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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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.

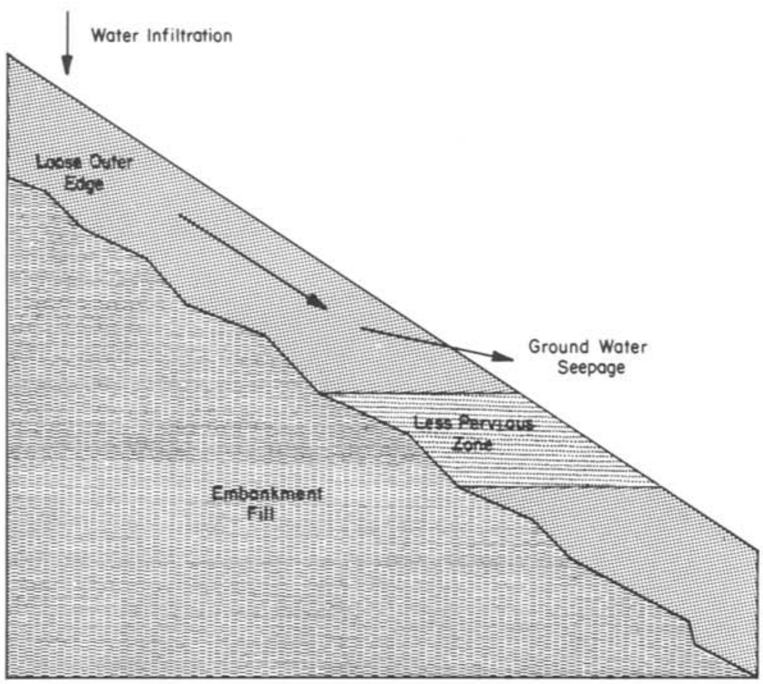

Figure 1. – Schematic illustrates variables controlling shallow slope failures in compacted fill slope.

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.

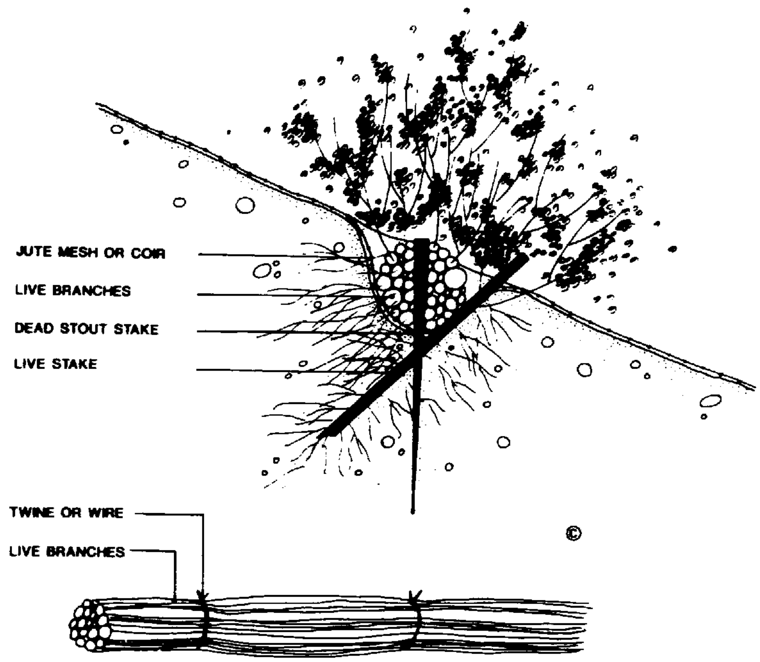

Figure 2. – Live fascine with jute or coir. Rooted/leafed condition of the living plant material is not representative at time of installation.

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

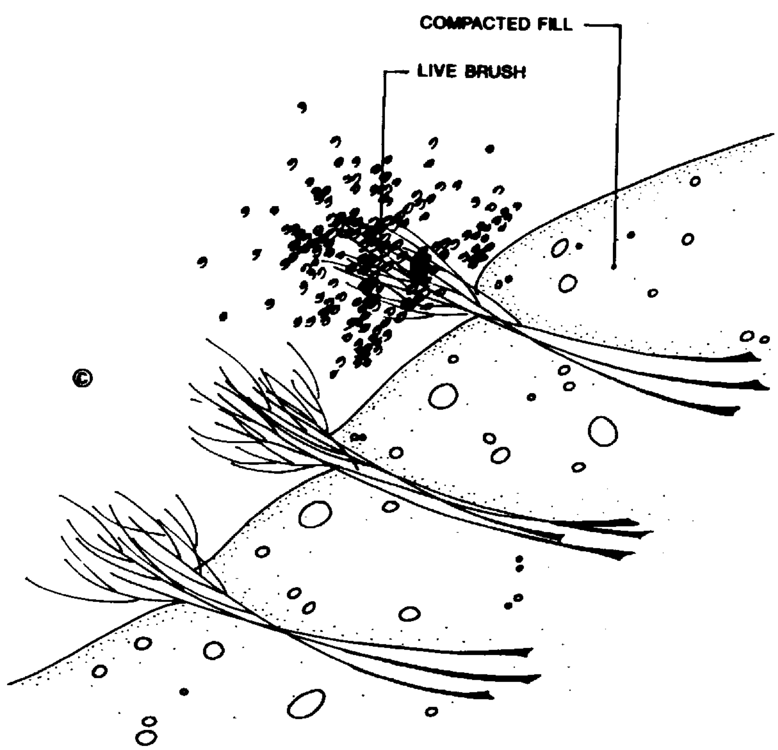

Figure 3. – Schematic representation of a brushlayer fill installation.

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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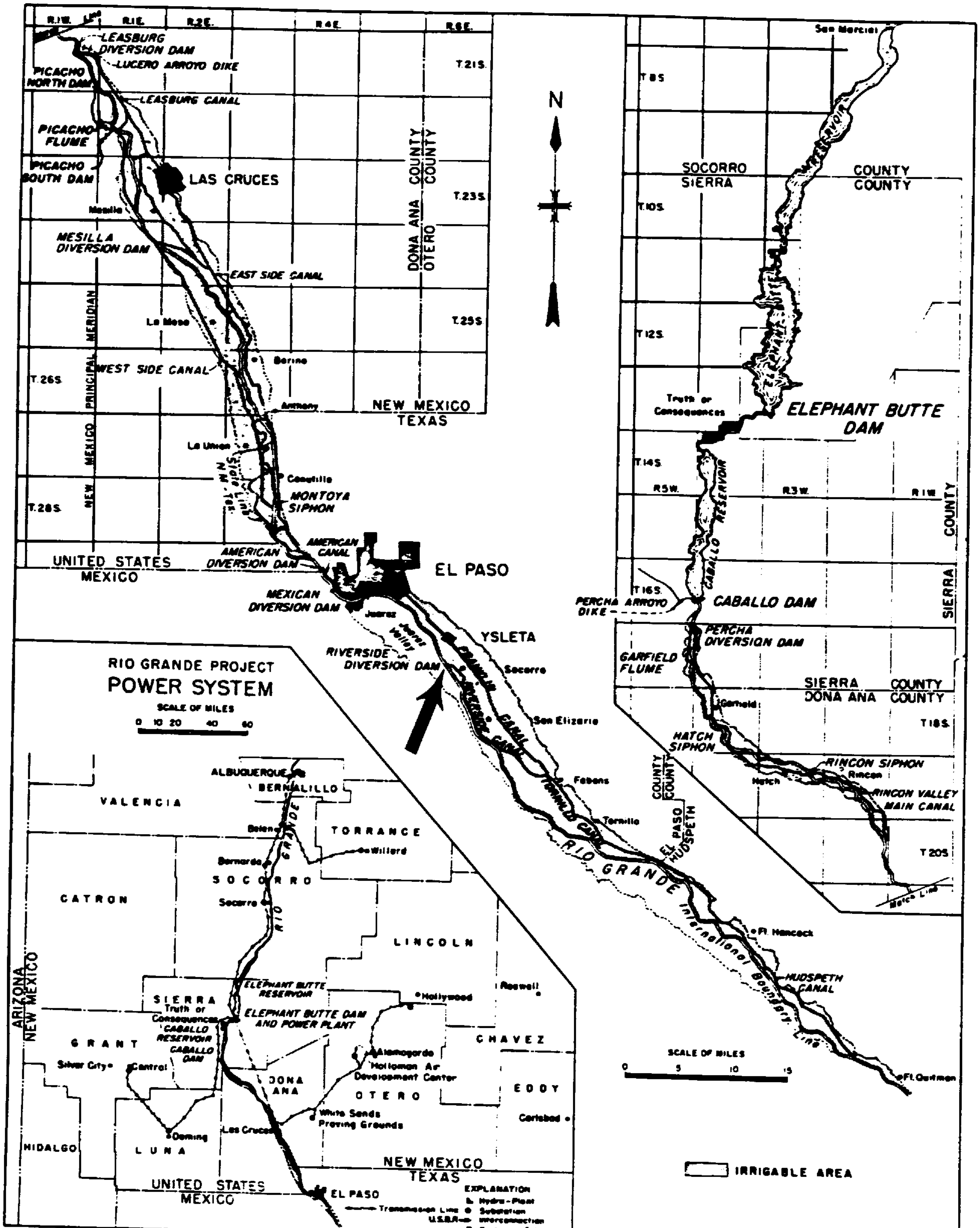


Rio Grande Project

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.

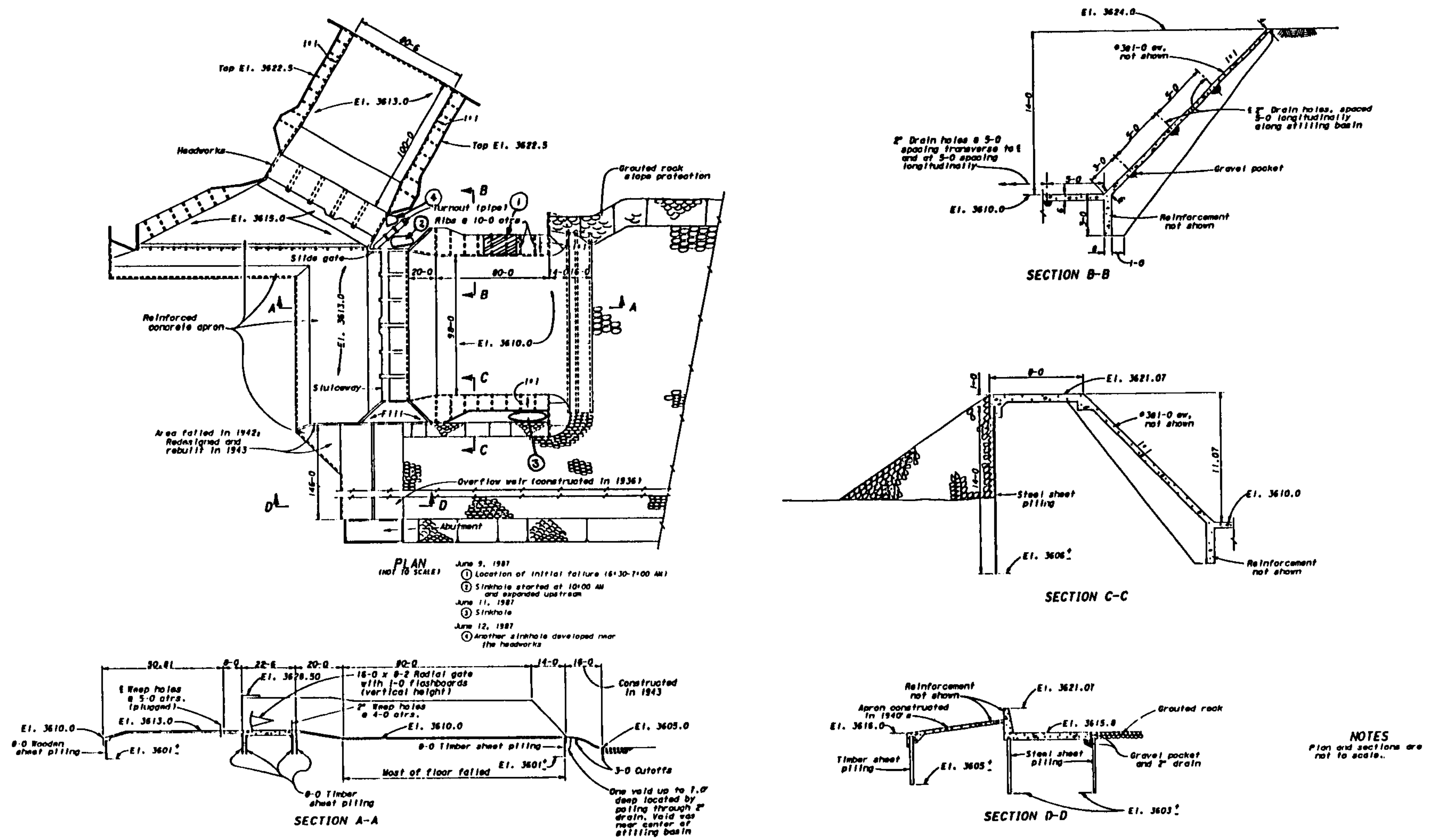
Conditions Immediately Prior to Failure

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:

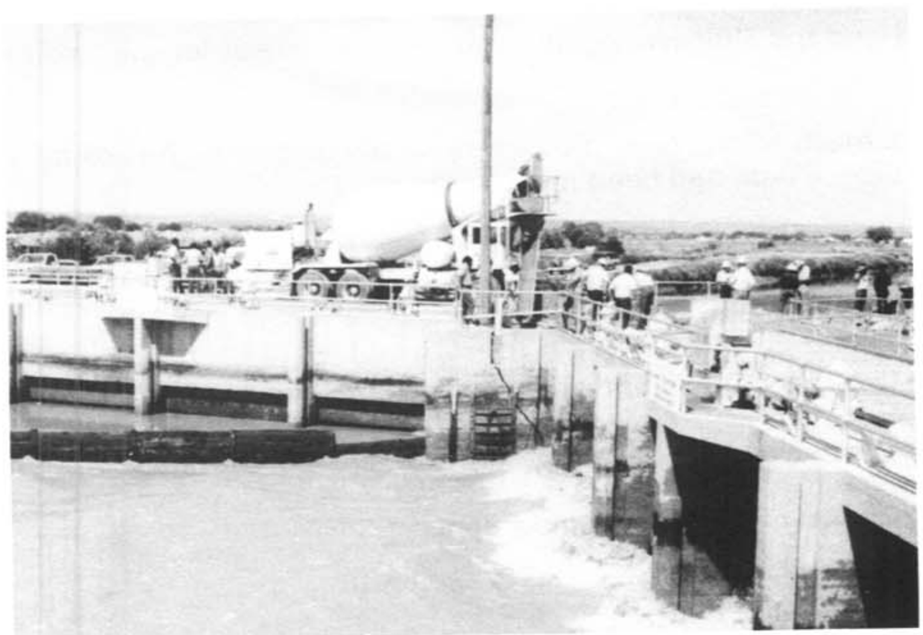
1. Another sinkhole developed near the right abutment of headworks.

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

Temporary Modifications To Enable Diversion

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.

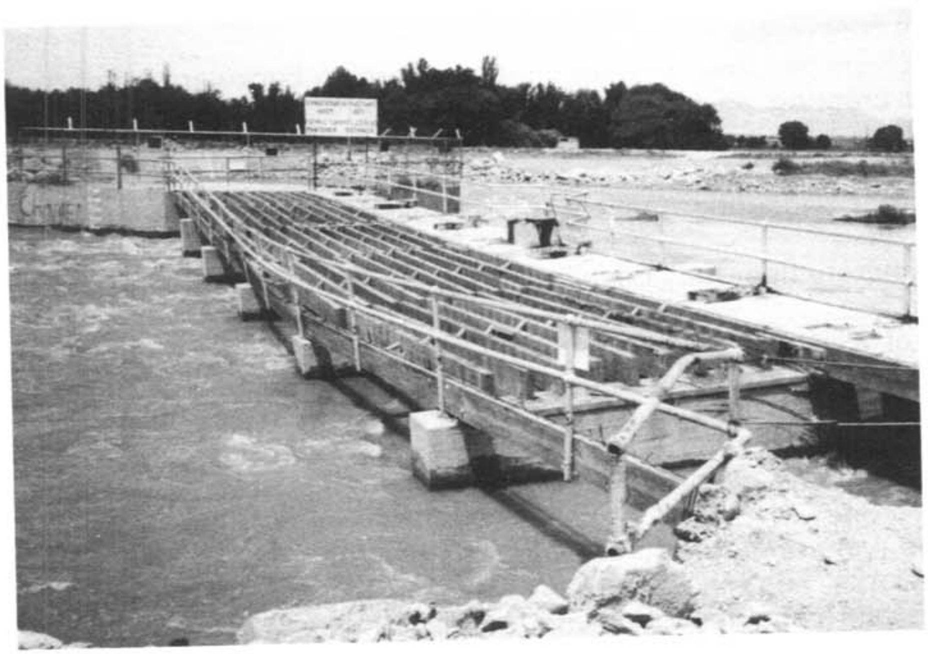
4. Rusting of reinforcing bars may have contributed to failure of a section of the stilling basin. No evidence of rusting of the apron reinforcement was found.

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

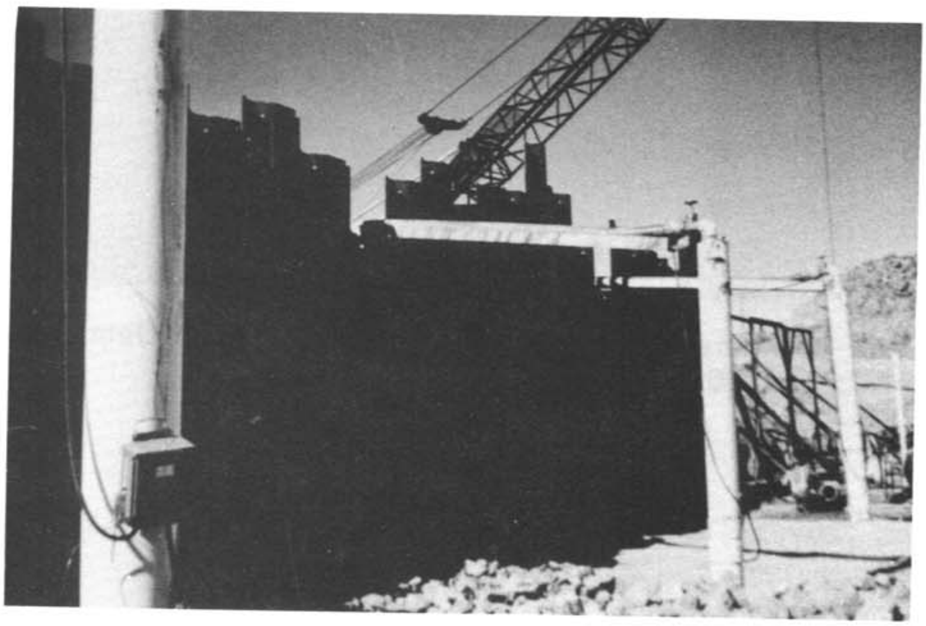

Figure 3. – Work area showing the dewatering wells just upstream of the steel sheet piling. 7/27/89

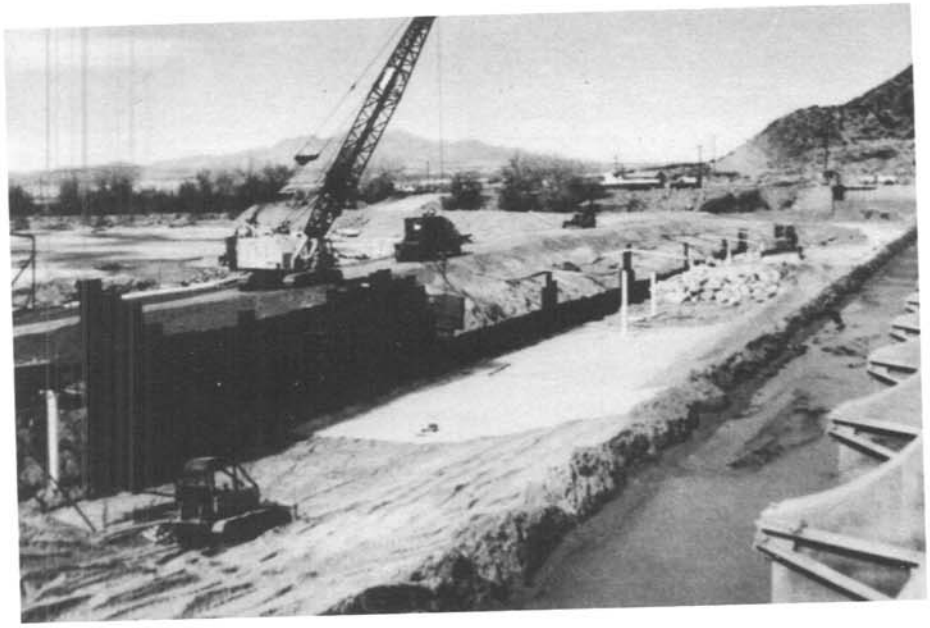

Figure 4. – Work area as viewed from the diversion dam. Some dewatering wells have been relocated to allow for geotextile placement and bedding material (center of photograph). 7/27/89

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

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The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

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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