable weir where flow over the gate varies as the three-halves power of the head, says UMA engineer Dale Miller. "This means that fluctuations in flow rate are reflected only as nominal fluctuations in water level. With water level held constant by the overshot gate, any diversions from the channel are assured of steady-state conditions. For a typical sluice gate diversion, the water manager needs to set his gate position only once with the knowledge that the flow through the gate will remain stable," says Miller.

How did the overshot gate come into being? Originally patented in 1890, the gate never did seem to catch on with design engineers. A quick review of modern day design manuals mentions no word about it. It was not until the SMRID (St. Mary River Irrigation District) needed a new inlet structure into Sauder Reservoir and manager Jim Brown's displeasure with existing structures that engineer Jozef Prozniak (then with UMA) began to develop the design. Only 2 years later, when trying to patent their product did UMA learn that 90 years previously someone had designed and patented a structure similar to theirs. No sooner was the gate designed than automation was pursued by UMA's Ian Daniluk to give the water manager around-the-clock control. All was not lost however, as UMA was able to copyright their automated controls and programs.

This smaller automated Overshot Gate is solar-powered, but these gates can also be connected to the local electrical grid system.

Financial assistance for further developing the automated overshot gate was received from the National Research Council of Canada, Farming For the Future and the Irrigation Council of Alberta. With financial assistance and the recent advances in computer and communications technology, UMA has reduced the size of the overshot gate and is now providing economical around-the-clock water level control on small canals as well.

"The overshot gate permits ease of operation by the water manager; a water level change of 10 cm (4 inches) is accomplished with a gate change of 10 cm (4 inches). The increment of control is very small; precise gate adjustments of as little as 5 mm (0.20 inch) are possible," adds Miller.

As simple as it may seem, the new design took many days to bring it from the idea stage to a fully functional irrigation structure.

For example, to prevent vibrations (both mechanical and auditory) and uneven flows over the gate, the nappe or overflow portion of the flow requires venting to atmosphere to prevent negative pressures from occurring underneath the gate panel. Venting is accomplished in two ways: by embedding vent pipes in the sidewalls of the larger structures; or by encasing one of the hoist cables with a vent hose on the smaller modular structures.

UMA enlisted the services of Armtec Inc. (a major gate and metal pipe manufacturer), to prototype the gate designs on a trial basis. Today Armtec has added the overshot gate to its product line and is selling them in both Canada and the United States.

Certainly it can be said the UMA development of the award winning automated overshot gate is one more tool in the design engineer's bag. With the control provided by this new structure, more efficient use can be made of a finite resource — water. When coupled with an automated control system, the overshot gate will contribute to improved conservation and management, at a time when the public is more aware of water issues and water managers are demanding more of their operating personnel and the tools they have at their command.

For more information please contact Dale Miller, P.Eng., UMA Engineering Ltd., Stafford Drive North, Lethbridge, Alberta T1H 2B2; telephone (403) 329-4822.

PLUGGING OF AIR VENTS DOWNSTREAM OF OUTLET WORKS GUARD GATES

by Darrel E. Krause[1]

Generally, air vents are provided between the guard (also sometimes referred to as "emergency") gate and the regulating gate (or valve) of an outlet works configuration. The vent is normally connected to the downstream side of the guard gate body in a manifold to distribute the airflow. Without an adequate air supply, negative pressures can develop while the guard gate is being closed under "unbalanced head" conditions, which can cause cavitation damage or water column separation; or as discussed later, lack of an adequate air supply can result in collapse of the pipe downstream.

Background

During normal operations, the guard gate is kept completely open while water is being released through the outlet works. The regulating gate (or valve) at the end of the pipe is used to control or stop the flow of water. With the regulating gate/valve closed, the guard gate can then be closed under "balanced head" conditions. Under these normal operating conditions, the air vent is needed only to release air from the pipe between the gates (or gate and valve) during the filling process or to supply air to the pipe during draining to perform maintenance or inspections within.

However, if the regulating gate/valve cannot be closed for some reason, the guard gate may need to be operated (under "unbalanced head" conditions) to stop the flow during an emergency situation. If an adequate air supply is not provided to the pipe downstream of the gate, negative pressures may cause the pipe to collapse.

The size of the air vent can vary and is determined from the diameter and length of the downstream pipeline, design flow, and design head. The vent is usually small for relatively close-coupled gates/valves and basically is used to vent air while filling or draining the pipe between them. Larger vents are normally required when the guard gate is located some distance upstream of the regulating gate/valve and there is the danger of high negative pressures developing and collapsing the pipe during an emergency closure of the guard gate. Where collapse of the pipe is not a concern, larger vents may be provided to reduce vibrations and noise which will occur during an emergency closure. Air vents are not normally sized to prevent minor and infrequent cavitation damage which will occur during the short period required for an emergency closure.

Most air vents are provided with a combination (air vacuum/air release) air valve assembly (see figure 1) to allow automatic operation, particularly during an emergency closure. Such a valve assembly allows the release of air from the pipe during the filling process and allows the admission of air during the draining or emergency closure process. In addition to the air valve assembly, most air vents consist of steel pipe embedded in concrete and terminating in a type of air manifold on the downstream side of the guard gate. The air manifold contains a number of holes or air passages across the width of the gate body to allow for equal air distribution (see figure 2). A properly operating

[1] Darrel E. Krause is a General Engineer employed by the Bureau of Reclamation, Facilities Engineering Branch, Denver, Colorado.

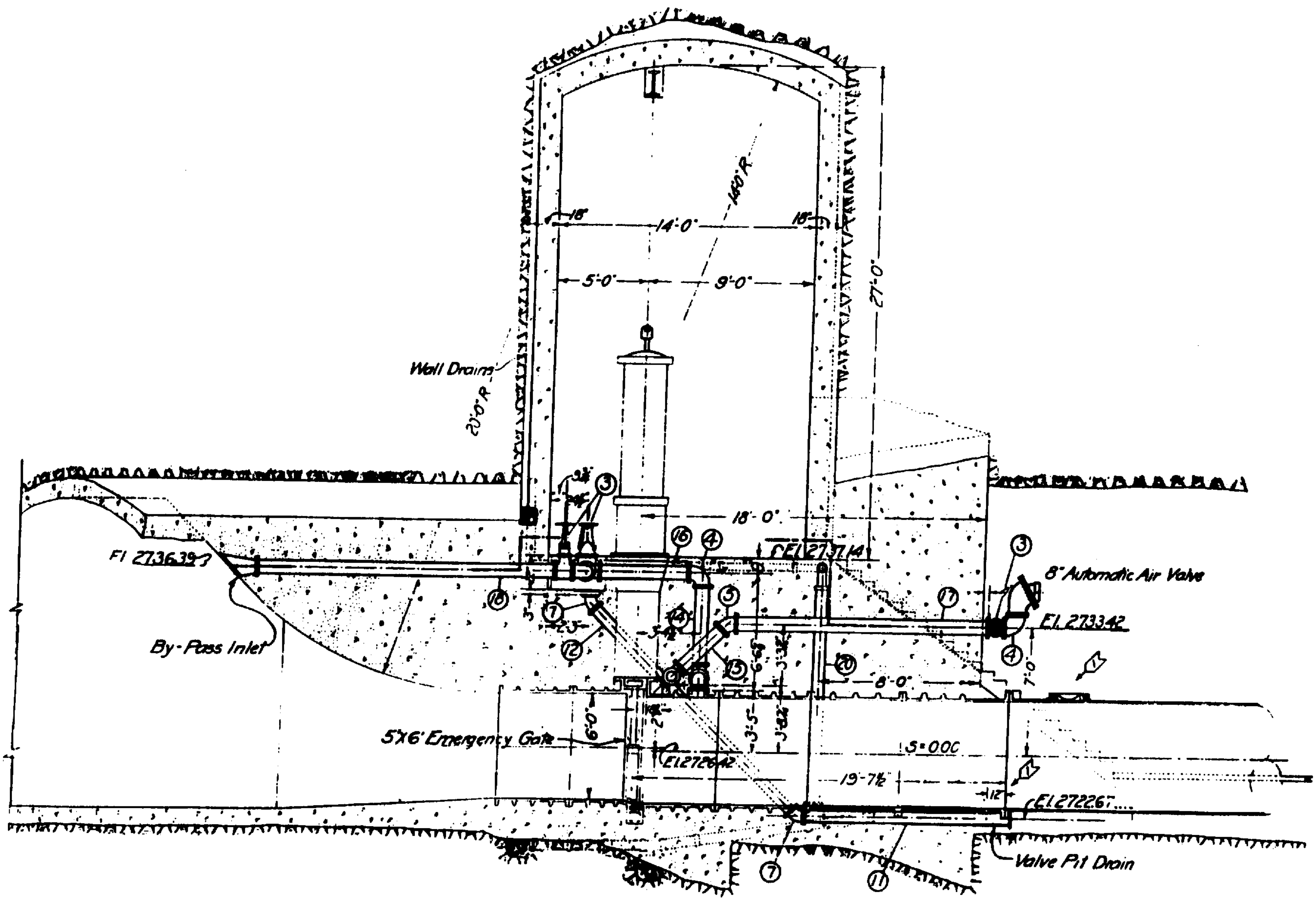


Figure 1. - Example of automatic air valve assembly provided on air vent downstream of emergency (guard) gate.

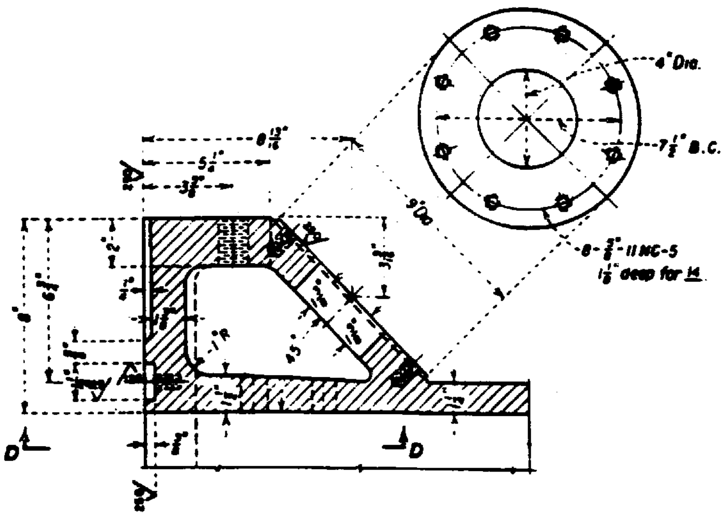
air valve assembly, as well as a clear and unobstructed air manifold and pipe, is essential to permit safe operation of the outlet works, particularly during an emergency closure.

Plugging Incident

During the past several years, Reclamation research personnel have been conducting field tests to determine an appropriate standardized procedure to be used for ensuring reliable emergency closure capability of guard gates. One such field test was conducted at Silver Jack Dam in southwestern Colorado. During the course of the test, it was observed that the air manifold downstream of the guard gate was almost plugged with silt and debris, with the airflow area through the air passages/holes reduced by approximately 75 percent. This plugging caused a dramatic effect on air demand during the emergency closure process. A similar problem has been observed and corrected at other similar locations where the small vent holes in the manifold were obstructed with rust and silt.

Precautions To Be Taken

Operating personnel at all Reclamation facilities having similar types of outlet works configurations and air vents should be made aware of this plugging possibility and its consequences. Silt, debris, and rust can accumulate through normal operations, as was observed at Silver Jack Dam. Thorough and frequent (at least annual) inspections of the air manifold and passages/holes, as well as the air valve assembly, should be included as part of the operating personnel's routine duties and responsibilities at the dam to ensure safe and proper functioning of the air vent and outlet works. This is particularly important in the event of an emergency closure when the pipe could collapse due to an inadequate air supply.



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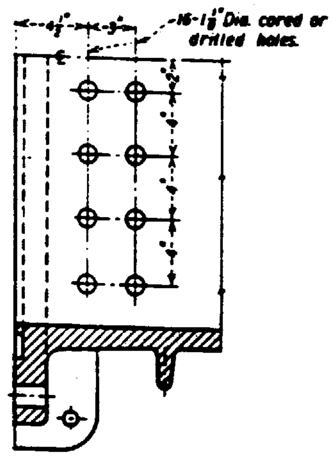

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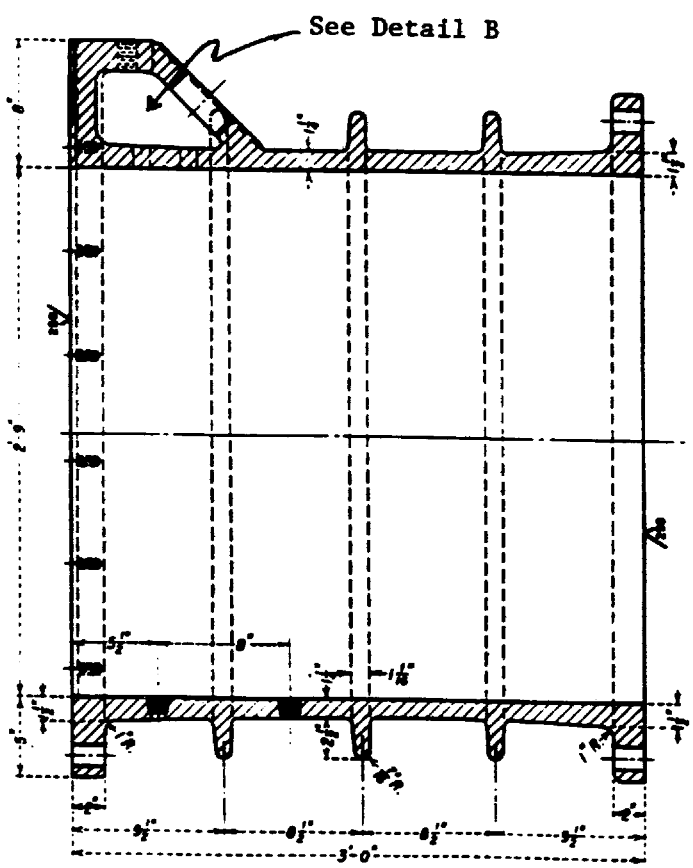

Downstream frame of gate

Figure 2. - Details of a typical air vent manifold.

NEW METHOD OF STREAM BANK PROTECTION[1]

By Clifford Baber, P.E. [2]

With five creeks flowing through its residential and commercial developments, the property owners in Saint Charles, Missouri, are all too familiar with creek bank erosion. Over the years, a variety of methods have been employed by the city and private owners in an attempt to curb this erosion.

Cole Creek, having the largest drainage area of the five creeks, experiences the greatest fluctuation in flow. The serious degree of bank erosion at one residence along Cole Creek resulted in the city investigating and evaluating various erosion control methods.

Investigation began with a grid confinement system, but nearly vertical creek banks caused the manufacturer not to recommend its use. Interlocking sheetpiling was the next method to be researched. With an estimated cost of between $125,000 to $130,000, this method was not recommended. A concrete retaining wall that later could be incorporated into a vertical wall channel was a third alternative investigated. This 18-foot-high by 62-foot-long wall was estimated to cost $76,000 in 1987.

In the fall of 1987, it was decided to receive bids for a gabion basket wall. The low bid of $75,500 for the 80-foot-long by 18-foot-high wall caused the city to continue to search for a less costly alternative.

A new method of creek bank stabilization was introduced to the city in late 1987. The new method, the Waterloffel, is a variation of the Loffelstein wall system that originated in Europe. This retaining wall system consists of trough shaped concrete modules or units with interlocking wings. Each module is 18 inches wide, 26 inches long, 7 inches tall, and weighs approximately 176 pounds. The Waterloffel modules are produced by Seagren Industries, Saint Louis, Missouri. It was estimated this retaining wall could be constructed for approximately $54,000.

Construction of the Waterloffel retaining wall began with a 3-foot-thick footing located 1 foot below the creek bed. Poor soil conditions and sudden thunderstorms made construction of the footing difficult. During installation of the footing, the first layer of modules was embedded in the fresh concrete to ensure no slippage between the modules and the footing would occur. Construction of the wall progressed by setting the next layer of modules on the newly completed layer. Each layer of modules was set back approximately 8 inches to create a 40° slope from the vertical.

After completion of the first six or seven layers, a scour protection system was installed. This system, consisting of concrete-filled grid confinement system, was constructed at the toe of the retaining wall. The scour protection system was designed to help prevent scour along the footing.

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[1] Reprinted with permission from the Editor, Public Works, December 1989 issue.
[2] Mr. Baber is Assistant City Engineer, Saint Charles, Missouri.

Creek bank stabilization system is built by placing layers of trough shaped modules on the steep slope. Interlocking wings tie units together, as shown in detail at left. Later a scour protection system was placed at the toe.

To ensure positive drainage through the wall, a 1-foot-thick layer of clean rock was placed behind the wall and the troughs of the modules were filled with clean rock. A filter fabric was placed between the rock layer and the excavated creek bank to reduce contamination of the clean rock.

The configuration of the modules will allow vegetation to grow along the wall eventually, blending the wall into its natural surroundings.

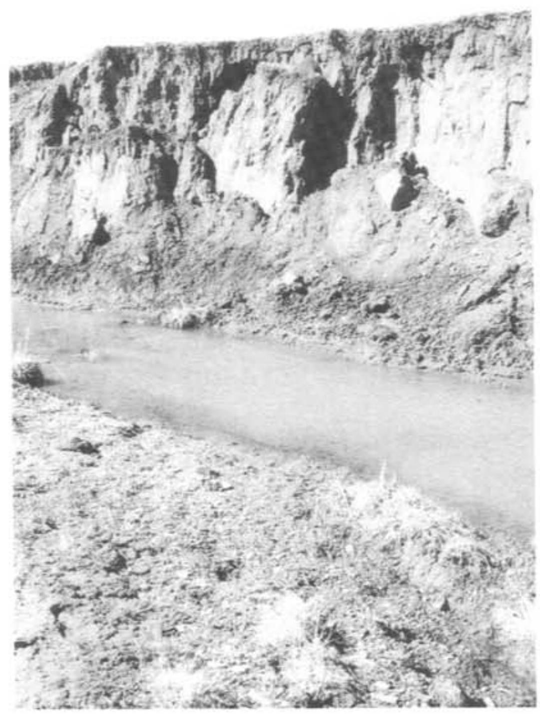
The completed Waterloffel wall is 85 feet long and 20 feet high, containing 1,210 modules. An average of 130 modules were placed per day with construction of the wall taking approximately 1 month. The bids for the wall ranged from $56,492 to $88,365 with the low bid representing a 34 percent saving over the gabion basket wall, the next least expensive option considered.

NEW REVETMENT DESIGN CONTROLS STREAMBANK EROSION[1]

by Russell A. LaFayette and David W. Pawelek[2]

A watershed condition analysis of the Bluewater Creek watershed near Grants, New Mexico, showed that although most of the uplands were in at least satisfactory condition, stream channel meander cutting continued to provide excessive sediments to the fluvial system. An innovative revetment system eventually solved the problem.

The project site is located along the main channel of Bluewater Creek, one of two major streams contributing most of the flow into Bluewater Lake, a 2,350-acre impoundment in northwestern New Mexico. Located in the Zuni Mountains, the watershed contains 52,000 acres, 86 percent of which is managed by the Mt. Taylor Ranger District, Cibola National Forest, Southwestern Region, USDA Forest Service. Average annual precipitation varies from 14 inches at Bluewater Lake to 24 inches at the highest elevation. Precipitation falls predominantly in the summer in short-duration high-intensity thunderstorms.



Construction view of revetment shows main baffle segment parallel to flow, one perpendicular baffle, and posts in place for a second. At right, bank slumping along the main stem of Bluewater Creek prior to the project.

The watershed strongly reflects its land use over the last 200 years. Hispanic and anglo settlement, accompanied by extensive grazing and lumbering, reduced ground cover, decreased water infiltration, increased runoff and surface erosion, and initiated channel degradation.[1][3] Timber harvest took place over most of the watershed between 1890 and 1940. Grazing of cattle and sheep continued, and fire scars on remaining trees and stumps testify to extensive wildfires. Since the USDA Forest Service acquired the

1 Reprinted with permission from the Editor, Public Works, December 1989 issue.

2 Russell A. LaFayette is a Hydrologist, USDA Forest Service, Southwestern Region; and David W. Pawelek is a Hydrologist, Cibola National Forest, Albuquerque, New Mexico.

3 Numbers in brackets pertain to References at end of article.

land in the early 1940's, better grazing and timber management have markedly improved the land.[2]

A hydrologic function analysis was begun in 1984.[3,4] Results were summarized in 1987 by Hanes and LaFayette.[5] Generally speaking, most subwatersheds were in at least satisfactory condition. Improved land use had allowed most land surfaces to regain sufficient cover to arrest surface erosion and moderate most normal rainfall and snowmelt runoff.

Problems Remain

Problems persisted, however. Recovery of the drainage network has been slow. Headward gully erosion continues into broad meadows, lowering their water table, producing sediment, and decreasing productivity. Channel meandering causes very active streambank erosion, resulting in sedimentation of the stream and lake. Poorly located or unneeded roads channel water into drainages, as do livestock trails. Flood peaks remain unacceptably high while base flows remain small, limiting the extent of perennial streams in the watershed. Riparian vegetation remains far below potential, resulting in unstable streambanks, higher water temperatures, and low fish productivity.

Watershed study findings prompted a long-term program to improve hydrologic function and resultant benefits. Treatment goals include: moderate flood peaks, prolong base flows, and store water within the soil mantle; reduce sedimentation of the stream and lake; increase wildlife and fish productivity; boost timber and forage productivity; demonstrate watershed analysis and treatment methods; and preserve archeological resources.

Treatment methods include: gully headout control, road closures and improved channel crossings, livestock management, increasing riparian vegetation, better timber management, controlling channel base levels, increasing fish and wildlife management, and controlling excessive streambank erosion. The remainder of this article addresses a method to achieve this last objective.

Streambank Erosion Control

Channel degradation and stream meandering had left several actively eroding streambanks along the main Bluewater Creek channel. Lateral water movement in the channel would undercut banks ranging from 10 to 20 feet high, causing a large prism of soil to fall directly into the active stream. Riparian vegetation was unable to become established on these banks and sediment production was substantial, particularly during high runoff years. Fourteen such banks and many smaller ones were measured in a 4-mile length of channel between private lands to the center of the watershed and a critical road crossing above the lake. Ways to control this excessive streambank erosion were analyzed and a preferred method chosen.

The method selected must stop streambank failures, thus limiting channel and lake sedimentation. It must work with the stream system rather than against it to preserve and promote maximum stream length and maintain channel gradient. The system should promote onsite sediment storage and development of riparian vegetation and overall ground cover.

Excellent cover was established less than a year after revetment installation.

Various streambank erosion control measures were evaluated for their advantages and disadvantages:

Livestock control in riparian areas.–This would provide a cure only in the very long term. Water flowing along the base of high cut walls could continue to undermine vertical banks.

Riparian planting.–Planting alone without livestock control would be inexpensive but ineffective. Progress would require many years and may not control water at the base of existing cut banks.

Bank shaping.–Although shaping would reduce the prism of soil available to enter the stream as well as provide a site for plant growth, problems included where to dispose the cut slope soil and erosion of the exposed surface before plant growth. It would not prevent water from running along raw banks or arrest meandering.

Gabions.–Although effective in preventing bank erosion, problems include high cost, complex construction, and failure to promote sediment deposition onsite and riparian plant establishment.

Kellner jacks or tetrahedrons.–These methods come close to meeting project goals since they follow stream contours, promote sediment deposition and riparian plant establishment. Negatives include relatively high cost, complex construction, and little esthetic appeal. Also, most jack systems are designed for larger stream systems.

Porous fence revetment.–This alternative was chosen for several reasons. Fence materials allow water to pass through the system, thus reducing water velocity and promoting sediment deposition. Sediments provide a growth medium for riparian vegetation. Plant growth, particularly woody species, strengthen the total system while making it less visible to the public. The system largely preserves the meander length and gradient. Temporary livestock exclosure is needed to speed recovery, but controlled grazing can be resumed after several years

Two streams meanders were chosen to apply the porous fence revetment system. Designated in an earlier examination of meanders as sites G and H, the two locations had a combined streambank length of 1,400 feet. Eroding meander scarps measured up to 18 feet vertically. Based on three USGS streamflow gauges around the Zuni Mountains, design flows for the 100- and 50-year recurrence interval floods were 1,420 ft^3/s and 890 ft^3/s, respectively, from the 39,000-acre watershed above the sites (figure 1).

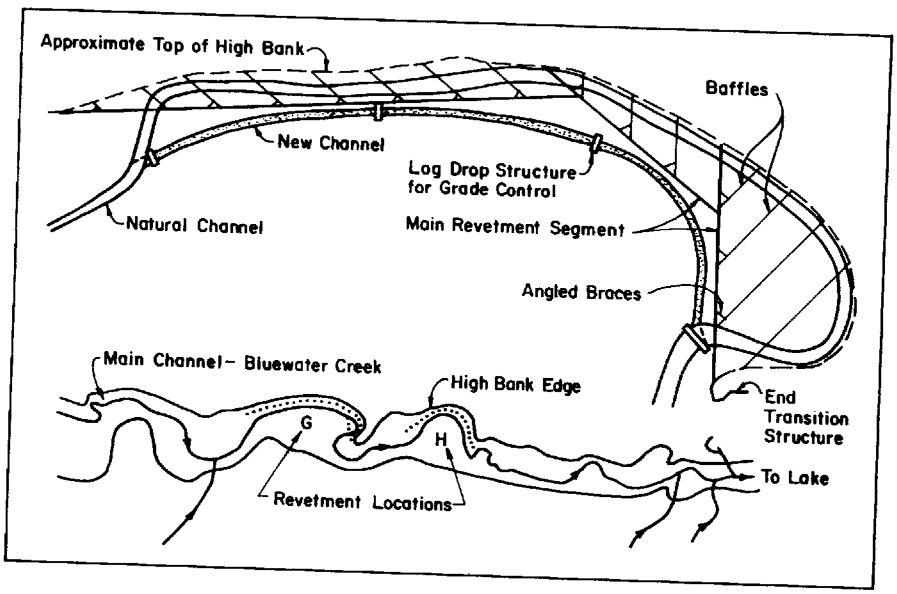


Figure 1. – Revetment locations on the creek and details of revetment G design.

Soils are a deep alluvium, with the streambed entrenched into this material. Plants dominating high benches are rabbitbrush and snakeweed. Kentucky blue grass is found on lower terraces next to the stream course. Many old terraces and meander scars are evident, depicting a history of channel degradation and meandering. Before the arrival of European settlers, we theorize that Bluewater Creek flowed in a wide and shallow alluvial valley growing significant riparian vegetation and providing home to numerous beaver and other wildlife.

Project Design and Materials

Materials are the key to this design. Other designs we had studied used posts of treated telephone pole, various diameter piping, old railroad rails, and similar materials.[6] All were difficult to handle and required drilling or other means to attach fencing materials. Fence materials typically were galvanized chain link fence or welded wire.

The design at Bluewater Creek uses pre-drilled galvanized steel U-channel sign posts commonly used to erect highway signs (figure 2). Posts can be bought in various lengths in 2-foot increments, in weights from 2-1/2 to 4 lb/ft. Two or more posts can be bolted together for added strength if desired. These posts are easy to handle, can be driven by hand or machine, and pre-drilling provides handy places to attach bolts, cable, and wire.

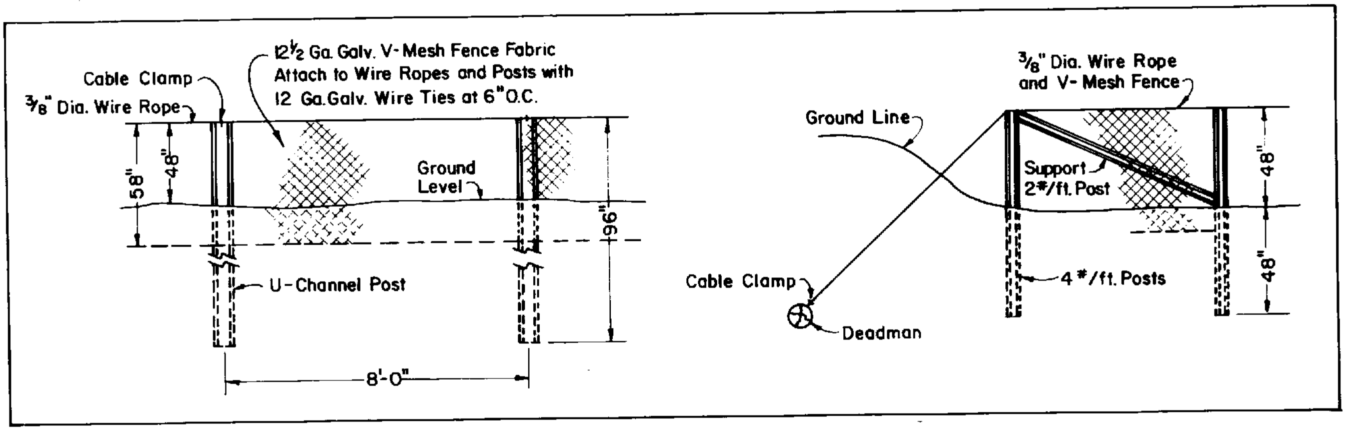


Figure 2. – Details of main revetment and baffle sections and the deadman and support assembly.

We chose fence material of 12-1/2-gauge galvanized V-mesh woven wire. Horizontal strands are twisted double wire. Woven V-mesh verticals are wrapped around horizontals rather than welded, a stronger and more flexible system since spotwelds can break. Wire rope used for top support and deadmen are galvanized, as are all fasteners.

Each revetment section consists of several components. A new permanent channel is dug to keep water away from the eroding bank, provide work space for construction, and a site for sediment deposition and plant growth. The new channel must be maximized to maintain overall stream length and gradient.

The revetment section consists of two elements: one or more main segments aligned parallel to flow, and a series of baffles oriented perpendicular to flow, extending from the main segments back into the streambank. Main segments and baffles are faced with the fence material, attached so that water and debris will force the fence material against the posts. Galvanized wire rope extends along the top of the main segments, held at tension and secured in the ground via deadmen made of post segments. Wire rope is similarly attached along the top of the baffle posts and secured in the banks. Fence material is secured to the posts and wire rope by 12-gauge galvanized tie wire and galvanized U-bolts.

In this design, 8-foot posts were driven 4 feet into the ground, leaving 4 feet exposed. Posts for the main segments and baffles were on 8-foot centers. Fence material 58 inches high was used, with 10 inches being buried and the remaining 48 inches above ground. Burying was done to provide below-ground protection should the channel meander or widen against the revetment.

At several locations, 24-inch-diameter Ponderosa pine logs were buried across the new channel with the top edge of the log level with the stream bottom. This feature was designed to provide temporary grade control while the revetments became firmly established.

To enhance the revetments' effectiveness, several features were added to the project. All disturbed soil was seeded to promote ground cover and reduce erosion. Local willow cuttings were planted along banks of the new channel to provide bank strength and eventual shade. Cottonwood poles were planted to provide a future over-story component and seed source. Livestock grazing was eliminated for at least 5 years to give all vegetation a chance to become established, after which well-managed grazing will be allowed.

Project Results

The revetments were installed by a contractor at sites G and H in late fall 1986. An above-average snowfall provided significant snowmelt runoff in spring 1987 before any vegetation could become established. The structures functioned flawlessly, however, capturing significant sediment deposits and small woody debris. The new channel widened considerably, but not enough to endanger the structures.

Vegetation growth in and around the structures did well during 1987, providing additional flow roughness to enhance sediment capture and provide ground and bank cover. Willow cuttings were planted on several occasions by various volunteer groups and cottonwood poles were also put in place. The second winter saw little snow and little spring flow but significant summer thunderstorm runoff. Plant growth in and around the revetments was excellent. Willow plantings were partially successful. Most cottonwood poles lost their top vegetation but sprouted from the root collar in summer 1988.

Due to improper installation and channel widening beyond expectations, several log grade-control structures were unsuccessful. Those logs properly installed continue to work well and remain submerged except during the lowest flows.

Project costs for materials, labor, and equipment totaled $27,700. With a bank length of 1,400 feet, cost averaged $19.79 per foot of protected bank. As plant growth continues, maintenance costs should prove minimal, since each passing season provides additional root strength and bank cover.

The project designers feel confident the project will continue to succeed. It has survived both high spring runoff and summer runoff events. Bank erosion is reduced, sediment deposition is occurring as planned, and plant growth is excellent. High-intensity storms in summer 1989 put the revetments to the test, and they did their job as designed.

Based upon the success of these first two structures, the forest service has installed six additional revetment segments. Several design advances aided project success. The basic design is simple, easy to install, and requires minimal equipment. Materials are readily available in various sizes and strengths to meet anticipated stream forces. It provides an integrated solution, combining revetment fencing, planting, seeding, and livestock management.

The designers feel several adjustments in design will make future projects more successful. Fences will be buried deeper, with only 1-1/2 to 2 feet of fence exposed. This will be less obtrusive above ground yet provide necessary control above and below the surface. The constructed channel will be further from the main revetment segments, providing additional insurance against undercutting. Grade control logs may be eliminated since change in grade along the stream profile is minimal. Cottonwood cuttings will be grown in a nursery for 1 year before planting since rooted stock survive better than bare poles sunk to ground-water level. Willows will be cut from local stock and planted as early as possible. These adjustments should strengthen the success of the initial design.

It is encouraging to note that local private landowners, previously skeptical of the forest service work along Bluewater Creek, are making inquiries about the project and considering similar work on their own adjacent lands.

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Acknowledgment.-The preceding article is based on a paper presented by the authors at the International Erosion Control Association Conference XX, February 15 to 18, 1989, Vancouver, British Columbia, Canada.

The following article offers a solution to erosion problems on access road fill sections and is not intended to be used on storage dams or canal fill sections where tree roots would create seepage paths.

FILL SLOPE REPAIR USING SOIL BIOENGINEERING SYSTEMS[1]

by Robbin B. Sotir and Donald H. Gray, Ph.D.[2]

SOIL BIOENGINEERING is an applied science combining mechanical, biological, and ecological concepts to construct living structures for erosion, sediment, and flood control. Plant parts are used as the major structural components to reinforce the soil mantle. The live plants or cuttings may be used in conjunction with inert structural members such as wood, stone, or synthetic materials.

This technology offers natural and effective solutions to instability problems along streams and rivers, highway cut and fill slopes, in wetland recovery, and recreational site rehabilitation. In repairing disturbed or damaged lands, soil bioengineering allows the land to recover at a faster rate and become stable, self-supporting, and productive.

Soil bioengineering systems function immediately as soil reinforcing units and as barriers to surface erosion. In time, roots and shoots develop to further enhance stability. The shoots and associated foliage form a protective vegetative cover that mitigates the effects of rainfall erosion and improves mass stability by removing excess soil moisture through transpiration. The roots permeate and reinforce the soil, thereby increasing its strength and resistance to sliding. This living system grows stronger and more effective with age. The stabilization of the surface soils encourage the natural invasion of a diverse and stable plant community. Detailed information about soil bioengineering systems and construction procedures can be found in publications by Schiechtl [1][3] and Gray and Leiser [2], respectively.

The North Carolina Department of Transportation chose a failing slope site about 60 miles east of Ashville as the test or demonstration site for soil bioengineering systems. North Carolina, similar to other states, has been plagued with numerous shallow landslides, slumps, sloughing, and erosion on both cut and fill slopes along roadways. In March 1984, the Division of Highways Geotechnical Unit contracted with Robbin B. Sotir & Associates to develop a preliminary reconnaissance report for soil bioengineering solutions at 10 sites in North Carolina typical of the shallow slide and erosion problems.

In December 1985, after reviewing the consultant's preliminary report, it was decided to establish a demonstration project to repair a fill slide on NC 126, using soil bioengineering technology. The demonstration site was an 870-foot-long fill slope that generally runs east-west and has a southern exposure. The embankment has a maximum

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[1] Reprinted with permission from the Editor, Public Works, December 1989 issue.
[2] Ms. Sotir is President of Robbin B. Sotir & Associates (formerly Soil Bioengineering Corporation), Marietta, Georgia. Dr. Gray is a Professor, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.
[3] Numbers in brackets pertain to References at end of article.

height of 60 feet with a slope face ranging from 100 to 200 feet long. This fill section was constructed in 1980-81 with the slope varying from 1.5:1 to 2:1. The slope began to fail and erode shortly after construction and was repaired several times using conventional treatments. When the decision was made to repair the slope with soil bioengineering systems, face sliding and erosion had started to undermine the guard rail posts at several locations and encroach on the pavement structure.

Installation of fill brushlayer during North Carolina embankment stabilization project, top; and a view of the slope before repairs began.

Stability Analyses

Factor of safety.–The relative security or factor of safety of an earthen slope is normally expressed as the ratio of the shear strength to the shear stress along a critical surface. A slope fails when the shear stress on this critical surface equals the shear strength (i.e., when the safety factor approaches unity). Different types of mass stability analyses or models have been developed to predict the factor of safety.[3] The so-called infinite slope model is appropriate for analyzing transitional slides in which the failure surface is planar and parallel to the slope over most of its length. An infinite slope analysis is suitable for evaluating the stability of a compacted fill when failure occurs primarily by shallow sloughing along a surface roughly parallel to the face of the slope. The failure surface tends to be located in the looser, less compacted soil near the outer edge of the fill, as depicted in Figure 1. Water infiltrating into the slope from the top flows along this looser, more permeable layer parallel to the slope. Less permeable zones along the way or at the bottom of the slope tend to divert this seepage laterally outward and cause it to emerge at the face as shown schematically in Figure 1. This condition not only decreases mass stability, it can lead to piping and seepage erosion as well.

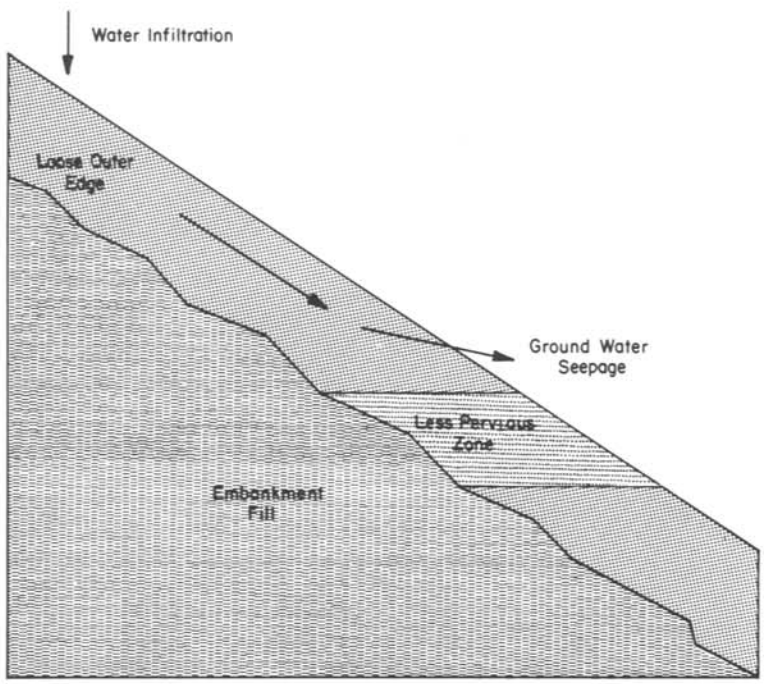

Figure 1. – Schematic illustrates variables controlling shallow slope failures in compacted fill slope.

Effect of plant roots on soil strength.–The main effect of the presence of roots in a soil, insofar as strength is concerned, is to provide a measure of apparent cohesion.[4,5] This root cohesion can make a significant difference in the resistance to shallow sliding or shear displacement in more sandy soils with little or no intrinsic cohesion. Actual shear tests in the laboratory and field on root permeated sands indicate a shear strength increase per unit root concentration ranging from 7.4 to 8.7 $lb/in^2/lb$ or $root/ft^3$ soil.[4,6] Root concentration relationships reported in the technical literature were used to estimate likely root cohesion as a function of depth.[7] A low-to-medium root concentration with depth was used in the stability analyses to determine the likely influence of slope vegetation on mass stability of a compacted fill embankment.

Infinite slope analyses.–Infinite slope analyses were conducted on a 1.5:1 and 2:1 embankment slope. Factors of safety were computed as a function of the vertical depth and seepage direction with respect to a horizontal plane. Root cohesion was computed as a function of the root concentration in the soil, which varied with depth as explained previously. The intrinsic soil cohesion was set low at 0.2 lb/in^2 assuming that the fill was composed primarily of granular soil or borrow. The friction angle was set to either 35° (an expected maximum for sandy fill material) or 30° (an expected minimum for loose sandy fill).

The factor of safety dropped below unity ($F<1$) when the seepage either paralleled or emerged from the slope face at depths of 2 to 3 feet for a friction angle of 30°. The results of the stability analyses show that both the seepage direction and presence of root cohesion have a significant effect on the factor of safety. Even a small amount of root cohesion can increase the factor of safety substantially. This influence is very pronounced at shallow depths where root concentrations are highest and reinforcement effects therefore greatest.

The effect of seepage direction on stability also deserves some comment. Vertical seepage greatly increases the factor of safety. This condition, in fact, yields the same factor of safety as a dry slope. Accordingly, to the extent that slope vegetation and soil bioengineering systems promote downward seepage and infiltration, they also enhance mass stability.

Slope Repair/Rehabilitation

Plan development.–Robbin B. Sotir & Associates was contracted to develop preliminary construction plans, procedures, and specifications for the project. The "Construction Document," Phase II, submitted June 1986, included the final construction plans. From these documents, the project was constructed. The documents included: scope of work, definition of terms, description of the various soil bioengineering systems, plant harvesting, installation procedures, and detailed cross sections and plans of the soil bioengineering systems. The site was divided into three major areas with particular soil bioengineering systems to be installed in each area. The following is a brief description of the areas and the soil bioengineering systems installed:

- **Area 1**.–The slope at the western end is 2:1 with a maximum height of approximately 40 feet, and was moderately stable except for a seepage zone and a circular slump.

  Soil bioengineering systems installed included live staking, live cribwall, cut brushlayers, and live fascines (figure 2).

- **Area 2**.–The slope was 1.5:1 with some vertical sections and had a maximum height of about 60 feet. It was heavily eroded with raveled and sliding sections.

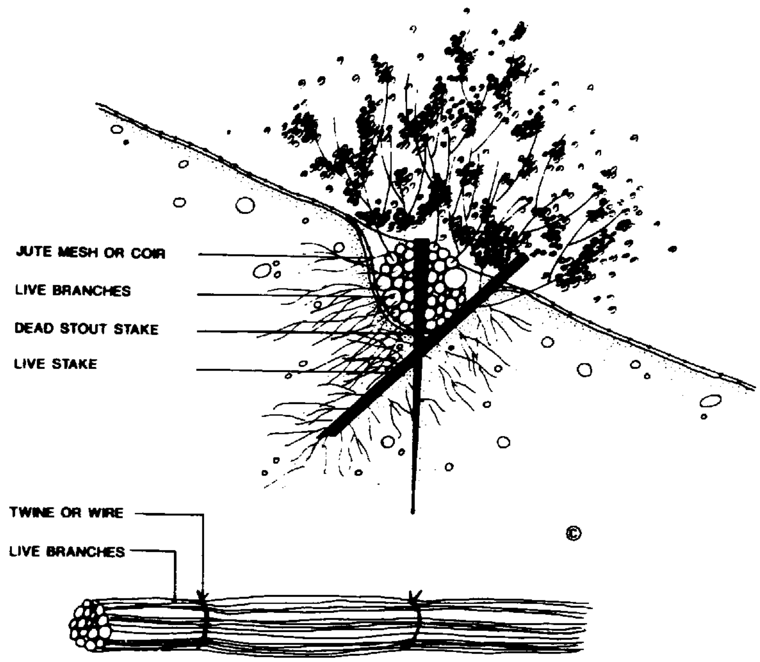

Figure 2. – Live fascine with jute or coir. Rooted/ leafed condition of the living plant material is not representative at time of installation.

Soil bioengineering systems installed were fill brushlayers (figure 3), reinforced brushlayers, live staking, and rooted plants.

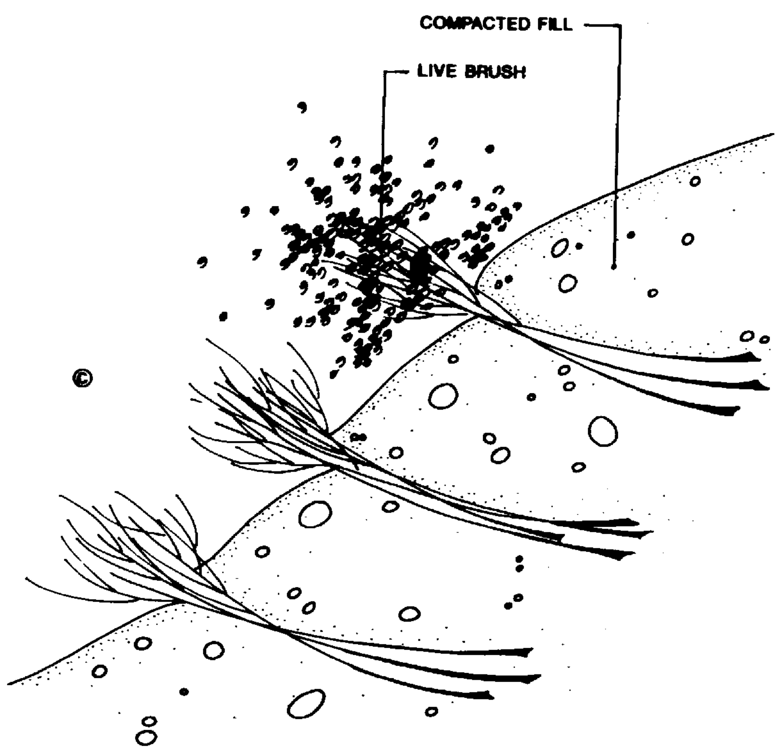

Figure 3. – Schematic representation of a brush-layer fill installation.

Area 3.–The slope at the eastern extremity was 1.5:1 to 2:1 with a maximum height of 25 feet. This area had cracks in the upper slope paralleling the roadway, about 3 to 5 feet outside the guardrail, and 1 foot deep. The toe of the slope was stabilized and buttressed by existing trees.

Soil bioengineering systems installed were cut brushlayers, live fascines, and live staking.

Construction.–Actual soil bioengineering production work began October 27, 1986. Harvesting of the cut plant material began for placement of the first fill brushlayers in Area 2. This operation requires the use of chain saws as well as machines during both harvesting and installation. Daily workers, including on-site supervisory personnel, averaged 24, but varied between 12 and 35 for most of the construction period.

Harvesting-related activities required about half of the man-hours consumed. These activities included securing permission to harvest, sometimes improving access to the harvesting site, and the actual harvesting of the plant stems and their transportation to the construction site.

A small bulldozer was used to prepare the fill brushlayer terrace lifts in Area 2. Terraces for cut brushlayers and live fascine trenches were dug by hand. Typically, plant material delivered to the site consisted of bundles containing long live stems of brush. The average truck load contained 40 bundles. Based on these estimates, the quantities of plants and cuttings installed in each area are given in Table 1. In addition to live stems, quantities of lime, fertilizer, grass seed, and jute mesh were used at the project site.

Table 1. - Live Material Requirements

Location	Quantity or length total	Unit	Number of bundles
Area 1			
Cut brushlayers	600 lineal feet	0.7 bundle/foot	420
Live fascines	570 lineal feet	0.2 bundle/foot	115
Live stakes	1,500 stakes (est)		
Live cribwall	40 feet long	8 layers-stepped	80
Area 2			
Fill brushlayers	5,483 lineal feet	1.36 bundles/foot	7,450
Rooted plants	3,000		
Live stakes	2,500 stakes (est)		
Area 3			
Cut brushlayers	1,315 lineal feet	0.7 bundle/foot	920
Live fascines	1,514 lineal feet	0.2 bundle/foot	300
Live stakes	3,000 stakes (est)		

Selected fill brushlayers in Area 2 in the toe area were reinforced by placing a 6-1/2-foot-wide layer of geogrid manufactured by Tensar Corporation, Morrow, Georgia, on the terrace beneath the brush. The large openings in the grids allow root development through the grid apertures and into the soil. These grids provided additional reinforcement to the soil in the slope. Brushlayers behave like and are installed in a similar manner (vis-a-vis orientation, spacing, and width of layer) as geotextile and/or geogrid layers. Relationships developed to determine spacing/width requirements for a desired level of slope security for geotextiles/geogrids can also be adapted for brushlayers as explained in the next section.

Spacing and width requirements for brushlayers.–The vertical spacing between reinforcements and the width of the layer (or length of the reinforcements) are critical design parameters in constructing reinforced embankments. The reinforcements must be spaced sufficiently close to each other so that they do not break in tension. They must also be long enough to resist pull-out. The same spacing/length considerations apply to brushlayers. Consequently, procedures developed for calculating the required spacing/width of grids and fabrics can be adopted for brushlayers as well.

Results of the calculations showed that under conservative assumptions, required vertical spacings between brushlayers would range from 3.2 to 1.6 feet. Assumption of higher friction angle in the hill and higher concentrations of brush stems in each layer yielded more liberal spacing and width requirements.

System Evaluation

All the soil bioengineering systems employed in the project can be used to control erosion and to stabilize heavily eroding slopes. An analysis to determine the proper system to use in a given situation is critical. Equally critical is the proper installation of the systems. Even the simplest method, live staking, can be installed incorrectly.

Live staking system.–This is the simplest and least expensive system to install and should be used on a slope before erosion problems start. It should be part of a routine slope maintenance program.

Brushlayering system.–This system (figure 3) includes brushlayers cut into a slope and those used in a new or repair fill that abuts the slope face. The branches are placed as the fill is raised in the latter case. This system works very well as porous filter units to control surface erosion from heavy rains during and after construction. This filtering action was observed during construction when 4.5 inches of rain fell in a 3-day period. The brushlayers prevented erosion of the outer edge of the loosely compacted fill material placed in the fill brushlayer system in Area 2. Those areas of the slope where such systems had not been installed experienced large soil flows and earth failures.

In addition to mechanical reinforcement from stems and adventitious roots, the brushlayers also favorably modify the hydrologic regime near the surface of the slope. The brushlayers act as lateral drains. They intercept ground-water seepage along the loose outer edge of a compacted fill, divert the flow downward, and then convey it out laterally through the brushlayer itself. Redirection of seepage flow downward greatly improves mass stability as noted previously in the stability analysis section.

Installing the brushlayer system requires careful planning and supervision. The application is complicated but offers immediate soil reinforcement, drainage, and surface protection benefits. It causes the slope to become its own self-supporting structure.

Live fascine system.–This system serves as a pole drain immediately after installation to control and direct surface runoff. It is useful for preventing erosion at specific locations. The root system that eventually develops from the live fascine permeates the soil and helps to stabilize the slope. These adventitious roots reinforce the loose soil layer at the outer edge of a compacted fill. Live fascines also work well immediately to stop head-cutting up the face of a slope. Live fascines have specific site planning, preparation, and installation requirements.

Live cribwall system.–A live cribwall is a very site-specific system requiring detailed planning and design. Installation is somewhat more complicated than the other systems. Under certain conditions, this system could replace a conventional cribwall. It is useful in areas where space is limited and where immediate structural stability is needed. The stems and living roots that eventually permeate the interior fill of the wall, bind and tie the cribfill together into a unitary, monolithic mass that increases resistance to internal stresses acting on the structure. Live materials placed in the crib also root behind the unit and improve resistance to external forces; e.g., overturning. Also, because of this conjunctive vegetative treatment, the crib structure eventually blends into and becomes part of the natural landscape.

Conclusions and Recommendations

Soil bioengineering is an excellent way to repair many shallow mass wasting and slope erosion problems. It can be especially useful in areas where access with heavy equipment is difficult. Large projects such as the NC 126 repair require careful site analysis, detailed plans and specifications, cross sections, quantities, and contract provisions.

Soil bioengineering systems installed on this demonstration project permanently stabilized the fill slope. Soil bioengineering methods in general and brushlayering in particular provide soil reinforcement from plant stems, induration and reinforcement from adventitious roots, and favorable modifications of the soil moisture regime near the face of the slope.

Soil bioengineering systems blend naturally into the landscape and do not intrude visually, which makes them highly environmentally compatible. They offer important upland buffers in the overall land unit or watershed. The effectiveness, completeness, and suitability of a soil bioengineering project actually improves with time. Once the vegetation is established, it becomes self-repairing, supporting, and maintaining through constant regeneration. This attribute leads to low maintenance requirements. A properly designed, well-planned, and constructed soil bioengineering project has shown in many instances to be more cost-effective and to yield more than conventional approaches.

There are several factors to consider regarding a soil bioengineering land stabilization project:

- A soil bioengineering project is labor intensive. There is handwork, requiring use of shovels, mauls, chain saws, and picks. This may be an asset on sites where heavy equipment access is poor or nonexistent. Large fill projects require conventional mechanized equipment.

- The live plant material should only be installed during the dormant season, usually September through April. This is the season of snow and rain, which can cause a loss of project time. However, it is also a time when labor is most plentiful and, therefore, the least expensive.

- A readily available source of proper plant material is especially important. The plant material should be easily accessible and available in large quantities. As projects are completed, the project locations themselves can become harvesting sites for new projects.

- Personnel installing a soil bioengineering system must be instructed and supervised very carefully during installation. They should be properly informed and made to feel confident about the system's success.

- It is imperative to have a project designed, planned, and supervised by a trained and experienced soil bioengineering practitioner.

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THE SLUICEWAY FAILURE OF RIVERSIDE DIVERSION DAM

by Bill Bouley and Arthur Glickman[1]

Introduction

Riverside Diversion Dam, constructed in 1927, is the southernmost of the four diversion dams on the Rio Grande Project (figure 1). It provides the headworks for Riverside and Tornillo Canals, which serve 40,000 acres in the El Paso and Hudspeth Counties of Texas. The El Paso County Water Improvement District No. 1 operates and maintains the diversion dam. The diversion dam is located 15 miles southeast of El Paso, Texas, on the Rio Grande which forms the international boundary with Mexico. The diversion dam consists of a headworks, with five radial gates, having a design capacity of 900 ft^3/s; a sluiceway, with six radial gates; and a concrete overflow weir (photograph 1). Because of the international boundary, certain aspects of the operation and maintenance are subject to the approval of the International Boundary and Water Commission (IBWC).

Photograph 1. – Riverside Diversion Dam. Sluiceway stilling basin. 3/17/87

Flows along the lower Rio Grande are mainly controlled by Elephant Butte Dam and Caballo Dam. Caballo Dam is 115 miles upstream from Riverside Diversion Dam, and Elephant Butte Dam is 25 miles upstream from Caballo Dam. Stormflows and city and farm drainage flows are the other contributing flows. Between Caballo Dam and Riverside Diversion Dam are five other Bureau of Reclamation diversion dams which are the main turnout flows from this reach of the Rio Grande.

On June 9, 1987, while high flows were occurring in the Rio Grande, the sluiceway failed. An investigation was initiated to determine the cause or causes of failure, the

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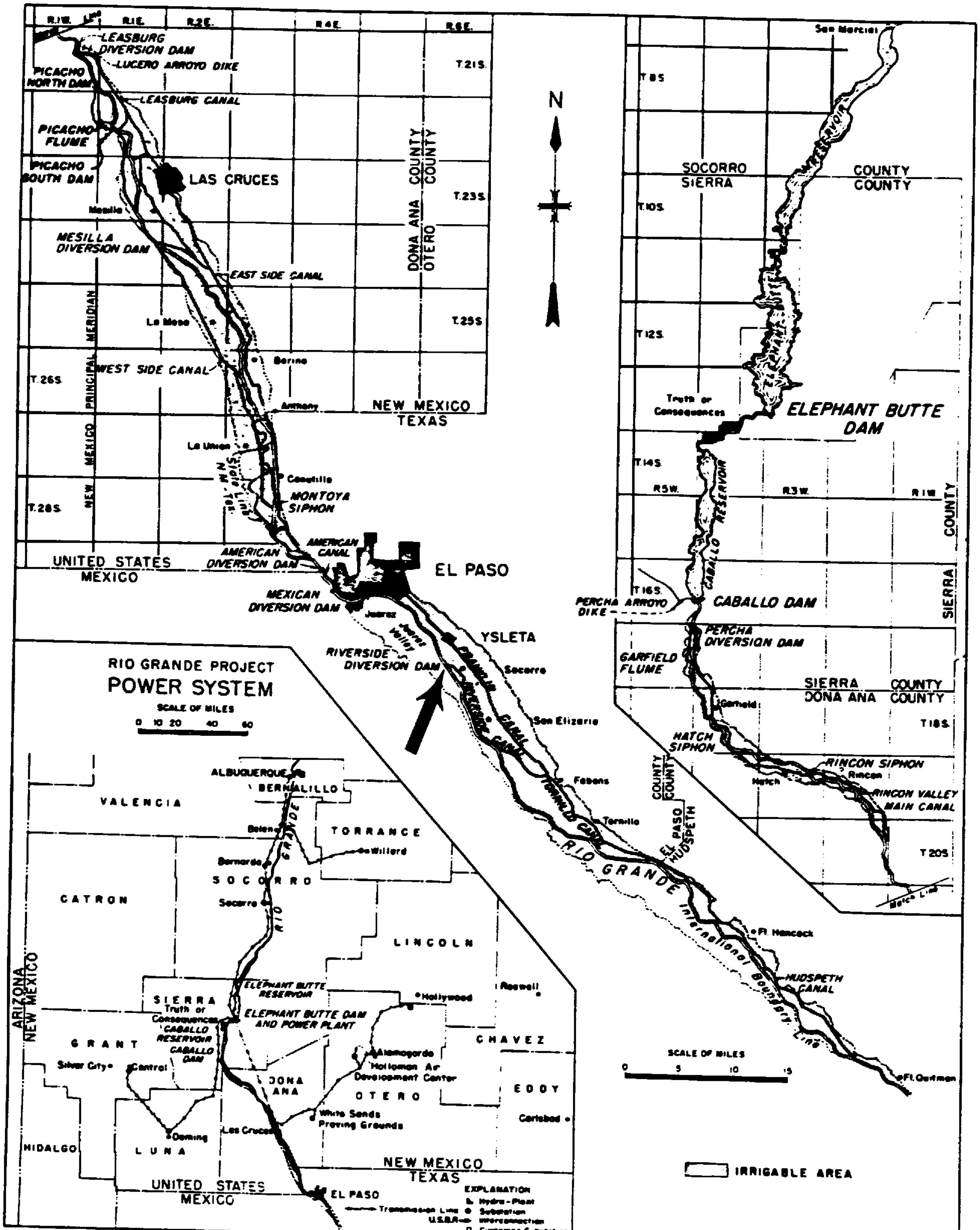


Rio Grande Project

Figure 1

method of failure, recommend any actions to be undertaken to prevent failures at other diversion dams in the project area similarly constructed to the Riverside Diversion Dam, and any possible changes to be made to Reclamation Design Standard No. 3, "Canals and Related Structures."

Historical Operations

Riverside Diversion Dam had been operational for approximately 60 years prior to the sluiceway failure. Most of the years had low riverflows which created a situation whereby sluicing of river sediments was not performed in order to take advantage of the little water supply available. Usually the riverbed was dry below the diversion dam, except for minor gate leakage. There were few noticeable problems identified with the dam.

The headworks and sluiceway were constructed in 1927. In 1936, the concrete weir was added to increase the overflow capacity of the structure to 11,000 ft^3/s. The construction proved itself when flooding occurred in the lower Rio Grande throughout 1942. The flow through the sluiceway radial gates in May 1942 was as high as 3,600 ft^3/s.

From the period of flooding in the 1940's to 1986, there were little or no excess flow releases from Caballo Dam causing the riverbed downstream from Riverside Diversion Dam to aggrade.

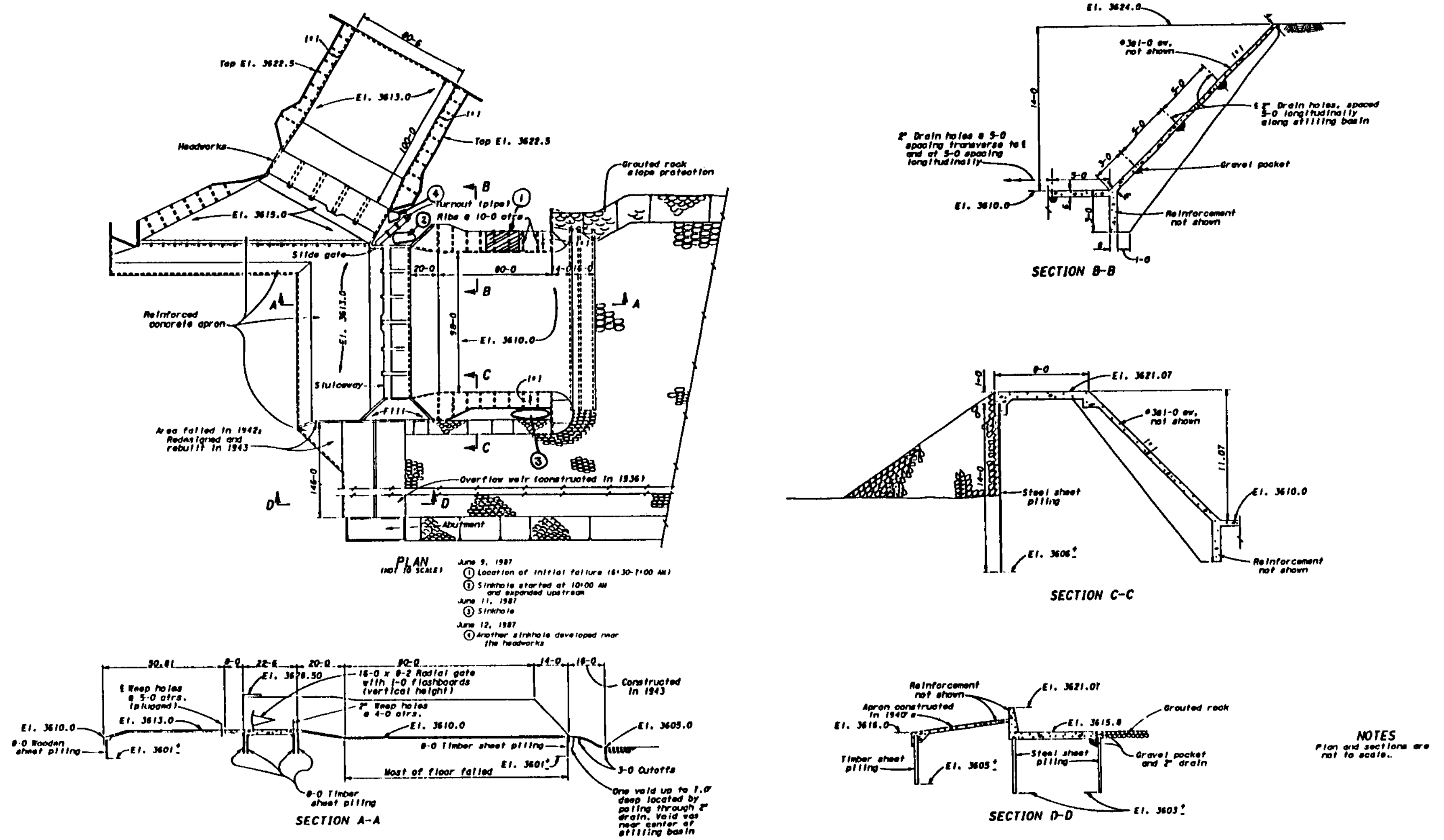
Conditions Immediately Prior to Failure

The sluiceway of the diversion dam failed on the morning of June 9, 1987. The total riverflow upstream of the diversion dam was approximately 3,900 to 4,600 ft^3/s. The flow through the headworks was approximately 430 ft^3/s and the flow through the sluiceway and over the concrete weir is estimated to be 3,500 to 4,200 ft^3/s. The water surface in the canal just downstream of the headworks was approximately at elevation 3621.3 feet.

Three gates of the sluiceway and three gates of the headworks were open. In the sluiceway, each end gate and one of the middle gates were open; and in the headworks, each end gate and one of the middle gates were open. Water flowed over the three closed sluiceway gates and under the three open sluiceway gates. Boards were previously added to the top of the sluiceway gates to add an extra 1 foot of vertical height. The elevation of the water surface upstream of the sluiceway was at or above elevation 3622.17 feet. For a flow through the sluiceway of 3,500 ft^3/s, the water velocity under the gates would be approximately 21 ft/s and the water velocity in the stilling basin would be as high as 25 ft/s, if a hydraulic jump did not occur.

Failure of the Sluiceway

The sequence of events (shown on figure 2) occurred as follows:



RIVERSIDE DIVERSION DAM

Figure 2

June 9, 1987:

1. A section of left downstream side slope failed at 6:30 to 7 a.m.

2. Most of side slope failed by 9 a.m. (photograph 2).

3. Closed sluice gate on left side by 10 a.m.

4. Upstream sinkhole, near headworks, started by 10 a.m. to noon.

5. Crack in left upstream sluiceway abutment noticed at noon (photograph 3).

6. Filled upstream sinkhole with concrete.

7. Dumped rock in failed embankment area.

8. Excess flow from Caballo Dam stopped.

9. All sluiceway gates raised to lower upstream water surface.

June 10, 1987:

1. At 2 p.m., an attempt was made to restart the diversions. The headworks gates were opened and the sluiceway gates were closed except for the two gates on the right side.

2. At 4:30 p.m., an additional movement of about one-eighth of an inch at the crack on left side of dam had been measured.

June 11, 1987:

1. Had additional structural movement.

2. Sinkhole (5 by 30 feet) behind downstream lining on right side of sluiceway.

3. Two center gates opened and two right-side gates of the sluice way were closed.

4. Excess flows from upstream dams stopped.

June 12, 1987:

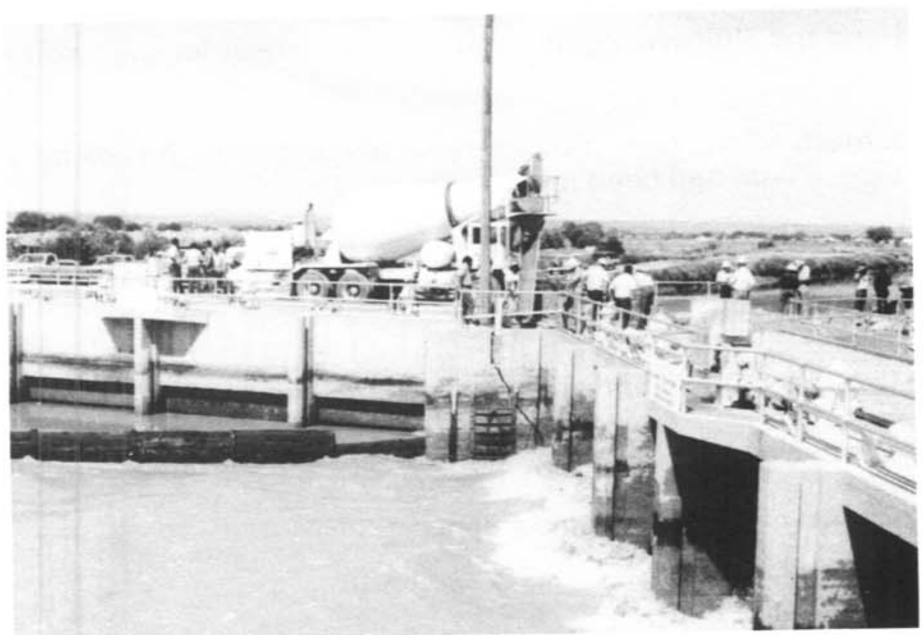
1. Another sinkhole developed near the right abutment of headworks.

2. Right downstream abutment of sluiceway failed.

3. Sluiceway gates were raised and diversions were stopped.

4. Excess riverflows were reduced.

Photograph 2. - Riverside Diversion Dam. Left abutment of sluiceway stilling basin. 6/9/87



Photograph 3. - Riverside Diversion Dam. Notice crack in left upstream abutment wall of sluiceway. 6/9/87

In addition to the above, it was noticed the floor of the sluiceway was destroyed and much of the area where the floor used to be was undercut. Photograph 4 shows the concrete drop, constructed in 1943 at the downstream end of the stilling basin, which remained in place.

When flows were reduced, it was noticed that some of the grouted rock protection downstream of the concrete weir had settled and failed.

Photograph 4. – Riverside Diversion Dam. Concrete drop at downstream end of sluiceway stilling basin. 2/11/88

<u>Temporary Modifications To Enable Diversion</u>

1. Filled sinkhole near headworks with concrete. Concrete had polymer fibers (photograph 5).

2. Rebuilt right downstream abutment of sluiceway with rock, which was then covered with concrete.

3. Constructed a rock cofferdam across downstream concrete apron of sluiceway. Covered upstream face and crest with concrete. Top of crest is at elevation of top of concrete weir.

4. Placed rock in sediment cover on upstream concrete apron of concrete weir.

5. Placed rock on downstream concrete apron of concrete weir. This rock was subsequently washed away.

6. Added rock protection to right bank of concrete weir.

Photograph 5. - Riverside Diversion Dam. Filling sinkhole at left side of sluiceway. 6/9/87

Possible Causes of Failure

One of the main contributing factors to the failure of the sluiceway was the degradation of the river channel downstream of the diversion dam. Normally in dry years, there are no flows past the Riverside Diversion Dam. In the years 1986 and 1987, the storage capacity of the Elephant Butte and Caballo Reservoirs had filled and excess flows were released downstream. The excess flows released in 1986 and 1987 were as high as 2,600 and 2,500 ft^3/s, respectively. The estimated flow past the diversion dam at the time of failure was 3,500 to 4,200 ft^3/s. The flow past the diversion dam includes 2,000 ft^3/s excess flow release plus stormflow.

The Rio Grande channel in the area of the Riverside Diversion Dam is composed of a silty sand material. During periods of low riverflows, the downstream riverbed aggrades, and during periods of high riverflows, the downstream riverbed degrades. The sluiceway stilling basin is filled with sediment during the low flow years. Operators of the diversion dam observed that the sediment deposits in the stilling basin had eroded away by March 1987.

As much as 17 feet of degradation occurred downstream of the diversion dam during the high flow years. This degradation resulted in a loss of tailwater in the stilling basin. The low tailwater was insufficient to force a hydraulic jump in the stilling basin.

The condition of high riverflows and resulting degradation downstream of the diversion dam also occurred in 1942. At that time, high flows caused the river channel downstream from the diversion dam to scour a considerable amount, damaged the upstream right abutment wall and floor to failure, severely eroded the left bank downstream from the sluiceway stilling basin, almost failed the right downstream abutment of the sluiceway, and uplifted sections of the sluiceway stilling basin as much as 0.4 foot.

Repairs and modifications were made to the diversion dam after the flooding in 1942. These corrective actions consisted of: a new guide wall being constructed upstream of the sluiceway on the right side, a concrete apron (5-foot drop in elevation) being constructed downstream from the sluiceway with riprap being replaced, repairing the left embankment downstream of the sluiceway and adding grouted riprap in the side slope protection, adding a concrete apron and steel sheet piling cutoff downstream of the concrete weir, plugging 2-inch-diameter drain holes upstream of the sluiceway gates, and cleaning out the 2-inch-diameter drain holes in the sluiceway stilling basin. The plugging of the 2-inch-diameter drain holes in the sluiceway stilling basin was caused by deposits of sediments and rusting of the iron pipe inserts.

The following are the most probable failure modes:

1. The left downstream embankment of the sluiceway became saturated and failed. This includes failure of the adjacent floor and then undermining of the remaining structure. Hydrostatic uplift underneath the apron may have contributed to this mode of failure.

2. The concrete sluiceway floor failed first and that resulted in failure of the left embankment. The sluiceway floor failure could have been caused by one of the following events:

   a. Scour from the downstream channel working backward, underneath the 5-foot drop, and creating a void underneath the floor. There is no indication that this happened.

   b. Excessive uplift underneath the floor caused by seepage underneath the structure.

   c. The concrete floor may have broken up during the high-velocity flows.

3. Failure of a section of the floor and side slope in the stilling basin by either of the above means led to rapid undermining of the remainder of the stilling basin and the gate structure.

Factors Which May Have Contributed to the Failure

1. Facilities were operated to provide a higher than design water surface upstream of the headworks. The high upstream water surface resulted in higher ground water and uplift pressure against the downstream stilling basin.

2. Seepage from the canal may have also contributed to the high ground water behind the left downstream sluiceway abutment.

3. Plugging of the drain holes by sediment or rusting in the sluiceway abutment and floor may have prevented relief of uplift pressure downstream of the radial gates.

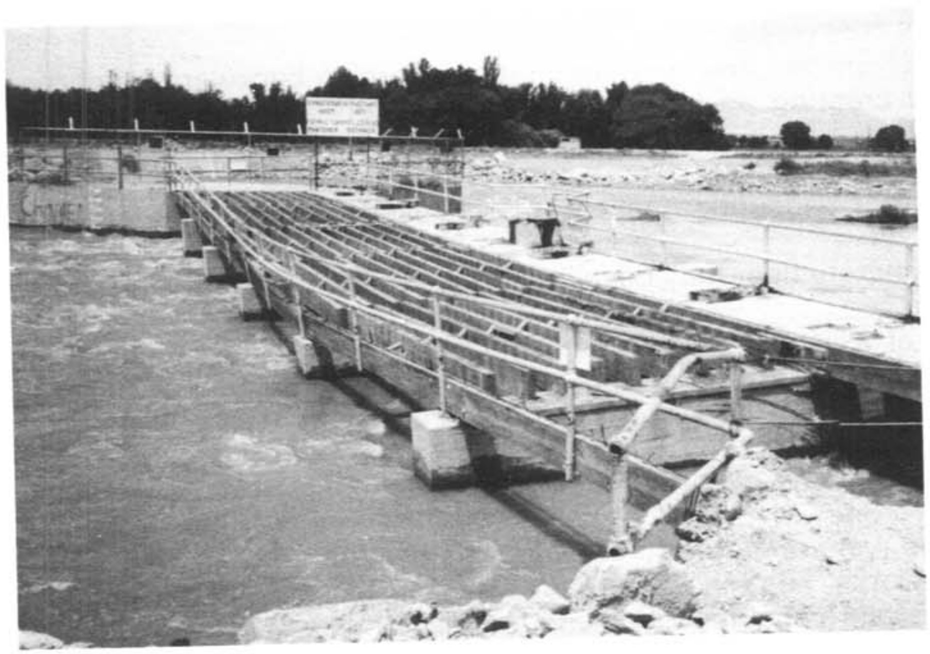
4. Rusting of reinforcing bars may have contributed to failure of a section of the stilling basin. No evidence of rusting of the apron reinforcement was found.

5. Repairs of the damage, which occurred in 1942, may not have been complete. Voids may have been created, underneath the structure or adjacent abutments, which were not detected and subsequently filled.

6. The high-velocity flow and turbulence in the sluiceway stilling basin could have set up a vibration in the concrete slab which either eventually damaged the slab or pumped out foundation material through the 2-inch-diameter drain holes.

Epilogue

Since the failure of the Riverside Diversion Dam sluiceway, all hoist motors and equipment were removed to the El Paso County Water Improvement District No. 1 service yard. The wooden operating deck was dismantled to restrict access onto the twisted deck (photograph 6). Sluiceway radial gates were left in place to provide additional upstream protection to the temporary rockfill diversion. Plans to replace the failed structure are being determined. Costs for reconstructing the sluiceway were estimated at $4.2 million in 1987 by the Water Conveyance Branch. IBWC will design and construct the replacement structure over the international boundary if that option is chosen. Another option being considered is extending the American Canal to join the existing Riverside Canal.



Photograph 6. – Riverside Diversion Dam. Dismantled sluiceway structure from left abutment after the failure. 7/15/87

Because upstream project diversion dams were similarly constructed on the same riverbed material, radar surveys and geotechnical investigations were performed by Precision Engineering, Inc., under contract with the Elephant Butte Irrigation District on the concrete aprons of Percha, Leasburg, and Mesilla Diversion Dams. The ground penetrating radar survey was used to locate any anomalies which were then investigated further by coring and drilling. These diversion dams were found to be in good condition. Considering the failure at Riverside Diversion Dam, the Elephant Butte Irrigation District intends to follow this procedure at 5-year intervals to update the database on the anomalous areas. Although no voids were encountered, the potential for piping beneath the dams does exist. The engineering consultants recommended piezometers be installed to better monitor ground-water pressures around the structures.

REHABILITATION OF DOWNSTREAM APRON OF SAN ACACIA DIVERSION DAM

by Viola Sanchez[1]

During the 1980's, high flows in the Rio Grande created large seasonal fluctuations in the riverbed downstream of San Acacia Diversion Dam. The dam is located about 60 miles south of Albuquerque, New Mexico.

The riverbed is considerably scoured immediately downstream of the dam during the spring runoff. The scour condition, which is similar to erosion, is worsened by summer thunderstorm activity, which creates high peak runoff flows in the river. The scour condition persists until diversion for irrigation ceases in November.

The average riverbed elevation, the summer of 1988, was 6 to 10 feet below the dam's concrete apron, with localized scour holes of 18 feet immediately downstream of the dam. After an RO&M (review of operation and maintenance) examination, the threat of foundation erosion resulted in a Category 1 recommendation for correction. This meant repairs were critical for stability of the structure and must be completed prior to the 1989 runoff season.

Repair work consists of placing riprap, bedding material, and geotextile for a distance of 65 feet downstream of the dam along the entire 700-foot width to an elevation of 7.5 feet below the apron elevation. The riprap is held at the downstream end by a line of sheet piles driven up to 28 feet below the top of the riprap and capped with concrete. The riprap is part of a filter system which consists of a 5-foot-thick layer of 3/4-yd^3-size riprap placed over a l-foot thickness of 6-inch-size riprap which was placed over a 1-foot layer of gravel. Geotextile was placed between the gravel and the underlying fine sand material. To accomplish this work, most of the 1,000 to 1,800 ft^3/s riverflows were diverted through the adjacent Low Flow Conveyance Channel for approximately 2 miles. The conveyance channel and river levee were breached at this point to return flows back to the Rio Grande. A minimum flow of 150 ft^3/s was required to be passed through the dam to support an endangered species of fish. Storage was not required at upstream reservoirs.

Dewatering of the work area was accomplished with 40-foot-deep 800-gal/min wells spaced as close as 30 feet. The dewatering operation lowered the water table in the riverbed 21 feet, allowing the 17-foot excavation to be done under dry conditions. Hydraulic jetting methods were used successfully in constructing the dewatering wells in the riverbed material. The ease in constructing the wells by this method allowed successful installation of wells outside of two-thirds of the excavation. Having the dewatering wells installed outside the excavation and still maintaining ground water at the design level significantly accelerated construction.

The completion date for this project was March 31, 1989, for a total construction time of 3 months. Previous attempts to accomplish this work during the 1980's by two separate

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contractors failed mainly due to dewatering problems. This time, the work was accomplished by the Bureau of Reclamation's operation and maintenance crews from the Socorro Field Division of the Albuquerque Projects Office.

Figure 1. – Downstream view of San Acacia Diversion Dam from canal headworks. 6/30/66

Figure 2. - Downstream view of San Acacia Diversion Dam showing depth of scour prior to rehabilitation. 11/88

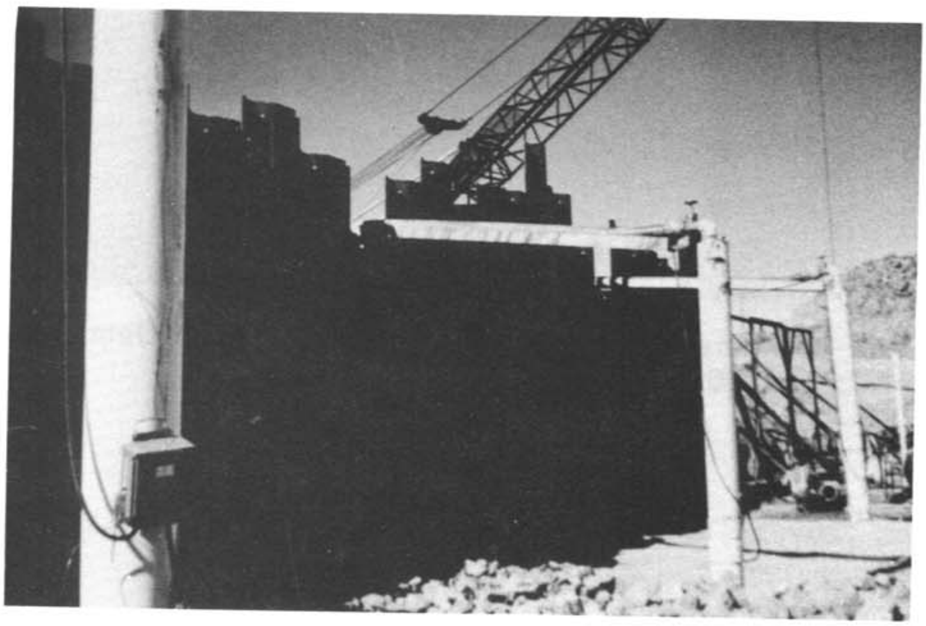

Figure 3. - Work area showing the dewatering wells just upstream of the steel sheet piling. 7/27/89

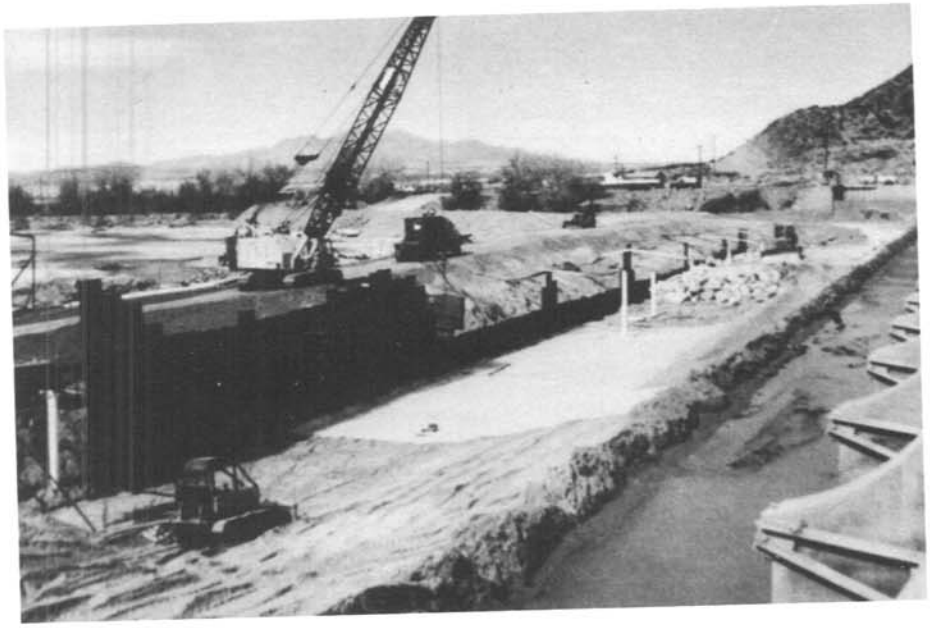

Figure 4. – Work area as viewed from the diversion dam. Some dewatering wells have been relocated to allow for geotextile placement and bedding material (center of photograph). 7/27/89

Figure 5. – Work area showing sheet piling dewatering wells, geotextile, bedding, and riprap placement. 7/27/89

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The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.

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