# Attachment E

# Feasibility Design and Estimate for the ALP Project

*Attachment E to the ALP Project Final Supplemental Environmental Impact Statement (FSEIS) includes a description of the design assumptions and a summary of the design calculations, drawings, and cost estimates of the principal features of the ALP Project Preferred Alternative. Attachment E reflects the current estimates and projections of Reclamation, using standard methodologies and practices.*

*A discussion of potential cost allocations for the structural components of the Preferred Alternative is included as Attachment L to Volume 2 of this FSEIS.*

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# FEASIBILITY DESIGN AND COST ESTIMATE FOR THE ANIMAS-LA PLATA PROJECT

The Department of the Interior (Interior), through the Bureau of Reclamation (Reclamation), and in cooperation with the United States Environmental Protection Agency (EPA) and the Ute Mountain Ute Tribe and the Southern Ute Indian Tribe (Colorado Ute Tribes), has prepared a Final Supplemental Environmental Impact Statement (FSEIS) for the Animas-La Plata Project (ALP Project). This attachment to the FSEIS provides design assumptions and cost estimates for the Preferred Alternative (Refined Alternative 4) for the ALP Project.

## 1.0 PREFERRED ALTERNATIVE AND FEATURES

As discussed in Chapter 5 of the FSEIS, following evaluation of the ALP Project alternatives, Refined Alternative 4 was selected as the Preferred Alternative. This attachment summarizes design assumptions and calculations, provides drawings, and presents cost estimates for the components of the Preferred Alternative. The design and cost estimates have been prepared at the feasibility level. As such, they provide greater detail than an appraisal-level analysis, but less detail than that required for final construction drawings and design specifications.

### 1.1 Preferred Alternative

#### *1.1.1 Dam, Pumping Plant, and Inlet Conduit*

The Preferred Alternative would provide a water supply of 112,000 acre-feet/year (afy) to implement the Colorado Ute Water Rights Settlement Act (Settlement Act) and serve additional municipal and industrial (M&I) water needs in the project area of southwest Colorado and northwest New Mexico. The Preferred Alternative would regulate project water supply with the off-stream, 120,000 acre-feet (af) capacity, Ridges Basin Reservoir. The reservoir would be filled with water pumped into Ridges Basin from the Animas River, two miles south of the center of Durango, Colorado. From this Durango Pumping Plant, water would flow through a 2.2-mile conduit to the reservoir formed behind Ridges Basin Dam that would be constructed across Basin Creek. The Durango Pumping Plant could also supply untreated water to a raw water supply line. The 120,000 af capacity of Ridges Basin Reservoir would include 30,000 af of recreational or fishery storage which would be maintained as a minimum inactive pool. The balance of 90,000 af would be active storage. The reservoir would be operated to regulate the pumped inflow to meet M&I water supply demands and comply with the requirements of the San Juan River Basin Recovery Implementation Plan (SJRBRIP). Water would be available for distribution from the reservoir by connection to the outlet works at the base of the dam, by discharge into Basin Creek where it would flow into the Animas River for downstream use, and by direct pumping from the reservoir to higher elevations.

#### *1.1.2 Navajo Nation Municipal Pipeline*

The Navajo Nation Municipal Pipeline (NNMP) would deliver the Navajo Nation water supply entitlement of 4,680 afy from the ALP Project. Starting at the west boundary of the City of Farmington, New Mexico, it would extend 28.9 miles to Shiprock, New Mexico. The pipeline would replace an existing 30-year old pipeline and would provide increased capacity to deliver water to the Navajo Nation

Chapters of Upper Fruitland, San Juan, Nenahnezad, Hogback and to Shiprock where existing water mains extend to Cudei, and Beclaibito. The pipeline would be operated and maintained by the Navajo Tribal Utility Authority (NTUA), the operating agency for the existing water facilities.

The Intergovernmental Relations Committee of the Navajo Nation Council endorsed the new pipeline project on May 18, 1998. The Navajo Nation Department of Water Resources appraised the pipeline project (Navajo Nation 1998), and it was reviewed by Reclamation (Reclamation 1999).

#### *1.1.3 Water Acquisition Fund*

The non-structural component of Refined Alternative 4 would consist of the creation of a water acquisition fund ($40 million) that could be used by the Colorado Ute Tribes to acquire water rights on a willing buyer/seller basis in an amount sufficient to allow the Tribes approximately 13,000 afy of depletion in addition to the depletion from the structural portion of the project. However, to provide flexibility in the use of the fund, authorization would allow some or all of the funds to be redirected for on-farm development, water delivery infrastructure, and other economic development activities.

### 1.2 Structural Features

#### *1.2.1 Dam, Pumping Plant, and Inlet Conduit*

Major structural features of the Preferred Alternative are the Ridges Basin Dam and Reservoir, an intake structure and pumping plant of 280 cubic feet per second (cfs) capacity on the Animas River below Santa Rita Park (formerly, Gateway Park) in Durango, and an inlet conduit to the reservoir from the pumping plant along a route south of Bodo Creek and County Road (CR) 211. The feasibility design for Ridges Basin Dam proposes a zoned earthfill dam containing a thick impervious core bordered by filters and drains and supported by sloping pervious shells upstream and downstream. The height of the dam, streambed to crest would be 217 feet and the length of the crest 1,670 feet. The crest elevation would be 6,892 feet, the full level would be 6,882 feet; and bottom of active storage would be 6,801 feet. Upstream and downstream slopes would be 2:1 (horizontal to vertical) in the active storage range with a bench at the bottom level of active storage and below that level; 3:1 upstream and 2-1/2:1 downstream. The core would bear directly on the foundation rock and the compressible alluvium would be removed both upstream and downstream for placement of the shell of the dam. Foundation excavation would amount to 2,600,000 cubic yards (cy) and dam fill would require 5,700,000 cy.

The Durango Pumping Plant feasibility design includes an intake structure with trash grate, fish screens and fish bypass conduit to the river; a pumping plant building housing five 56 cfs horizontal centrifugal pumps with a total capacity of 280 cfs. The design would also include four smaller pumps to handle lower river flows, to trim between the larger units, and to provide redundancy for reliability; and an electrical switchyard next to the building to receive power from the Western Area Power Administration (WAPA) transmission line and supply electricity for the motors and plant. The Ridges Basin Inlet Conduit feasibility design proposes a 66-inch diameter pipeline buried along a route from the pumping plant south across CR 211 and up Bodo Draw south of the creek line, on the alignment described in the Final Supplement to the Final Environmental Statement (1996 FSFES) (Reclamation 1996), terminating at a discharge structure over the crest of the ridge. Water from the discharge structure would flow in a rock-lined channel leading to the reservoir. Hydraulic surge in the conduit would be controlled with a spherical surge chamber located next to the pumping plant, away from the river.

### *1.2.2 Navajo Nation Municipal Pipeline*

The Navajo Nation has requested that a water conveyance pipeline (the NNMP) be included as a structural component of the ALP Project to upgrade the service now being provided for Navajo Nation Chapters in the Farmington and Shiprock areas, and to replace the existing 30-year old pipeline. The NTUA delivers water to seven Navajo Nation Chapters: Upper Fruitland, San Juan, Nenahnezad, Hogback, Shiprock, Cudei, and Beclaibito. The pipeline would be operated and maintained by the NTUA, the operating agency for the existing water facilities.

The new pipeline would deliver 4,680 af (2,340 af of depletion) of M&I water from the ALP Project to supplement the water supply to these seven chapters. Existing M&I water requirements are currently being provided through an existing pipeline from the City of Farmington's water treatment plant.

## 1.3 Non-Structural Features

The Colorado Ute Tribes would still be approximately 13,000 afy short of the total quantity of depletion recognized in the Settlement Agreement. Therefore, the non-structural component of the project would establish and utilize a $40 million water acquisition fund which the Colorado Ute Tribes could use over time to acquire water rights on a willing buyer/willing seller basis in an amount sufficient to allow the Tribes to purchase approximately 13,000 afy of historical depletions in addition to the depletions available from the structural portion of the project. To provide flexibility in the use of the fund, authorization would allow some or all of the funds to be redirected for on-farm development, water delivery infrastructure, and other economic development activities.

## 1.4 Prior Designs

### *1.4.1 Dam, Pumping Plant, and Inlet Conduit*

The Preferred Alternative represents a large reduction from the facility proposed in the Definite Plan Report (Reclamation 1979). At that time, Reclamation described a project that would provide 80,100 afy of M&I water supply and 118,100 afy of irrigation water. The proposed dam and reservoir capacity was 273,000 af and the pumping plant flow rate was 480 cfs. Comparisons with prior designs in this report relate to the structures described in Appendix A, Designs and Estimates, of the Definite Plan Report (Reclamation 1979), and to the modifications in dam alignment and pumping plant configuration described in the 1996 FSFES.

### *1.4.2 Navajo Nation Municipal Pipeline*

The City of Farmington has supplied treated water to NTUA since completion of the existing Farmington to Shiprock pipeline in 1969. The original 30-year water contract was recently renewed for five years with an optional additional five years. It provides for a maximum supply of 3.0 million gallons per day (MGD). Metered flow during the peak month of July 1998 averaged 1.7 MGD. Water supplied to NTUA in the existing pipeline amounted to 1,168 afy in 1977. The proposed pipeline would receive up to 4,680 afy of ALP Project water also handled through the city system and treatment plant. The system would be designed for a peak flow of 8.1 MGD (equal to 12.6 cfs), which is two times the average annual flow.

The need for water has been projected based on increased population connected to the system and on per person water use. The need for water is projected to be 2,400 to 2,600 afy in 2013 (Molzen-Corbin 1993) and 8,245 afy for the Navajo Nation Chapters in 2040 (Navajo Nation 1998).

## 1.5 Relocation of Interferences

### *1.5.1 Gas Pipelines*

Four gas pipelines lie within the reservoir area. Three owned by Northwest Pipeline Corporation (Northwest) and one pipeline owned by Mid-American Pipeline Company (MAPCO) would need to be relocated in order for dam construction to proceed. The two MAPCO gas product lines that parallel the existing 26-inch Northwest natural gas pipeline would be relocated in the same right-of-way acquired for the Northwest relocation.

In anticipation of the need for a relocation corridor around the Ridges Basin area, Reclamation performed an “Alternate Route Analysis for Gas Pipeline Relocations in Ridges Basin.” This evaluation examined 17 different relocation alternatives to assist in evaluating the relocation resource and environmental impacts (see Volume 2, Attachment K). The evaluation criteria used included biological resources, cultural resources, recreation, geologic, aesthetic considerations, land use, operation and maintenance, and construction costs. Based on these criteria, relocation routes near the proposed dam and reservoir ranked highest, but these alternatives required further refinements to answer concerns about the potential effect on aquatic life and dam safety. One of the recommendations that the analysis listed was to investigate the conflict of having utilities installed near dams, water storage, diversions, or conveyance structures. The conclusion of the analysis was that the relocation routes that were within the reservoir or near the dam should not be considered as viable alternatives. The highest ranked alternatives then became those crossing Southern Ute Indian Tribe lands south of the proposed reservoir area.

Reclamation then consulted with the Southern Ute Indian Tribe on the acceptability of selecting a relocation alignment that would cross Tribal lands and was informed that these alignments would be considered by the Tribe.

After the Preferred Alternative for the ALP Project is finalized, a relocation corridor will be selected using the results of the route analysis report, the ALP Project requirements, an evaluation of the right-of-way acquisition requirements, and any new environmental and cultural resource information available. Additional environmental compliance may be necessary, depending on the actual relocation corridor selected for the gas pipeline relocations.

No known gas pipelines would require relocation for the NNMP, but the pipeline would cross several petroleum pipelines.

### *1.5.2 County Road 211*

Portions of the existing CR 211 would be inundated by the reservoir and would be relocated above the reservoir’s high water level. Two routes were investigated. Each would begin at CR 211 on the west side of the crest of Bodo Draw and proceed west about 1.3 miles along the low hills north of the proposed reservoir and near the 115-kilovolt (kV) Tri-State Generation and Transmission Association, Inc. (Tri-State) transmission line. One alternative would be to relocate a portion of the road up a draw and continue westerly on top of the ridge 1.8 miles to an intersection with Wildcat Canyon Road (State Highway 141) at the entrance to the Rafter J residential area. The other alternative would be to continue west, cross the electric transmission line and continue 1.2 miles on the uphill (north) side of the transmission line to junction with existing CR 211 west of the future high water level.

There are no known county road relocations required for the construction of the NNMP

### *1.5.3 Electrical Transmission Line*

A 0.6-mile portion of the 115-kV Tri-State transmission line would be affected by the planned full reservoir water surface levels at 6,882 feet elevation. Six structures that would otherwise stand in up to 12 feet of water at their present location would be relocated westerly to higher ground.

Electrical transmission line relocations would not be required for the NNMP.

## 1.6 Access Roads

### *1.6.1 Dam, Pumping Plant, and Inlet Conduit*

Access for project construction activities would be from CR 211, a dirt road. County Road 211 branches from South Camino del Rio close to a signaled intersection with State Highway 160/550 in Durango. Space for construction equipment and supplies and for worker parking would be available in the reservoir basin and to the south of the building site.

Road access for future operation and maintenance of the dam would connect with CR 213, La Posta Road, and proceed along the general alignment of existing private roads to the materials extraction site and haul road that was used for the Uranium Mill Tailings Remedial Action (UMTRA) Program in 1990. This site has been designated Borrow Area B and would be expanded as a select materials source for Ridges Basin Dam. Between CR 213 and Borrow Area B, improvement or construction of 1.6 to 2.0 miles of road would be required, depending on the route agreed upon, and Borrow Area B to the dam would require 0.8 mile of improvement and 0.6 mile of new construction. A roadway across the downstream slope of the dam and 0.5 mile of new road on the right abutment would provide access to the dam crest.

### *1.6.2 Navajo Nation Municipal Pipeline*

The new pipeline would generally follow the alignment of the existing pipeline for nearly two-thirds of the route from Farmington to Hogback, with a route deviation on the western portion from Hogback to Shiprock. The requirement for new construction access and maintenance roads would be minimal.

## 1.7 Land Acquisition for Structural Features

### *1.7.1 Dam, Pumping Plant, and Inlet Conduit*

Reclamation currently owns 4,638 acres of land in the Ridges Basin area. For project construction, proposed acquisitions include about 680 acres to complete the land needed for the reservoir, about 830 acres needed for the borrow area and access, and an easement of 20 acres needed for the pumping plant, and a temporary pumping plant construction easement. Easements for the inlet conduit (approximately 12 acres), increased flows in Basin Creek, and for improvement and future use of access roads from CR 213 would be required.

### *1.7.2 Navajo Nation Municipal Pipeline*

Because the majority of construction would be along the existing pipeline, minimal additional right-of-way would be needed for NNMP. A temporary construction easement would be in addition to the permanent needs.

### 1.8 Construction Program

#### *1.8.1 Dam, Pumping Plant, and Inlet Conduit*

Project construction would span a period of seven years. Beginning with final design engineering, the relocation of gas pipelines would start while the specifications and construction documents are being completed for the dam. At the dam site, excavation of the tunnel portals and tunnel construction would start once the gas lines are removed, about 18 months from project start. While tunnel construction is underway, the cut-off wall and dewatering wells would be installed, the outlet works stilling basin and channel constructed with the objective of completing stream diversion into the tunnel within 18 months under the dam contract, or about 3 years into the project schedule. The pumping plant construction would start approximately 30 months into the schedule, and the inlet conduit construction would begin about 9 months later. Equipment delivery times on the order of 12 to 14 months are anticipated for the pumping plant and conduit. Foundation excavation at the dam may take 8 to 10 months and embankment construction 20 to 30 months depending on whether double shifts are used.

#### *1.8.2 Navajo Nation Municipal Pipeline*

It is anticipated that two years would be required for NNMP construction. Design would be completed in the fifth year and construction completed in the seventh year of the seven-year construction program.

### 1.9 Project Operation

#### *1.9.1 Dam, Pumping Plant, and Inlet Conduit*

Operation of the pumping plant and dam outlet works would be conducted from the control room of the Durango Pumping Plant. The control room would be in communication with the Reclamation office in Durango where operation of southwestern Colorado projects is coordinated. River flow, reservoir level, outlet flows and upstream watershed gage data indicative of changes in river flow, would be directed to an operational model to advise of the best combination of pumping units to meet the reservoir and downstream demands and comply with the river bypass requirements and downstream commitments. Equipment maintenance duties and inspection patrols of the dam and reservoir would be directed from the pumping plant. Equipment and facility repair tasks beyond the scope of periodic maintenance duties would be assigned to specialized contractors.

#### *1.9.2 Navajo Nation Municipal Pipeline*

The pipeline would be operated and maintained by the NTUA, the operating agency for the existing water facilities. Treated water would be purchased from the City of Farmington.

## 2.0 RIDGES BASIN DAM AND RESERVOIR

### 2.1 Regional Geology And Seismicity

The proposed Ridges Basin Dam and Reservoir site is located near the eastern margin of the Colorado Plateau in the Navajo physiographic section. To the north of the site area, the high peaks of the Needle, La Plata, and San Juan Mountains form the picturesque Southern Rockies physiographic province. The Upper Cretaceous sedimentary rocks that underlie the site area form part of the northern margin of San Juan River Basin, a structural basin approximately 100 miles in diameter. The bedrock near the dam site consists primarily of Upper Cretaceous age (70 million years ago) sandstone, shale, siltstone, and coal.

The formational names of these upper cretaceous sediments, in order of oldest to youngest include: Point Lookout Sandstone, Menefee Formations, Cliff House Sandstone, Lewis Shale, Pictured Cliffs Sandstone, Fruitland Formation, and the Kirkland Shale.

The major structural features of the region were formed millions of years ago during the late Cretaceous to early Tertiary Laramide orogeny (Kilgore 1955; Kelly 1957; Ridgley et al. 1978). The Upper Cretaceous rocks of the dam site area form a hogback monocline as part of the northern boundary of the San Juan Basin. These rocks are inclined 8 to 32 degrees towards the southeast into the basin and gradually flatten until they are nearly horizontal in the center of the basin 50 miles to the southeast. Much younger surficial deposits in the dam site area include alluvium, alluvial fans, colluvium, landslides, and glacial outwash deposits. The glacial outwash deposits are in the vicinity of Borrow Area B, about 2 miles southeast of the dam site.

A detailed discussion of seismicity of the region is contained in Reclamation's Seismotectonic Report 92-2 (Reclamation 1992a). The report concluded that the potential seismic source closest to the Ridges Basin Dam site is a random (floating) earthquake, unrelated to any mapped faults. This random source is assigned a maximum credible, local magnitude of $6.5M_L$ and would include any surface-rupturing earthquakes on faults within the vicinity of the dam. Using historic seismicity data for the region, a probabilistic analysis resulted in an annual probability of occurrence of $2x10^{-5}$ for a $6.5M_L$ earthquake about 14 kilometers from the site. Other potential seismic sources were considered; however, none of these exceeded the assigned magnitude for the random earthquake or were closer than 14 kilometers from the site.

## 2.2 Site Geology and Seismicity

The principal bedrock units exposed at the dam site include Lewis Shale, Pictured Cliffs Sandstone, and the Fruitland Formation in ascending order. The Lewis Shale consists predominately of siltstone of deep-water marine origin. Siltstone logged by Reclamation (Reclamation 1992b) in drill holes is typically calcareous, nonfissile, well indurated, dark gray to black, laminated to very thinly bedded with predominately convoluted bedding, and moderately soft. In surface outcrops, the Lewis Shale weathers to rusty, olive gray platy pieces surrounded by silty, clayey soils.

The Pictured Cliffs Sandstone consists of a massive sandstone ledge (approximately 90 feet thick) ledge underlain by interbedded sandstone and siltstone. The massive sandstone unit forms predominant cliffs at the dam site, particularly on the left abutment. This sandstone unit is typically fine-grained, quartzose, moderately cemented, light gray, laminated to massively bedded and hard, with interbedded sandstone having similar properties. In surface outcrops, the Pictured Cliffs Sandstone weathers to nonfissile, well indurated, dark gray to yellowish brown, moderately hard to moderately soft, blocky ledges.

The Fruitland Formation consists of a non-marine sequence of interbedded sandstone, siltstone, shale and coal. The sandstone is typically fine-grained, quartzose, light gray, laminated to thickly bedded and hard. The siltstone and shale are typically carbonaceous, occasionally sandy, dark gray, laminated to thinly bedded with convoluted bedding common, and moderately hard to moderately soft. The coal is black, brittle, laminated to thinly bedded, soft and generally moderately fractured.

Within the proposed dam embankment footprint, the surficial deposits consist of colluvial veneers on both abutments, alluvial fan deposits that interfinger with predominantly fine-grained fluvial and alluvial deposits of clay and clayey silt. The clay deposits are up to 90 feet thick in the central portion of the dam. Further upstream within the reservoir area, these clay deposits have been designated by as Borrow Area A Reclamation (1992b). Downstream, about two miles to the southeast, a fairly large deposit of glacial outwash materials consisting of cobbles, gravels, sand, silt and some minor clay exists. These coarse grained deposits have been designated by Reclamation as Borrow Area B (Reclamation 1992b).

### *2.2.1 Abutments*

The right abutment is covered by up to 40 feet of colluvium interfingering into alluvium at the base of the slope where the maximum thickness occurs. Bedrock to be encountered consists of sandstone and interbedded sandstone and siltstone units of the Pictured Cliffs Sandstone and siltstone with sandstone interbeds of the Lewis Shale. The right abutment of the planned embankment does not encounter the coal-bearing Fruitland Formation. Depth to groundwater ranges from about 220 feet near the dam crest to about 45 feet near its base.

Sandstones within the right abutment would require only minimal surface stripping of loose blocks and intensely to moderately weathered areas to obtain suitable foundation. Some minimal drilling and blasting or dental concrete would be required in areas of overhangs and ledges. The siltstones and shales have been identified as being moisture-sensitive and susceptible to slaking upon exposure. Prolonged exposure may require secondary excavation and clean-up.

The left abutment is covered by approximately 10 feet of colluvium in the higher elevations with up to 15 feet of colluvium and alluvial fan deposits near the base of the slope. Bedrock consists of sandstone, siltstone and interbedded sandstone, and siltstone units of the Pictured Cliffs Sandstone and siltstone with sandstone interbeds of the Lewis Shale. The Lewis Shale would be encountered only during excavation at the base of the left abutment. Groundwater would be encountered during excavation at and near the base of the left abutment and would require dewatering.

The blocky sandstone overhangs on the left abutment would require line drilling and blasting with the use of a hoe-ram in other areas for shaping. Some of the sandstone units are very hard and will require close space drilling and blasting. The Lewis Shale can be excavated by common methods and may be susceptible to air slaking requiring secondary cleanup.

### *2.2.2 Valley Foundation*

The valley alluvium extends up to 90 feet thick and consists of sandy clay, clayey sand, and lean clay with varying amounts of gravel. Gravel lenses are encountered more along the margins of the valley as the alluvial fans interfinger with the finer-grained deposits. The boring logs also show the presence of more gravel beneath the upstream shell of the dam (Reclamation 1992b, 1996). The dry density of the valley deposits range from about 85 to greater than 100 pounds per cubic foot (lbs/$ft^3$), with the higher densities occur where more sand and gravel exists.

Groundwater occurs under unconfined conditions within the fine-grained alluvium and within secondary fractures systems of the bedrock. The groundwater table is 30 to 40 feet below ground surface, except within the deeply eroded creek channel where groundwater is near the surface.

For the 273,000 af dam and reservoir proposed by Reclamation (Reclamation 1996), the soft, compressible upstream alluvium would be left in place beneath the dam embankment and would be consolidated over a two and one-half year period with a wick-drain/surcharge fill system. With the smaller 120,000 af reservoir, this approach becomes less cost-effective and less technically feasible for the following reasons:

1. Permeability of foundation soil (alluvium) is one of the important factors that may affect the consolidation times or effectiveness of the wick drains. This parameter is assumed to be $10^{-6}$ cm/sec for both horizontal and vertical directions. This assumption may not be true, because horizontal permeability is always higher than vertical permeability. As Reclamation points out in its analyses, the consolidation time would be affected by the alluvium permeability. Higher soil permeability tends to prolong the consolidation time (refer to Reclamation technical memorandum No RB-3620-22, Table 2, 1996) (Reclamation 1992c). The laboratory consolidation tests on samples taken from foundation soils indicate that upstream alluvium has a higher Cv (higher permeability) than downstream or central zones. Sandy and gravelly lenses are suspected to be present in the upstream alluvium. If this is true, the use of a coefficient of permeability of $10^{-6}$ cm/sec for both horizontal and vertical directions for upstream alluvium may not be representative of field conditions. Consequently, the design calculation may underestimate the consolidation time from three to nine years if the in-situ permeability is changed from $10^{-6}$ to $10^{-5}$ cm/sec. In addition, in-situ soil permeability is best obtained from a field pumping test, rather than extrapolating from non-representative laboratory consolidation tests.

2. The anticipated loading conditions would extend beyond the limits of previous projects that have used wick drains to accelerate consolidation. The height of embankment which would be placed above the wick drains is considerably greater than typical applications, and the Consultant Review Board members recommended the maximum surcharge embankment height be limited to 125 feet. With this limitation, only the upper one-third of the embankment foundation could be surcharged.

3. Even with the 125-foot surcharge height limit, the possibility of buckling and lateral shear of the wick drains would still exist. The flexible wick drain may potentially buckle under high surcharge load on the order of 125 feet as stipulated by the Consultant Review Board. In addition to high axial load, the flexible wick would also be subjected to high lateral loads induced by the embankment. This could potentially tear the wick drains and develop a (weak) failure plane in the foundation soil.

4. Even if lower permeabilities are present, the 2-1/2-year period to achieve 90 percent consolidation for the upper one-third of embankment foundation would not be a practical consideration, when this material could simply be removed.

5. The excess pore pressures expected from the wick drain/surcharge system would require costly instrumentation and monitoring of the foundation soils and embankment during construction and for a period after construction. The concept of a staged construction method to strengthen the alluvium foundation is good. It would eliminate the potential settlement of foundation soil and increase the alluvium soil strength. However, this construction method may require an expensive geotechnical instrumentation, monitoring program, and engineering evaluation to verify if the increased soil strengths meet the design criteria. Potential changes in the construction schedule may occur if the expected field performances do not meet the design criteria.

6. Installation of wick drain may encounter refusal due to the potential presence of stiff clay and dense sand or gravel. The presence of more gravel and dense clay within the upstream valley alluvium would limit the installation of wick drains without a lot of pre-drilling.

For these reasons, the proposed design requires removal of the alluvial materials beneath the dam embankment both downstream and upstream. The planned excavation below the water table requires a soil-bentonite cut-off wall just upstream of the upstream toe of the dam embankment and downstream of the excavation for the approach channel. This groundwater barrier would be from 40 to 95 feet deep and extend across the valley floor about 1,750 feet. A series of dewatering wells upstream of the soil-bentonite cut-off wall is also planned. Dewatering trenches and sumps would be required throughout the valley floor excavation in bedrock.

Most of the bedrock to be exposed by the valley floor excavation is Lewis Shale with lesser amounts of Pictured Cliffs Sandstone in the downstream left and right sides of the channel. Excessively weathered and soft areas of the Lewis Shale would require over-excavation and backfill with lean concrete or compacted materials.

### *2.2.3 Faults, Joints, Fractures, and Other Bedrock Defects*

No significant evidence of faulting was encountered in any of the Reclamation exploration drill holes or geologic mapping at the site (Reclamation 1992b). One insignificant, bedrock fault was observed in a road cut about 580 feet downstream of the toe of the dam. No other evidence of faulting, such as surface offsets or a break in correlation of lithology was found. If a fault does exist, other evidence surrounding the site suggests it would be at least of Tertiary age and would not have any impact on dam design.

Sedimentary bedding at the dam site ranges from approximately 20˚ to 22˚ towards the southeast near the top of the left abutment to 28˚ to 35˚ toward the southeast on the right abutment. This steeping of dip is thought to reflect a point of flexure on the monoclonal ridge that forms Basin Mountain.

Left abutment bedding joints strike N32˚ to 57˚E with dips ranging from 12˚ to 32˚SE. Joint spacing ranges from a minimum of one-quarter inch near the surface to about seven feet at depth. Most joints are slightly open to tight and often have clay or other material coating and infilling. The joints are moderately rough surfaced.

On the right abutment, bedding joints are oriented N49˚E and dip 27˚SE. These joints are spaced from 1/32 inch to 1 foot apart near the surface to up to 21 feet at depth.

Three predominant joint sets are described by Reclamation as “A,” “B,” and “C” for the two abutments and valley floor bedrock, with the “A” set subdivided into two subsets as follows (Reclamation 1992b):

| | |
|---|---|
| Bedding: | N43˚ to 49˚E, 23˚ to 27˚SE |
| Joint Set A-NE: | N07˚ to W, 60˚ to 82˚NE |
| Joint Set A-SW: | N07˚ to 24˚W, 75˚ to 76˚SW |
| Joint Set B: | N66˚ to 76˚W, 75˚ to 77˚NE |
| Joint Set C: | N75˚ to 78˚E, 56˚ to 71˚NW |

The joint lengths range from a few feet to up to 100 feet long, spaced very close (less than ¼-inch) at the surface to 50 feet at depth. The average joint spacing is 5 to 10 feet. Most have coatings or in-filling of clay, silt, or other materials.

### *2.2.4 Potential Seepage*

Seepage through the bedrock is not expected to be excessive, particularly in the Lewis Shale. However, for feasibility-level planning, grouting is considered to control seepage within the bedrock. The proposed design includes a single line grout curtain placed 30 feet upstream of the dam centerline. Primary curtain holes would be drilled to a depth of about 95 feet and spaced on 20-foot centers. In some areas, secondary grout holes spaced on 10-foot centers may be needed. Grout holes would be under a grout cap in 30-foot stages. All grout holes would be inclined about 30 degrees from the vertical in a northwest direction.

Additional blanket grouting may be required in localized areas of both of the abutments, particularly in the more fractured and closely jointed sandstone units of the Lewis Shale and Pictured Cliffs Sandstone and at the contact between these two units. The right abutment does not encounter the Fruitland Formation, so the more extensive grouting planned for the prior higher dam embankments would not be needed; however, the contact between the Lewis Shale and Pictured Cliffs Sandstone would be grouted.

Grout takes are expected to be relatively low, probably in the range of .5 half-bags per foot of drill hole. Isolated grout takes of up to two bags per foot can be expected where open fractures/joints are encountered.

#### *2.2.5 Reservoir Leakage*

Reservoir leakage is expected to be minimal because the majority of the reservoir would overlie the Lewis Shale, weathered portions of the Lewis Shale, and the clayey soil deposits formed from weathering of the Lewis Shale. These materials are expected to be relatively impermeable. There is a gas well drill hole located just upstream of the embankment toe. This drill hole would be closed and sealed in accordance with state and federal guidelines.

#### *2.2.6 Landslide Potential and Rim Stability*

Several small, shallow earth flows have occurred on the slopes of the right abutment. These shallow slumps occurred along the soil/bedrock boundary and would be removed or treated during construction.

A large rotational rock slump occurred about 1,000 feet upstream of the dam axis on the left side of the reservoir. This failure occurred along the “A” set joint set and moved downslope below the sandstone cliff. Although no age has been determined for this failure, there is no evidence for recent movement. This area would not be impacted by dam construction, but would be inundated, in part, during reservoir filling. During the preliminary design stage, further evaluation of the potential landslide impacts during reservoir filling and lowering would be conducted.

#### *2.2.7 Site Seismicity and Design Earthquakes*

Seismic activity in the vicinity of Ridges Basin Dam is low. U.S. Geological Survey (USGS) catalogs of active faults for the State of Colorado do not indicate any active faults within a radius of 176 kilometers of the site. Based on the probabilistic earthquake ground motion map published by the National Earthquake Hazards Reduction Programs (NEHRP) in 1997, the dam site is considered to have a low spectral acceleration (Sa) on the order of 0.3 g at a period of 0.2 seconds for a 2,500-year seismic event. This correlates to a peak bedrock acceleration of about 0.2 for a 2,500-year earthquake. The design pseudo-static seismic coefficient is taken at half the peak bedrock acceleration (Hynes et al. 1987) under long-term conditions (steady state and partial rapid drawdown). Under short-term conditions (end of construction), the recommended pseudo-static seismic coefficient is 0.05 g.

Based on the Reclamation studies (Reclamation 1992a), the random event was assigned a maximum credible earthquake (MCE) local magnitude ($M_L$) of 6.5 at a distance of 14 kilometers from the dam site.

Using the Toro, et al. (Toro 1997) attenuation relationship for earthquakes in the Central United States, the estimated peak bedrock acceleration at the dam site is about 0.3 g, with bedrock response spectra of 5 percent damping. These design parameters are used to conduct the permanent seismic deformation analysis of the embankment during a MCE event.

## 2.3 Construction Materials and Their Assumed Properties

Borrow Area A upstream of the dam site is the primary source for the impervious clay core. Reclamation subdivided Borrow Area A into four subareas and summarized the average physical properties of the soils within the subareas (Reclamation 1996). To maintain a plasticity index (PI) of 15 or higher, subareas A1 and A2 appear best suited as potential sources. There may be localized pockets of clay within the embankment foundation excavation, as well as the surficial clay deposits in Borrow Area B, that would be suitable for the impervious clay core. Handling, scheduling, haul distances and other factors would govern which sources are used; regardless, the quantity is sufficient to build a wide, impervious clay core embankment with pervious upstream and downstream shells.

For the impervious clay core material, the effective strength of Ø´ = 30˚ and C´ = 0 is adopted for the static stability "end of construction" effective stress analysis (Reclamation 1996). In addition, the wet density of 126 lbs/ft$^3$ was adopted from the Reclamation laboratory results.

Borrow Area B downstream of the dam site is the main source for the upstream and downstream pervious shells as well as the internal drain, blanket drain, and transition filter zone between the impervious clay, drain, and pervious shells. The upstream and downstream shells would be constructed from pit run material that is a mixture of minus 24 inches of boulders, cobbles, gravel, sand, silt, and minor clay with a maximum of silt and clay approximately 10 percent by weight passing the No. 200 sieve (based on the minus 3-inch fraction). Based on previous projects, professional experience, engineering judgment, and empirical correlations, these materials should yield an effective strength of Ø´ = 42˚ and C´ = 0. A wet unit weight of 135 lbs/ft$^3$ and a saturated unit weight of 140 lb/ft$^3$ were assumed for the stability analysis.

Borrow Area B can also provide processed (washed and screened) material for the transition filter zone and the internal drain/blanket drain. The gradation of the transition filter zone material and the drain material would follow the current design standards of the Reclamation, U.S. Army Corps of Engineers, and U.S. Soil Conservation Service as outlined in a recent U.S. Committee of Large Dams publication (Kleiner 1999).

The transition filter zone material would be poorly graded sand (SP) with 100 percent passing the one-half inch screen and 5 percent passing the No. 200 sieve. An inplace or wet density of 120 lbs/ft$^3$ and a saturated density of 130 lbs/ft$^3$ are assumed for the transition filter material. Based on engineering judgment and experience, this material probably has an effective strength of Ø´ = 36˚ and C´ = 0. The internal drain material and blanket drain would be poorly graded gravel (GP) with 100 percent passing the 2-inch screen and 0 to 5 percent passing the No. 4 sieve. A maximum dry density for the drain gravel is approximately 110 lbs/ft$^3$. A wet unit weight of 130 lbs/ft$^3$ is assumed. An effective strength of Ø´ = 40˚ and C´ = 0 is also assumed.

## 2.4 Dam Embankment Design

Material specified for the embankment design of the 273,000 af dam proposed by Reclamation in 1996, was a mixture of gravel, sand, silt and clay, having minimum 40 percent passing a No. 200 sieve. This material is relatively impervious; therefore, high seepage water pressure would not be expected to dissipate during normal drawdown operation.

The upstream slope was designed to be 2:1 from the dam crest (El. 6,973) to the bottom of active storage (El. 6,898). Based on the Reclamation stability calculations (Reclamation 1996), the computed minimum factors of safety under partial draw-down from top to the bottom of active storage (approximately 66 feet draw down) was about 0.59 to 0.93, assuming the upstream shell did not have any cohesion (C=0). Judging from these results, Reclamation considered the use of C=0 for shell and core material as very conservative. Subsequently, Reclamation increased the soil strength of upstream shell and impervious clay core by adding cohesion factors of C= 1,500 pounds per square foot (lbs/ft$^2$) for upstream shell and C= 216 lbs/ft$^2$ for clay core. Based on this assumption, the computed safety factors under partial drawdown was higher than 1 and considered acceptable.

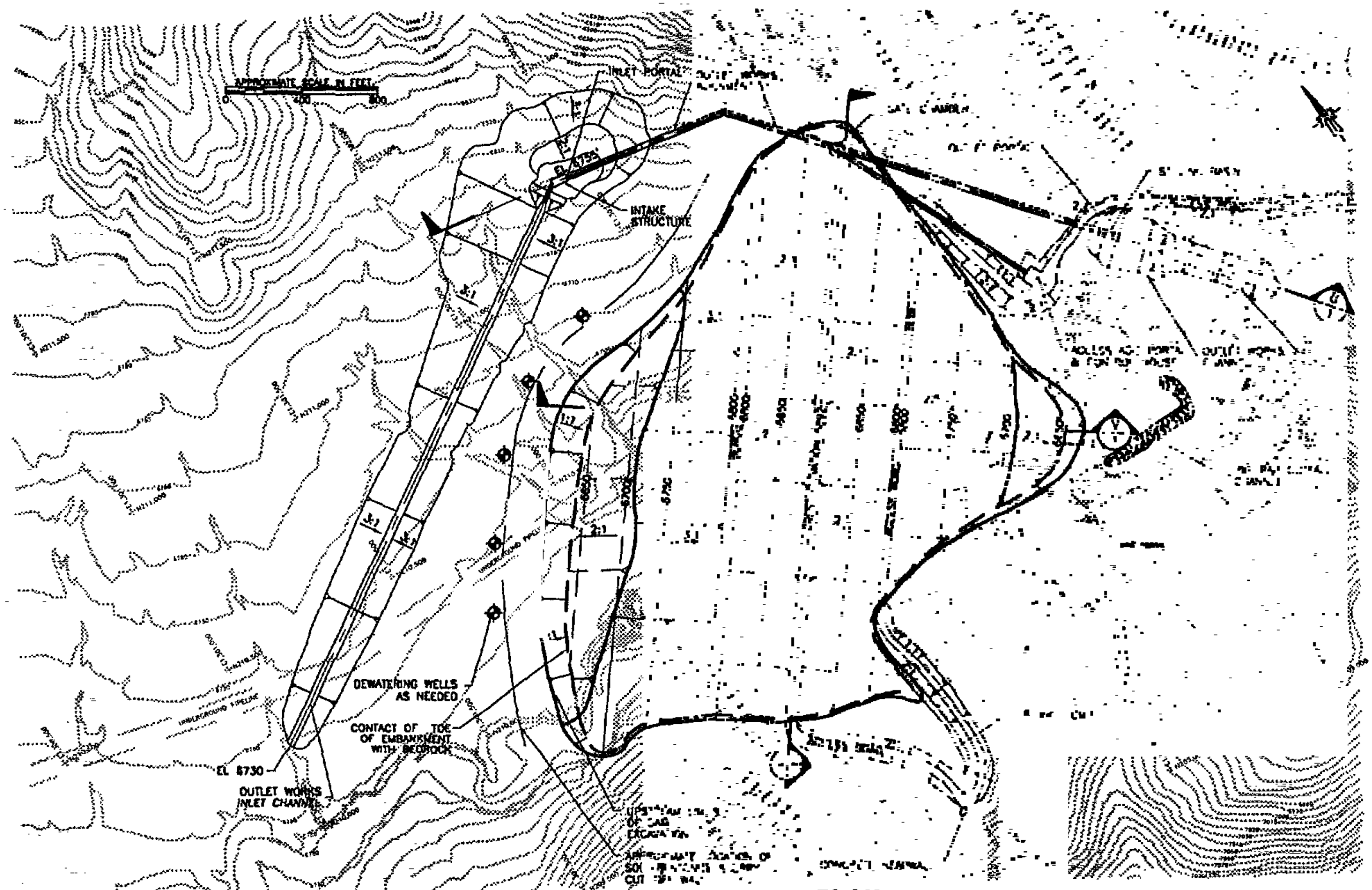
The common practice among geotechnical engineers is to use a drained soil strength (C=0) approach for long-term loading cases, especially if the shell material is predominantly a cohesionless soil. Therefore, the approach to increase the cohesion factors for shell and clay core is not justified as cohesion cannot be relied on for long term stability and under conditions of increased pore pressures during rapid draw down. The previously proposed upstream slope of 2:1, above the bottom of active storage level, may be potentially unstable during partial and rapid drawdown conditions.

Flattening the upstream slope is probably not an effective way to increase the safety factor of the upstream slope. A better solution is to use pervious shell material especially in the active storage zone, so that high seepage pressures would not be developed in the upstream slope during partial drawdown.

Because of the stability issues with the previous conceptual design and the problems with using the wick drains/surcharge consolidation of the clayey alluvium to be left in place under the upstream portion of the dam, a simpler embankment design is proposed. This preferred embankment design consists of excavating all the soft, loose alluvial materials beneath the entire dam embankment and building a wide, impervious clay core. This clay core would be surrounded by filter transition material and an internal chimney drain connected to a blanket drain on the downstream side. Pervious, boulder/gravel/sand rock fill material would be used for both the upstream and downstream shells. To take advantage of the more than 10 years of Reclamation exploration, laboratory and field testing, and engineering design, the proposed embankment is placed in the valley so the dam centerline is the same as the previous Reclamation alignment for larger dams. **Figures 2-1 and 2-2** illustrate the proposed dam in plan view and section.

### *2.4.1 Foundation Excavation and Treatment*

The alluvial materials beneath the planned dam embankment can be excavated by common methods. As noted under the section, Valley Foundation, a soil-bentonite cutoff wall upstream of the upstream toe of embankments is required. This would allow temporary 1 horizontal to 1 vertical (1H:1V) cut slopes in the alluvium. In places, these temporary slopes may have to be laid back at 2H:1V depending on the effectiveness of the dewatering and inplace strength of the alluvium. Within the embankment excavation, dewatering trenches and sumps in the bedrock should be anticipated. As noted earlier, a grout curtain would be placed 30 feet upstream of the dam centerline with primary holes drilled to 95 feet deep, spaced 20 feet apart. The drill holes would be inclined 30 degrees from vertical in a northwest direction to intersect the majority of bedding planes and other rock defects.




PLAN

ATTACHMENT E
FEASIBILITY DESIGN AND ESTIMATE

ANIMAS – LA PLATA PROJECT

**RIDGES BASIN DAM**
**PLAN**

| | | |
|---|---|---|
| DRAWN BY: ROC | REVIEWED BY: DD | FIGURE: 2-1 |
| APPROVED BY: DKR | DATE: 20DEC99 | |

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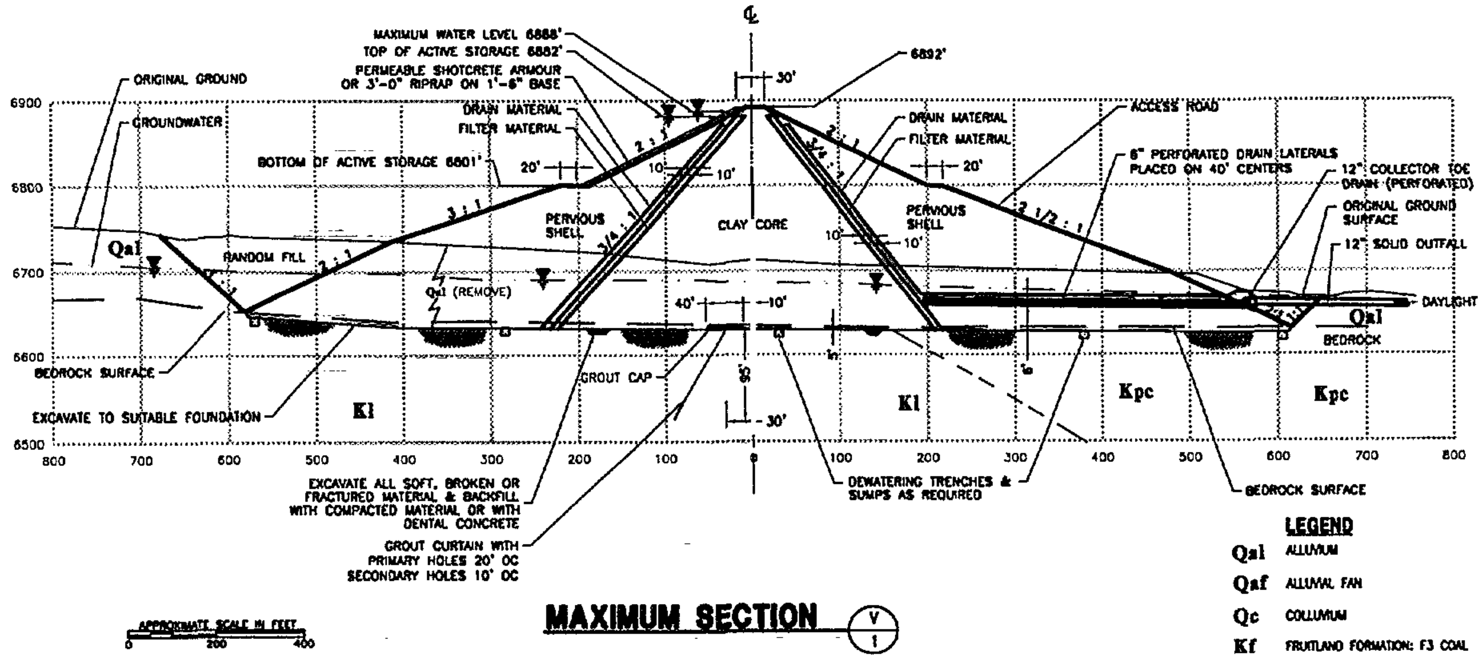


## MAXIMUM SECTION

**LEGEND**

- **Qal** ALLUVIUM
- **Qaf** ALLUVIAL FAN
- **Qc** COLLUVIUM
- **Kf** FRUITLAND FORMATION: F3 COAL
- **Kpc** PICTURED CLIFFS SANDSTONE
- **Kl** LEWIS SHALE

ATTACHMENT E
FEASIBILITY DESIGN AND ESTIMATE

ANIMAS – LA PLATA PROJECT

**RIDGES BASIN DAM**
**MAXIMIUM SECTION**

| | | |
|---|---|---|
| DRAWN BY: ROC | REVIEWED BY: DD | FIGURE: 2-2 |
| APPROVED BY: DKR | DATE: 20DEC99 | |

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### 2.4.2 Clay Core

The impermeable clay core is planned to be about 420 feet wide at the base of the maximum section and would continue to the crest at 3/4H:1V slopes on both the upstream and downstream sides. The crest width is 30 feet. Ten feet of clay would also cap the transition filter material at the dam crest. The crest elevation (without camber) is planned at elevation (El.) 6,892. During preliminary design, an evaluation of the anticipated embankment and foundation settlement would be conducted so that an appropriate camber for the dam can be established. Based on experience and engineering judgment, total settlement at the crest could be as much as 4 to 6 feet. Most of the clay core and upstream shell would be founded on Lewis Shale bedrock where more settlement is expected to occur than embankment founded on the Pictured Cliffs Sandstone.

The final crest elevation of El. 6,892 provides 4 feet of freeboard over the maximum water level from a probable maximum flood (PMF) and 10 feet of freeboard over the top of active storage at El. 6,882.

### 2.4.3 Transition Filters and Drains

Both the upstream and downstream faces of the impermeable clay core would be covered with 10 feet (horizontal distance) of transition filter material (SP) and then another 10 feet (horizontal distance) of drain material (SG). On the downstream side, the chimney drain would connect to a 6-foot thick blanket drain that encases 6-inch diameter, perforated polyvinyl chloride (PVC) drain laterals placed 40 feet on center. These laterals would then connect to a 12-inch diameter, perforated PVC collector toe drain. Three (one in the center and one on each abutment) solid, PVC outfall pipes would then daylight and drain the collector toe drain. Weirs would be constructed for the drain outfall discharge to facilitate flow measurements. In order to daylight the outfalls within a reasonable downstream distance, the blanket drain and laterals with a minimum 3 percent grade are placed about 5 feet below the existing water table and roughly 25 to 30 feet above the foundation bedrock.

### 2.4.4 Downstream Pervious Shell

From the crest down, the downstream pervious shell has a slope of 2H:1V to El. 6,801 where a 20-foot wide bench and access road is planned. Below this bench the slope flattens to 2.5H:1. Random fill is placed within the toe excavation to restore the original ground surface elevation. As the blanket drain is 5 feet below the existing water table and 25 to 30 feet above bedrock, the pervious shell below the blanket drain would be saturated.

### 2.4.5 Upstream Pervious Shell

From the crest down, the upstream pervious shell has a slope of 2H:1V to El. 6,801, the bottom of active storage, where a 20-foot wide bench is planned. This 2H:1V slope would be armored with 3 feet of rip rap on an 18-inch thick base or alternatively, shotcrete armor with back drains and weep holes. Below the bottom of active storage, the slope flattens to 3:1 to about where the original ground surface occurs, near El. 6,750. Below this, the temporary pervious fill slope would be 2H:1V to bedrock. The void at the toe of the dam would be backfilled with random material to the original ground surface.

### 2.4.6 Embankment Quantities

The inplace embankment quantities are estimated to be as follows:

| Raw Cut | |
|---|---|
| Alluvium Excavation | 2,323,250 cy |
| Bedrock Excavation (assumed 5-foot depth) | 248,340 |
| Total | 2,571,590 |

A 1.2 bulking factor can be expected for the alluvium, and the bedrock is assumed to bulk 1.4 times.

| Raw Fill | |
|---|---|
| Impervious Clay Core | 1,245,430 cy |
| Transition Filter Material | 253,010 |
| Chimney and Blanket Drain Material | 360,840 |
| Pervious Shells | 3,524,160 |
| Toe Fills (Random) | 286,900 |
| Total | 5,670,340 |

#### *2.4.7 Stability Analysis*

*General Assumption and Conditions*

Slope stability analyses were conducted for the upstream and downstream embankments using the computer model Stabl5M™. The soil strength parameters were taken mostly from the values that were used in previous Reclamation slope stability analyses (Reclamation 1996). Based on past experiences and engineering judgment for rock fill dams, a Ø of 42 degrees represents the friction angle of well compacted cohesionless (rock fill) material for the upstream and down stream shells. Separate model runs were conducted for static and earthquake loads (using pseudo-static seismic coefficient of 5 to 10 percent and permanent deformation analyses under MCE event). The analyses were performed for short-term (end-of construction) and long-term (steady state and partial or full rapid draw-down

*Condition A : End-of-Construction*

Condition A assumes the impervious clay in embankment core and alluvium foundation having undrained shear strength of 3,000 and 2, 000 lbs/ft$^2$. The calculated static factors of safety (FS) are 1.67 and 1.63 for upstream and downstream slopes, respectively. The calculated pseudo-static FS are 1.44 and 1.46 for 5 percent pseudo-static load.

*Condition B : High Level Steady State (El. 6,888)*

Condition B assumes the impervious clay in embankment core and alluvium foundation having drained friction angle of 28 and 27 degrees, respectively. The maximum water surface in the upstream slope is designed at El. 6,888. The phreatic surface is developed assuming no head loss occurs in the upstream shell and impervious clay core. This would result in higher pore water pressures in the embankments and alluvium foundation; and, hence, is a more conservative approach. Under this condition, the calculated static FS is 2.03 and 1.90 for upstream and downstream slopes, respectively. Under 10 percent horizontal pseudo-static load, the calculated pseudo-static FS are 1.28 and 1.48 for upstream and downstream slopes, respectively. Under the postulated MCE event (magnitude 6.5 with peak ground acceleration of 30 percent), the calculated permanent deformation is very small (less than 2 inches). The design free board of 4 feet against the probable maximum flood (PMF) and MCE event is acceptable.

*Condition C: Partial Rapid Drawdown (El. 6,882 to 6,801)*

During partial rapid draw down, condition C assumes the pheratic line having a partial head loss occurring in the pervious shell and no head loss occurring in the impervious clayey core. The effective and total stress analyses are used to model the soil strength of the impervious clay core under rapid draw down loading. The effective soil strength was used to model the pervious upstream shell material. The calculated static FS for upstream slope ranges from 1.5 to 1.65 using effective and total stress analyses. The calculated pseudo-static FS is 1.16 for 10 percent pseudo static load. The computed permanent deformation of the upstream slope under MCE loading is about 3 inches. Since the pheratic surface in downstream slope is similar to steady state loading, the calculated static and pseudo-static FS for both conditions are identical.

Under very extreme conditions, full rapid draw down is conducted from El. 6,882 to 6,701. Assuming partial head loss occurs in the pervious shell and no head loss occurs in the clay core, the calculated static FS is about 1.49. Under this remote case, the effect of seismic force is always ignored.

In conclusion, the proposed embankment slopes satisfy the minimum factor of safety during end-of construction (FS = 1.3); steady state and partial draw down (FS=1.5); pseudo-static seismic loading (FS=1.1) and full rapid draw down (FS=1.1) assuming the pervious cohesionless material is used in the upstream slope. During partial rapid draw down, the stability of upper slope (above El. 6,801) is primarily controlled by the position of pheratic surface rather than slope geometry. Hence the use of pervious cohesionless material is necessary to induce a significant head loss in the upstream slope.

## 2.5 Outlet Works

In the conceptual, appraisal level stage of design, a pressurized 78-inch diameter conduit outlet with a capacity of 1,530 cfs was considered to be benched into the left abutment for the 135,000 af reservoir and dam. Even after attempting several alignments, the bedrock cuts through the low knoll on the left abutment would be as high as 90 feet and there would be a short segment where bedrock would require lean concrete fill to maintain a suitable elevation and grade. Bending the conduit did not resolve these short-comings and could create adverse hydraulic conditions. Reclamation also rejected a conduit outlet for much of the same reasons as follows (Reclamation 1996):

- ❑ The thick alluvial layer in the valley requires the conduit to be placed close to the abutment to maintain the desired grade; and

- ❑ The dam footprint is similar in shape to an inverted cone with a long upstream toe and a short downstream toe. These combinations make it difficult to develop an alignment suitable for hydraulic considerations (minimum bends).

Reclamation considered both abutments suitable for a tunnel outlet, with the right abutment being originally favored because of economics (Reclamation 1992a; 1996). However, during final design, the right abutment alignment disclosed several problems, including problems with both portal cut excavations and difficulties in locating suitable foundations for both the intake structure and stilling basin. Also, the stilling basin would be located too close to the toe of the dam. For these reasons, the Reclamation tunnel outlet configuration on the left abutment is adopted as the preferred structure and alignment for this feasibility level design and cost estimate.

### *2.5.1 Summary Description of Outlet Works*

Outlet works would consist of an intake approach channel, intake structure, upstream tunnel, gate chamber, downstream tunnel, separate access adit, control building, stilling basin, and the discharge channel. A plan and section is presented on Figure 2-1.

The outlet works intake approach channel would begin at Basin Creek with an invert elevation of 6,730 feet, or about 20 feet above the existing water table. The channel, with a 20-foot wide base and 3H:1V side slopes, would extend 1,614 feet across the valley, terminating at the intake structure. All of the channel excavation would be in alluvium and common methods can be used. The flatter 3H:1V slopes are to provide stability under submerged conditions.

The intake structure consists of a drop inlet structure with its entrance at El. 6,760 and a centerline El. 6,733.75 to insure the drop inlet structure is founded on bedrock. The drop inlet structure would have a temporary opening at El. 6,730 for diversion, which would be plugged with concrete after diversion. A small diversion inlet pipe with a bulkhead gate located in the inlet channel at El. 6,730 and connected to the drop inlet, would eliminate 60 feet of dead storage.

From the inlet structure to the gate chamber, a short 7.5-foot diameter conduit section from the inlet would connect to a 7.5-foot diameter, reinforced concrete, pressurized tunnel about 665 feet long. The tunnel would change direction about mid-point with a 41 degree 38 minute bend.

The gate chamber would be located within the left abutment a few feet upstream of the dam axis. A pair of 4 foot by 6 foot bonneted gates in tandem would function as the main guard gate and regulating gate for emergency releases. A combination of a 20-inch pipeline, isolation ball valve, and a 14-inch jet flow gate would control releases of up to 100 cfs for the regular flows. A second 30-inch pipeline, isolation ball valve, and a 24-inch jet flow gate would be installed to increase the regular releases up to 250 cfs in the future. Both of the discharge pipes from the jet flow gates would be flanged so that connections to future distribution pipelines could be easily made. The working space within the gate chamber would consist of a domed structure with an inside diameter of about 22 feet. Access to the gate chamber working area from the control building would be provided by a 587-foot long access adit from the downstream left abutment of the dam.

The control building would house controls for the outlet gates and valves, flow meters and recorders for release flows, and dam safety instrumentation monitors. It would also house the propane gas-fueled standby generator. Operational and status data would be transmitted by telephone to the Durango Pumping Plant and valve position commands received. Electrical power would be supplied to the control building by an extension of La Plata Electric Association lines from west of CR 213 along the roadway up Basin Creek.

The downstream tunnel would be concrete lined, connecting the gate chamber to the stilling basin. This tunnel section would be flat-bottomed, 8 feet in diameter and about 696 feet long. Flow in the downstream tunnel would be open channel.

The stilling basin would have a length of about 60 feet and would be designed to provide energy dissipation for releases up to 500 cfs. Safe downstream channel capacity and maximum planned releases would be limited to 250 cfs. Emergency releases exceeding the stilling capacity of the basin would be released downstream of the structure; however, this would be an extreme operational condition.

The downstream channel conveys releases from the stilling basin to Basin Creek, a drop of about 40 feet. The downstream channel would consist of a length of rip rap lined channel; a headwall and a length of reinforced concrete pipe; an impact energy dissipater; and a short length of rip rap-lined channel.

### *2.5.2 Summary of Outlet Works Geology*

Tunnel cover varies from 63 feet at the inlet portal to 118 feet at the gate chamber, and 22 feet at the outlet portal, with a maximum cover of 170 feet. Although the entire tunnel is below the water table, groundwater inflow is not expected to exceed 75 gpm. The intake structure and inlet portal would be founded on Lewis Shale and would be excavated below the water table. Approximately 42 feet of alluvial fan deposits would be excavated to reach bedrock at El. 6,730. About 60 percent of the tunnel length would be excavated in Lewis Shale starting at the inlet portal. Pictured Cliffs Sandstone would be encountered about 200 feet downstream of the gate chamber. Methane gas should be anticipated throughout the tunnel excavation, particularly in the Pictured Cliffs Sandstone. The access adit would encounter about 495 feet of Pictured Cliffs Sandstone and about 109 feet of siltstone and shale beds of the Lewis Shale.

According to Reclamation (1992b), the Lewis Shale has the following properties:

| | |
|---|---|
| Compressive Strengths | 8,970-20,600 psi<br>10,150 psi median |
| RQD Value | 70-90 |

Likewise, the Pictured Cliffs Sandstone has the following properties:

| | |
|---|---|
| Compressive Strengths | |
| P2, P5, P7 and P9 Units | 9,560-22,200 psi<br>11,390 psi median |
| P1, P3, P4, P6 and PP Units | 2,544-9,330 psi<br>7,696 psi median |
| RQD Value | |
| P2, P5, P7 and P9 Units | 71-100<br>94 Average |
| P1, P3, P4, P6 and PP Units | 53-100<br>90 Average |

In general, the rock quality designation (RQD) suggests none, to occasional steel sets and occasional split sets with wire mesh and steel straps. Shotcrete would not be used extensively because of the wet conditions and bonding problems with the weaker sedimentary bedrock units. Tunneling advance could be 3 to 10 feet. Support would be commenced after each blast and using complete support 50 feet back from the face.

## 2.6 Basin Creek Improvements

Planned water supply releases from Ridges Basin Reservoir ranging from 25 to 130 cfs and future releases of up to 250 cfs are projected for non-binding Colorado Ute Tribe water use development. These releases would be controlled by valves at the outlet works of the dam and would flow down Basin Creek to the Animas River. The watercourse along the creek is about 3.2 miles in length with an elevation drop of about 420 feet from the dam to the river. The upper 2.5 miles of the creek is incised into a clayey sand formation, while the lower 0.7 miles passes over several natural rock controls. Planned releases are greater than the normal seasonal runoff and silt transport into the Animas River is a concern. A field survey of the creek channel indicated that a means of control would be necessary if the planned releases would not increase silt transport. Alternative means of control investigated included the following:

- Armoring the channel with rock;
- Replacing the streambed with a concrete lined channel;
- Installing a number of check or vortex weirs; or
- Releasing flows into a conduit laid along side the creek.

The steep slope of the streambed and the absence of a nearby source of heavy, durable rock makes possible use of channel armoring for erosion control less attractive than a concrete-lined channel. With the slope of 0.026, velocity in a concrete-lined channel would be high, and protective measures would be necessary for safety of wildlife and persons. Creating steps in the channel with a series of check and drop or vortex weirs would produce an increase in silt transport initially, but would stabilize with use. It would also create some wetlands. The steps would be placed about 150 feet apart. All three of these solutions involves realigning the streambed into gentle curves and grading it to create relatively flat slopes from the modified flow line to the original ground on either side. The lower 0.7 mile of the creek may accept the additional flow without significant modification.

The conduit alternative would require pipe of about 48 inches diameter. It would be laid in roadway and across private property, leaving the creek relatively undisturbed. Use of pressurized pipe would leave open the potential for energy recovery at the conduit discharge. The alternative of check or vortex weirs was used in the cost estimates of this analysis.

## 2.7 Cost Estimate

Feasibility level estimated construction costs are based on construction quantities measured on preliminary design drawings and on unit prices selected from similar work. Major equipment items were priced based on quotations provided by manufactures and included allowances for installation based on experience. Quantities for Ridges Basin Dam were measured with an earthwork model set up over the same dam axis and ground contours used by Reclamation for the large dam considered in the 1996 FSFES, and with the dimensions of the current feasibility design for the 120,000 af dam and reservoir. Unit prices based on previous estimates have been updated to April 1999 using the Reclamation Construction Cost Index weighted for earth dams, pumping plants, and steel pipelines.

In the estimated construction cost summary, a construction contingency amount of 20 percent, considered appropriate for preliminary design level, is listed separately. Reclamation estimates include this percentage within the itemized costs rather that listing it separately. The total field cost includes the 20 percent construction contingency. To obtain the total construction cost, 30 percent was added to the total field cost based on the following estimate of work remaining to be carried out:

| Item | Percent |
|---|---|
| Investigations | 4 |
| Designs and specifications | 8 |
| Construction inspection | 12 |
| Legal and Administrative | 2 |
| Environmental compliance | 4 |
| **Total** | **30** |

**Table 2-1** presents a summary of the Ridges Basin Dam and Reservoir construction cost estimate. For the total project cost, several items must be added to the estimated feature construction costs, including cultural resources, recreation facilities, fish and wildlife mitigation, wetlands mitigation, and interest during construction.

**Table 2-1**
**Ridges Basin Dam**
**Estimated Cost in Millions of Dollars (April 1999)**

| Item | Cost ($ million) |
|---|---|
| Land Acquisition | 6.9 |
| Relocation Gas Pipelines & Electric | 10.5 |
| Relocation County Road | 3.2 |
| Access Roads | 2.7 |
| Clearing & Stock Fencing | 0.8 |
| Basin Creek Improvement | 2.1 |
| Dam Structure | 57.6 |
| Outlet Works | 6.9 |
| Initial Filling | 2.0 |
| **Ridges Basin Dam and Reservoir** | **92.7** |
| Construction Contingency (20%) | 18.5 |
| **Total Field Cost** | **111.2** |
| Engineering Design, Inspection and Administrative, Legal (30%) | 33.4 |
| **Total Construction Cost** | **144.6** |
| **Total Ridges Basin Dam and Reservoir Construction Cost Rounded to Nearest Million Dollars** | **145.0** |

## 3.0 DURANGO PUMPING PLANT

### 3.1 Location and Access

The Durango Pumping Plant is the proposed intake pumping plant for the ALP Project. The proposed pumping plant site is two miles south of the center of Durango, on the west side of the Animas River across from the downstream end of Santa Rita Park, formerly Gateway Park. This is the same location proposed in the 1996 FSFES and is presented in Map 2-6 in Chapter 2 of the FSEIS. Access to the site is from CR 211, immediately north of Centennial Mall. A plant entrance road, with 30 feet of gravel surface and drainage ditches would branch from the existing roadway along the base of Smelter Mountain about 400 feet north of its intersection with CR 211, and proceed a distance of 1,400 feet at a maximum grade of less than 6 percent to the pumping plant.

### 3.2 Pumping Plant Structure

The pumping plant is designed as an indoor structure that houses 9 horizontal single stage centrifugal pumps of varying capacities. Oriented with the long side parallel with the river, the pump and equipment portion of the plant would be below the finished ground surface with an interior height of 43 feet, a width of 57 feet, and a length of 250 feet. Above the below surface portion of the plant, the crane housing would extend 24 feet above the ground to facilitate loading, unloading and maintenance of the pumping units and equipment. The crane housing would be 40 feet wide and 250 feet long. Construction would use cast-in-place and precast concrete. A spherical air chamber would be partially buried behind the plant away from the river. Transmission lines and an electrical switchyard would be located to the south, between the plant and CR 211. Fill slopes between the plant and the intake structure, and between the intake structure and the river, leave space for landscaping.

#### *3.2.1 Foundation Conditions*

Overburden at the pumping plant site is generally impervious clayey sand about 15 feet thick underlain by 8 feet of river terrace deposits. Bedrock at the site is siltstone, sandstone and shale of the Point Lookout Sandstone formation. It occurs at a depth of about 25 feet and would form the foundation for the plant. Groundwater at a fault about 150 feet southeast of the proposed plant may carry slightly elevated levels of contaminants, however, its location is well beyond the planned limits of construction excavation. Backfill around the structure would employ drains or pervious material placed to restore natural groundwater movement after construction.

#### *3.2.2 Construction Dewatering Monitoring*

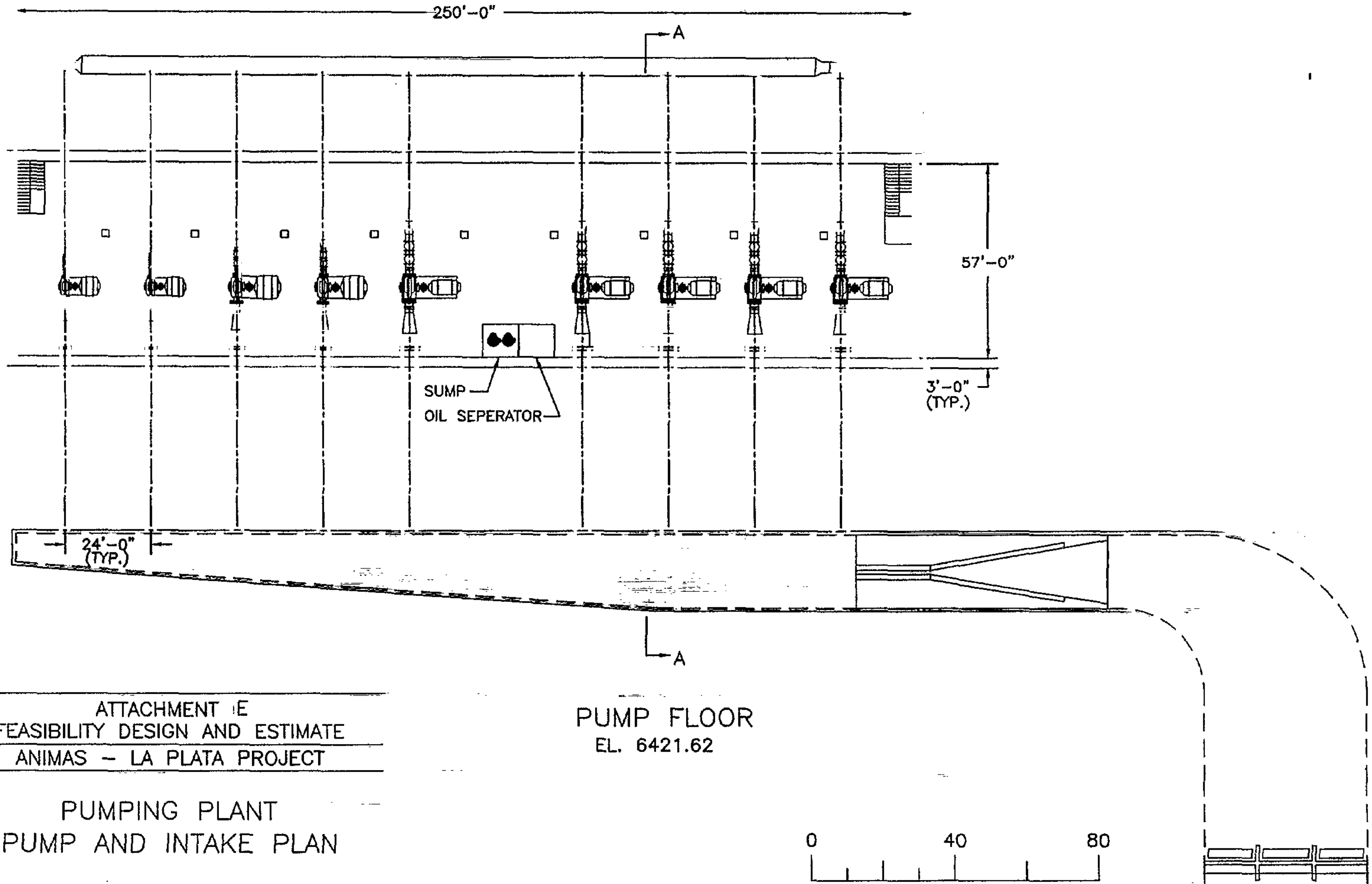
The Durango Pumping Plant site is a former UMTRA site and regular monitoring of the water removed during construction dewatering would be required. The contractor would be required to prepare and implement, if necessary, a contingency plan for treating the water removed during excavation in the event that groundwater contamination levels exceed anticipated limits.

### 3.3 Intake Structure

The intake structure feasibility design is based on reducing the flow path dimensions resolved by the 1996 model test (Reclamation 1997) in proportion to the reduction in design flow rate. Of the two channels in the 1996 design, only the longer radius channel is employed in the present design. It would consist of a grated structure which would extend 48 feet along the riverbank. The grated structures would pass flow through three control gates into a covered channel that extends 90 feet back from the river then turns to pass through a V-type fish screen after passing through the fish screen, the flow would enter a covered forebay chamber that would supply the pump intake pipes. The design approach velocity for the screen is less than 0.5 feet per second. The fish screen area would open for cleaning and maintenance access. A fish bypass conduit at the base of the screen, designed to carry 15 cfs back to the river, would extend downstream about 300 feet to develop 1 foot of differential head. A plan view of the intake structure and the pump floor is presented in **Figure 3-1**.

### 3.4 Pumping Unit Selection

The pumping plant capacity, number of pumps and individual pump capacities were selected to permit the integration of available flow over the duration shown on the probability curve derived from the hydrological model and presented in Exhibit 3-1. Maximum pumping capacity is linked to the reservoir active storage through the hydrological model. For an active storage quantity of 90,000 af, the maximum pumping rate was determined to be 280 cfs or 125,700 gpm. Static lift from the river level over Bodo




ATTACHMENT E
FEASIBILITY DESIGN AND ESTIMATE
ANIMAS – LA PLATA PROJECT

PUMPING PLANT
PUMP AND INTAKE PLAN

| | | |
|---|---|---|
| DRAWN: D.F.T. | CHECKED: V.P.N. | FIGURE |
| APPROVED: D.D. | DATE: 9/14/99 | 3 – 1 |

Draw is 511 feet and maximum dynamic losses amount to 40 feet for a maximum total dynamic head of 550 feet. The capacity selection of individual pumps was restricted to single stage, low speed pumps of proven design that were commercially available.

Applying the above criteria, and using data provided by manufacturers, it was determined that the maximum flow could be attained with five horizontal centrifugal pumps, each rated at approximately 25,000 gpm and 550 feet of total head. Commercially competitive pumps are available in this range. To accommodate lower flows, two intermediate size pumps (approximately one-half of the capacity of the large pumps) and two small pumps (approximately one-half of the capacity of the intermediate pumps) were also selected. These pumps would provide standby capacity should one of the larger pumps be unavailable for service.

### *3.4.1 Pump Configuration*

Large, single stage horizontal pumps are similar in silt handling capability to the vertical spiral case pumps proposed for the higher capacity pumping plant of the 1996 ALP Project design and the horizontal configuration is more accessible for maintenance. The horizontal pumps would also require less structure height than the former vertical pumps. The forebay and the pump intake piping arrangement would provide positive suction head at the pumps. These features are presented in **Figure 3-2**. Operating characteristics of the selected pumps are presented in **Table 3-1**. For a pumping plant with multiple pumps, the superimposition of the pump operating curve and the piping system elevation and friction curve provides a system head curve. This illustrates the flow produced by a single pump or any grouping of pumps. Exhibit 3-2 presents the system head curves for the large pumps.

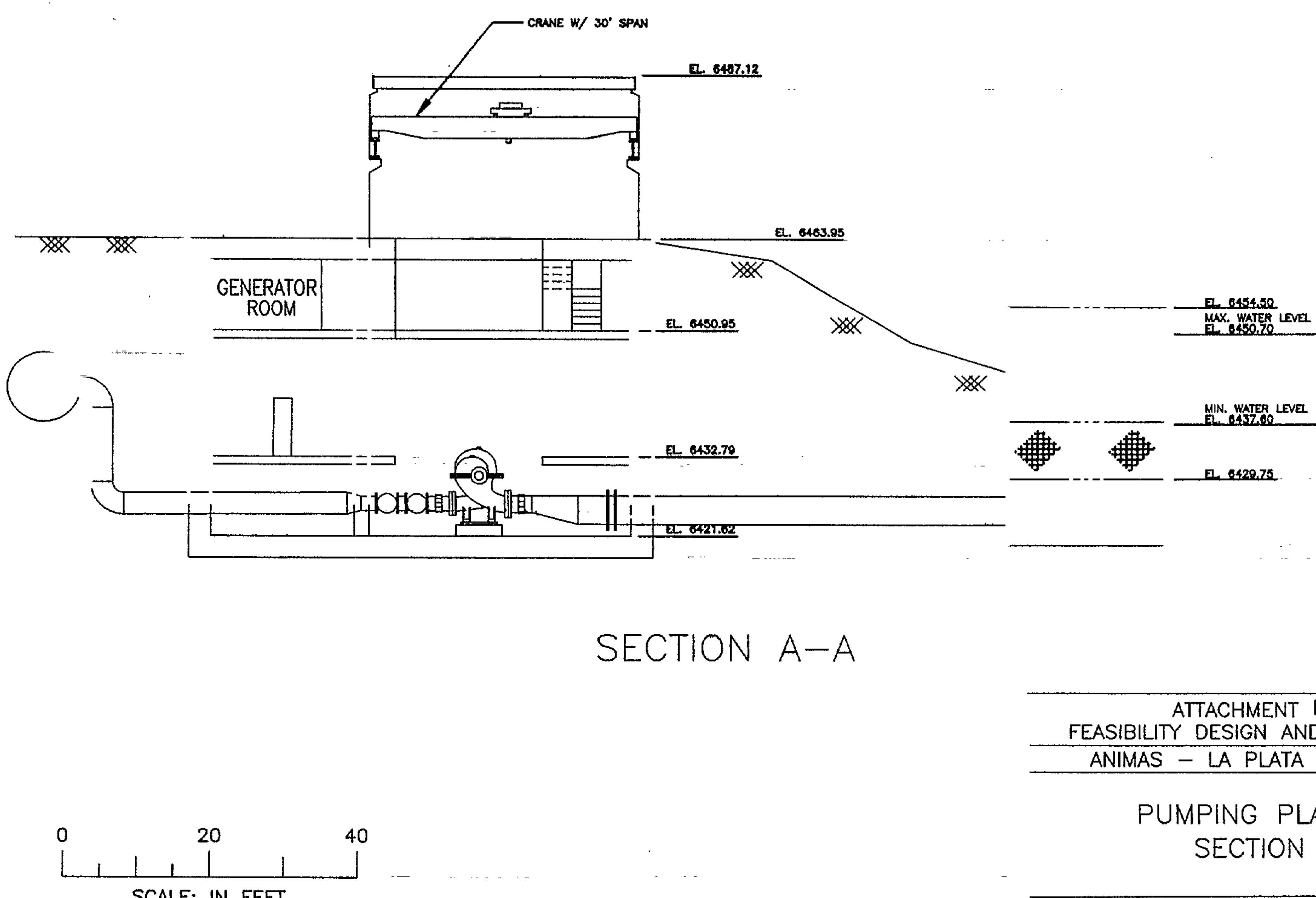
**Table 3-1**
**Pump Operating Characteristics**

| Characteristic | Unit | Pump Size | | |
|---|---|---|---|---|
| | | **Large** | **Intermediate** | **Small** |
| Flow | cfs | 56 | 28 | 14 |
| Flow | gpm | 25,000 | 12,500 | 6,250 |
| Total Dynamic Head | feet | 550 | 550 | 511 |
| One Pump Run-out Flow | gpm | 32,000 | 15,000 | 7,000 |
| One Pump Run-out Head | feet | 500 | 500 | 500 |
| Driver Speed | rpm | 900 | 1200 | 1800 |
| Driver Power | HP | 5000 | 2500 | 1,250 |

### *3.4.2 Materials and Manufacture*

The following materials and manufacture standards would be specified for service conditions at the intake pumping plant:

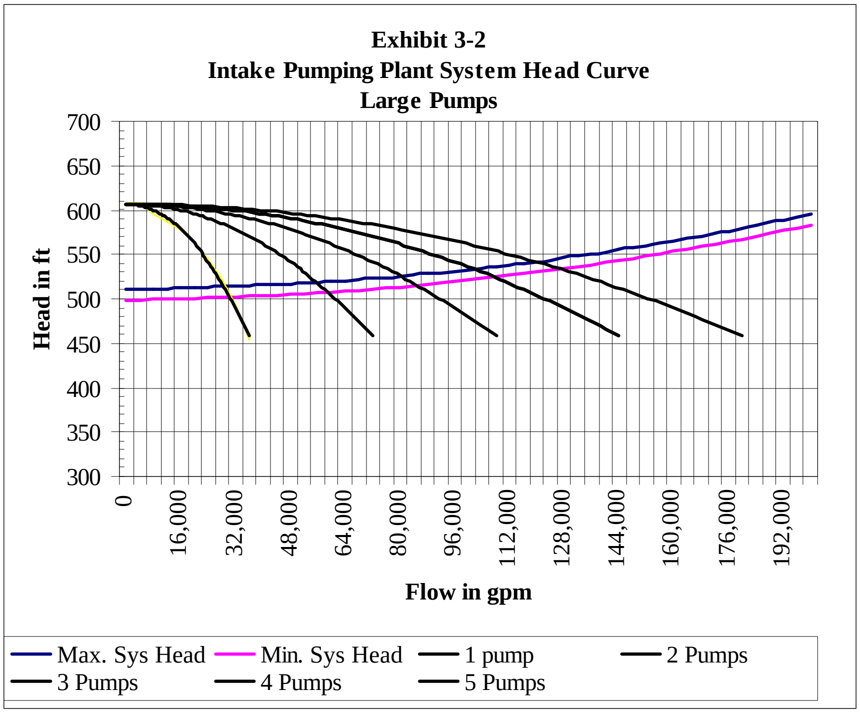
❑ Casing: Axially split casing with suction and discharge flanges in the lower half. This allows access to the rotating element without disturbing piping or motor drive and also provides rigidity for pipe loads, reducing coupling and bearing misalignment. Upgrading the casing to steel, rather than cast iron, will allow welding should wear result from silt in the water. Upgrade of the casing wearing ring to 13 percent chrome stainless steel for wear resistance to silt.




SECTION A–A

| ATTACHMENT E<br>FEASIBILITY DESIGN AND ESTIMATE | | |
|---|---|---|
| ANIMAS – LA PLATA PROJECT | | |
| PUMPING PLANT<br>SECTION | | |
| DRAWN: D.F.T.<br>APPROVED: D.D. | CHECKED: V.P.N.<br>DATE: 9/14/99 | FIGURE<br>3 – 2 |

❑ Impeller: Double suction type for hydraulic balance. Upgrade from bronze to 13 percent chrome stainless steel for wear resistance to silt. Provide optional hardened wear rings for better service.




❑ Shaft and Sleeves: Heat-treated carbon steel and protected from wear and erosion by removable sleeves.

❑ Seals: Gland packing type, PTFE impregnated. Seal water, if required would be provided from the pump discharge by way of a filtration system.

❑ Bearings: Heavy duty, anti-friction type pump bearings arranged for oil lubrication without external cooling.

## 3.5 Major Valves

Major flow control and isolating valves would include the following:

❑ Pump Discharge Control Valves: Metal to metal seated American Water Works Association (AWWA) Standard Specification C507 Class 250 ball valve. Operated by an air/oil cylinder operator capable of operating the valve with a minimum of 100 psig air pressure to the cylinder. Controls provide for independent opening and closing speed controls as well as an emergency high speed close feature with a separate speed control valve.

❑ Pump Discharge Isolating Valves: Metal to metal seated AWWA C507 Class 250 ball valve.

❑ Pump Suction Isolation Valves: Rubber seated AWWA Class 25 butterfly valves.

### *3.6 Plant Auxiliary Systems*

The following plant auxiliary systems would be provided to support and operate the pumping units and valves and serve the plant operating and maintenance personnel.

#### *3.6.1 Compressed Air System*

Compressed air at a nominal 125 psig pressure would supply the air pneumatic valve operators of the pump discharge control valves, the sewage ejector and utility hose connections throughout the plant for operation of pneumatic tools. The system would include:

- ❑ Two station service air compressors (one duty and one stand-by): Two stage, multi-cylinder, automatic unloading, air-cooled, horizontal type, belt driven by an electric motor suitable for operation on 460-V, 3-phase, 60 Hz. Operation normally automatic, controlled by pressure switches mounted on the main air receiver with manual operation available at the motor control center.

- ❑ Air receivers: Vertical type with floor supporting stand, inlet and outlet connections, pressure relief valve, pressure gage, automatic condensate trap, drain valve and manhole. Designed for 150 psi working pressure in accordance with Section VIII of the ASME Boiler and Pressure Vessel Code. An air dryer with bypass would be provided on the discharge piping of the main air receiver.

#### *3.6.2 Oil Recovery System*

An oil recovery system would guard against pollution from accidental oil spills in the plant. Designed to meet a 10 parts per million (ppm) maximum oil-in-water effluent criterion, it would provide a margin of safety over the EPA requirement of 15 ppm stipulated in paragraph 423.12 of the "Environmental Protection Agency Effluent Guidelines and Standards for Steam Electric Power Generating." Oil separators would be installed to recover any oil spilled and collected by the plant drainage system with capacity to contain the largest expected single spill that could occur in the at the hydraulic power units or in the oil storage area. A recovered oil storage tank would store oil for removal for disposal.

#### *3.6.3 Plant Drainage System*

The plant drainage system would be designed to collect water from areas where wash down and seepage may collect, from watercooled equipment, and from pump seals and convey it to a sump where it would pass through the oil separator. Any separated oil would go to the recovered oil storage tank and the water would be pumped to discharge into the intake bay.

#### *3.6.4 Service and Domestic Water Systems*

The service water system would distribute water to hose connections throughout the plant, supply cooling water for air compressors and aftercoolers and for air conditioning equipment. The domestic water system would supply the restroom facilities and drinking fountains. Both systems would be supplied by a connection with the City of Durango treated water main along the west side of Highway 160/550, about 300 feet east of the pumping plant.

### *3.6.5 Sanitary Waste*

Sanitary waste plumbing from the restroom facilities and drinking fountains would drain to the plant sewage ejectors. The sewage ejectors would pump to a connection with the City of Durango sewer system on the west side of Highway 160/550, about 300 feet east of the pumping plant.

### *3.6.6 Fire Protection System*

Fire protection for the pumping plant includes:

- Fire suppression sprinkler system in conjunction with portable fire extinguishing and fire hose reel.

- Fire hose stations inside the plant.

- An extension from the City of Durango water main along the west side of Highway 160/550, to provide outside fire hydrants near each end of the plant and near the switchyard.

- Portable 20-pound wall-mounted fire extinguishers, and 100-pound wheeled units.

In addition, heat detectors would be placed in areas of the pumping station protected by manual extinguishing equipment and ionization detectors would be placed to serve as an early warning system. Fire detection and early warning alarms would be displayed on an enunciator system panel in the control room.

### *3.6.7 Stand-by Power*

In the event of normal service power loss, diesel generating sets would provide emergency power for essential emergency loads including control room, sump pumps, elevator, limited area lighting, and the unit auxiliaries necessary for safe shutdown of the pumping units.

### *3.6.8 Discharge Pipeline Dewater*

Provisions will be included in the design to permit dewatering of the pump discharge, or reservoir inlet conduit. Piping, sleeve valve and isolation valves would bypass the pumps and lead back to the intake works and the river.

## 3.7 Servicing Equipment

### *3.7.1 Crane*

The crane provided over the plant would be a single trolley 20-ton bridge crane designed to comply with OSHA standards and adequate to handle the weight of the heaviest load. The crane would be used for handling materials and equipment, including loading, assembly, installation, and future maintenance operations.

### *3.7.2 Elevator*

A hydraulic elevator would be provided to transport personnel between floors.

## 3.8 Heating, Ventilation and Air Conditioning

The pumping plant would be equipped with heating, ventilating and air conditioning systems to meet the various temperature and ventilating requirements. Continuous air conditioning would be provided for the control room only, while all other rooms and spaces would be ventilated by means of forced air systems. The criteria for design would be to use the largest volume computed as (1) the amount of air required to maintain the design temperatures considering heat gains from solar sources, equipment, lights, and personnel, and (2) the number of air changes per hour.

### *3.8.1 Control Room*

The control room air conditioning system would maintain a slight positive pressure by adding make-up air equal to the exfiltration losses plus constant exhaust air quantities. In the summer, the make-up would be mixed with return air, filtered, cooled by the cooling coils, and distributed to the conditioned rooms through ducts. A thermostat in the return air duct or in the room would start and stop the refrigeration compressors to maintain the desired average room temperature. A standby unit would be provided for emergency cooling during maintenance or breakdown of the main unit.

In the winter, the system fan would circulate air to each zone and uncontaminated air would be recirculated. Electric duct heaters would provide for heating and for tempering the make-up air.

### *3.8.2 Exhaust Systems*

Exhaust fans would pull air from the plant through the ventilated spaces in the sewage pump area, oil recovery, oil and storage room and diesel generator room and exhaust it directly outside. The oil storage room exhaust duct would be equipped with an automatic fire damper.

### *3.8.3 Pump and Switchgear Gallery*

Heating and ventilating systems for the pumping and switchgear gallery would consist of two air handling units and dampers, electric heater, ducts, registers, and control equipment. Each air handling unit would be packaged, including air filters, air plenum and centrifugal type cabinet fan. The ventilation system would include power roof ventilators to exhaust air to the atmosphere.

The air handling unit supply fans would run continuously all year and have manual start/stop controls. The power roof ventilators would cycle on and off as a function of pump room temperature. During summer operation, the system would take in 100 percent outdoor air, supply it to the rooms and exhaust outside. During winter operation, all uncontaminated air would be recirculated and the ventilators would not operate. Heating would be primarily by recirculation of equipment heat within the plant.

## 3.9 Electrical Features

### *3.9.1 Pump Motors*

Motors for driving the centrifugal type pumps would be horizontal-shaft induction type, five-5,000 horsepower (hp) at 13,200 volts, and two-2,500 hp and two-1,250 hp at 4,000 volts. The enclosure would be specified as Weather Protected Type II with ventilating passages at both intake and discharge to permit passage of external cooling air over and around the windings of the motor. Motor enclosures would also be constructed to limit the motor noise level to 85 decibels or lower. Control and protective devices would be supplied for alarm and shutdown of the motors for problem conditions.

### *3.9.2 Switchyard*

The switchyard to bring power into the pumping plant would be designed and built by WAPA. A single transmission line and 13.8-kV circuit would feed the plant load. Purchasing and storing a spare transformer would provide additional reliability.

### *3.9.3 Bus and Switchgear*

Cables rated at 15-kV would carry power from the utility structure located outside the plant to the 15-kV switchgear located inside the plant. A 15,000-volt bus would run inside the plant for the 13,200-volt motor starters and a 5,000-volt bus for the 4,000-volt motors starters. The 5-kV switchgear, 13,800 to 4,160-volt transformer, 4,160 to 480-volt plant service transformer and 480 volt distribution equipment would all be located inside the plant building.

### *3.9.4 Motor Starter and Control Equipment*

The plant design connected load would consist of five-5,000 hp motors, two-3,000 hp motors, two-1,315 hp motors, and 500 kVA of miscellaneous load. The plant full load current would be 3560 amps at 4,160 volts. A search of manufacturers indicated that the largest vacuum circuit breaker available for 5-kV metal clad switchgear is 3,000 amps. Since that is not sufficient for the connected load of 3560 amps, the larger 15-kV metal-clad switchgear was selected. A 5,000 hp motor is available for 4,160 volts, however, the 13,200-volt motor was selected to suit the switchgear.

Motor control equipment would contain draw out fuses, starters, control power transformers, selector switches, pushbuttons, and all unit protective and control devices. A programmable motor protective relay would be used to centralize all unit control and protective features.

*5-kV Motor Starting and Control Equipment*

Cables from the 15-kV metal clad switchgear circuit breaker would provide power to a 7,000 kVA, 13,800 to 4,160 volts, 60 Hz transformer in the 5-kV switchgear assembly. The secondary side of the 7,000 kVA transformer would be connected to the 5-kV switchgear main circuit breaker using 5-kV cables. A 5-kV auto-transformer with motor control equipment would serve the starting and control functions of the 3,000 hp and 1,315 hp motors.

*15-kV Motor Starting Equipment*

A-15 kV metal-clad circuit breaker along with a 15-kV auto-transformer would function as the starter for the large 5,000 hp units. Reduced voltage starting would be used to limit the power system voltage drop.

### *3.9.5 Main Plant Control*

Main plant control cubicles and console would serve to centralize all plant operations including indicating, recording, operating, communication, and protective functions. Manual, automatic and supervisory type functions would be provided to allow full flexibility in plant operations. A computer programmable logic controller housed in this control board would provide automatic features such as automatic restart after power failure and selection of different size units to optimize plant performance based on available river flow. Control console personal computers would be furnished with the necessary hardware and software to field program each ladder logic diagram.

### *3.9.6 125 Volt DC Battery and Battery Charger*

A 125-volt DC battery and battery charger would be provided to supply control power for 15-kV and 5-kV metal clad switchgear and main plant control cubicles.

### *3.9.7 Plant Service*

Plant service power supply would be obtained by connecting to the 5-kV switchgear fused disconnect switch and providing a 480-volt unit substation.

### *3.9.8 Auxiliary Control Boards and 480-Volt Motor Control Centers*

Auxiliary control boards and 480-volt motor control centers would be provided in the plant for operating the auxiliary systems such as hydraulic pumps, water cooling pumps, electrically driven valves, and air compressors.  Auxiliary control boards and 480-volt motor control centers would obtain power from a unit substation.

## 3.10 Cost Estimate

Feasibility level estimated construction costs are based on construction quantities measured on preliminary design drawings and on unit prices selected from similar work.  Major equipment items were priced based on manufacture quotations with allowances for installation.  Pumping plant quantities were determined from current feasibility design drawings. Unit prices based on earlier years have been updated to April 1999 using the Reclamation Construction Cost Index weighted for earth dams, pumping plants, and steel pipelines.

**Table 3-2**
**Durango Pumping Plant**
**Estimated Cost in Millions of Dollars (April 1999)**

| Item | Cost ($ million) |
|---|---|
| Land Acquisition | 1.1 |
| Access and Site Improvements | 0.46 |
| Intake Works | 2.4 |
| Plant Structure | 5.6 |
| Pumps and Motors | 5.7 |
| Manifold Piping | 1.7 |
| Auxiliary Equipment | 1.4 |
| Electrical | 40.0 |
| Surge Control | 9.0 |
| **Durango Pumping Plant** | **23.26** |
| Construction Contingency (20%) | 4.65 |
| **Total Field Cost** | **27.91** |
| Engineering Design, Inspection and Administrative, Legal (30%) | 8.37 |
| **Total Construction Cost** | **36.28** |
| **Total Durango Pumping Plant Construction Cost Rounded to Nearest Million Dollars** | **36.3** |

## 4.0 RIDGES BASIN INLET CONDUIT %

### 4.1 Route and Features

The conduit route from the Animas River to Ridges Basin was selected to provide the lowest pumping lift with reasonable construction access and to minimize alteration of natural terrain contours. Inspection of the reservoir site topography indicates two lower level inlet routes with reasonable proximity to the river. One route would enter the reservoir from Basin Creek and the other would enter the reservoir from Bodo draw. For the design reservoir size of 120,000 af, the lift from the river to the elevation of the ridge at the top of Bodo draw is less than the lift from the river to the reservoir at the downstream locations accessible to Basin Creek. Placing the conduit at a lower elevation by excavating the crest of the ridge further reduces the pumping lift.

The conduit would be buried in a trench and backfilled so that upon completion of construction the terrain would be returned to natural contours. The route of the conduit from the pumping plant to the reservoir is along the trace defined during redesign by Reclamation in 1995. It proceeds southerly from the pumping plant, turns southwest to cross CR 211 and the Bodo Creek line, passes south of Hill 6966 approximately 1,200 feet south of CR 211 then turns up Bodo draw, and finally, approaches CR 211 and crosses the ridge along side CR 211. An air vent of about 12 inches in diameter would stand about eight feet above ground just before the crest of the ridge. The conduit would terminate on the reservoir side of the ridge with a stilling structure from which the flow would continue down to the reservoir in a rock-lined ditch.

### 4.2 Conduit Profile

#### *4.2.1 Concept of 1996 FSFES with Tunnel*

The conduit described by Reclamation in The Definite Design Report, 1980, included a tunnel across Bodo Ridge at elevation 6,898 feet, some 80 feet below the crest of the ridge at 6,878 feet. The tunnel entered the proposed 273,000 af reservoir at the level of the bottom of the 111,000 af active storage pool. The conduit at 108 inches diameter was designed for a flow of 480 cfs. Stored water could flow back down the inlet conduit, although the design criteria only required a small flow for the Animas-La Plata Water Conservancy District and the City of Durango.

#### *4.2.2 Crest Elevation and Power Consumption*

To determine the optimum elevation across Bodo Ridge, the cost of construction of a tunnel at the elevation proposed in the 1996 FSFES, and the cost of construction in open-cut at various depths of excavation were compared with future power consumption. Normal trenching was assumed to reach a maximum depth of 16 feet. Open-cut construction in excess of normal trench depth was calculated based on removing earth to provide 20 feet of pipe handling space on one side of the trench area and 30 feet of earth handling space on the other. The pipe trench area would include 1/2:1 construction slopes and the handling area, 1:1 slopes. Earthwork in excess of normal trenching included open-cut excavation, movement to stockpile, retrieval from stockpile, and compacting into place. Current earthwork prices were applied.

Power consumption was based on the present worth of a projected pattern of future use. The projected pattern of future use consists of initial reservoir filling over a two-year period followed by a period of uniformly increasing use from 1/20th of full use the first year, to full use at the end of 20 years and then full use thereafter. For comparing costs, the initial filling was assumed to start the second year after the midpoint of construction expenditures. With a discount rate of 6.5 percent and a power cost inflation rate of 2.5 percent, additional excavation to a trench bottom elevation of 6,945 feet resulted in the lowest cost. **Table 4-1** presents costs for the complete inlet conduit including the extra cost involved with crossing Bodo Ridge at the elevations investigated.

**Table 4-1**
**Conduit Construction Cost and Present Value of Power Cost**
**for Various Excavation Elevations Across Bodo Ridge**
**Discount Rate 6.5 Percent, Cost Escalation Rate 2.5 Percent**

| Excavation Elevation (feet) | Construction Cost ($ million) | Present Value Power ($ million) | Total Present Value ($ million) |
|---|---|---|---|
| 6,962 | 7.2 | 19.3 | 26.5 |
| 6,950 | 7.35 | 18.89 | 26.24 |
| 6,945 | 7.5 | 18.72 | 26.22 |
| 6,940 | 7.7 | 18.55 | 26.25 |
| 6,930 | 8.13 | 18.2 | 26.33 |
| 6,890 (Tunnel) | 10.42 | 16.83 | 27.25 |

### *4.2.3 Conduit Diameter and Material*

The design maximum flow rate for the inlet conduit is 280 cfs. The pipe would contain a pressure, or head of water, ranging from 0 to about 540 feet plus a surge pressure of about 15 percent of the head. This is in the economic pressure and diameter range of steel pipe as compared to an alternative reinforced concrete pipe. **Table 4-2** presents the cost and power value investigation of alternative diameters. Surge chamber cost is included. As the pipe diameter decreases, the velocity increases and the size of the chamber required for controlling surge increases. Comparison was made at the 280 cfs flow rate because most of the reservoir refill is accomplished in the spring and early summer at the peak rate and the demand portion of the power cost is determined by this rate. A diameter of 66 inches was selected. The pipe would be coated inside and outside to protect against corrosion and equipped to accept cathodic protection if post construction measurements indicate it advisable. The approximate length of the inlet conduit would be 11,200 feet.

**Table 4-2**
**Conduit Diameter, Construction Cost and Present Value of Power Cost**
**Discount Rate 6.5 Percent, Cost Escalation Rate 2.5 Percent**

| Diameter (inches) | Flow Rate (cfs) | Construction Cost Conduit/Surge | Present Value Power ($ millions) | Total Present Value ($ millions) |
|---|---|---|---|---|
| 60 | 280 | 6.6/1.61 | 19.58 | 27.29 |
| 66 | 280 | 7.5/1.34 | 18.72 | 27.56 |
| 72 | 280 | 8.4/1.13 | 17.27 | 27.8 |
| 78 | 280 | 9.525/0.97 | 17.965 | 28.46 |

## 4.3 Surge Provisions

Pressure variations caused by increases and decreases of flow during normal starting and stopping of pumps can be kept small by controlling the rate of movement of pump discharge valves. Pressure down-surge due to power failure at the pumps, however, is a condition that requires analysis and a reliable control mechanism. Conventional means of controlling surge include air chambers, constant head tanks, and air inlet-controlled release valves.

An estimate was made of down surge following power failure to the pumps, with no surge protection on the pipeline. The result was negative or vacuum pressure extending the full length of the pipeline. This indicated the need for a large volume of air and an air chamber was selected as a reliable means of surge control. A spherical air chamber was selected for its economy of shape and with the consideration that it may also present a more pleasing appearance than an elongated tank. Investigation using Parmakian charts (which assume a frictionless system), and with no reverse flow permitted through the pumps, indicated a diameter of 38 feet. This diameter was selected because the air volume would limit upsurge to an acceptable 15 percent and down-surge would not create negative pressures.

## 4.4 Cost Estimate

Estimated construction costs are based on construction quantities measured on preliminary design drawings and on unit prices selected from similar work. Major equipment items were priced based on manufacture quotations with allowances for installation. Unit prices, based on previous years, were updated to April 1999 using the Reclamation Construction Cost Index weighted for earth dams, pumping plants and steel pipelines.

**Table 4.3**
**Ridges Basin Dam Inlet Conduit**
**Estimated Cost in Millions of Dollars (April 99)**

| Item | Cost ($ million) |
|---|---|
| Land Acquisition | 0.3 |
| Pipeline | 4.8 |
| Discharge Channel | 0.5 |
| **Inlet Conduit** | **5.6** |
| Construction Contingency (20%) | 1.1 |
| **Total Field Cost** | **6.7** |
| Engineering Design, Inspection and Administrative, Legal (30%) | 2 |
| **Total Ridges Basin Inlet Conduit Construction Cost** | **8.7** |

# 5.0 NAVAJO NATION MUNICIPAL PIPELINE

## 5.1 Plan

The NNMP would deliver the Navajo Nation water supply entitlement of 4,680 afy from the ALP Project. Starting at the west boundary of the City of Farmington, it would extend 28.9 miles to Shiprock. The pipeline would replace an existing 30-year old pipeline and provide increased capacity to deliver water to the Navajo Nation Chapters of Upper Fruitland, San Juan, Nenahnezad, Hogback and to

Shiprock where existing water mains extend to Cudei, and Beclaibito. The pipeline would be operated and maintained by the NTUA, the operating agency for the existing water facilities.

## 5.2 Current and Projected Use

The City of Farmington has supplied treated water to NTUA since completion of the existing Farmington to Shiprock pipeline in 1969. The original 30-year water contract was recently renewed for five years with an optional additional five years. It provides for a maximum supply of 3.0 MGD. Metered flow during the peak month of July 1998 averaged 1.7 MGD. Water supplied to NTUA in the existing pipeline amounted to 1,168 afy in 1977. The proposed pipeline would receive up to 4,680 afy of ALP Project water also handled through the city system and treatment plant. The pipeline would be designed for a peak flow of 8.1 MGD (equal to 12.6 cfs), which is two times the average annual flow.

Need for water has been projected based on increased population connected to the system and on per person water use. The need for water is estimated to be 2,400 to 2,600 afy in 2013 (Molzen-Corbin 1993) and 8,245 afy for the above Chapters in 2040 (Navajo Nation 1998).

## 5.3 Project Elements

### *5.3.1 Pipeline*

Topography and principle water demands divide the proposed pipeline into four reaches. The first reach, of about 13.4 miles, would extend from Farmington to a high point north of Morgan Lake. It would connect with the Farmington system, cross the San Juan River and then be placed along the same right-of-way as the existing pipeline, to the south of the river. Major turnouts would supply the Upper Fruitland and San Juan Chapters.

Preliminary design for this reach involves the ground elevation and supply pressure at the connection with Farmington and the ground elevation and desired system pressure at the high point in the Nenahnezad-Morgan Lake area. The existing contract with Farmington calls for a minimum pressure of 60 psi, which, added to the ground elevation of 5,220 feet, provides a hydraulic elevation, or grade, of 5,355 feet. The ground elevation at the high area is 5,360 feet. To provide pressure to the connected distribution piping and overcome pipe friction requires a higher hydraulic grade. When the pressure at the connection point approaches the contract minimum, pumping is required.

Note that the existing pipeline only functions without a pumping because the pressure at the Farmington connection is normally higher than 60 psi. Pressures recorded at the connection in March and May, 1999 and ranged from 79 to 96 psi (Dykstra 1999). As use within Farmington grows pressure at the connection point will decline. A pipe diameter of 24 inches in this reach would supply turnouts along the route with adequate pressure and allow the pumping plant to be located on higher ground between Farmington and Nenahnezad, about 1.8 miles west of Ojo Amarillo Canyon.

The second reach of 4.3 miles would extend from Round Knob, north of Morgan Lake to the eastern boundary of the Hogback Chapter. At the end of this reach, the pipeline would cross from the south side to the north side of the San Juan River. The diameter would be 20 inches.

The third reach of 5.0 miles would be routed on a new right-of-way north of the Hogback Canal, on the south side of Highway 550 until it rejoins the existing route east of Shiprock. The existing pipeline would remain in place to serve homes south of Hogback canal. The diameter would be 20 inches.

The fourth reach of 6.2 miles would cross Highway 550 and extend to Cortez Tank at Shiprock at a top elevation of 5,170 feet. Turnouts would serve the area around Shiprock including the Cudei and Beclaibito Chapters. The storage tanks would provide local peaking flows and the pipeline diameter would be 16 inches.

The pipe diameters and the design pressure requirement are within the price competitive range of ductile iron; steel, cement mortar lined; steel cylinder, rod wrapped, cement mortar lined; and plastic pipe. The steel pipes would be coated with cement mortar or epoxy, or both. Steel and ductile iron materials would require cathodic protection. All, if properly installed and maintained, would provide long-lasting service. Final design investigations, including soil conditions and maintenance practices, would indicate if specifications should be written to invite bids on more than one type of pipe.

### *5.3.2 Storage Tanks*

Storage tanks would be located at the high point and end of the pipeline to provide service when the pumps are not operating, to meet local peak flow needs, and to add reliability to the system for line interruptions, maintenance shut downs or fire supression. The total recommended storage capacity of 7.0 million gallons (Navajo Nation 1998) would provide approximately 1.7 days of reserve. Near Nenahnezad, the tank would be elevated approximately 90 feet and have a capacity of 1.5 million gallons. At Shiprock, on the hill near Cortez Tank, an additional 5.5 million gallons of ground based storage would be constructed.

### *5.3.3 Pumping Plant*

A new pumping plant would be constructed to provide sufficient pressure in the future when growth increases and water use in Farmington reduces available delivery pressure. It would be located on a hillside near Nenahnezad. The pumping plant would include variable speed pumps that would conserve energy when Farmington delivery pressure is adequate and full pumping is not required.

## 5.4 Cost Estimate

Estimated construction costs are based on construction quantities and prices cited in the technical memoranda of the Navajo Nation Department of Water Resources (Navajo Nation 1998) and Reclamation (Reclamation 1999) and on unit prices selected from similar work. Construction quantities along the 153,000 feet of pipeline would include 510,000 cy of trench excavation, pipe and appurtenances purchase and installation, 31,900 cy of pipe zone material backfill and compaction, and 412,000 cy of trench backfill. In addition there would be river and road crossings, and pumping plant and storage tank construction. Unit prices based on years were updated to April 1999 using the Reclamation Construction Cost Index.

In the estimated construction cost summary, a construction contingency amount of 20 percent considered appropriate for preliminary design level, was added to obtain the total field cost. To obtain the total construction cost, 30 percent was added to the total field cost to cover additional technical investigation, engineering design, construction inspection, environmental compliance, administrative and legal costs. **Table 5-1** presents the estimated construction summary.

**Table 5-1**
**Estimated Construction Cost Summary (April 1999)**
**Navajo Nation Municipal Pipeline**

| Item | Quantity | Unit Cost | Amount |
|---|---|---|---|
| Water line, 24 in | 71,100 ft | 92.00 | 6,541,200 |
| Water line, 20 in | 49,100 ft | 70.00 | 3,437,000 |
| Water line, 16 in | 32,800 ft | 52.00 | 1,705,600 |
| Valves and Appurtenances | Lump Sum | 100,000 | 100,000 |
| Outlets and Transfer Connections | 60 Each | 800 | 48,000 |
| Crossings, River | 2 Each | 450,000 | 900,000 |
| Crossings, Roads | Lump Sum | 120,000 | 120,000 |
| Cathodic Protection | Lump Sum | 45,000 | 45,000 |
| Pumping Plant | Lump Sum | 440,000 | 440,000 |
| Surge Control | Lump Sum | 130,000 | 130,000 |
| Storage, Ground Tank | 4.0 Mgal | 0.24 | 960,000 |
| Storage, Elevated Tanks | 1.5 Mgal | 0.66 | 990,000 |
| **Subtotal Field Costs, April '99** | | | $15,416,800 |
| Construction Contingency (20%) | | | 3,083,200 |
| **Total Field Cost** | | | $18,500,000 |
| Engineering Design, Inspection and Administrative, Legal (30%) | | | 5,500,000 |
| **Total Construction Cost** | | | $24,000,000 |

# 6.0 WATER ACQUISITION FUND

## 6.1 Plan

The non-structural component of Refined Alternative 4 would consist of the creation of a water acquisition fund (a $40 million fund) that could be used by the Colorado Ute Tribes to acquire water rights on a willing buyer/willing seller basis in an amount sufficient to allow the Tribes approximately 13,000 afy of depletion in addition to the depletion from the structural portion of the project. However, to provide flexibility in the use of the fund, authorization would allow some or all of the funds to be redirected for on-farm development, water delivery infrastructure, and other economic development activities. (See Section 2.3.2.1.2, non-structural components discussion of Alternative 1).

## 6.2 Land Locations

It is estimated that purchase of 10,300 acres of irrigated land, distributed in four river basins, would be necessary to obtain the 13,000 afy of depletion described above. The acreage is distributed among the four basins as follows:

- ❑ Pine River Basin - Purchase 2,300 acres of land and leave water on the land
- ❑ La Plata River Basin- Purchase 2,400 acres of land and leave water on the land
- ❑ Animas/Florida River Basins - Purchase 2,300 acres of land and leave all the water on the land
- ❑ Mancos River Basin - Purchase 3,300 acres of land leave water on the land

The cost of acquiring the water rights would include the purchase price of the land and the cost of transferring water rights.

### 6.3 Cost Estimate

The water acquisition fund cost was based on the acquisition of water rights on a willing buyer/willing seller basis in an amount sufficient to allow the Colorado Ute Indian Tribes approximately 13,000 afy of depletion in addition to the depletion from the structural portion of the project. It is estimated that purchase of 10,300 acres of irrigated land, distributed in four river basins, would be necessary to obtain the 13,000 afy of depletion. The land cost was determined through the process described in Attachment D and results in the estimated $40 million capital cost. This cost includes the purchase price of the land, the cost of transferring water rights, and the cost of measures to avoid or mitigate impacts to wetlands and cultural resources.

## 7.0 CULTURAL RESOURCE MITIGATION

### 7.1 Plan

Mitigation measures are discussed in Section 3.9.4. Mitigation measures would include a program to compensate for losses of archaeological sites that would occur as a result of construction and operation. This program would be undertaken in coordination with the Colorado and New Mexico State Historic Preservation Officers and the Advisory Council on Historic Preservation. The proposed program would consist of data recovery, analysis, technical publication, and providing for storage and curation facilities for permanent maintenance of the artifact collection and other related information. A cultural preservation plan would help identify specific actions to preserve the cultural resource values. In addition to the scientific value, this would produce information of considerable public interest.

To address adverse impacts to exposed human remains at sacred sites, a Native American Graves Protection and Repatriation Act (NAGPRA) has been prepared and is included as Attachment H. The plan describes the procedures that would be followed in the event that human remains or cultural items are encountered during the course of project activities.

### 7.2 Cost Estimate

Costs to conduct the cultural resources mitigation program for Refined Alternative 4 include survey, recovery, protection, preservation, and display components. Included in these costs are contingencies and non contract costs.

| | |
|---|---|
| Ridges Basin Archeological District Increment | $ 7.5 million |
| Wetlands, Fish and Wildlife Increment | $ 0.6 million |
| NNMP Increment | $ 0.9 million |
| Total | $ 9.0 million |

The costs associated with recreation feature cultural resource mitigation are not included in the previous listing. Also not included is the estimated $2.9 million included within the water acquisition fund.

## 8.0 WETLANDS AND FISH AND WILDLIFE MITIGATION

### 8.1 Plan

Mitigation measures are discussed in the various resource sections of Chapter 3 and in the Indian Trust Asset and Environmental Justice section of Chapter 4 of the FSEIS. Mitigation measures are proposed for all adverse impacts, when possible, to reduce or avoid the impacts identified. Chapter 5 discusses Reclamation's and Interior's commitments to implement these mitigation and other impact avoidance measures.

Where appropriate, mitigation implementation plans would be developed to carry out selected mitigation measures identified in Chapter 5. Mitigation implementation plans would include: measurable goals and objectives; criteria for training of staff; mechanisms for field monitoring and oversight; definition of management authority to correct errors and "stop work" procedures; reporting; and fiscal and administrative accountability. These plans would include a timeline for performance, and would develop alternative approaches to implementation as appropriate. Mitigation implementation plans would be developed, reviewed and approved by Reclamation and other appropriate federal and/or state agencies prior to implementation. Accountability of implementation would be made through frequent reporting and consultation with these agencies for the life of the mitigation implementation program.

A wildlife and wetland mitigation plan is included in the project to replace losses. It includes land acquisition and development and land management plans around Ridges Basin Reservoir. Fishery plans include bypass flows, stocking, and other measures. These are discussed in more detail in Chapters 3 and 5.

### 8.2 Cost Estimate

Estimated costs for this feature are $12.8 million which includes a $2.1 million recreation increment (fish hatchery component and fisherman access). Mitigation plans include land acquisition and development. Included in these costs are contingencies and non-contract costs.

## 9.0 COSTS INCURRED THROUGH 1998

Since the ALP Project was authorized, approximately $68.0 million of costs associated with previous investigations, planning, design, and environmental compliance activities were expended through Fiscal Year 1998. Project expenditures incurred in Fiscal Years 1999 and 2000 are included in the non-contract (30 percent other costs) that are a part of the cost estimate for each feature of Refined Alternative 4.

## 10.0 COST SUMMARY

Table 10-1 defines the total cost for Refined Alternative 4.

**Table 10-1**
**Total Costs - Refined Alternative 4**

| Item | Present Worth Cost |
|---|---|
| **Project Components** | |
| Ridges Basin Dam | $145.0 |
| Durango Pumping Plant | $ 36.3 |
| Ridges Basin Inlet Conduit | $ 8.7 |
| Water Acquisition Costs | $ 40.0 |
| Cultural Resources Mitigation | $ 9.0 |
| Wetlands and Fish and Wildlife Mitigation | $12.8 |
| **Total Project Components** | **$251.8** |
| **Other Components** | |
| Navajo Nation Municipal Pipeline | $ 24.0 |
| Other Project Costs Through FY 1998 | $ 68.0 |
| **Costs to Implement the Preferred Alternative** | **$275.8** |
| **Total for Other Components** | **$ 92.0** |
| **TOTAL COSTS FOR PROJECT** | **$343.8** |

## 11.0 ANNUAL OPERATION COST

### 11.1 Dam, Pumping Plant, and Inlet Conduit

Operating costs for the dam, pumping plant, and inlet conduit include operating and maintenance personnel, equipment operating and repair cost and electrical power for pumping. For future full project operation, pesonnel requirements were estimated to include a supervisor, records clerk, four pumping plant operators, and two maintenance workers for the pumping plant, dam, and reservoir. In initial years, fewer personnel would be employed. Computerized supervisory control may reduce the number of pumping plant operators.

Repairs and services include annual payments made to a fund for pumping and electrical equipment repair and replacement, and dam maintenance expenses that is beyond the capacity of the regular maintenance personnel. Operating costs for Ridges Basin are summarized in **Table 11-1.**

**Table 11-1**
**Summary of Annual Operating Costs for Future Full Project Operation**
**Ridges Basin Dam, Reservoir and Pumping Plant**

| | Quantity | Cost |
|---|---|---|
| **Pumping Power** | | |
| Summer Demand | 18,700 kW | $386,000 |
| Winter Demand | 11,700 kW | $241,500 |
| Energy Use | 67,100,000 kWh | $543,500 |
| **Annual Pumping Costs** | | **$1,171,000** |
| **Other Operating Costs** | | |
| Personnel | 8 persons | $320,000 |
| Maintenance Equipment Operation | | $30,000 |
| Repairs and Services | | $70,000 |
| **Subtotal** | | **$420,000** |
| **Total Annual Operating Cost** | | **$1,591,000** |
| **Power Cost $/af** | | **$13.51** |
| **Project Operating $/af** | | **$14.21** |

Notes:
Power cost for pumping hydrological model derived average of 84,090 afy to Ridges Basin plus 2,560 afy to Durango Terminal Reservoir. Rates based on Colorado River Storage Project: $3.44 per month per kW demand, 8.1 mils per kWh.
Project operating cost based on apportioning power cost, personnel, maintenance and repair cost to the project diversion of 111,965 afy.

## 11.2 Navajo Municipal Pipeline

Operating cost includes operating and maintenance personnel, equipment operating and repair cost, electrical power for pumping and contract services. Service of regular NTUA maintenance personnel was estimated to include part time of a foreman and records clerk, and full time of two maintenance workers. Repairs and services include annual payments made to a fund for pumping and electrical equipment repair, tank painting, and right-of-way maintenance expense that is beyond the capacity of the regular maintenance personnel. Operating costs for the NNMP are summarized in **Table 11-2**.

**Table 11-2**
**Summary of Annual Operating Costs**
**Navajo Nation Municipal Pipeline**

| | **Quantity** | **Cost** |
|---|---|---|
| **Pumping Power** | | |
| Demand | 290 kW | $53,600 |
| Energy Use, 35% operation | 445,000 kWh | 8,000 |
| Annual Power Cost | | $61,600 |
| Treated Water Cost | | |
| Purchase from City of Farmington | 4.0 MGD Average | $1,664,600 |
| **Other Operating Costs** | | |
| Personnel | 2 part, 2 full time | $100,000 |
| Maintenance Equipment Operation | | 14,000 |
| Repairs and Services | | 20,000 |
| **Subtotal Other Costs** | | **134,000** |
| **Subtotal Operating Cost** | | **$1,860,000** |
| Escalation Contingency, 15% | | 280,000 |
| **Total Annual Operating Cost** | | **$2,140,000** |
| **Unit Operating Cost** | **Cost per 1000 gallons** | **$1.46** |

Note:
Power rates applied: $15.40 per month per kW demand, 18 mils per kWh.
Purchase of treated water from City of Farmington at 1999 rate of $1.14 per 1000 gallons.

## 11.3 Water Acquisition Fund

There are no projected operation and maintenance costs associated with the water the $40 million acquisition fund. However, to provide flexibility in the use of the fund, authorization would allow some or all of the funds to be redirected for on-farm development, water delivery infrastructure, and other economic development activities.

## 11.4 Cultural Resource Mitigation

Annual program operation costs for the Ridges Basin Archeological District are estimated to be $100,000. This estimate includes staff support for meeting Section 110 of NHPA requirements, recovered materials' preservation, and NAGPRA work activities.

## 11.5 Wetlands and Fish and Wildlife Mitigation

Annual costs for the wetlands and fish and wildlife mitigation program commitments are estimated to be $108,000. Individual commitments are included in Chapter 5.

# 13.0 REFERENCES

Barnes, H., E.H. Blatry, Jr., and P.T. Haynes. 1954. Geology and Fuel Resources of the Red Mesa Area, La Plata and Mountain Counties, Colorado. U.S. Geol. Survey Oil and Gas Investigation Map OM-149.

Dykstra, Steven, J. 1999. Hydraulic Analysis of the Farmington-Shiprock Water Transmission Line. 1999. University of New Mexico. Pp. 17-18.

Fassett, J.E., and J.S. Hinds. 1971. Geology and Fuel Resources of the Fruitland Formation and Kirtland Shale of the San Juan Basin, New Mexico and Colorado. U.S. Geol. Survey Prof. Pap. 676, 76 pp.

Hynes-Griffin and Franklin, A.G. 1984. Rationalizing the Seismic Coefficient Method. Vicksburg, Mississippi. U.S. Army Engineer, Waterways Experiment Stations, Miscellaneous paper. GL/84/13.

Kleiner, D.E. 1999. Granular Filters and Drains for Embankment Dams: U.S. Committee on Large Dams Bulletin, 20 pp.

Kelley, V.C. 1957. Tectonics of the San Juan Basin and Surrounding Areas: Four Corners Geological Society Guidebook. pp. 44-52.

Kilgore, L.W. 1955. Geology of the Durango Area, La Plata County, Colorado: Four Corners Geological Society Guidebook. pp. 118-124.

Molzen-Corbin. 1993. Navajo Tribal Utility Authority Shiprock Water Supply Study.

Navajo Nation. 1998. An Appraisal Level Study of the Proposed Farmington to Shiprock Municipal Water Line, Department of Water Resources, Water Management Branch Technical Memorandum. June 19, 1998. 19 pp.

Ridgley, J.L., M.W. Green, C.T. Pierson, W.I. Finch, and R.D. Lupe. 1978. Summary of the Geology and Resources of Uranium in the San Juan Basin and Adjacent Region, New Mexico, Arizona, Utah, and Colorado: U.S. Geological survey Open-File Report 78-964, 107 p.

Toro, G.R., N.A. Abrahamson, and J.F. Schneidger. 1997. Model of Strong Ground Motions from Earthquakes in Central and Eastern North American: Best Estimates and Uncertainties: Seismol. Research Ltrs., Vol. 68, No. 1., Jan/Feb 1997. pp. 41-72.

U.S. Bureau of Reclamation. 1992a. Seismic Evaluation for Ridges Basin Dam Site, Animas-La Plata Project, Southwestern Colorado. U.S. Bureau of Reclamation Seismotectonic Report 92-2, June 1992, 48 pp w. append.

U.S. Bureau of Reclamation. 1992b. Ridges Basin Dam, Geologic Design Data Report G-499. U.S. Bureau of Reclamation, Volume 1, December 1992. 144 pp.

U.S. Bureau of Reclamation. 1992c. Wick Drain Design for Upstream Alluvial Foundation Ridges. Basin Dam, Animas-La Plata Project, Upper Colorado. U.S. Bureau of Reclamation Tec. Memo No. RB-3620-22.

U.S. Bureau of Reclamation. 1995. Concept Design Summary Animas-La Plata Project Durango Pumping Plant, U.S. Bureau of Reclamation, September 19, 1995. 16 pp.

U.S. Bureau of Reclamation. 1996. Design Summary. Ridges Basin Dam, Animas-La Plata Project, Colorado-New Mexico, Vol. I, III and VII B (partial). U.S. Bureau of Reclamation, August 1996.

U.S. Bureau of Reclamation. 1997. Durango Pumping Plant Model Studies Fiscal Year 1996, U.S. Bureau of Reclamation, Transmittal Memorandum Tony Wahl to Manager Durango, September 30, 1997. 23 pp.

U.S. Bureau of Reclamation. 1999. Farmington to Shiprock Pipeline Technical Memorandum Review. U.S. Bureau of Reclamation, 11 pp.