The Great Salt Lake West Desert Pumping Project

Its Design, Development, and Operation

Utah Division of Water Resources
Utah Department of Natural Resources

June 1999

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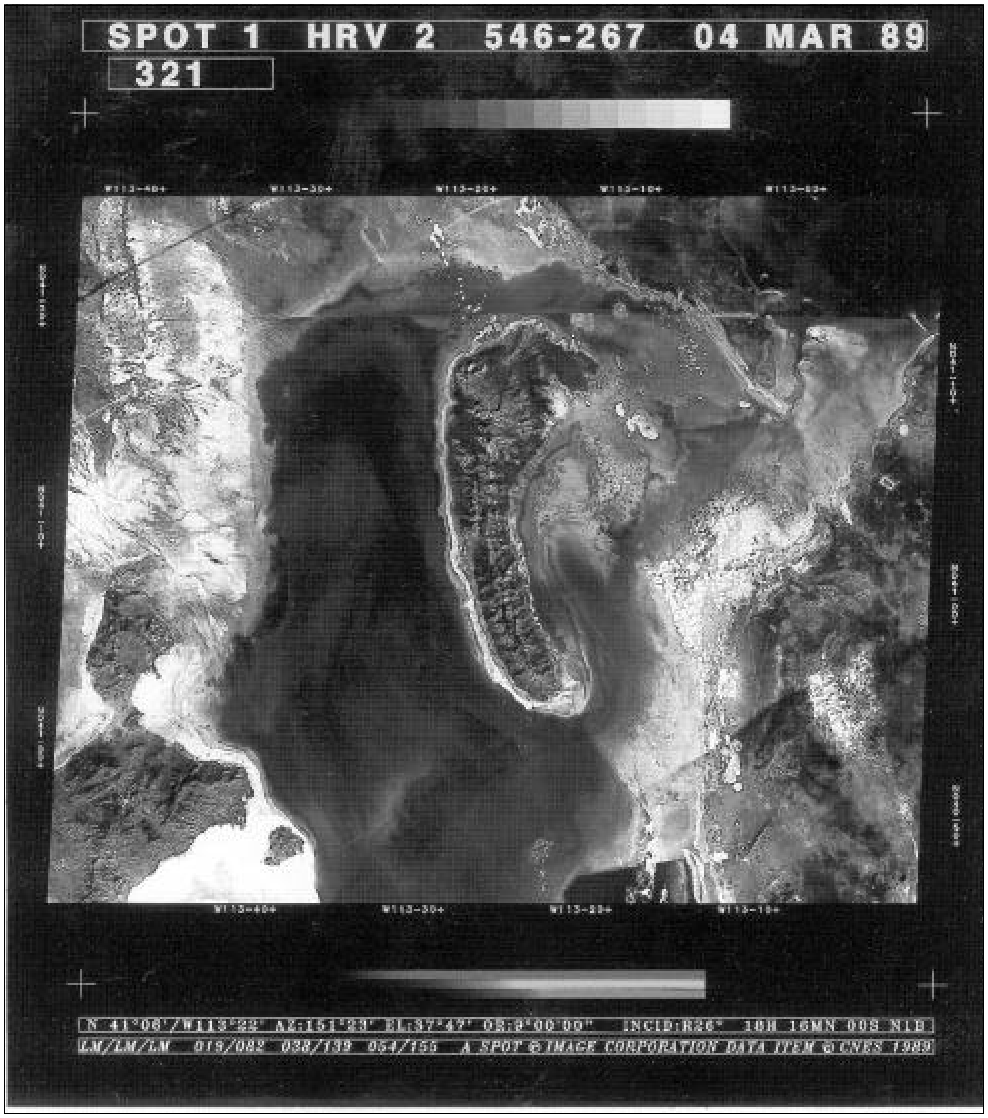

Satellite View of the West Desert Pumping Project

Awarded the 1988 Civil Engineering Achievement of Merit by the American Society of Civil Engineers

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The Great Salt Lake West Desert Pumping Project

Contents

Contents (continued)

Chapter 1

Introduction

Construction of the West Desert Pumping Project was an unprecedented flood control action on the Great Salt Lake, the largest body of water in the Western Hemisphere without an outlet to a sea. The project, designed to enhance the lake's natural evaporation process was conceived and constructed in record time. Construction began on July 7, 1986. The first of the project's three pumps began operating on April 10, 1987. The project was fully operating on June 3, 1987.

Between fall 1982 and June 1987, the level of the Great Salt Lake rose over 12 feet, the tail-end of a steady rise of nearly 20 feet between 1963 and 1987. The lake had more than doubled its surface area and increased its volume three-fold. The lake level reached a modern-day record 4211.85 feet above mean sea level in 1986 and 1987, surpassing the historic high of 4211.60 set in June 1873. At the new record level, the lake covered almost 2,400 square miles and contained over 30 million acre-feet of water. The Great Salt Lake went on a costly, destructive rampage with its horded inflow from record amounts of snow and rain in northern Utah. Shoreline flooding caused an estimated $240 million in damages to Interstate Highway 80, mineral industries, railway systems, sewage treatment plants, wildlife habitat, recreation areas, and public and private property.

West Desert Pumping Project facility 10 miles west of Lakeside in Box Elder County

Weather experts could predict no immediate change in the weather, which led to fears that Interstate 80 would be lost to flooding, requiring a new, rerouted freeway. The Southern

Pacific and Union Pacific railroads considered shutting down operations because of flood damage. Fears grea that the Salt Lake International Airport would stop flights because runway drains were starting to fill up. The Great Salt Lake was out of control.

Construction and operation of the West Desert Pumping Project was controversial, and it spawned considerable public and political debate about costs and alternatives to pumping lake brine. Concern about the damage caused by the Great Salt Lake was widespread, but many people harbored hope the lake would heal itself. The project, however, eventually won approval from the Utah State Legislature by a substantial margin as the most cost-effective and technically sound solution with the greatest public benefit. Project engineers faced and overcame unique challenges, including the harsh environment of the Great Salt Lake, remoteness of the Pumping Plant, and difficult access to construction areas. The project was nominated for the prestigious Outstanding Civil Engineering Achievement Award from the American Society of Civil Engineers and won the society's Civil Engineering Achievement of Merit Award.

A total of $71.1 million was authorized to flood control efforts during a special session of the 1986 Utah State Legislature, including $60 million to the Utah Division of Water Resources, Utah Department of Natural Resources, to implement the project to pump water from the Great Salt Lake into the desert area west of the lake.

The pumping project was shut down on June 30, 1989, after more than two years of successful operation. The project pumped about 2.73 million acre-feet of brines from the lake. The shutdown process took about eight weeks, requiring the Pumping Plant to be secured and dismantling, preserving and storing tools and system control devices.

Since the project was shut down, the Pumping Plant has been inspected periodically and maintained as insurance against future flooding around the Great Salt Lake. It is a permanent facility that cannont be dismantled for others uses.

This historical review traces the development, design, implementation, operation and eventual shutdown of the West Desert Pumping Project by the state of Utah to combat the costly flooding of the Great Salt Lake, and recounts events and critical decisions that led to the project's construction and operation.

Valuable information about instrumentation, climate, evaporation and wind tides concerning the Great Salt Lake has been gained from operating the pumping project and is presented in the appendices of this review. ⌂

Chapter 2

Utah's Wet Weather Problems 1982-1986

“A Heck of a Way to Run a Desert”

Governor Scott M. Matheson

September 1982 was very wet. A 100-year storm on September 26, 1982, unloaded 2.27 inches of rain at the Salt Lake International Airport. Reportedly, it was the most precipitation ever measured in one day during 108 years of weather record-keeping in Salt Lake City. Precipitation totaled 7.04 inches that September, making it the wettest on record in Utah. Total precipitation at the Salt Lake City International Airport during the 1982 water year was 22.86 inches, compared to an annual average of 15.63 inches.

Snowfall between autumn 1982 and May 1983 in north and central mountain areas of the state was well above average. Alta, about 20 miles southeast of Salt Lake City at the top of Little Cottonwood Canyon, reported a total 845 inches of snowfall. The snow cover on June 1, 1983, ranged from 2.4 to 3.4 times greater than average in the Bear River Basin, about 4.2 to 5.2 times greater in the Weber River Basin and about 3.7 to 5.2 times greater in the Jordan-Provo River Basin. Soil moisture in drainage basins was considerably more than average. Skier days and ski industry revenues jumped to new records. The elevation of the Great Salt Lake in June 1983 was 4200.70 feet above sea level.

Hints of Flooding Problems

The Thistle landslide on April 12, 1983, was the first clue that the high precipitation would cause problems in the state. The slow-moving landslide in Spanish Fork Canyon in Utah County severed the Denver and Rio Grande Railroad line between Denver and Salt Lake City, breached U.S. Highway 50 and 6, and dammed off the Spanish Fork River. The backed-up river subsequently inundated the small town of Thistle. Eventually the slide became a dam 200 feet high that held back an estimated 65,000 acre-feet of water. Gov. Scott M. Matheson declared the landslide a disaster and asked for $7.8 million in federal disaster aid.

Snowpack in the mountains normally peaks by the first of April, but the weather stayed cold and rain and snow continued until the middle of 1983. A landslide occurred in Payson Canyon, Utah County, on April 19. The Great Salt Lake rose nearly four feet, prompting state officials to consider closing the road to Antelope Island State Park. A landslide closed the road through Emigration Canyon east of Salt Lake City, and city officials " . . . were keeping their fingers crossed on City Creek in the canyon behind the State Capitol Building."

An emergency drawdown of water in Mountain Dell Reservoir east of Salt Lake City in Parley's Canyon was ordered to make room for the expected heavy runoff from melting snow. Other slides occurred in the Canyon Cove area near the Holladay Gun Club on the eastern edge of the Salt Lake Valley.

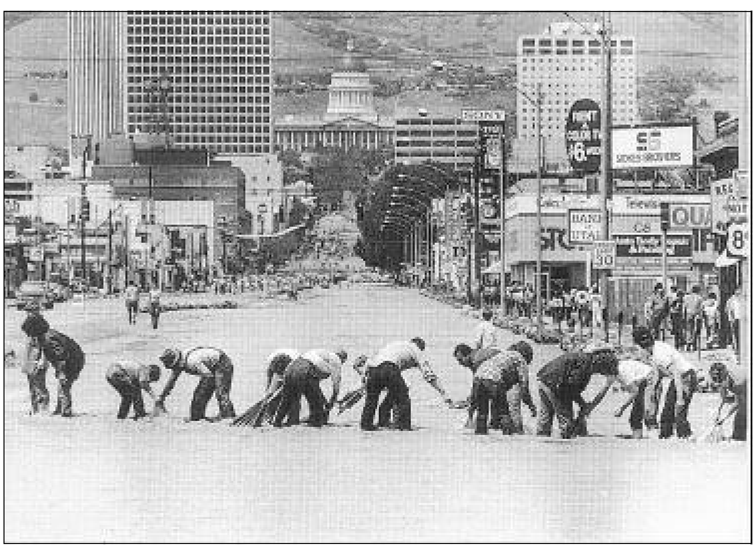
Waves on the Great Salt Lake, tossed by 60 mile-per-hour winds, battered the Southern Pacific and Union Pacific railroad lines. By May 14, 1983, Utah Lake in Utah County was three feet above flood stage and more than 10,000 acres of farmland were under water. Talk began about the return of ancient Lake Bonneville to the Great Basin.

Memorial Day Meltdown

The major snowmelt in 1983 started during a heat wave on the Memorial Day weekend. Runoff gouged down canyon stream beds, especially in City Creek Canyon, into downtown Salt Lake City. The city's State Street eventually became a river banked by sandbags placed by thousands of volunteers. On May 31, a 30-foot high mud slide oozed down Rudd Creek into Farmington, Davis County, burying four homes and forcing people to evacuate several blocks. That same day, 1,100 people fled a landslide in Fairview, Sanpete County. A flood from Chicken Creek washed out I-15 near Levan, Juab County. Someone caught a trout on May 31 in the river that was State Street in downtown Salt Lake City.

Runoff into the Sevier River flowed more than 500 percent of normal by June 1. A flood of mud cascaded down Stone Creek in Bountiful, Davis County, destroying six homes and damaging 50 others. Another 1,100 people were evacuated. By then, half the state was on emergency disaster status. Then the spillway of the DMAD Reservoir, a regulating reservoir near the end of the Sevier River in Millard County, failed on June 23, 1983, and released approximately 14,000 acre-feet of water, flooding the farming community of Deseret and cutting off irrigation water supplies to over 60,000 acres of farmland.

Bridges that had been erected to funnel foot and vehicle traffic across the State Street river in Salt Lake City were being taken down by the end of June, but spillways of dams along the Colorado River were being opened in anticipation of heavy runoff.



State Street flooding in Salt Lake City on May 30, 1983

Wildlife in Trouble

Utah's big game herds foundered in the heavy mountain snowpack that had accumulated by December 1983. Deer and elk abandoned their snowbound brouse areas and browsed in man's domain. Thousands of deer and elk moved onto farmland and into residential areas and community streets looking for food.

Prodded by nationwide public sympathy, and about $314,000 in donations from across the U.S., the Utah Division of Wildlife Resources, other public and private agencies, wildlife interest groups, and volunteers started feeding specially formulated food pellets to deer and hay to elk. The state legislature also authorized $172,000 to the big game feeding program. An estimated 40,000 to 50,000 deer and 4,000 elk were fed at hundreds of feeding stations. Big game losses were dramatically reduced through the feeding program that continued through April 30, 1984.


Utah's big game herds were fed during the '83-'84 winter

The Great Salt Lake

The Great Salt Lake in northwestern Utah is North America's unique inland sea. The lake's terminal desert setting, Pleistocene heritage, and the nearly five billion tons of salt and other minerals it contains suggest a dead sea. The Great Salt Lake is old, but it is not dead.

The saline lake brims with diverse life. Algae, brine flies, brine shrimp and their billions of bright red eggs, salt marsh bushes and grasses, and millions of shorebirds and waterfowl are part of its life. Of the lake's eight named islands, Antelope Island and Fremont Island are inhabitable. Antelope Island has a state park and transplanted buffalo, deer, elk and antelope. Smaller, more inaccessible islands are nesting grounds for migratory birds. The lake has hunting, sailing, mineral industries, micro burst storms and, of course, exquisite sunsets.

The surface level and volume of the Great Salt Lake change continuously, primarily in response to climatic factors. Man's activities have had a lesser, but still important, effect on the level and volume of the lake. The lake level generally declines in the spring and summer when the weather is hot enough that the loss of water by evaporation from the lake surface is greater than the inflow from surface streams, groundwater and precipitation directly on the lake. The lake level rises in the autumn when the temperature cools and the inflow exceeds the loss of water by evaporation.

Because the Great Salt Lake is a closed basin and its only outflow is evaporation from its surface, the change in the lake's surface area, volume and level reflects the integrated effect of all processes of the hydrologic cycle within the drainage basin. Historically, these effects have been displayed by changes to the inflow to the lake which has caused wide fluctuations in the surface area, volume and stage of the lake. Since 1847, when the historical record of lake level fluctuations began, the annual inflow to the lake has ranged from 1.1 to 9.1 million acre-feet. The stage reached its first modern period high of 4211.5 feet in 1873 and a low of 4,191.35 feet in 1963.

The major surface flows that enter the Great Salt Lake are from the Bear, Weber and Jordan rivers, the headwaters of which occur within 50 miles of each other. Smaller streams that discharge into the Great Salt Lake are Farmington, Centerville, Holmes, Ricks, Parrish, Stone and Mill creeks.

The Bear River drains a 6,800 square-mile area in Utah, Wyoming and Idaho and provides the largest surface inflow to the Great Salt Lake. Some of the heaviest precipitation in the state of Utah occurs in the Weber River Basin, which covers about 2,060 square miles. Flows entering the Great Salt Lake from the Jordan River originate in the Uinta

Mountains east of Salt Lake City. The main tributaries above Utah Lake are the Provo and Spanish Fork rivers that rise at 11,000 and 9,500 foot elevations respectively. The Jordan River begins at the outflow from Utah Lake and flows northward through Salt Lake County to the lake. Several streams from the west slopes of the Wasatch Mountains enter the Jordan River along its path to the Great Salt Lake. The major streams are Little Cottonwood, Big Cottonwood, Mill, Parley's, Emigration and City creeks.

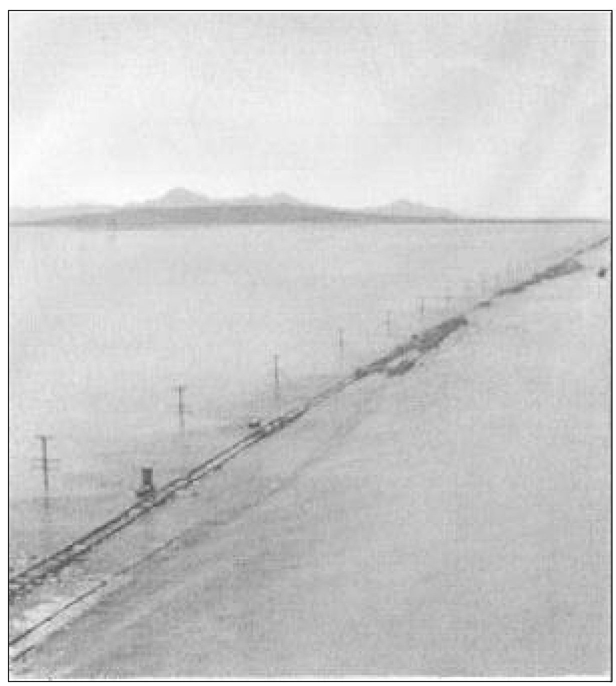
These runoff swollen rivers and streams harassed homes, businesses, reservoirs and roads between 1982 and 1986, then dumped their enormous flows into the Great Salt Lake.

The Lake Catches It All

From 1963 through 1986, the Great Salt Lake rose nearly 20 feet, more than doubled its surface area, and increased its volume nearly three-fold. Almost 12 feet of the rise occurred since the beginning of 1982, attributed to excessive precipitation in northern Utah drainage areas that feed the Great Salt Lake. Inflow to the lake in 1986 was more than double the normal average. On June 5, 1986, the level of the south arm of the Great Salt Lake reached a new record historic high elevation of 4211.85 feet above sea level. The lake reached the same level again in 1987. At this modern record level, the lake covered approximately 2,400 square miles and contained more than 30 million acre-feet of water. For perspective, its expanse was only about 487 square miles less than the states of Delaware and Rhode Island combined, and the lake contained an acre-foot of water for every resident of Utah, Idaho, Wyoming, Montana, Colorado, North Dakota, South Dakota, Nebraska, Iowa, Kansas, Oklahoma, New Mexico, Arizona, Nevada, Oregon and Washington.

In its bloated size from the horded inflow from record amounts of snow and rain in northern Utah, the lake went on a destructive rampage. By 1986 flood damage estimates around the lake to public and private land, industries, major transportation routes, public facilities and wildlife habitat totaled more than $240 million. Potential cost of damages was estimated at $1 billion, figuring affected company payrolls, tax payments, capital expenditures and purchases.

Flooding problems existed all around the lake. On the south shore, Interstate 80, the Union Pacific Railroad, the Great Salt Lake State Park and Marina, and beaches were inundated. Water washing over the freeway during one May 1986 storm backed traffic for miles. The Union Pacific Railroad that parallels Interstate 80 along an 11.5 mile stretch on the lake's south shore acts as a breakwater for I-80. The railroad raised its tracks a number of times between Kennecott and Burmester in Tooele County, starting in 1983, that railroad officials estimated cost about $24 million. The state park facilities access roads and marina were washed away; a remnant restroom building withstood the heavy waves the longest, but it too was pulled apart and spread along the shoreline. The Saltair resort and adjoining amusement park that had been reopened in the spring of 1983 after being restored by several Salt Lake City businessmen was engulfed; its dance floor and parking lot ended up under four feet of



Southern Pacific Railroad causeway near Lakeside after June 7, 1986, storm.

water. Beaches popular with local residents and out-of-state visitors were scoured away by the lake's high waves.

AMAX Magnesium Corp., one of 11 mineral industries around the lake, lost a 200-yard section of a 13-mile-long outer dike to pounding waves. In two days, nearly 500,000 acre-feet of lake water flowed through the breach to inundate the company's mineral extraction ponds. The company employed 750 people and indirectly affected 1,150 jobs. The company paid about $1 million in taxes each year. The dike breach stopped a $5 million joint venture with Diamond Crystal Salt Co. to produce solar salt in ponds planned for the area that was flooded.

The lake level threatened health problems to groundwater in low-lying areas. In Erda, Tooele County, 40 septic tanks failed and 300 more were endangered. In some low-lying areas such as Rose Park on the east side of the lake, as many as 1,000 homes were threatened. The elevation of Rose Park was actually three feet below the level of the lake.

Serious concern was shown for groundwater problems that could affect the Salt Lake International Airport. The airport is at elevation 4213, while the runways are at 4220.

The state and federal waterfowl management areas were devastated by the lake's intruding salt water. Facilities at Farmington Bay, Howard Slough, Ogden Bay state waterfowl management areas and the historic federal Bear River Migratory Bird Refuge were completely destroyed along with facilities of many private duck and goose clubs. Power and communication towers along the east shore of the lake were hundreds of yards offshore. Eventually the state park facilities on Antelope Island shorelines were washed away, and the causeway from the Syracuse area to the island was covered by nearly six feet of water. Productive farmland in this area was submerged.

Fresh water management areas at Locomotive Springs National Waterfowl Management Area 50 miles west of Brigham City were topped by wind-driven lake water, damaging about 85 percent of the 4,000-acre management area. It had been a prime fall duck and goose hunting area since 1935. Luckily, the fresh water springs were undamaged.

Several sewage treatment plants became islands in the lake, protected by walls of sandbags. A radio station, setting several feet below the shoreline water level, surrounded itself with dikes.

Great Salt Lake Minerals and Chemicals Corp., a company that operates a huge system of ponds at the lake's north edge to extract potassium sulfate, potash and salt, diverted employees from mineral production to dike building to shore up against the inexorably creeping lake level.

The floods of 1983 were probably the most widespread the state has ever experienced. By the end of 1983, flood damage throughout the state exceeded $478 million, according to the Utah Department of Public Safety. Total damage in 1984 was estimated at $190 million. Of Utah's 29 counties, 22 were approved by the Federal Emergency Management Administration (FEMA) as disaster areas.

Despite the devastation, the situation spawned some humor. Proponents of a so-called "Think Lake Bonneville Society" announced rain dances to prod Mother Nature to bring back prehistoric Lake Bonneville that once covered most of northern Utah and parts of Idaho and Nevada. The society jokingly claimed the new Lake Bonneville would boost tourism, create a building boom (people would have to move to the mountains) and allow planting of fresh-water sturgeon to make Utah caviar. And interestingly, Utah County commissioners, asking for a day of prayer to stop the flooding, ignited a protest from the "Freedom From Religion Foundation" in Madison, Wisconsin, In a letter to Utah Attorney General David L. Wilkinson, the group called the commissioner's plea an "unacceptable abuse of separation of church and state."

The State Reacts

Before 1983 the state left most of the responsibility for flood control with the counties. The 1983 spring floods, however, were so extensive and serious that the state became deeply involved in flood mitigation and prevention.

Historians say Brigham Young, leader of the Mormon pioneers who arrived in the Salt Lake Valley in 1847, explored the possibility of spilling the lake into the west desert area when the lake peaked at 4211.5 feet in 1873. But the lake receded on its own after 1873. During the early 1970s, several researchers and state and federal agencies defined the hydrology of the lake, developed computer models of it, and investigated alternatives for dealing with high lake levels. Summaries of much of this work were published in 1973 and 1974 by the Utah Division of Water Resources titled *Great Salt Lake Climate and Hydrologic System* and *Hydrologic System Management Alternatives Report.* The 1977 and 1978 drought years slowed interest in preparing for problems with high levels of the Great Salt Lake. But a flurry of legislation was passed in 1983 and 1984 legislative sessions which committed the state to help alleviate flood damage. With the continual rise of the lake during those years, a breach of the Southern Pacific Causeway gained support. The elevation of the lake was over three feet higher on the south side of the causeway than on the north side. The causeway had become a dam. Great Salt Lake Minerals provided $200,000 for the state to conduct a feasibility study of the proposed breach. The state's technical position was that the breach would lower the level of the south arm of the lake nearly a foot and would be cost effective. In January 1984, after being defeated in two legislative sessions, the breach of the Southern Pacific Causeway was funded. Costing about $3 million, a 300-foot wide bridge was constructed in the causeway near the west side of the lake and the causeway was opened on Aug. 1, 1984.

During a two-day special session of the Utah State Legislature that ended on May 14, 1986, a $71.7 million flood control plan was approved to pump water and build more emergency shore diking. The pumping project, originally proposed during the administration of Gov. Scott M. Matheson, was one of several proposals that were spawned by flooding of the Great Salt Lake. The "last resort" pumping plan, sponsored by Republican Sen. Fred Finlinson, passed with two-thirds support and was expected to lower the lake by about 16 inches after a year of pumping. Engineers hoped pumping would start in February 1987.

The flood control bill, HB 6, cautiously backed by Gov. Norman H. Bangerter, provided $60 million to the Utah Division of Water Resources, Utah Department of Natural Resources for pumping the lake water and $10 million to the Disaster Relief Board to implement finger diking in Salt Lake County, raising breakwaters around the Great Salt Lake Marina, raising dikes at the AMAX Magnesium Plant and American Salt Co. to protect Interstate 80, and further dike protection of sewage treatment facilities on the lake's east shore. Tagged to the bill was $1.2 million for engineering design of an interisland diking proposal and $500,000 for preconstruction design studies for upstream storage dams, principally on the Bear River, which would require 10 to 20 years to complete. Flood control funding included $30 million from an existing flood mitigation fund deposited in the Conservation and Development Fund managed by the Utah Board of Water Resources, and $41.7 million obtained from a general obligation bond. The bond was to paid off with a one-eighth cent share of the state's sales tax retained through 1989. ⌂

Chapter 3

Great Salt Lake Basin

General Aspects

The Great Salt Lake is the drainage sink for an area approximately 22,000 square miles. Most of its inflows originate in mountainous areas near the lake. The major river systems that sustain the lake derive mostly from the Wasatch Mountains (See Figure 1).

Extremely low precipitation over many parts of the Great Basin, particularly that part occupied by the Great Salt Lake, in conjunction with the large amounts of precipitation experienced in the higher mountain elevations of the watershed give rise to the hydrologic characteristics of the lake and its environs. Typically, the annual precipitation over the lake is less that 12 inches. Snow accumulations in the adjacent Wasatch Mountains often exceed 200 inches in early spring and 40 inches of total precipitation in a year. The following information is helpful to understand conditions in the Great Salt Lake Basin that led to the design, development and operation of the West Desert Pumping Project.

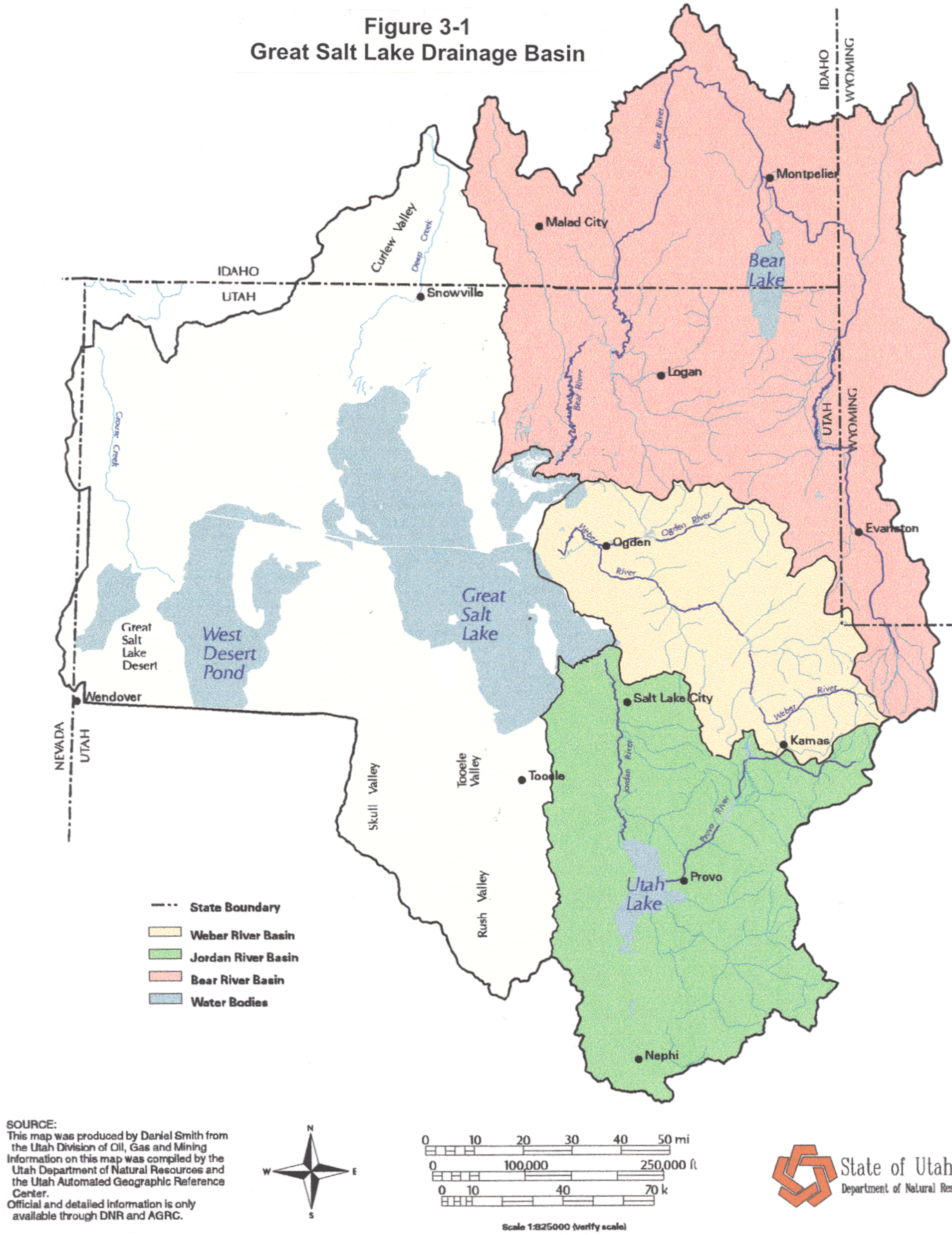
Historical Aspects

Significant exposures of formations from every geological period exist within a 25-mile radius of Salt Lake City. This fact underlies the fundamental reason for the presence of the Great Salt Lake.

Major geotechnic events of the past 10 to 20 million years have created the bowl now occupied in part by the lake and have exposed ancient and extremely resistant materials as stingy sources of sediment to the lake. The eastern perimeter of the Great Salt Lake consists of the Wasatch Front, a range of block-faulted mountains which represent the eastern edge of the basin and range province. The stretching of the earth's crust in this zone resulted in a characteristic signature of north-south trending block-fault mountains interspersed with sediment laden basins. Although the predominant structures are tilt faults, in the vicinity of the Great Salt Lake the majority of features are horsts and grabens. Great Salt Lake occupies a graben complex.

The Wasatch Range that confines the lake on the east is a straight, narrow horst of resistant Precambrian rocks (Stockes 1980). Parallel to this horst is the Wasatch structural trough which is filled by tertiary and quaternary sediments. The trough is

Figure 3-1
Great Salt Lake Drainage Basin



complex, consisting of a number of subsidiary blocks. Sedimentation has not kept pace with subsidence, and the situation is nearly a mirror image of the eastern face of the Sierra Nevada Mountains on the west side of the basin and range province.

The next westerly uplift feature is the Antelope-Promontory Horst, which includes Antelope Island and the Promontory Peninsula. The later is historically significant for the meeting of the transcontinental railroads in the last century. West of this chain lies the main body of the Great Salt Lake, occupying a complex designated the Great Salt Lake Graben. Boundary mountains west of the lake are the Lakeside Range on the southwest and the Terrace-Hogup ranges on the northwest. Hogup Cave has yielded some of the oldest relics of human habitation in the Great Basin. The lake bed area between Lakeside and Hogup ranges has been called the "threshold" through which the Great Salt Lake would flow into the West Desert. Recent studies have shown the actual threshold extends in a southerly direction from the Newfoundland Range across the lake bed area at an elevation of approximately 4,215 feet above sea level.

West of the Wasatch Range and south of the Great Salt Lake lies the Oquirrh Range, famous for the largest open pit copper mine in the world. The predominant formation is the Oquirrh Group, consisting of about 11,000 feet of Middle Paleozoic limestones and related calcareous rocks.

To the northwest of Great Salt Lake, the Lakeside Mountains stretch for about 30 miles. Although this range is less impressive than the Oquirrh or Wasatch ranges, a Paleozoic section more than 43,000 feet thick has been identified in these mountains.

The Promontory Range is the remaining significant mountain in close proximity to the Great Salt Lake. It protrudes into the lake for about 30 miles, separating Bear River Bay from the so-called "north arm" which was virtually hydraulically isolated from the rest of the lake by construction of the Southern Pacific Railroad Causeway. The Promontory Range displays more than 33,000 feet of sedimentary rocks of Precambrian and Paleozoic origin. The area occupied by the Great Salt Lake has been sediment starved. Sediments transported to the lake by three major tributaries, the Bear, Weber and Jordan rivers, are extremely small size. The rivers arrive at the Great Salt Lake at very low gradients after traversing some extremely efficient sediment traps. This factor, as much as any other, accounts for the Great Salt Lake being where it is. Stokes (Stokes 1980) summarized the situation as follows:

"The fact that this area of faulted crystalline rock appears to have been the center of a long succession of Cenozoic water bodies, including Lake Bonneville and the Great Salt Lake, suggests that bedrock types, erosion products, and sedimentation have been fairly constant for several million years, perhaps ever since the Miocene. Although depressed areas are being filled by deposition, the Great Salt Lake Graben, because of the nature of the bedrock in surrounding uplifts (and because of the sparse and fine-grained nature of the sediments in the tributary stream), has always lagged behind so as to be the low spot of the drainage system."

Ancient Lake Bonneville

The shorelines of Lake Bonneville, the immediate predecessor to the Great Salt Lake, are conspicuous topographical features of northern Utah. They exist at elevations which are now nearly 1,000 feet above the surface of the Great Salt Lake, and they attest to substantial changes in the climatological and hydrological regimes of the Pleistocene. The more visable shorelines were highlighted in National Geographic Magazine (Gore 1985). These manifestations of Lake Bonneville are relatively young, having been established within the last 25,000 years. Strong indications exist that there were many marked rises and declines of the lake in the last 70,000 to 100,000 years (Morrison 1966). Furthermore, these late Pleistocene oscillations were preceded by at least two earlier periods when lake levels were generally high in middle Pleistocene, and there are indications that earlier lakes existed in this region in Tertiary time. The sedimentary records and shoreline information of these earlier predecessors are either buried beneath younger sediments or have been obscured and obliterated by subsequent erosion and deposition events. Not much evidence exists on the nature or extent of pre-Bonneville lakes.

Lake Bonneville was the largest late Pleistocene pluvial lake in the Western Hemisphere. At its peak it inundated an area of nearly 20,000 square miles, as illustrated in Figure 2. It extended nearly 285 miles north to south and 140 miles east to west, and several times it attained a depth of approximately 1,000 feet. It was principally a Utah lake, with parts of it in Nevada and Idaho.

Another factor in the predominance of Lake Bonneville is related to a series of ancient volcanic eruptions and lava flows in the Soda Springs area of Idaho. Evidence shows these lava flows diverted the Bear River from its previous course to the Snake River and then to the Columbia River and the Pacific Ocean, and redirected it southward into Lake Bonneville. Based on today's conditions, this new tributary to the lake would have increased the total lake surface inflow by about 25 percent.

Conditions of the Great Salt Lake

The surface level and volume of the Great Salt Lake changes continuously, primarily in response to climatic factors. Man's activities have had a lesser, but still important, effect on the level and volume of the lake.

The lake has a yearly cycle and a long-range fluctuation. The yearly cycle begins to decline in the spring and summer when the weather is hot enough that the loss of water by evaporation from the lake surface is greater than the inflow from surface streams, groundwater and precipitation directly on the lake. It begins to rise in the autumn when the temperature decreases and the loss of water by evaporation is exceeded by the inflow. According to records, the rise can be expected to begin between September and December and the decline to begin any time between March and July.

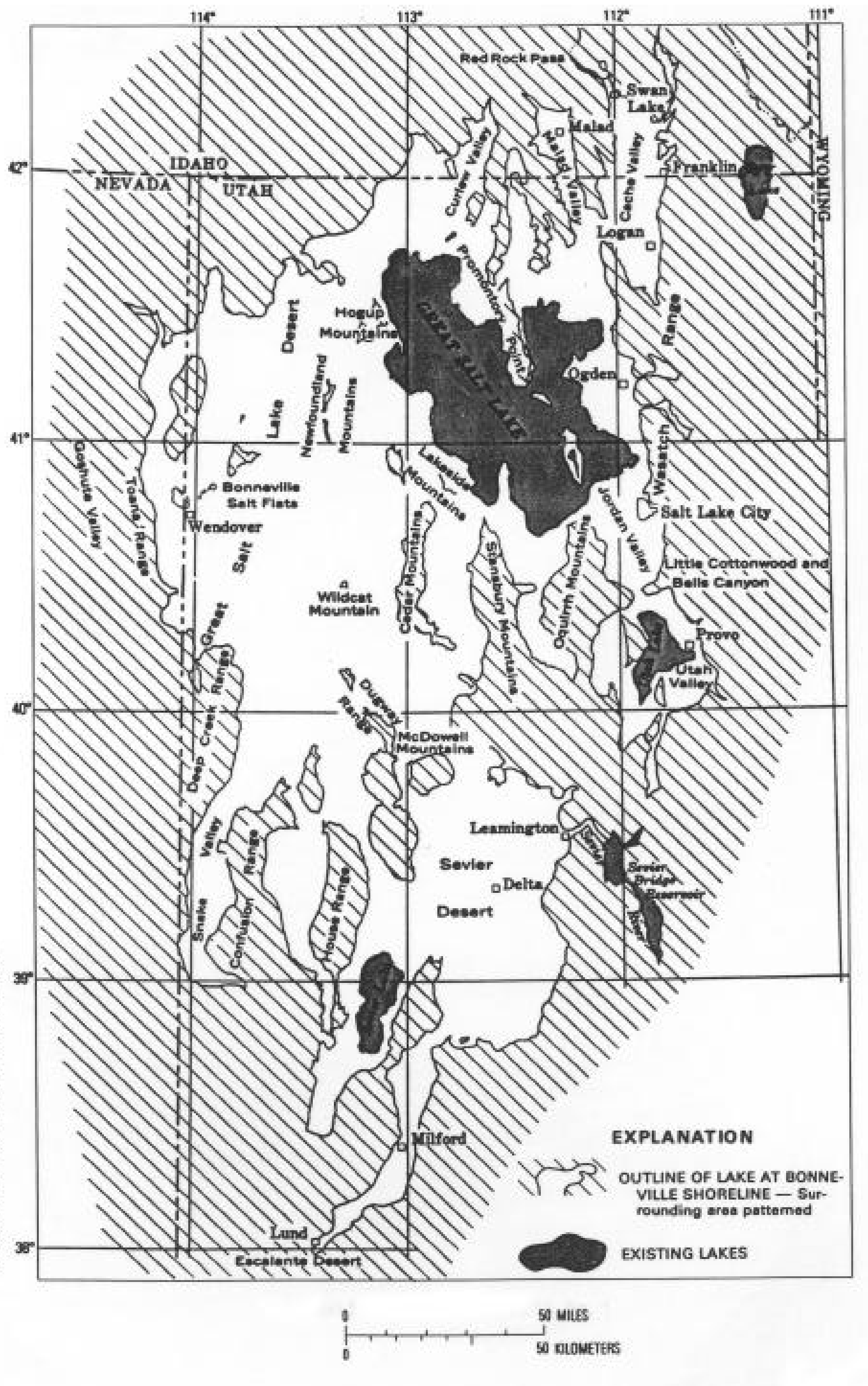
Because the Great Salt Lake is a closed basin and its only outflow is evaporation from its surface, the change in the lake's surface area, volume and level reflects the integrated effect of all processes of the hydrologic cycle within the drainage basin. Historically, these effects have been displayed by wide fluctuations in the inflow to the lake which has caused wide fluctuations in the surface area, volume and stage of the lake.

The historical record of lake level fluctuations, shown in Figure 3, began in 1847. The level was determined indirectly by Gilbert (1890) for 1847-75 on the basis of reported observations of the depth of water over the sandbars between the mainland and Antelope and Stansbury islands. Gilbert related these oral reports to later measurements by determining the elevations of Antelope and Stansbury islands sandbars, making soundings on the Antelope Island bar, and relating water level there to gage readings near Black Rock and Farmington. The highest level in the modern period (4211.85 feet) was reached in June 1986 and duplicated in June 1987. Level elevations for the north and south arms of the lake are shown in Figure 4.

Periodic gage readings after 1875 were made by various observers at Black Rock, Farmington, Lakeshore, Garfield, Midlake and Saltair. These gage readings were summarized in various USGS water supply reports. Since October 1, 1912, gage heights have been published in annual water supply papers, and corresponding elevations have been published since 1958. The lake level has been measured continuously at the Salt Lake County Boat Harbor (south arm) since 1939 and at Saline (north arm) since 1966 when the elevation difference between the north and south arms of the lake was first noticed. The rapid rise of the lake level in 1984 and 1985 prompted relocation of the gage twice near the harbor. Semi-monthly records have been published in annual USGS water supply papers and in state reports. The gaging sites and the chronology of the records are shown in Figure 5.

Research at the University of Utah (Glenne and Eckhoff 1976) developed a mathematical model to quantify the relationship between precipitation and the level of the lake. The research showed an increase of as little as two inches in the sustained average precipitation at Salt Lake City would result in a 10-foot rise in the lake level. Events between 1982 and 1987 show how accurate that scenario was. Additionally, Glenne and Eckhoff developed a Markov model to be used to estimate the statistical

Figure 3-2
Lake Bonneville Area



Adapted from Crittenden, 1963, fig. 1

Figure 3-3
Historical Great Salt Lake Hydrograph

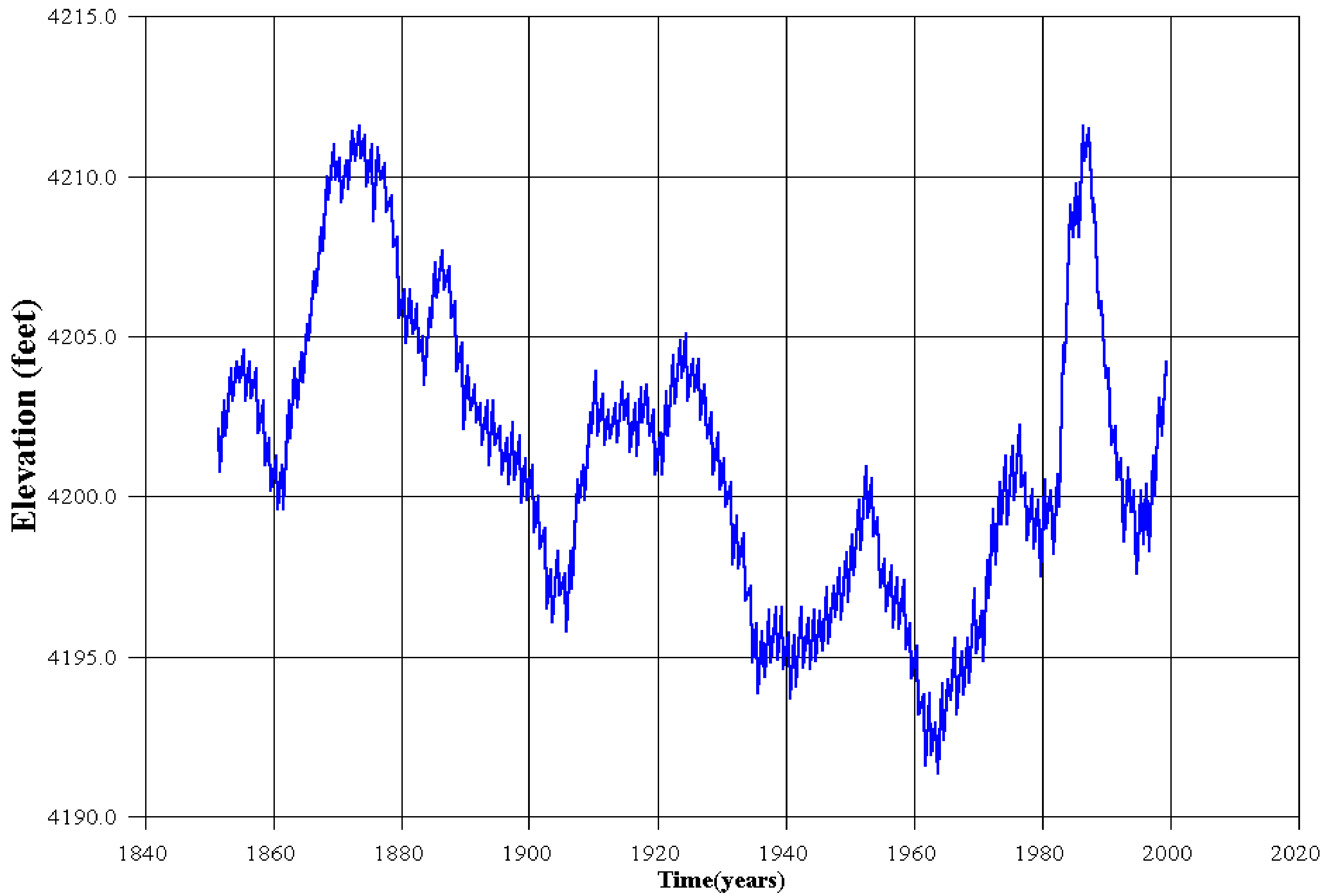


Figure 3-4
North and South Arm Elevations of the Great Salt Lake, 1981-1999

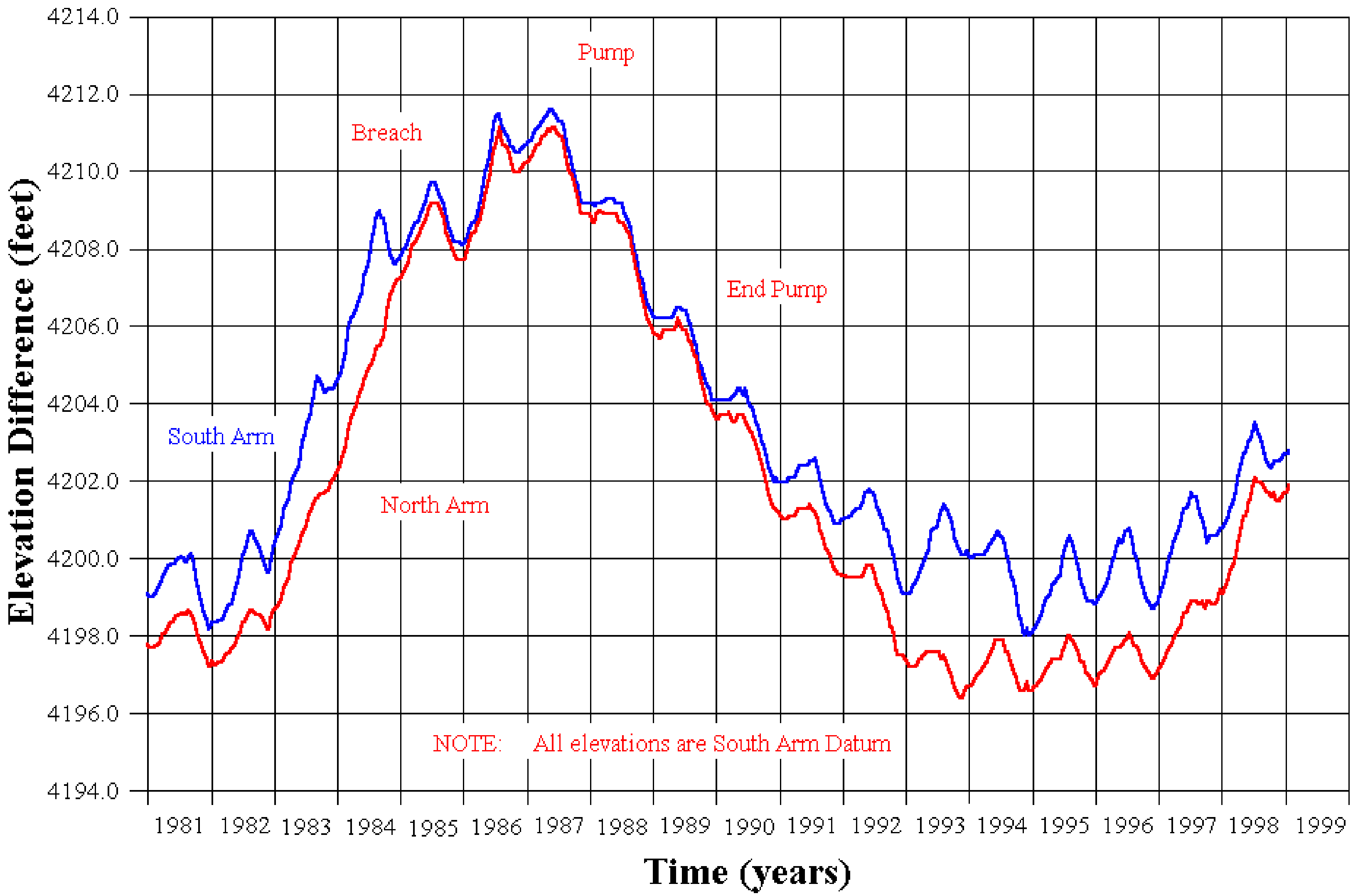
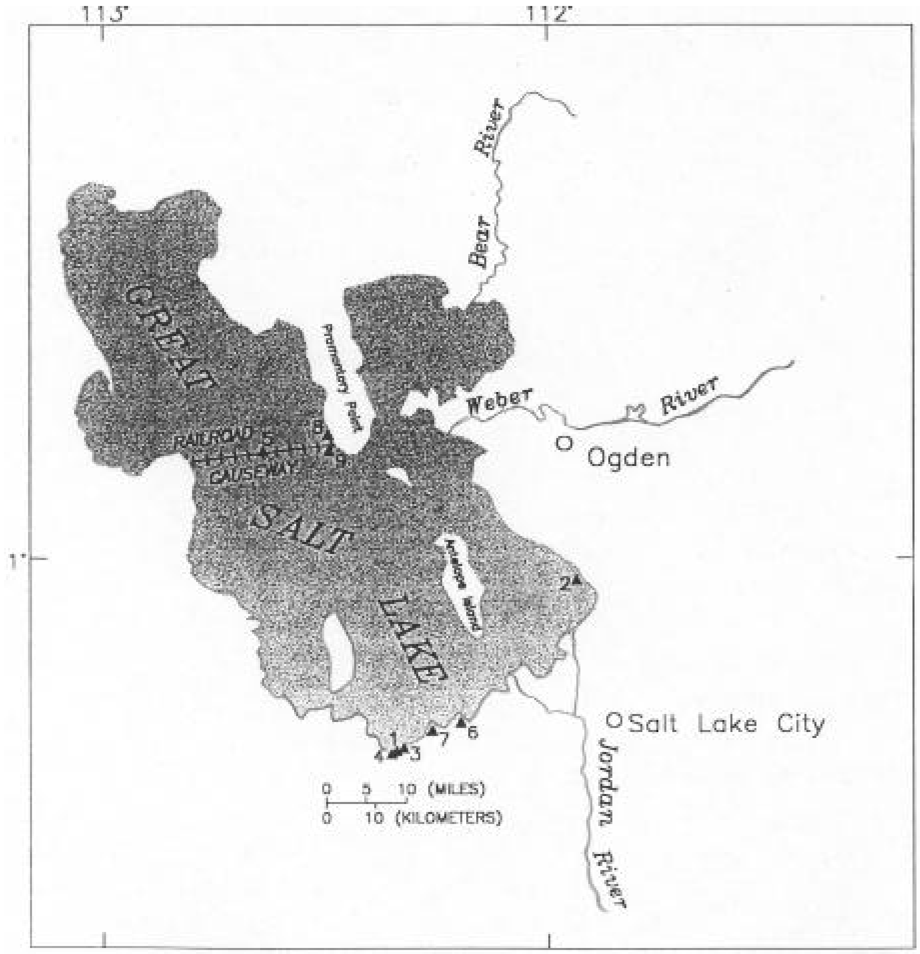


Figure 3-5
Gage Locations Used to Determine the Level of the Great Salt Lake, 1875-1983



EXPLANATION

NUMBER	GAGE	PERIOD OF RECORD
1	Black Rock	1875–1876
2	Farmington	1876–1879
3	Lakeshore	1879–1881
4	Garfield	1881–1901
5	Midlake	1902–1903
6	Saltair	1903–1938
7	Salt Lake County Boat Harbor	1939–Present
8	Saline	1966–Present
9	Promontory Point	1968–1982

properties (return periods) of high lake levels. Using a two-year logged precipitation runoff function, they estimated the lake level would climb above elevation 4208 about once in 500 years.

Hydrologic Model For The Lake

To facilitate analysis of the hydrologic system of the Great Salt Lake with a minimal amount of input data, the Utah Division of Water Resources (Stauffer 1985) developed a water balance model of the lake based only on annual input data. The model is able to predict June 1st as well as end-of-the-year lake elevations either for present water use conditions, or with projected future additions to or depletions from the system.

As a result of the lake's rising levels, the Division of Water Resources and the Utah Water Research Laboratory at Utah State University jointly updated the water balance model and developed a stage damage model and stochastic generation model of the lake. The water balance model was modified to include the effects of the Southern Pacific Transportation Company causeway, using recalibrated data for the 1944-83 period. The updated model has been used extensively for studying lake management/control alternatives for the lake. A summary of results of this effort was published by James, et.al (1984) at the Utah Water Research Laboratory.

Drainage and Import-Export for the Great Salt Lake Basin

The Great Salt Lake drainage basin is a closed basin covering approximately 22,000 square miles surrounding the lake with the lake as the terminal point. Because the basin is closed, the only water supply to the basin is precipitation or imports from outside the basin. Outflow from the basin is either evapotranspiration (consumptive use) or exports. Because surface exports from the basin are very small and imports are less than 100,000 acre-feet annually, virtually all of the precipitation that occurs in the basin is either consumptively used, evaporated or stored in the basin.

The portion of the precipitation in the Great Salt Lake Basin that eventually enters the lake as surface flow collects into water courses until the water enters the lake. The major surface flows that enter the Great Salt Lake are from the Bear, Weber and Jordan rivers, the headwaters of which occur within 50 miles of each other.

Bear River

The Bear River rises on the northern slope of the Uinta Mountains in Utah at about elevation 11,000 feet above sea level. It flows a 500-mile horseshoe-shape course northward through Utah, Wyoming and Idaho, then southward back through Idaho into Utah and to the Great Salt Lake. Principal tributaries to the Bear River are Smiths Fork in Wyoming, the Malad and Cub rivers in Idaho, and the Logan River in Utah.

The Bear River is the largest surface inflow to the Great Salt Lake. It drains an area of about 6,800 square miles in Utah, Wyoming and Idaho. It is a vital supply of agricultural water in the Bear River Basin and its flows are regulated by several dams and diversions.

The nearest gaged station to the Great Salt Lake on the Bear River is near Corrine, Utah, located just north of the Bear River Refuge. The flow at Corinne is considered the contribution of flow in the Bear River to the Great Salt Lake. Flow records have been kept at Corinne since 1949 (with a period from 1957-63 missing). The missing flow data were estimated by direct correlation based on a gaging station 18 miles upstream from Corinne at Collinston, Utah.

Weber River

The headwaters of the Weber River begin in the Wasatch Mountains just south of the headwaters of the Bear River at the 10,500 foot elevation. The river flows in a northwest direction toward the Great Salt Lake. Some of the heaviest precipitation in the state of Utah occurs in the Weber River Basin. The drainage area of the entire basin is about 2,060 square miles. The natural flow varies markedly from year to year due to wide fluctuations in precipitation. The Ogden River is the major tributary to the Weber River in addition to many creeks which add to the flow. The flow has been stabilized by reservoirs that are part of the Ogden River, Weber River and Weber Basin projects.

A gaging station near Plain City is about six miles from the Great Salt Lake. Below this station the river branches into three distinct channels with numerous side channels. Because there is little data available below Plain City, the flow at Plain City is used for the Weber River contribution to the surface flow. The historical record for this station is from 1904 to the present.

Jordan River

The flow entering the Great Salt Lake from the Jordan River originates in the Uinta Mountains east of Salt Lake City. The main tributaries above Utah Lake are the Provo and Spanish Fork rivers that rise at 11,000 and 9,500 foot elevations respectively. The Jordan River begins at the outflow from Utah Lake and flows northward through Salt Lake County to the lake. Several streams from the west slopes of the Wasatch Mountains enter the Jordan River along its path to the Great Salt Lake. The basin area is 3,490 square miles. Heavy water demands are placed on the Provo-Jordan River Basin system, including municipal and industrial waters for the cities from

Payson to Bountiful along the Wasatch Front, and considerable agriculture in Utah and Salt Lake counties.

Flow records of the Jordan River are the poorest of the major inflows to the Great Salt Lake. The flow of the Jordan River near the Great Salt Lake was not measured until 1965, and the gaging at 2100 South Street in Salt Lake City began in 1943. The flow at the Jordan Narrows has been monitored since 1913. The major tributaries, Little Cottonwood, Big Cottonwood, Mill, Parleys, Emigration and City creeks along the Wasatch Front have been monitored for the last 50 years. But the diversion patterns for municipal, industrial and agricultural purposes are so complex that correlation between the sum of these flows and the flow at the lake is extremely difficult. For this reason, the combined flows of the Jordan River and surplus canal at 2100 South Street in Salt Lake City are used as the inflow to the Great Salt Lake.

Estimates vary on the amount of ungaged flow that enters the lake from other sources. Based on computer correlations, however, other surface flow entering the lake is approximately 8 percent of the combined Bear, Weber and Jordan rivers gaged flow (Stauffer 1985).

Dissolved Solids Inflow to the Lake

Studies by the USGS have indicated the Bear, Weber and Jordan rivers systems contribute approximately 60-80 percent of the surficial dissolved solids load entering the Great Salt Lake. The remaining dissolved solids come from small streams and canals. This inflow of dissolved solids amounts to approximately 2,150,000 tons per year. The water that enters the Great Salt Lake in the three main streams is quite different in chemical quality from the water in the headwaters of these streams. Most of the runoff in the three streams originates as snowmelt or rainfall on the Uinta Mountains and Wasatch Mountain Range, and this runoff is low in dissolved solids and of the calcium bicarbonate type, suitable for most any use.

In the lower reaches of the Bear and Jordan rivers, however, the dissolved solids increase because of evapotranspiration, return flow from irrigated lands, discharge of industrial and municipal wastes, and groundwater inflow: and the water type changes in these two streams as the major dissolved constituents become sodium, chloride and sulfate. In the Weber River, however, the dissolved solids do not greatly increase and the water type remains the same.

Table 1 shows the quality of water entering the Great Salt Lake from the three major tributaries during the water year 1975, and illustrates the ranges of some of the more common ion constituents.

Estimates of subsurface inflow volume vary from 275,000 acre-feet, reported by Peck and Richardson (1966) as the average for years 1937-61, to an average of 75,000 acre-feet for the years 1937-73, reported by Arnow (1978). Handy and Hahl (1966) reported a dissolved solids load of 1.2 million tons contained in 200,000 acre-feet of subsurface inflow during 1964. This flow for 1964 would have an average concentration of 700 mg/l total dissolved solids. The volume of subsurface inflow to the lake and the dissolved solids load are vary small compared to the total volume and load of the lake.

Climate and precipitation patterns over the Great Salt Lake are complex. Whelan (1973) states: “The climate of the area ranges from temperate-arid west of the lake, with an annual precipitation of 4.5 inches, to temperate semi-arid east of the lake, with an annual precipitation of 16 inches.” Precipitation on the surface of the Great Salt Lake is estimated to contribute 25 to 30 percent of the total inflow to the lake.

Climatological Differences

The climate of the Great Salt Lake Basin is dominated by the Sierra Nevadas some 500 miles to the west and the Rockies several hundred miles to the east. The mountain ranges forming the west coast chain modify the character of winter storms which move across the Great Salt Lake Basin. Most of the moist Pacific air which brings winter precipitation to the basin must move across these mountain barriers with consequent moisture loss. This climatic factor accounts for the semi-arid nature of the Great Salt Lake Basin.

Table 3-1
Ranges of Some Dissolved Solids Constituents of Major Tributaries Entering the Great Salt Lake Water Year 1975 (Values in mg/l)

Tributary	Silica		Calcium		Magnesium		Sodium		Potassium		Bicarbonate		Sulfate		Chloride	
	High	Low	High	Low	High	Low	High	Low	High	Low	High	Low	High	Low	High	Low
Bear River at Corrine	16	8	69	51	65	20	300	59	22	7	372	211	67	31	460	85
Weber River at Plain City	12	8	65	35	23	9	49	12	7	2	303	135	34	15	64	16
Jordan River at 5800 So. SLC	31	20	180	83	74	54	220	140	21	13	354	251	460	240	310	190

Data from *Water Resources Data For Utah*, U.S. Geological Survey Water Data Report UT-75-1, Oct. 1974 - Sept. 1975

The Rocky Mountains to the east of the Great Salt Lake Basin also have a marked moderating influence on the climate of the basin. The mountains prevent the westward penetration of all but exceptionally strong outbreaks of cold continental air. These cold masses which sweep across the central states in winter have their source in the snow-covered plains of northern Canada and the ice-covered wastes of the Arctic.

Topography of the basin influences the climate in another way. Air over the surrounding mountain slopes is cooled during the long winter nights by relatively colder surfaces of the slopes. This colder air flows down the slopes and canyons and collects at the bottom of the basin just as the water of old Lake Bonneville once did.

The semi-arid continental climate of the Salt Lake Basin is characterized by large variations in mean temperatures because of the effects of local topography. Mean average annual temperatures range from 53.2 degrees F. at Antelope Island located in the Great Salt Lake to 44.9 degrees F. at Snowville, Utah, about 20 miles north of the lake. Temperatures along the southern and western shores of the lake range between 41 and 52 degrees with slightly colder mean temperatures of 49 and 50 degrees F.

The warmest temperature recorded at an official station in the basin was 112 degrees F. at Wendover, Utah, on July 13, 1939. The second highest, 111 degrees F., was observed at Antelope Island on July 25, 1959. The lowest temperature recorded near the lake was -32 degrees F. at Corinne, Utah, on Christmas Day 1924. The record low temperature was measured at -44 degrees F. at Lewiston, Utah, in Cache Valley. ⌂

Chapter 4

Great Salt Lake Contingency Plan

The Department of Natural Resources and Energy published a report in January 1983 titled, *Recommendations for a Great Salt Lake Contingency Plan for Influencing High and Low Levels of the Great Salt Lake.* The contingency plan was prepared within parameters of the December 16, 1976, *Comprehensive Plan for Managing the Great Salt Lake.* The contingency plan of the Department of Natural Resources and Energy contained recommendations to meet the then legislative mandate for maintaining the level of the lake below 4202 feet of elevation (*Utah Code Annotated, 1953*, as amended in 1979 Title 65-8a-7).

At the time, the news media and others made light of the law that required the lake to be maintained below elevation 4202. The contingency plan, however, focused on the concern from industry that resulted in recommendations to the governor that something needed to be done to protect important investments and resources near the lake. In retrospect, it is interesting to note that the contingency plan was not prepared because of the abnormal high precipitation and flooding that occurred in September 1982. The plan was prepared because the lake since 1963 had been rising at an alarming rate and was predicted to reach elevation 4202 feet by 1983.

As early as 1976, the Great Salt Lake Technical Team started looking at the rising lake level. Ideas like pumping lake water into the West Desert, diking, and upstream development were being discussed.

The 1983 contingency plan basically gave a brief history of lake fluctuations and analyzed what it called the three most likely trends for future lake levels. They were:

1. Most likely lake level trend: elevation 4207 feet by the year 2025.
2. Most likely low lake level trend: elevation 4189 feet by the year 2010.
3. Most extreme, yet possible, high lake level trend: elevation 4210 feet by the year 1998.

The third trend, elevation 4210 feet by the year 1998, viewed as the most extreme, was actually quite close in elevation but not accurate in timing. The lake reached elevation 4211.85 by June 5, 1986.

The plan also assembled information about alternatives that had been previously studied, such as pumping water from the lake into the West Desert, breaching the Southern Pacific Railroad Causeway, diking low areas around the lake, and storing/developing water upstream to the Great Salt Lake before it enters he lake.

The contingency plan was the first report to recommend pumping water into the West Desert as the best short-term solution to lake flooding. It further suggested that long-term solutions might include development of the Bear River, creation of fresh water ponds in the north end of Bear River Bay, or possible development of a peak power project in Puddle Valley.

The plan concluded, ". . . there are presently insufficient data on which to base firm action recommendations," and urged that additional feasibility analyses be completed.

This conclusion led to donations from private capital to begin a feasibility analysis of pumping water from the Great Salt Lake into the West Desert. ⌂

Chapter 5

Investigation of Great Salt Lake Flood Protection Alternatives

Studies and investigations to find alternatives to protect development around the Great Salt Lake did not start with the high levels of the 1980s. The Great Salt Lake has historically experienced wide cyclic fluctuation of its surface elevation, which has continually plagued those who have utilized its shores.

During the period from 1940-1965, when the lake was relatively low, it was thought by many that the lake would remain low or even dry up. Development around the lake during this period included large wildlife management areas at the mouths of the rivers, large evaporation ponds in low areas for the salt extraction industries, major roads and railroads across and along the shores, recreation facilities, and a causeway connecting the east shore to Antelope Island.

A peak elevation of 4202.3 feet was reached in 1976 that prompted a renewal of public awareness of the lake and problems associated with high levels of the lake. This public awareness provided new legislative support to state agencies and universities to address problems related to flooding problems around the lake.

Industries along the lake's shore at this time were experiencing a financial strain due to productivity losses and structural damage. Concern also existed for wildlife and recreational areas around the lake. In the three years following 1976, the lake level receded more than two feet (1977-78 was one of the lowest precipitation years on record). In September 1982, however, the lake began rising rapidly again due to abnormal high rainfall and an abrupt ending of the evaporating season. The continued high precipitation caused inflows of 7.5 million acre-feet in 1983 and 9.0 million acre-feet in 1984. This caused the lake to peak at elevation 4204.70 feet in June 1983 and at elevation 4209.25 feet in June 1984. The two successive rises of the lake (approximately five feet each) were the two largest rises of the lake in historical record.

Alternatives were proposed by the Division of Water Resources and many others were suggested by the public. Alternatives were grouped as follows:

1. Export flood flows from the Great Salt Lake drainage basin, mainly to the Bear River and Sevier River drainages.
2. Store water within the basin before it reaches the Great Salt Lake, mainly in reservoirs associated with the Bear River. Most of the reservoirs were part of ongoing studies on the Bear River to develop some of the Bear River flows.
3. Consume (through evapotranspiration) large amounts of water within the basin. This type of alternative would require large diversions to new agriculture lands. In some cases, water would be supplied from one of the reservoirs involving the Bear River. Only a couple of proposals matched this concept; one was the Utah Lake/Cedar Valley Pumping Project.
4. Continue letting the flood flow collect in the Great Salt Lake.

Many of the proposed alternatives dealt with flood flows and high lake level once the water was in the lake. These ideas included:

(a) breach the Southern Pacific Railroad Causeway to lower the south arm and raise the north arm,
(b) dike around the lake to protect major facilities and resources, close the Southern Pacific Causeway and divert the Bear River into the north arm to maintain the south arm lower than the north arm,
(c) build a pump/storage/power project in Puddle Valley, and
(d) pump water from the lake to the West Desert (West Desert Pumping Project).

The Division of Water Resources published a report in January 1984 entitled, *Great Salt Lake Summary of Technical Investigations for Water Level Control Alternatives* that summarized alternatives associated with the Bear River, Utah Lake and the Great Salt Lake, except diking. Alternatives addressed in the report are shown in Figure 1, and Table 1 lists alternatives studied and primary investigators.

A major study, *Great Salt Lake Diking Feasibility Study*, was prepared in December 1984. In 1984-85, these ongoing investigations were summarized in the short report, *Great Salt Lake Summary of Lake Control/Management Alternatives.*

Table 2, among other things, lists projects, proposals, schemes, etc. as of 1984-85 which were identified for possible control/ management of the high levels of the Great Salt Lake. It shows the status of various investigations, shows construction time, identifies the type of control alternatives would provide, gives information about impacts to various uses, and shows effectiveness of alternatives to lower the lake level.

The table also shows which projects/schemes could be constructed on a short-term schedule and the effect they would have on lowering the level of the south arm of the lake. It shows that projects such as damming the north arm of the lake and pumping brine into Puddle Valley had a large potential to lower the lake level, but they were very costly and would take more than five years to construct. The table shows the West Pumping Project as the best alternative to deal with the existing problems with the high lake level.

During 1983, and to a limited extent in 1984, the Division of Water Resources under special assignment from the Executive Director of the Department of Natural Resources conducted technical studies on several alternatives to supplement existing data and to assess the feasibility of the alternatives. The studies, undertaken by the division in 1983, were summarized in its report *Great Salt Lake Summary of Technical Investigations Water Level Control Alternatives.*

The ongoing work in 1984 relates mainly to directions given to the Division of Water Resources through Senate Bill 97. Engineering studies being conducted by the division included water quality studies on the Bear River; investigations related to the South Fork, Avon and Oneida Narrows Reservoirs on the Bear River; the Cedar Valley Project; work on the West Desert Pumping

Figure 5-1
Great Salt Lake Basin with Locations of Water Level Control Alternatives

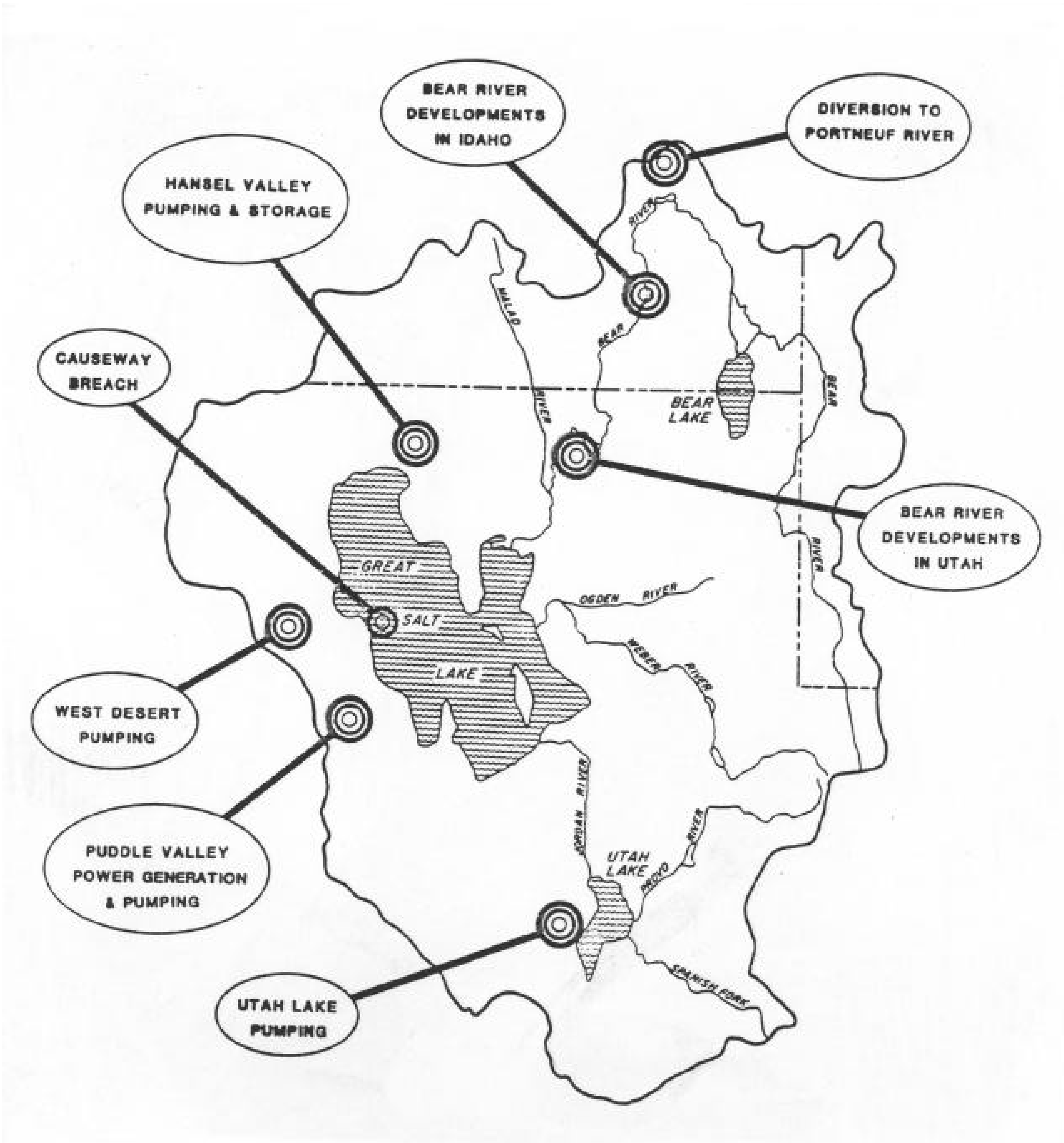


Table 1
Study Title and Primary Investigator (1983)

Study	Investigator
Bear River Basin	
Mill Creek Reservoir	PRC Engineering, Inc.
Amalga Reservoir	Dames and Moore
Honeyville Reservoir	Palmer-Wilding Consulting Engineers
Washakie Reservoir	Horrocks-Carollo Engineering
East Promontory/West Bay	PRC Engineering, Inc.
Lampo Reservoir	Palmer-Wilding Consulting Engineers
Oneida Narrows Reservoir	Division of Water Resources
Cutler Reservoir Enlargement	Division of Water Resources[a]
Soda Springs Reservoir	Idaho Water Resources Board[b]
Diversion to Portneuf River	Higginson-Barnett Consultants
Hansel Valley Storage	Division of Water Resources
Great Salt Lake	
West Desert Pumping Alternative	Eckhoff, Watson & Preator
Puddle Valley Pumping & Power Generation	Utah Water Research Lab. - USU
S.P.R.R. Causeway Breach	Division of Water Resources[c]
Evaporation & Precipitation Effects of WDP	No. American Weather Consultants
Damages Incurred Due to Rising Lake Level	Bureau of Economic and Business Research - U of U
Hydrologic Modeling	Utah Water Research Lab. - USU
Utah Lake	
Pumping to Maintain Compromise Elevation	Division of Water Resources
Pumping for Irrigation Projects	Division of Water Resources[d]

[a]Some information provided by Harza Engineering through courtesy of Utah Power and Light Company.
[b]Study done in 1981 for state of Idaho by J-U-B Engineers, Inc.
[c]Information provided by Southern Pacific Transportation Company.
[d]Cooperative Studies with Bureau of Reclamation, July 1971-April 1972.

Table 2
Summary of Great Salt Lake Control/Management Alternatives, 1984-85

	Status of Project Investigations[1]				Schedule for Construction, Etc.			Type of Control in Terms of Storage, Evap., Etc.						Impact to Various Uses. Positive or Negative[2]							Status of Cost Information[3]			Effect on Lowering the S.A. Peak in inches	
PROJECTS/SCHEMES	Reconnaissance	Feasibility	Final Design	Constructed	Short, 0-2 yrs.	Medium, 2-5 yrs.	Long, 5- yrs.	Export from Basin	Storage above GSL	Storage in/near GSL	Consumptive Use	Evaporation	Lake Level Adjustment	Lake Level Control	Agr., M&I Uses	Recreation	Fishing-Wildlife	Major EIS Prob. Yes-No	S.A. Mineral Ind.	N.A. Mineral Ind.	Status of Data	Capital Costs in Million $	O & M Costs	First Year Operation	Reduction of Peak Levels under Conditions 1983
Bear River Basin																									
Mill Creek Reservoir		C					X		X		X			S	L	L	M	?	S	S	P	16.6	n/a	0.3	2.0
Amalga Reservoir		C					X		X		X			M	L	L	M	?	S	S	P	59.3	n/a	1.6	6.0
Honeyville Reservoir		C					X		X		X			S	L	L	M	?	S	S	P	35.6	n/a	1.1	4.0
East Promontory/West Bay Res.		C			?	X				X		X		M	?	L	M	?	M	M	P	31.5	n/a	3.2	6.0
Oneida Narrows Reservoir	C	U					X			X		X		S	L	L	?	?	S	S	R	15.6	n/a	1.3	3.0
Cutler Reservoir Enlargement	C	P					X			X		X		M	L	L	M	?	S	S	R	194.8	n/a	2.8	9.0
Soda Springs Reservoir		C					X			X		X		S	L	L	?	?	S	S	P	16.4	n/a	0.4	2.0
Hansel Valley Storage	C						X			X		X		S	L	?	?	?	S	S	R	141.3	n/a	7.0	3.0
Diversion to Portneuf River	C	P			?	X		X						M	?	?	?	Y	S	S	R	9.1	n/a	2.0	12.0
Avon Reservoir		U					X		X		X			S	L	L	?	?	S	S	-	n/a	n/a	0.3	2.0
Div. Bear River Water to S.L.C.		U					X				X			S	L	?	?	?	S	S		n/a	n/a	n/a	4.0
Washakie Reservoir		C					X		X		X			S	L	L	M	?	S	S	P	75.0	n/a	1.7	5.0
Lampo Reservoir		C					X		X		X			S	L	L	M	?	S	S	P	14.7	n/a	0.2	1.0
Rozel Irrigation Project	U						X				X			S	L	O	O	?	S	S	-	n/a	n/a	n/a	3.0
Utah Lake/Cedar-Rush Valley Project																									
Pumping to Maintain Compromise	C	U			?	X			X		X			S	O	?	M	?	S	S	R	70.1	n/a	3.7	
Pumping for Irrigation Project	C	U					X		?	X				S	L	?	?	?	S	S	R	37.6	n/a	0.8	6.0
Great Salt Lake																									
S.P.R.R. Causeway Breach				C	*						X	X		M	O	O	?	?	L	N	C	3.7	n/a	10.0	10.0
West Desert Pumping		C	P		X					X		X	?	L	O	O	O	Y	L	L	F	40.0	4.0	20.0	40.0
Puddle Valley Storage	C					X	?			X		X	?	L	O	?	?	Y	L	L	R	50.0	27.0	30.0	0.1
Puddle Valley Pump Storage	C						X			X		X	?	L	O	?	?	Y	L	L	R	130.0	47.0	30.0	0.2
Damming Bear River Bay	U	P				X				X		X	?	S	?	L	?	Y	L	N	-	30.0	n/a	12.0	0.1
Damming North Arm GSL	U						X			X		?	?	L	O	?	?	Y	N	L	-	400.0	n/a	60.0	45.0
Total Diking East Shore Areas	C	P				X	?			X		?		O	O	?	?	Y	N	N	R	200.0	n/a	-0.0	-0.0
Selective Diking East Shore Areas	U	P			X					X		?		O	O	?	?	Y	N	N	R	25.0	n/a	-0.0	-0.0

[1]C investigation complete at that status, U investigation underway at that level, and P proposed investigation at that level.
[2]L large benefit, M medium benefit, S small benefit, O no benefit, N negative benefit, and ? unknown benefit.
[3]R reconnaissance cost estimate, F feasibility level cost estimate, D final design cost estimate, C cost as constructed and n/a cost not available.

Alternative; and some in-house reconnaissance-level investigations of proposals to dam the north arm of the Great Salt Lake, dam the Bear River Bay, and selective diking along the east shore of the lake.

In general, the analysis of these alternatives, together with economic and political aspects, led to the following considerations.

(1) Export flood flows from the Great Salt Lake Drainage Basin - Diversion to Portneuf River was evaluated, but costs were three to four times more than West Desert pumping with only a third of the effect on the lake. Many other concerns were raised in any proposal to add flood flows to streams outside the basin which may also be at peak flows.
(2) Store water within the basin - Investigations showed that these alternatives (when analyzed as a Great Salt Lake flood alternative) required five-plus years to construct, would have very small impact on the level of the lake, and were more expensive than West Desert pumping.
(3) Evapotranspiration of new amounts of water within the basin - These alternative were included in the Utah Lake/Cedar-Rush Valley Project. They also were medium to high time for construction, costly compared to West Desert pumping, and would have very small effect on the level of the Great Salt Lake. Further, it was uncertain if such a project would have a water supply during average water years.

Eventually, the evaluation of alternatives led to the recommendation that West Desert pumping was the alternative that could be constructed in a short period (less than two years), would have a major impact on the level of the Great Salt Lake, and would have the best benefit-to-cost comparison.

Results of these collective investigations led to an overall concept for dealing with the flooding Great Salt Lake.

(1) Allow/use the natural terminal point of the lake to store flood flows and find ways to remove them once captured in the lake.
(2) Breach the causeway to reduce the head difference between the north and south arms of the Great Salt Lake. This, in effect, would lower the south arm where much of the flood damages occurred.
(3) Build the West Desert Pumping Project to remove water from the lake through evaporation. The project would evaporate up to 800,000 acre-feet of additional water from the Great Salt Lake.
(4) Build certain dikes to protect infrastructure around the lake. These included dikes around major sewage treatment plants. Raise parts of roads around the shore areas of the lake, including areas of I-80 and I-15. Also raise major causeways/dikes within the lake, including the railroad causeway. ⌂

References:

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Chapter 6

Preliminary Design of the West Desert Pumping Project

Introduction

A *Final Report West Desert Pumping Alternative - Great Salt Lake* was prepared in December 1983 for the Utah Division of Water Resources. This report documented the engineering feasibility studies conducted for the division to review and evaluate the proposed action plan and other information related to the West Desert area.

This chapter summarizes findings from that report. In general, the report concluded that the project was feasible and identified major elements of the project. The project consisted of a pumping plant, system of canals and ponds, containment dikes and return brine conveyance system.

West Desert Pumping - Basis of Design

The Great Salt Lake West Desert is in the Basin and Range province, which is characterized by flat basins separated by abrupt north-south tending mountain ranges. The basins were further flattened by deposition in Lake Bonneville, which covered this region. Uniquely, the West Desert basins are nearly the same elevation as the lake.

As described in the *Great Salt Lake Contingency Plan*, the West Desert Pumping Alternative was one of several alternatives identified to deal with controlling or managing high levels of the Great Salt Lake. The pumping alternative was the plan to pump water from the Great Salt Lake (south arm) into the desert west of the Great Salt Lake and evaporate it.

This alternative was first investigated by the Sacramento District, Corps of Engineers for the Great Salt Lake Hydrologic Subcommittee in July 1976. The report by the Corps of Engineers was later published as Appendix C in the *Great Salt Lake Hydrologic System Management Alternatives Report* by the Division of Water Resources in May 1977.

The Corps investigated three alternatives that might achieve net evaporative losses of 310,000, 380,000 and 850,000 acre-feet annually. Computer model evaluation of these alternatives indicated that larger amounts of net evaporation would be necessary for the pumping alternative to have some reasonable ability to “manage" high levels of the Great Salt Lake. The alternative (with its various options) considered in this investigation was, therefore, initially as large as possible. The goal was to identify a pumping alternative with a net evaporation in excess of 1,000,000 acre-feet per year.

In order to avoid filling the shallow desert basins with precipitated salt, it would be necessary to convey concentrated brines back to the lake. This step would also avoid mineral depletion in the lake. A similar salt problem mandated that the south arm brines of the Great Salt Lake be the source for pumping. Because south arm brines are a lighter density (less salty) than the north arm brines, they can be evaporated/concentrated easier and to a larger extent before salt crystals begin to form.

Although the Great Salt Lake is lower in elevation than the area identified for the West Desert Pond, lift pumping could help transfer large quantities of water from the lake to the pond. Under natural conditions, about 300,000 acre-free of water could be contained in the West Desert without flooding facilities in the Bonneville Basin east of Wendover, or without flow returning to the Great Salt Lake. Using dikes and levees could substantially increase the storage volume of the system.

An ideal arrangement would be to construct one retention dike near the west edge of the lake. This would minimize construction and pumping costs. However, the Hill Air Force Range uses large portions of the desert for weapons testing, and these facilities must be protected and maintained. Railroad tracks and embankments must be likewise protected from flooding and wave action. Similar protection must be provided for I-80.

Elements of the Project

Major elements of the system, Figure 6-1, are:

A. A breach of the Southern Pacific Transportation Company (SPTC) railroad **Causeway** to allow Great Salt Lake south arm brines to flow through an **Intake Canal** originating near railroad facilities at Lakeside to a **Pumping Plant** about 10 miles west on the east side of Hogup Ridge. An **Isolation Dike** would protect the canal from north arm wave damage.

B. An **Outlet Canal** or **Discharge Canal** cut through Hogup Ridge would carry the brine flow from the **Pumping Plant** to a **West Pond** that would have a surface area of about 375,000 acres and the capacity to evaporate approximately 840,000 acre-feet of water per year.

C. **Railroad Dikes** to protect the SPTC facilities from **West Pond** wave and water damage.

D. A **Bonneville Dike** to keep **West Pond** water out of the Bonneville Salt Flats and off I-80.

E. A **Newfoundland Dike** to retain the **West Pond**, with a **Flow Control Weir**.

F. An **Overflow Canal** that would deliver **West Pond** overflow to a secondary evaporation pond, the **East Pond**, with a surface area of nearly 88,000 acres and the capacity to evaporate about 220,000 acre-feet of water per year.

G. An **East Pond Dike** would lie parallel to the **Intake Canal** and serve as a final retention dam. It would also contain a **Flow Control Structure** to regulate the level of the pond and the return flow to the lake.

H. A **Return Brine Canal** would deliver concentrated brines back to the lake.

Comparison of Pumping Plant Alternatives

Summary

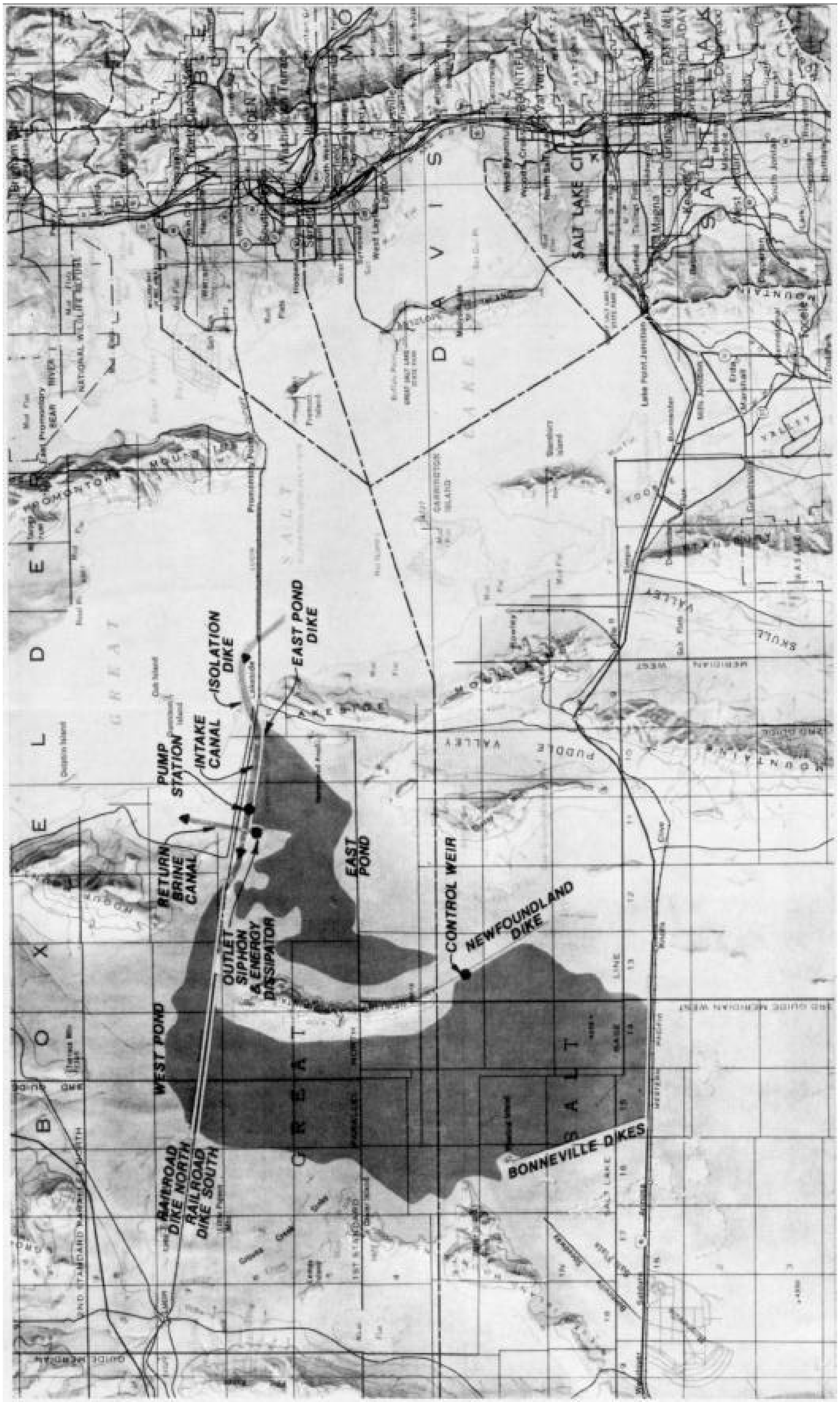
A feasibility study of the Pumping Plant was made in 1983.

A study and evaluation of diesel and electric pump drives indicated that diesel drives offered lower capital costs and better operating characteristics, but they had higher operating costs.

Design of the Pumping Plant was basic, and no special construction difficulties were anticipated. Construction of the Pumping Plant would take an estimated 18 months from the date of pump and engine order to placement.

Preparation of contract drawings and specifications would require eight months. Bidding would require about two months. The capital cost of the Pumping Plant was estimated at $24.52 million for the electric motor drive and $15.88 million for the diesel drive alternative. Pumping salt water with the density and chemical composition similar or equal to that of the Great Salt Lake is common. On the Great Salt Lake, pumps have successfully operated over long periods of time with routine maintenance adapted to the specific pumping

Figure 6-1 West Desert Pumping Plan, Preferred Alternative



conditions. Existing designs of pumps of the type and capacities as may be required for this project were available from several U.S. and foreign manufacturers.

The same held true for electric motors and/or diesel engines which were considered for this project. Special attention, however, was given to filtering the combustion air of the engines and cleaning their heat exchangers. If motors and/or engines were to be installed within a building, no unusual maintenance problems were anticipated, even with long periods of shutdown.

Pumps - Pumps could be made from several suitable materials which had been used under similar conditions. For the bowls, the following materials were acceptable:

- Stainless steel castings
- Ni-resist castings
- Aluminum bronze castings
- Coated cast-iron castings

Column pipes and discharge heads are commonly made from carbon steel, epoxy coated on the outside and neoprene coated inside.

Shafts were to be made from stainless steel, and impellers could be cast from either stainless steel, Ni-resist, nickel-aluminum, or aluminum bronze. Auxiliary mechanical equipment, such as valves, sump pumps, gates, fans or air compressors, have successfully operated in similar environments.

Engines - Four types of pump drives were considered in this study: gas turbines, diesel-electric generators, electric motors and diesel engines.

- Gas turbines are less efficient than diesels and not economical. In addition, their gear speed reducers with ratio of roughly 1:50 are expensive and not entirely trouble-free. These findings were based on a study of similar pumping plants in California.

- Diesel-electric generators, while fairly efficient when compared to direct diesel drives, are more expensive in capital cost . Additional electric motors and switchgear do not offer any special advantages.

- Electric motors are the preferred drivers for pumps whenever electric power is available. They offer the advantages of simplicity of installation and economy of operation and are easily controlled. However, in this project, the cost of an electricity transmission line would be very high, about the same cost as the Pumping Plant itself.

- Use of diesel engine drives, while more expensive than electric motors, eliminates the need for a transmission line and results in a very large savings of capital cost. In addition, power outages are avoided, eliminating a major cause of transient surges in the canals, thereby improving their slope stability. The cost of pumping, however, is higher than in the case of electric motor drives.

Sizing of Pumps - The design basis required a total installed capacity of 4,000 cfs at a static head of 23.5 feet and total dynamic head of about 28 feet. Pumping requirements for this project did not mandate a wide range of pumping capacity, nor was there a need for additional standby capacity.

Information obtained from pump manufacturers as to price, technical data and overall dimensions of pumps and drives is shown in Tables 6-1 and 6-2. The size of pumps would be 800 cfs capacity. Pumps larger than 800 cfs are seldom built, very heavy, and not as readily available.

An attempt was made to cost-analyze units in the range 500-800 cfs capacity, combining the cost of equipment with the cost of the pumping plant structure. Initial studies used a total flow rate of Q = 2500 cfs for three, four and five pumps, and these results are shown in Table 6-3. Later it was determined that this flow rate should be increased to 4,000 cfs, but the comparison for optimum pump size still remained the same. It became clear that the total cost of the plant did not appreciably vary for different size units within the specified drive alternative. The final selection of the unit size was then based on the practicality of operation and maintenance. This, of course, favored the least number of units. The 800 cfs units were then adopted for the plant layout. Units of this size offered the right combination of design, availability, compact plant structure, and simple operation and maintenance. One pump manufacturer (EBARA) independently also recommended pumps of this size for the project.

In addition to economic consideration, the recommendation to select five units was based on the following:

Table 6-1
Alternative Pump Design Criteria and Cost Comparison

Vendors	Discharge Capacity (CFS)	Total Head (Ft.)	Speed (RPM)	Bore (Inches)	Req'd BHP (HP)	Req'd Submerg-ence (Ft)	Cost/Pump ($)	No. Pumps Req'd for 4000 cfs	Total Cost ($)
Allis Chalmers	800	30	208	108	3,278	11	7,000	5	3.500,000
EBARA	1,250 800	30 30	135 161	144 120	5,552 3,702	23 18	1,590,000 1,146,667	3 5	4,770,000 5,733,335
Hitachi America	1,250 800	30 30	111 136	160 128	4,920 3,300	13 12	2,600,000 1,750,000	3 5	7,800,000 8,750,000
Ingersoll Rand	800	30	205	108	3218	12	580,000	5	2,900,000
Mitsubishi	1,250 500	30 30	110 225	150 95	4,776 2,020	3 2	915,000 479,000	3 8	2,745,000 3,832,000
Peerless	267	30	360	80	1350	7	25,000	15	3,750,000
Johnson	250	23	295	75	1006	12	105,000	16	1,680,000
M & W Pump Co.	200	30	N/A	60	N/A	9	*500,000	20	*10,000,000
Voith	1,250	30	204	N/A	N/A	N/A	See Table 6-2		

* Includes cost of diesel/motor drive
Table reproduced from *Final Report, West Desert Pumping Alternative, Great Salt Lake*, EWP Engineering, et. al., December 1983.

Table 6-2
Alternative Motor and Drive Design Criteria and Cost Comparison

Vendor	No. Units Req'd for 4000 CFS	Electric Motor: Motor Voltage (V)	Electric Motor: Motor BHP (HP)	Electric Motor: Cost/Motor ($)	Electric Motor: Total Cost Motors ($)	Diesel Engine and Gear Reducer: Engine Speed (RPM)	Diesel Engine and Gear Reducer: Engine BHP (HP)	Diesel Engine and Gear Reducer: Cost/Engine ($)	Diesel Engine and Gear Reducer: Cost/Gear Reducer ($)	Total Cost ($)
Allis Chalmers	5	4,000	3,500	253,000	1,265,000	900	3,278	585,000	160,000	3,725,000
EBARA	3 5	4,150 4,160	7,000 5,000	1,865,000 1,340,000	5,595,000 6,700,000	750 600	7,500 5,000	1,410,000 953,333	650,000 546,666	6,180,000 7,499,995
Hitachi America	3 5	13,200 13,200	6,000 4,000	1,175,000 620,000	3,525,000 3,100,000	750 750	6,300 4,200	1,500,000 970,000	1,750,000 1,000,000	9,750,000 9,850,000
Ingersoll Rand	5	4,160	3,500	557,000	2,785,000	900	3,500	585,000	66,000	3,255,000
Mitsubishi	3 8	- -	- -	- -	- -	430 900	5,700 2,700	1,513,500 421,400	1,577,500 421,600	9,273,000 6,744,000
Peerless	15	4,160	1,400	125,000	1,875,000	1,800	1,410	140,000	55,000	2,925,000
Johnson	16	2,300	1,250	218,000	3,488,000	1,800	1,125	153,000	63,000	3,456,000
M & W Pump Co.	20	See Table 6-1								
Voith	3	N/A	N/A	*1,915,000	*5,745,000	750	N/A	*5,065,000		*15,195,000

* Includes cost of pump
Table reproduced from *Final Report, West Desert Pumping Alternative, Great Salt Lake*, EWP Engineering, et.al., December 1983.

Table 6-3
Comparison of Pumping Plant Capital Costs
Q = 2500 CFS
(in millions of dollars)

ITEM	DIESEL DRIVE PUMPS 3 UNITS	DIESEL DRIVE PUMPS 4 UNITS	DIESEL DRIVE PUMPS 5 UNITS	MOTOR DRIVE PUMPS 3 UNITS	MOTOR DRIVE PUMPS 4 UNITS	MOTOR DRIVE PUMPS 5 UNITS
1. MECHANICAL EQUIPMENT (Including pumps, drive units, valves, piping and all miscellaneous equipment inside the plant.	6.33	6.33	6.33	4.96	4.96	4.96
2. CIVIL CONSTRUCTION (Including all concrete, metal work and earthwork for the pump plant structure, forebay and afterbay.	3.66	3.49	3.47	3.40	3.26	3.26
3. APPURTENANT STRUCTURES (Including switchyard and transmission line items for electric motor drive and fuel storage and piping for diesel engine drive.	0.59	0.60	0.61	11.97	11.98	11.98
TOTAL CAPITAL COSTS	10.58	10.42	10.40	20.34	20.20	20.20

Table reproduced from *Final Report, West Desert Pumping Alternative, Great Salt Lake*, EWP Engineering, et.al., December 1983

• With five units, the horsepower requirement of each diesel engine would be approximately 3,800 BHP. This represents the upper range of standard design diesel engines. Engines larger than 4,000 HP are of special design, requiring longer delivery time for engine and replacement parts.

• Units of this size, besides being more readily available than larger units, can be handled by a mobile crane at times of installation and repair.

There was no advantage to selecting more than five units for the following reasons:

• Installation cost increased in proportion to the number of units.

• Required maintenance and total cost of replacement parts similarly tended to increase in direct relation to the number of units.

The cost of energy to pump the required total amount of water remains substantially the same, regardless of the number of units selected for this project, because overall deficiencies in unit sizes were comparable. Five units provided adequate flexibility for this type of operation.

If, and when, future conditions indicated a need for less pumping capacity, operating fewer units or for shorter periods would satisfy this requirement. If larger pumping capacity is required, an additional pump may have to be added.

Costs

Capital Costs - Capital costs were estimated for the 4,000 cfs diesel engine drive and electric drive pumping plant alternatives. These costs, including mechanical equipment, civil construction, and appurtenant structures for the pumping plant were estimated to be $15.88 million for diesel driven pumps and $24.52 million for motor driven pumps. The large difference between the capital cost of the diesel engine drive and the electric motor driven Pumping Plant alternative was the cost of a power transmission line. Cost of the transmission line was obtained from the Utah Power and Light Co. Cost of the major plant equipment was received from several domestic and foreign manufacturers. Cost of civil works was based on the then prevailing unit prices in the Salt Lake City area.

Cost of Operation - These costs were derived by separately estimating cost of energy and cost of operation and maintenance. Cost of electricity was based on the Utah Power and Light Co. Schedule No, 9, which was found to be most favorable for this type of operation. Cost of diesel fuel was obtained from several wholesale dealers in the Salt Lake City area.

The total annual costs for diesel driven pumps were estimated to be $6.21 million. Total estimated costs for motor driven pumps was $4.20 million.

Plant Economics - Economic analysis of the diesel engine drive and electric motor drive alternatives compared total present worth of both for various periods of operation. Present worth of capitalized operation cost was computed using a 30-year project life and a 7.5 percent discount rate.

These figures indicated that if the pumping operation started immediately after the plant completion, and lasted for more than five years, the electric motor drives were more economical.

Therefore, the final selection of the pump drives was to be based on the probability of the most likely hydrologic scenario. The electric drives possibly were more economical in every case if more favorable energy rates were negotiated with the electric utility.

Evaporation Pond Alternatives

The 1983 report evaluated five configurations of evaporation ponds, illustrated in Figures 6-2 through 6-6. They were:

- Range Highline Alternative
- Range Boundary Highline Alternative
- Lakeside Lowline Alternative
- North Railroad Lowline Alternative

- South Railroad Lowline Alternative

These pond configurations included the inundation of land belonging to the U.S. Air Force.

A 1984 update report evaluated pond configurations which did not inundate Air Force land. Illustrated in Figures 6-7 through 6-11, they were:

- Wendover Pond Alternative
- Bonneville Pond Alternative
- Newfoundland Pond Alternative (Counterclockwise Rotation)
- Newfoundland Pond Alternative (Clockwise Rotation)
- Boundary Ponds Alternative (Clockwise Rotation)

Basis of Comparison- - In the 1983 report the ponds were subjectively evaluated and compared against each other. Characteristics investigated for each configuration included:

- Amount of embankment required,
- Minimum U.S. Air Force property inundation,
- Minimum desert floor construction,
- Anticipated construction time,
- Actual survey data available

Recommended Alternative - The South Railroad Lowline Alternative, with west and east ponds, was preferred. Pond characteristics were:

	West Pond	East Pond
Water Surface Elevations		
High	4218.3	Not available
Low	4217.6	Not available
Average	4218	4214
Area	374,000 acres @ 4218	88,000 acres @ 4214
Volume	1,093,000 AF @4218	285,000 AF @4214
Average Depth	2.92 ft.	3.24 ft.
Discharge Structure		Weir
Radial gates	1,000 ft. long Elevation 4217.5	
Peak Discharge	1,700 CFS	1,700 CFS

Basis of Comparison - The five pond configurations evaluated in the 1984 report (Those that did not inundate U. S. Air Force property) are summarized in Table 6-4. They were compared on the following characteristics:

- Surface area
- Volume (capacity)
- Required inflow (pumping rate)
- Construction of embankments
- Excavation

Pumping alternatives which would allow full capacity operation for a GSL South Arm water surface elevation of 4205 and 4208 were evaluated. Estimated capital costs for each of the five pond configurations are shown on Table 6-5. The following are summaries of each pond configuration in the 1984 report which did not inundate Air Force land.

Wendover Pond Alternative

The Wendover Pond alternative was unique: it would result in a pond with the largest surface area and, hence, the largest evaporation potential then any other alternative considered. The Wendover Pond would inundate all of the West Desert's depression storage below elevation 4218, except for the area between Lakeside and the Newfoundland Mountains. This alternative would cover U.S. Air Force lands southwest of the Newfoundland Mountains, as well as east of Wendover (Wendover Bombing Range). It would also submerge the Bonneville Speedway and Salt Flats.

Bonneville Pond Alternative

The only difference between the Bonneville Pond alternative and the Wendover alternative was that the south of Highway 80 would not be inundated. The result was a loss of pond surface area and evaporation potential. This configuration did not cover the Wendover Bombing Range, nor did it require that Highway 80 or the Union Pacific Railroad be protected from inundation from the south. This alternative submerged the Bonneville Salt Flats.

Counterclockwise Newfoundland Pond Alternative

The Counterclockwise Newfoundland Pond alternative was merely a continued retreat in pond size from the preceding alternatives. It did not inundate the Bonneville Speedway or Salt Flats. As with the preceding two alternatives, it submerged the U.S. Air Force land southeast of the Newfoundland Mountains, and routed the return brine flow through the Air Force land east of the Newfoundlands.

Clockwise Newfoundland Pond Alternative

The Clockwise Newfoundland Pond Alternative was of the same physical size as the counterclockwise alternative. It differed only in that the return brine was not routed through Air Force land, but circulated back to the Great Salt Lake north of the Southern Pacific Railroad through a channel cut in the Hogup Mountains. This alternate continued to submerge Air Force land southwest of the Newfoundland Mountains.

The flow pattern in this alternative, which routed the return brine to the lake without traversing Air Force lands east of the Newfoundlands, was applicable to the Wendover Pond and Bonneville Pond alternatives as well, with only slight modifications. The difference in cost between the two Newfoundland Pond alternatives was a cost applicable to other alternatives for keeping the return brine flow off Air Force land.

It should be noted that by choosing this alternate return brine alignment, a cutoff dam for storm water would be created south of the Intake Canal Barrier Dike. Any seepage into this area would likewise be trapped. To alleviate this condition would require a storm water/seepage pump station. This would keep the entire area south of the Intake Canal basically dry, a condition which would benefit the Air Force, particularly as compared to the no-project alternative.

Boundary Ponds Alternative

The Boundary Ponds Alternative was of significant interest because it did not inundated nor allow return brine flow across Air Force land. This alternative had the smallest pond surface area and, hence, the smallest evaporation potential of any alternative considered.

Geotechnical Considerations

Geotechnical and groundwater investigations provided preliminary field and laboratory test information necessary to determine the engineering feasibility, preliminary design, cost estimates and construction scheduling for the proposed West Desert Pumping Project.

The following is a summary of the results of field investigations, laboratory testing, and preliminary design evaluations, and provides recommendations related to geotechnical and groundwater considerations. It should be recognized that this investigation was preliminary in nature; detailed field and laboratory information had not been developed.

Scope

In accomplishing the above purposes, the general scope of this investigation included the following:

1. A field program consisting of:
   a. The drilling, logging and sampling of 18 borings located within the proposed pond areas, along the proposed dike and canal alignments, and at proposed alternate pumping station locations. The borings were drilled to depths ranging from 12 to 35 feet below the existing ground surface. Soil sampling and rock coring were performed. Piezometers were installed in a majority of these borings at the completion of drilling.
   b. The excavation, logging and sampling of 38 tests pits to evaluate potential borrow areas and further evaluate dike and canal alignments.
   c. A geologic reconnaissance of the project area and geologic mapping of critical lake bottom, shoreline and terrace areas was performed to provide information prior to drilling and test pit excavations. This reconnaissance was performed to evaluate borrow materials and the alignments of the proposed canal and dikes, and to identify potential problems or hazards for the proposed project.
2. A laboratory testing program which provided preliminary information on the engineering characteristics of the existing embankment and foundation soils and bedrock, and the existing quality of the groundwater.

Figure 6-2
Range Highline Alternative

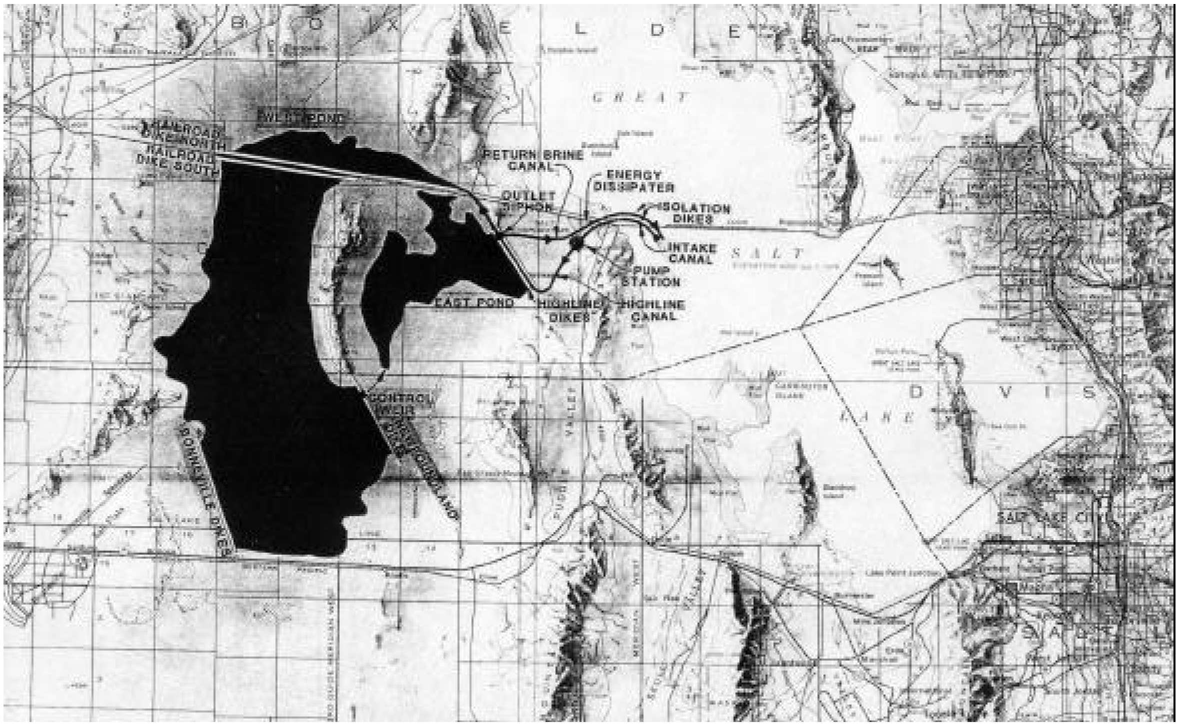


Figure 6-3
Range Boundary Highline Alternative

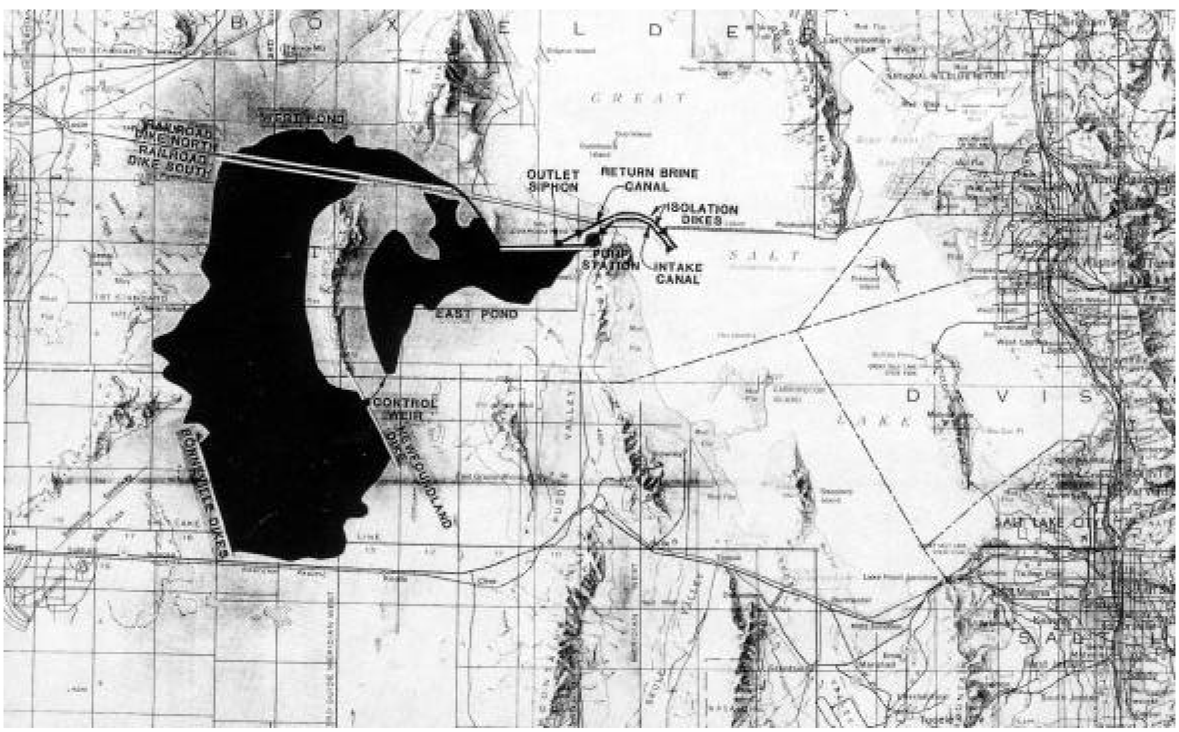


Figure 6-4
Lakeside Lowline Alternative

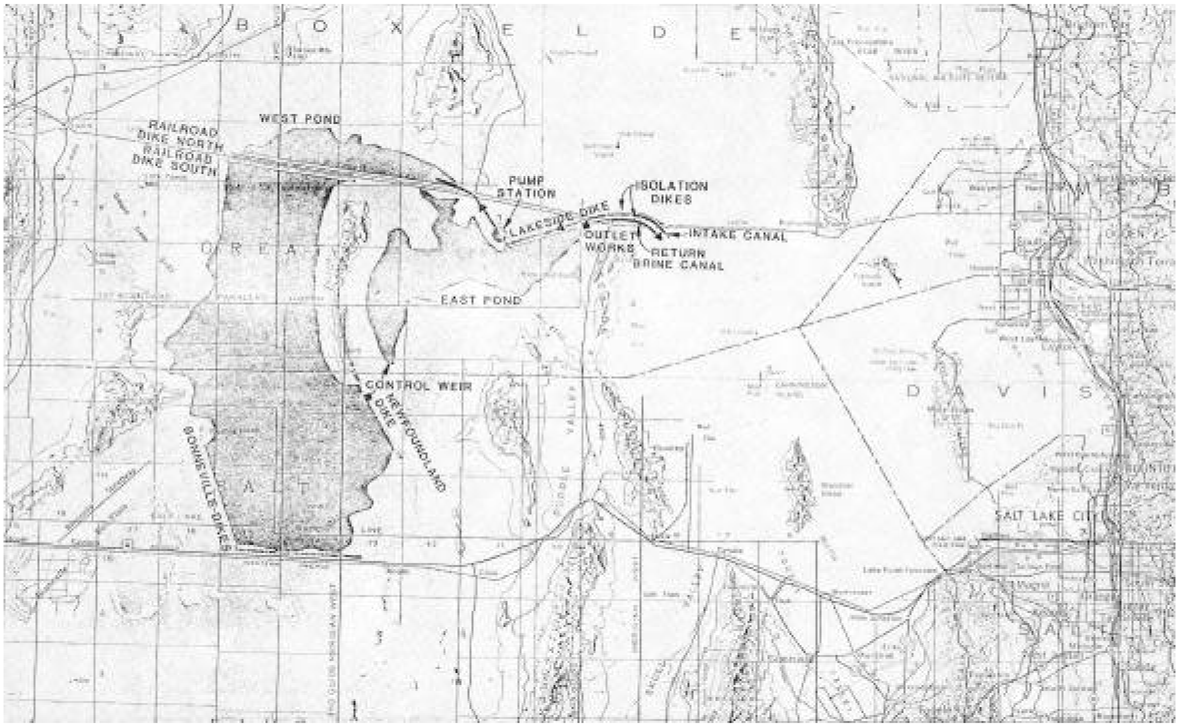


Figure 6-5
North Railroad Lowline Alternative

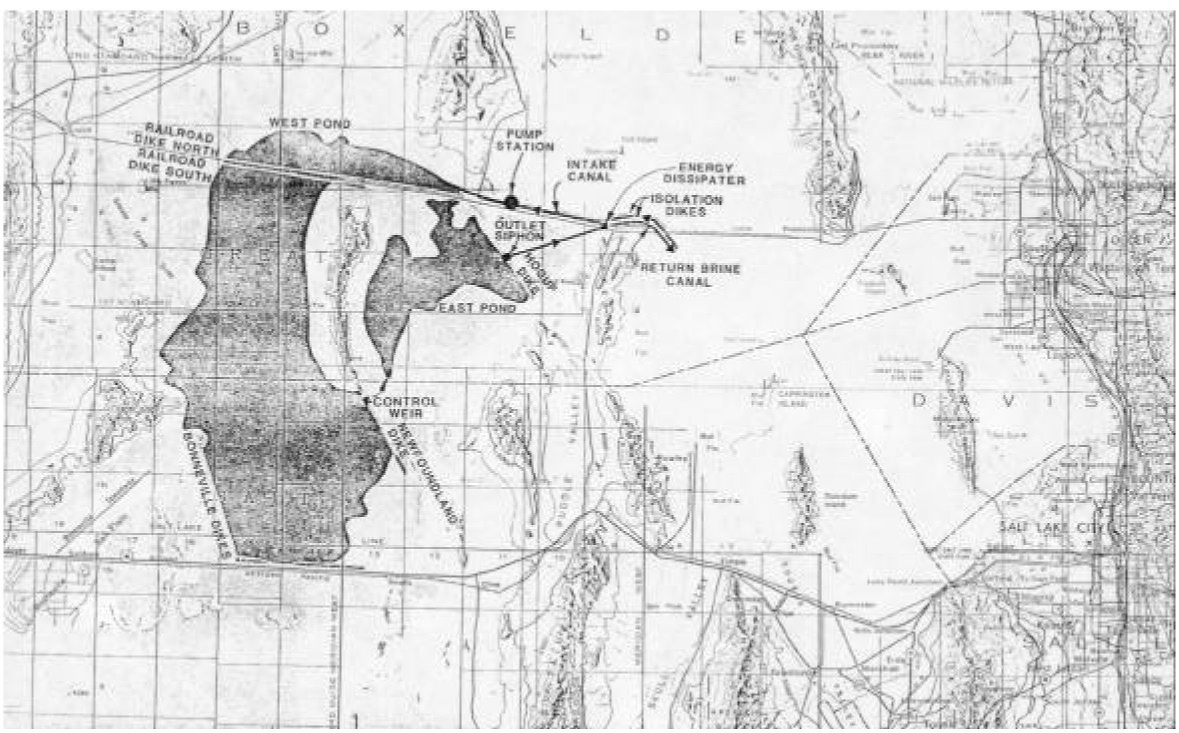


Figure 6-6
South Railroad Lowline Alternative

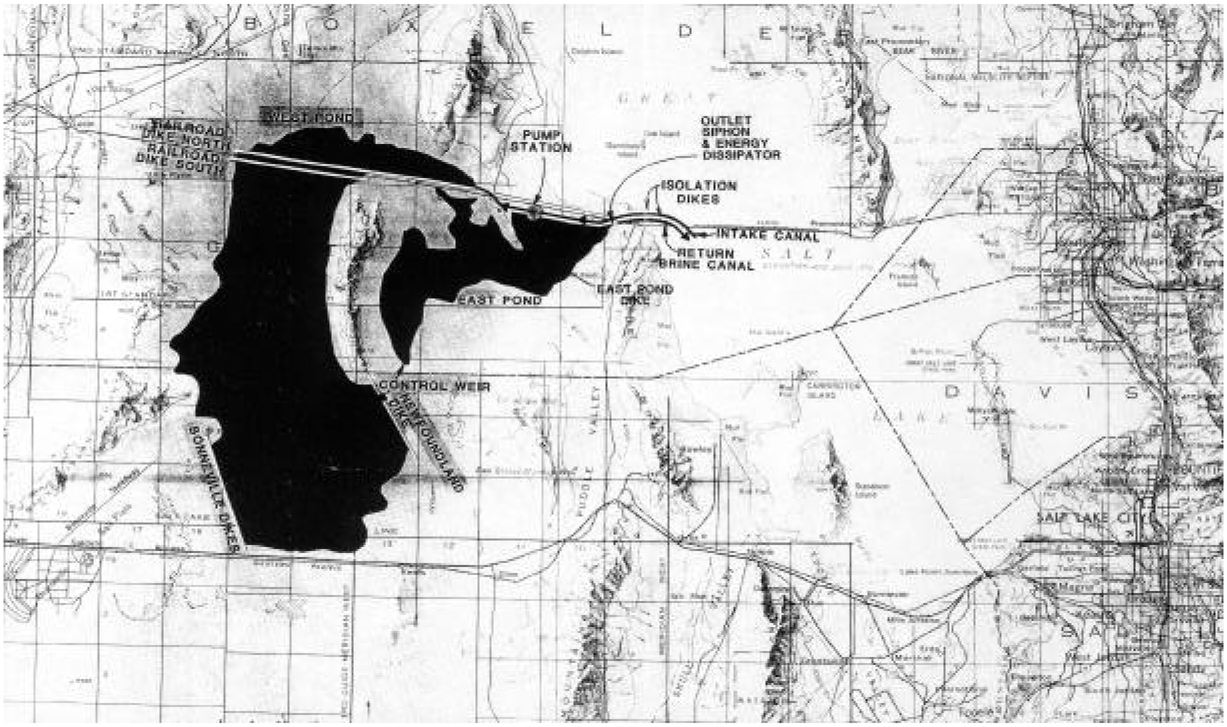


Figure 6-7
Wendover Pond Alternative

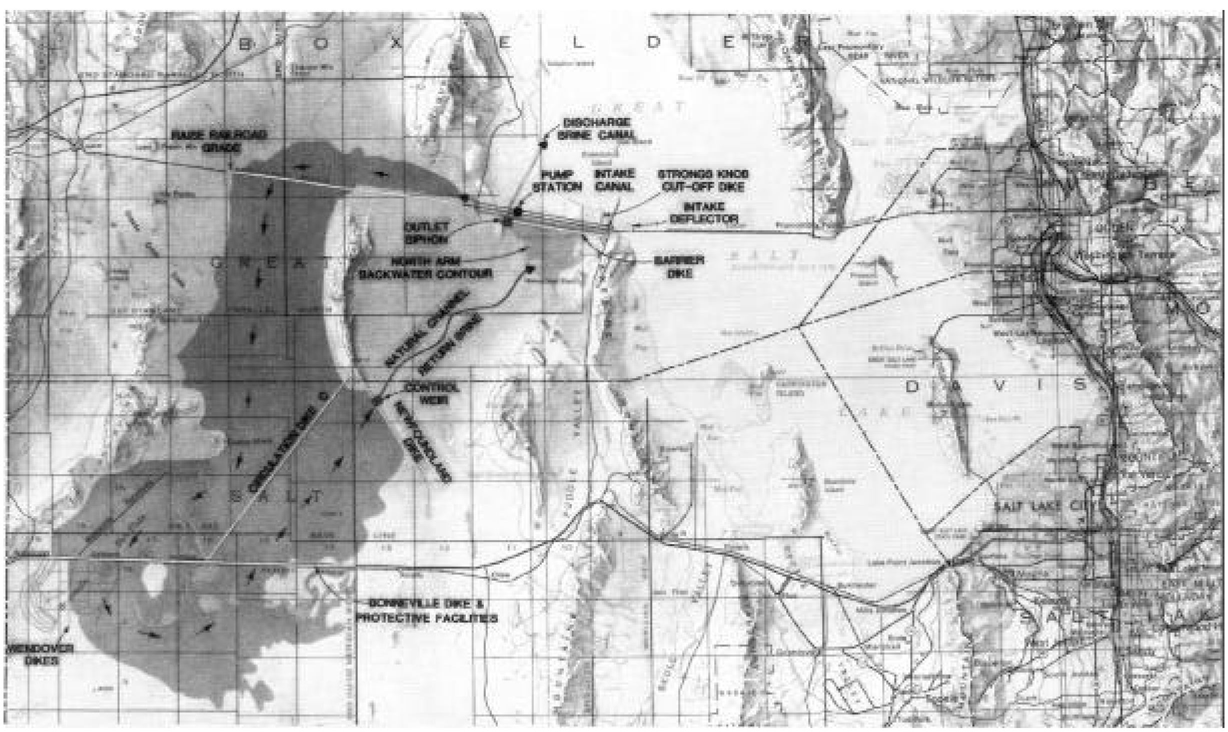


Figure 6-8
Bonneville Pond Alternative

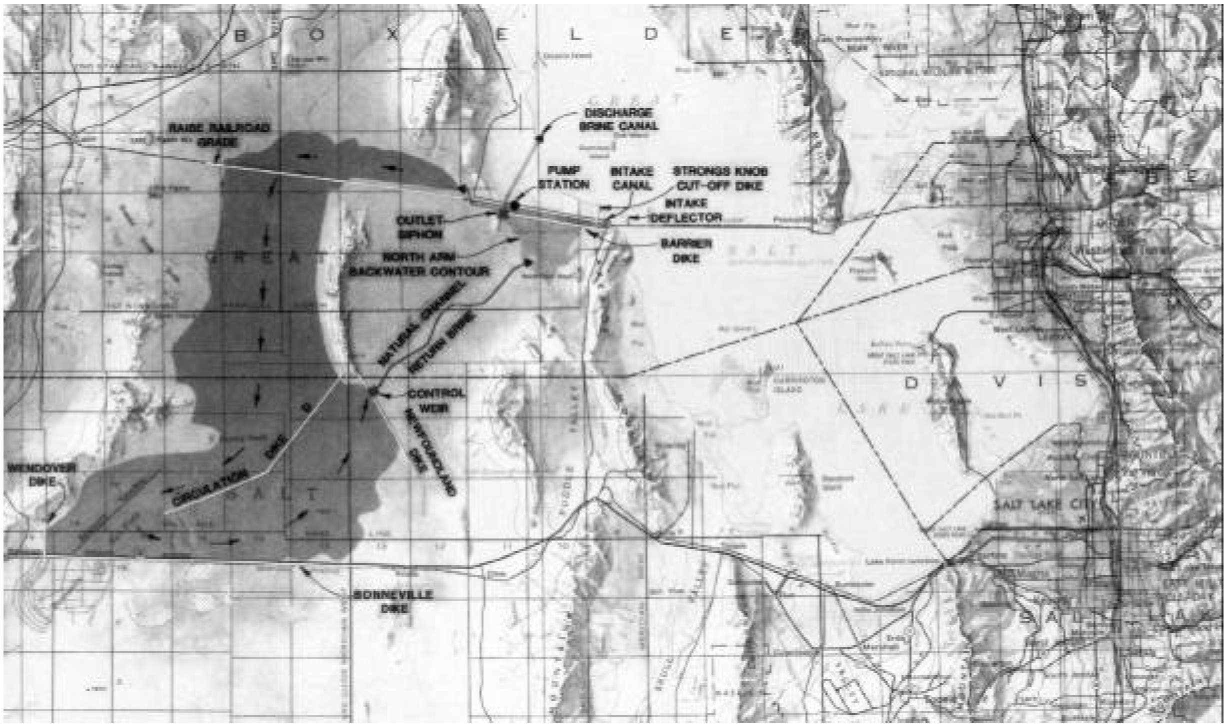


Figure 6-9
Newfoundland Pond Alternative, Counterclockwise Rotation

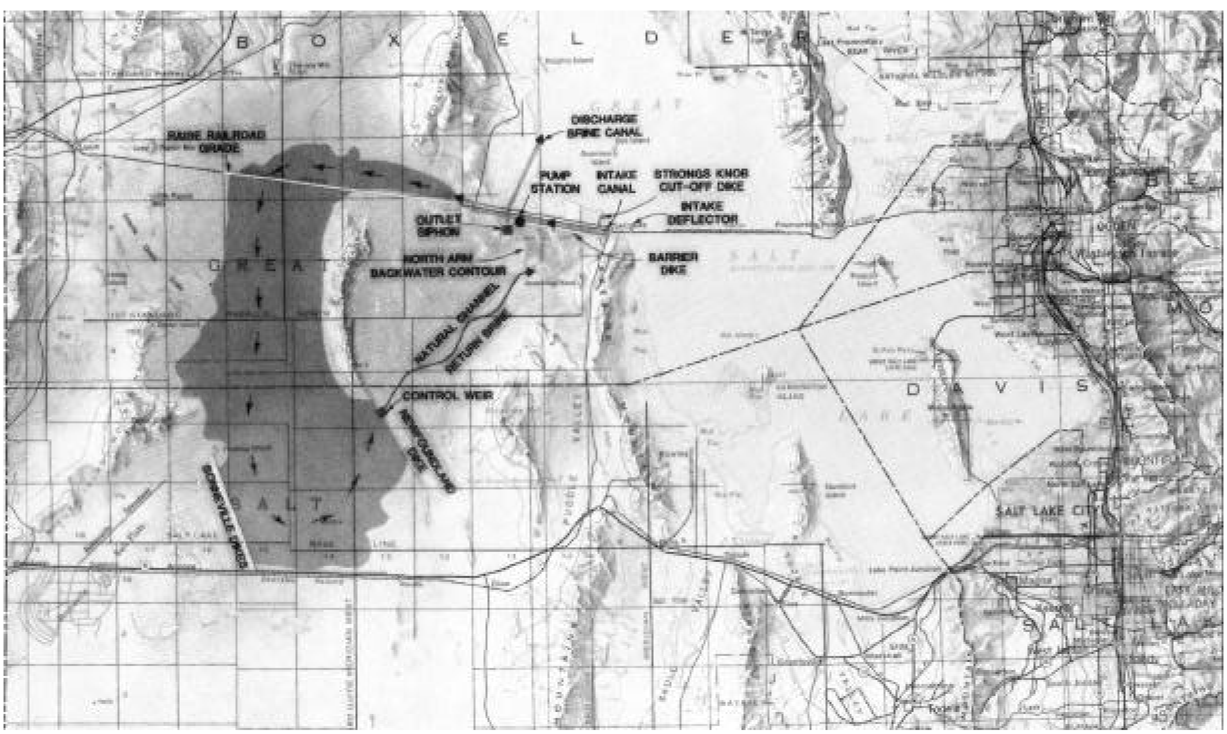


Figure 6-10
Newfoundland Pond Alternative, Clockwise Rotation

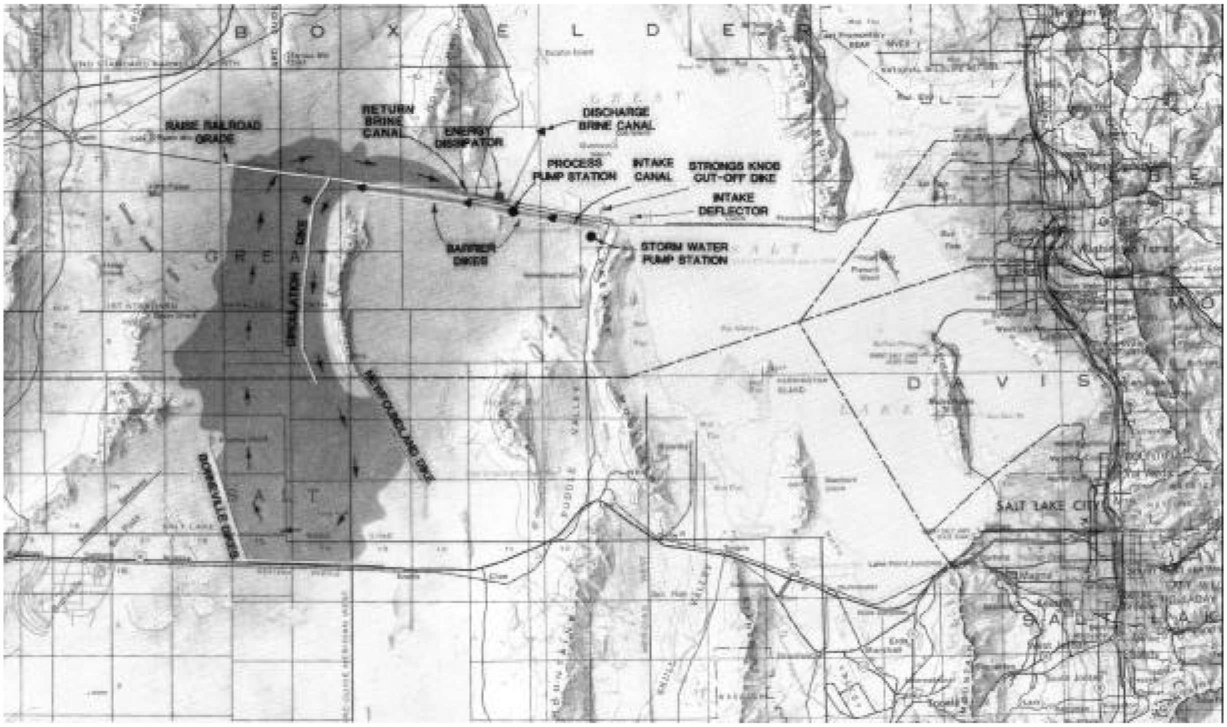


Figure 6-11
Boundary Ponds Alternative

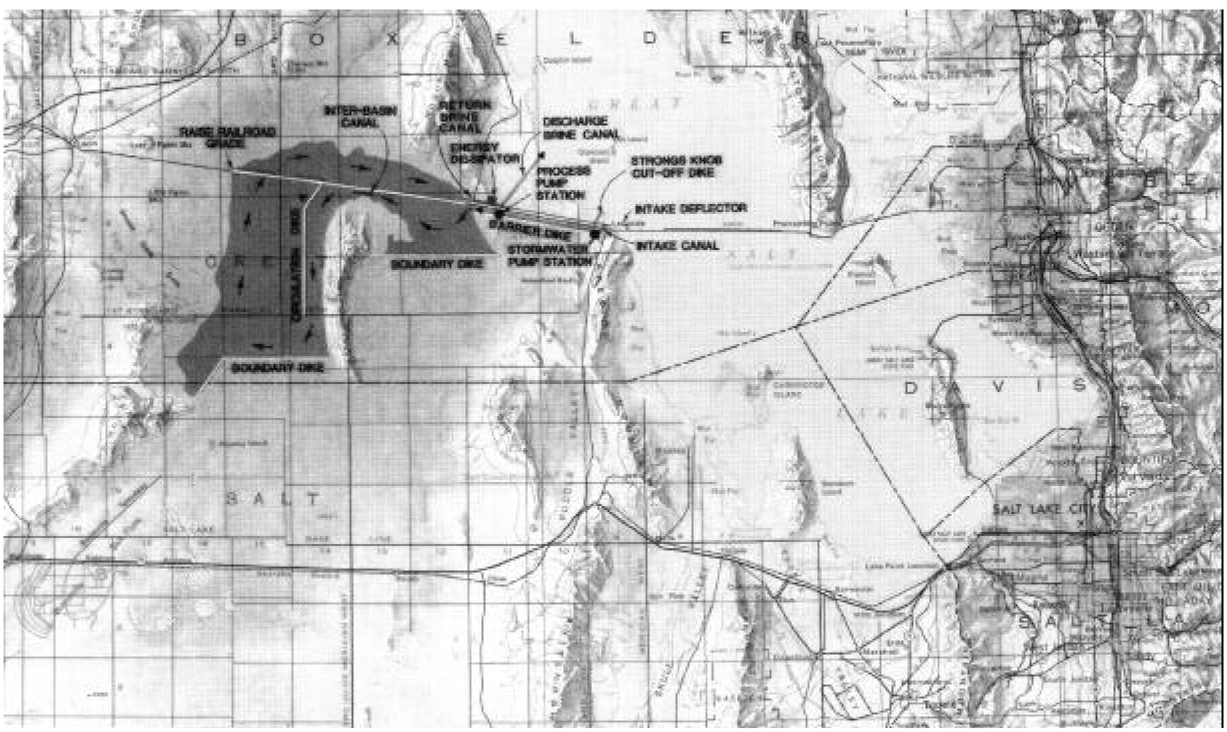


Table 6-4
Alternative Pond Configuration Alternatives
Characteristic Comparisons

ITEM	Wendover Pond	Bonneville Pond	Clockwise Newfoundland	Counterclockwise Newfoundland	Boundary Pond
Surface area @ Elev. 4218, acres	638,000	479,000	374,000	374,000	217,000
Volume @ Elev. 4218, acre-feet	1,760,000	1,440,000	1,090,000	1,090,000	680,000
Annual evaporation, acre-feet	1,570,000	1,140,000	910,000	910,000	530,000
Required pumping rate, CFS	4,000	2,800	2,400	2,400	1,400
Excavation required, cubic yards	8,530,000	6,700,000	8,610,000	5,360,000	6,920,000
Embankment required, cubic yards	8,475,000	7,220,000	6,000,000	4,775,000	6,400,000
Duty horsepower	12,800	8,600	7,400	7,400	4,300
Annual diesel consumption, gallons	6,600,000	4,400,000	3,800,000	3,800,000	2,200,000
Annual fuel cost	$5,600,000	$3,700,000	$3,200,000	$3,200,000	$1,900,000
COMPARISON FACTORS					
Average pond depth, feet	2.76	3.01	2.91	2.91	2.44
Annual evaporation	2.46	2.38	2.43	2.43	2.44
Fuel cost ($/acre-foot evaporated)	$3.57	$3.25	$3.52	$3.52	$3.59

Table reproduced from *Final Report, West Desert Pumping Alternative, Great Salt Lake*, EWP Engineering, et.al., December 1983

Table 6-5
Estimated Costs of Alternative Pond Configurations
South Arm Operating Elevation 4205, (in thousands of dollars)

ITEM	Wendover Pond	Bonneville Pond	Clockwise Newfoundland	Counterclockwise Newfoundland	Boundary Pond
Intake Defector Dike	412	412	412	412	412
Strong's Knob Cutoff	578	578	578	578	578
Intake Canal	4,693	3,519	2,666	2,666	2,352
East Barrier Dike	4,380	4,380	3,528	4,380	3,528
Pump Plant	13,200	9,920	8,770	8,770	5,700
Outlet Canal	5,427	4,010	2,968	3,571	2,197
West Barrier Dike	-	-	608	-	-
Inter-basin Canal	-	-	819	-	509
Boundary Dikes	-	-	-	-	4,228
Bridges	3,560	1,020	630	900	520
Raise SPTC Grade	5,320	5,320	5,320	5,320	5,320
Circulation Dikes	4,139	4,176	2,113	-	1,351
Wendover Dikes	2,117	194	-	-	-
Bonneville Dike	6,049	3,909	2,013	2,013	-
Newfoundland Dike	1,278	1,278	1,278	1,278	-
Return Brine Canal	-	-	5,058	-	4,032
Discharge Brine Canal	3,100	3,100	3,100	3,100	3,100
West Pond Weir	989	786	-	658	-
Energy Dissipator	-	-	672	-	450
Outlet Siphon	2,063	1,639	-	1,371	-
Drainage Pump Station	750	500	500	-	500
SUBTOTAL	$58,055	$44,741	$41,033	$35,017	$34,777
Contingency @ 25%	$14,514	$11,185	$10,258	$8,754	$8,694
TOTAL	$72,569	$55,926	$51,291	$43,771	$43,471
Unit Capital Cost ($)/acre-foot, evaporated	$46.22	$49.06	$56.36	$48.10	$82.02

Table reproduced from *Final Report, West Desert Pumping Alternative, Great Salt Lake*, EWP Engineering, et.al., December 1983.

Table 6-5 (Continued)
Estimated Costs of Alternative Pond Configurations
South Arm Operating Elevation 4208, (in thousands of dollars)

ITEM	Wendover Pond	Bonneville Pond	Clockwise Newfoundland	Counterclockwise Newfoundland	Boundary Pond
Intake Defector Dike	412	412	412	412	412
Strong's Knob Cutoff	578	578	578	578	578
Intake Canal	3,160	2,459	1,783	1,783	1,582
East Barrier Dike	4,380	4,380	3,528	4,380	3,528
Pump Plant	13,200	9,920	8,770	8,770	5,700
Outlet Canal	5,427	4,010	2,968	3,571	2,197
West Barrier Dike	-	-	608	-	-
Inter-basin Canal	-	-	819	-	509
Boundary Dikes	-	-	-	-	4,228
Bridges	3,560	1,020	630	900	520
Raise SPTC Grade	5,320	5,320	5,320	5,320	5,320
Circulation Dikes	4,139	4,176	2,113	-	1,351
Wendover Dikes	2,117	194	-	-	-
Bonneville Dike	6,049	3,909	2,013	2,013	-
Newfoundland Dike	1,278	1,278	1,278	1,278	-
Return Brine Canal	-	-	5,058	-	4,032
Discharge Brine Canal	5,616	5,616	5,616	5,616	5,616
West Pond Weir	989	786	-	658	-
Energy Dissipator	-	-	672	-	450
Outlet Siphon	2,063	1,639	-	1,371	-
Drainage Pump Station	750	500	500	-	500
SUBTOTAL	$59,038	$46,197	$42,666	$36,650	$36,523
Contingency @ 25%	$14,760	$11,549	$10,667	$9,163	$9,131
TOTAL	$73,798	$57,746	$53,333	$45,813	$45,654
Unit Capital Cost ($)/acre-foot, evaporated	$47.01	$50.65	$58.61	$50.34	$86.14

Table reproduced from *Final Report, West Desert Pumping Alternative, Great Salt Lake*, EWP Engineering, et. al., December 1983.

3. An office program which included reviewing the existing geological and engineering data, discussing data and canal construction with contractors and evaporation pond companies who were familiar with the proposed construction techniques, engineering analyses and the preparation of summary reports.

Summary of Conclusions

The following general conclusions were developed as a result of this preliminary geotechnical and groundwater investigation for the West Desert Pumping Alternative:

1. Based upon the geotechnical and groundwater considerations, the proposed West Desert Pumping Project was feasible.

2. Geotechnical considerations and construction scheduling indicated that the preferred alignments for the proposed Intake Canal and East Pond Dike would be locations which minimize dike heights due to the low strength clays in the area between Lakeside and Hogup Ridge.

3. The foundation soils upon which the proposed West Pond, East Pond, associated dikes and canals were established were primarily silty clay and clay soils which exhibited low strength and permeability and high compressibility characteristics. Due to the low permeability, seepage through the foundations would be very low.

4. It was recommended that, where possible, dikes be constructed of adjacent lakebed clay materials to keep seepage through the dikes to a minimum and to minimize importation of material. If the dikes could not be constructed of lakebed clays, imported granular dikes were recommended. An impermeable cutoff trench would be constructed through the center of the dikes keying into the lakebed foundation soils or by placing an impermeable clay blanket on the pond side of the dikes.

5. Preliminary evaluation of required freeboard for the dikes indicated that, based on wind velocity data, the necessary freeboard for setup and wave action would be four to six feet, depending on the dike location. Based on the fetch distances and the higher density of the pond water, slope protection was recommended on all dike slopes adjacent to the ponds. The slope protection material would consist of coarse, sandy gravel and cobble materials imported from borrow sources located at higher elevations adjacent to the lakebed deposits.

6. Potential problem soils identified along proposed canal and dike alignments consisted of low-strength clays, gypsiferous sand dunes and oolitic sand dunes. Generally, the lower strength clay occurred in areas of lower surface elevation. The dune materials generally were observed to be in a relatively loose condition and exhibited moderate permeability and compressibility characteristics. In addition, the gypsiferous dunes were somewhat water soluble.

7. Construction of the proposed dikes was very sensitive to the shear strengths of the foundation soils. In general, shear strengths of the foundation materials below all the dikes, except the Lakeside Dike, exhibited adequate strengths for the proposed embankment heights. However, due to the relatively low shear strengths near the Lakeside area and the proposed relatively high dikes which were necessary along that dike alignment, staged construction of that dikes would be necessary. It was recommended that the foundation soil shear strengths be further defined if the project was authorized, so that a more detailed stability evaluation could be performed in the Lakeside area.

8. Seepage through the proposed dikes constructed of lakebed soils and established upon lakebed foundation materials would be less than one gallon per minute per mile of dike. Ponded water and seepage loses from the proposed reservoirs would result in groundwater level increases of less than one to two feet below existing facilities adjacent to the proposed ponds.

Therefore, the impact of the seepage on existing facilities would be very minimal.

9. The installation of a Pumping Plant at any of the alternative pumping locations would probably require dewatering, and blasting would be necessary to excavate through bedrock materials where they were encountered.

10. Due to the extremely large project area and the difficulty to obtain access to significant and important critical areas for the project, the preliminary conclusions stated here were developed from a limited field and laboratory investigation and primarily based on experience in similar type soils. If this project were authorized, more detailed geotechnical subsurface elevations would have to be performed within critical areas. Specifically, these critical areas occurred at the proposed Pumping Plant location and along the alignment for the Lakeside Dike and Canal.

Hill Air Force Range

By way of background, the Hill Air Force Range was initially established in direct support of Air Force Logistics Command (AFLC) testing requirements and used extensively in support of Department of Defense (DOD) munition testing programs since it was withdrawn from the public domain in 1941. Munition testing on the range is divided into three categories: (1) munition flight testing, (2) munition ground testing, and (3) large and small missile motor firings. Service engineering test flights are also performed to test aircraft and ordinance modifications that generally result in weapon system improvements. The range is also used for tactical training for pilots in air-to- ground exercises.

The overall investment in range instrumentation and facilities alone was in excess of $40 million. Annual payroll and local contracts associated with the range were substantial. Many missions were top security efforts, and dry land and accessible target areas were essential. The range is also an essential part of a DOD Range Complex consisting of Hill, Wendover, and Dugway. This is one of the largest overland range complexes in the country. The non-accessibility/utilization of targets or reduction of available targets posed by wet target areas of access thereto would pose severe scheduling problems for the Air Force, the solution of which would be the curtailment, elimination or relocation of essential DOD tests.

The general area which would be affected by the West Desert Pumping Alternative was actively being used for the delivery of a large variety of munitions. The testing, training and hazardous operation accomplished was vital to maintaining a viable tactical/strategic force. The workload at the range taxed the available instrumentation and facilities to the limit thereby necessitating expansion to support ongoing and projected future operations.

The overall Hill Air Force Range has been used for different types of live munitions over the past 42 years. Many of these extremely hazardous munitions may not have exploded and lie at various depths below the surface of the ground. Dredging dikes, canals, etc., through this area would be a critical undertaking, and may seriously jeopardize the safety of personnel and equipment. Such activity would likely reduce the flexibility of the range to respond to the various operational, training and testing functions.

The West Desert Pumping alternative would inundate several non-critical targets. In addition to inundating these targets, it was possible that introducing water to adjacent areas would raise groundwater level due to capillary action and affect other targets. The need to minimize the impact to the Hill Air Force Range in the desert west of the Great Salt Lake was a major objective of the engineering feasibility study.

Coordination With Southern Pacific Transportation Company

General

Direct impacts on property owners in the project area were mostly focused on facilities owned and operated by the Southern Pacific Transportation Company (SPTC). In the area of anticipated heaviest construction, (that is, from the intake works near the existing breach to the West Desert Pond) there would be four crossings of the Southern Pacific railroad tracks and numerous other interactions with Southern Pacific's facilities. This section more clearly delineated the interaction with and participation of Southern Pacific Transportation Company in the project.

Specific Issues

Proposed design concepts were discussed with the SPTC Engineering Department. In general, SPTC indicated a strong interest in controlling the Great Salt Lake level and in implementing the West Desert Pumping Project. The position of the SPTC on specific project component issues was:

1. The arrangement of the intake at the causeway breach was acceptable to SPTC. The proximity of the deflector dike to the causeway would not cause right-of-way conflicts. A primary concern was with the replacement of riprap that might be lost due to wave action. Replacement could be handled by the SPTC on a cost reimbursable basis.

2. The SPTC would make its road along the railroad tracks available for access to the Pumping Plant on Hogup Ridge during construction and later for operation and maintenance. The railroad would provide, on a cost reimbursable basis, qualified people to control the access.

3. The SPTC did not anticipate difficulty in providing and maintaining a diesel fuel delivery facility on a cost-reimbursable basis. A double railroad spur could be built near the Pumping Plant to accommodate two trains of tank cars; SPTC was of the opinion that storage of fuel in tank cars would be more economical than storage in an underground tank. A week's supply could be stored. Alternately, the fuel could be stored in an underground tank. The final solution was to be adopted in the detailed design phase after the cost optimization of both alternatives was evaluated.

4. The SPTC was willing to design all bridges required to cross breaches in the railway embankment, based on spans as directed by project requirements on a cost reimbursable basis. Alternately, the bridges could be designed by the designers of the West Desert Pumping Project with SPTC participation and approval. Scheduling of design and construction, particularly rail traffic handling problems during construction, would need to be carefully evaluated and agreed upon.

5. Design of the protection of the railroad embankment through the West Pond would have to be coordinated with the SPTC and its geotechnical consultant. The SPTC indicated that dikes would be required versus raising the grade. Final design would depend on geotechnical and hydraulic criteria, ability of SPTC to raise grade and the long-term protection and stability of the railroad embankment.

6. The SPTC was prepared to act as construction manager for features of the project affecting its operation. Construction of the causeway breach would serve as a model for elements of the project which impacted the railroad.

7. The SPTC was prepared to furnish quarry materials from its Lakeside quarry on a cost reimbursable basis for use in the project. This material would meet riprap, ballast and quarry reject requirements for much of the embankment and dike work from Lakeside to the West Pond along the railroad alignment. The material was not suitable for concrete aggregate.

8. The SPTC did not see serious conflicts

if its facilities were used for construction access. This matter would be addressed during the detailed design stage of the project.

9. The SPTC was prepared to make space at Lakeside available for housing the construction crews on a cost reimbursable basis. Depending on requirements, the power supply for this camp might require upgrading by the construction of a transmission line from the Air Force base.

The above issues needed to be verified in detail with SPTC before start of final design. A detailed agreement between the SPTC and the state of Utah would probably be required to adequately define the responsibilities of both parties. Of particular interest were reimbursement provisions, schedules, review procedures, and common use of SPTC facilities during and after construction.

Summary and Conclusions

The engineering feasibility report completed in 1983 on the GSL/West Desert Pumping Alternative was based on work conducted by Eckhoff, Watson and Preator Engineering (EWP); International Engineering Company, Inc. (IECO); and Dames & Moore (D&M). Principal assignments were management and civil engineering, EWP; pumping and power systems, IECO; and geotechnical and GW hydrology, D&M.

Major conclusions and recommendations were:

1. Based on geotechnical and groundwater considerations, the proposed project was feasible.

2. The natural lake bed clay materials which predominate in the West Desert could be used for much of the canal construction, which would minimize the costs of much of the earthwork. They were also of relatively low permeability and would restrict the seepage of water through the dikes.

3. In areas of high dike construction, such as near Lakeside, staged construction would be necessary.

4. Seepage through dikes constructed of lake bed soils should be less than 1 GPM/mile. Ponding and seepage from the proposed ponds would result in groundwater increases of less than one to two feet. Therefore, the impacts of seepage on existing facilities would be minimal.

5. Because of access and time limitations, only preliminary geotechnical field studies and laboratory investigations were conducted. If the project was authorized, significant geotechnical investigations were to be performed in critical areas.

6. Although the required pumping facilities would be located in an unusual, remote and harsh environment, the facilities could be constructed without significant delays or additional costs, using conventional pumps, drivers and control systems.

7. Pumping Great Salt lake brines would not pose difficult problems.

8. Ordinarily, it would be preferable to utilize electric motors as the pump drivers, even with the large size of the recommended units. However, the study showed that diesel drive systems would have substantially lower capital costs, due to the need for long electrical transmission lines for this project. Diesel drivers were, therefore, recommended.

9. The recommended pumping facility would have a nominal capacity of 2,400/2,500 cubic feet per second and lift the GSL brine approximately 20 feet. Total installed power would be about 10,000 HP. Four to six pumps would be recommended.

10. There did not appear to be other factors which would invalidate the study's conclusions of the general and overall engineering feasibility of the proposed GSL/West Desert Pumping Alternative.

11. A variety of sizes and configurations of ponds, dikes, canals and control facilities were investigated in the study. Major evaluation factors invoked in the analyses were:

a. Effectiveness in evaporating GSL brine (maximize).
b. System costs (minimize).
c. Construction on Air Force Test Range properties (minimize).
d. Construction time (minimize).

12. The recommended alternative, based on the above evaluation factors, was designated the **Railroad Lowline Alternative (South Option)**, because the Intake Canal extended across the desert to the Pumping Plant at Hogup Ridge in an alignment which was parallel to and south of the Southern Pacific Railroad tracks.

13. Costs of the recommended alternative, component by component, are shown below.

14. Operating costs would be approximately $4 million per year, of which 90 percent would be for fuel costs for the pumps.

15. The proposed system would have the capability of evaporating 1,060,000 acre-feet per year. Including the effects of initial storage, the system would reduce the level of GSL by 3.1 to 3.2 feet within a three-year period.

16. Because of the extensive federal land which would be impacted by the proposed project, an Environmental Impact Statement was considered. The BLM and the Air Force strongly supported the preparation of an EIS. This process would take approximately one year to complete and cost an estimated $600,000.

17. An extremely long project schedule resulted from a decision to postpone other project activity until after completion of the EIS. Allowing for reasonable winter delays, the design/construction period following the EIS completion would be approximately 21 months, giving an overall project schedule of 33 months. This schedule assumed a "fast track" process that might entail extra costs because of inherent uncertainties.

18. If the engineering efforts were initiated at the same time as the NEPA (EIS) process, considerable time could be saved on the overall project schedule. It was estimated that the time could be reduced 25 months, providing relief for the 1986 GSL peak. This process would also assure adequate time to conduct requisite field studies and easement and right-of-way work. Costs would also be more under control.

19. The estimated cost of the engineering was $2.3 million.

20. An interim construction program was recommended that could:

a. Result in the retention of substantial runoff from the West Desert, and
b. Provide for the first stage of construction of the East Pond Dike.

These elements were part of the recommended program of construction.

Component Costs

Component	Return South ($1,000)	Return North ($1,000)
Pump Station Intake Canal	2,168	2,168
Pumping Plant	8,770	8,770
Pump Station Discharge Canal	6,546	6,546
GSL Isolation Dikes	5,269	2,247
Railroad Dikes	3,169	3,169
Bonneville Dikes	2,013	2,013
Newfoundland Dikes	1,278	1,278
East Pond Dike	5,188	5,188
Return Brine Canal	100	-
Railroad Bridges	3,680	2,430
West Pond Weir	658	658
East Pond Outlet	585	-
Highline Siphon	-	1,371
High Water Penalty	500	500
Subtotal	$39,924	$36,338
Contingency @ 25%	$9,981	$9,805
TOTAL	$49,905	$45,423

- Because of the nature of the West Desert construction environment and the remaining field work, the contingency factor had been set at 25 percent.

Approximately 300,000 acre-feet of runoff originated in the West Desert during 1983. Retaining and evaporating this amount of runoff could reduce the average level of GSL about 1/4-foot. The first impacts of the project could thus be advanced one year. ⌂

References

Final Report, West Desert Pumping Alternative, Great Salt Lake, Eckhoff, Watson and Preator Engineering, et. al., December 1983.

Final Report, West Desert Pumping Alternative, Eckhoff, Watson and Preator Engineering, October 1984.

1984 Update West Desert Pumping Alternative - Great Salt Lake, Eckhoff, Watson and Preator Engineering, October 1984.

Facilities and Appurtenances Study, Related to the E.I.S for West Desert Pumping Alternatives, Eckhoff, Watson and Preator Engineering, et. al., March 1985, Chapter 4.

Chapter 7

Environmental Impact Statement

Actions

The Utah Division of State Lands and Forestry entered into a contract with BIO/WEST, INCORPORATED, of Logan, Utah, on January 31, 1985, to prepare an Environmental Impact Statement (EIS) for the proposed Great Salt Lake West Desert Pumping Alternative.

In accordance with the National Environmental Policy Act (NEPA), the EIS was prepared as the decision document for the Bureau of Land Management (BLM) and the U.S. Air Force (USAF). The BLM was the lead agency for the EIS, and the USAF was the cooperating agency. In February 1986 the Draft EIS was released for public review and comment. The Final EIS was released in July 1986.

Bureau of Land Management

On June 20, 1986 (effective Date of Grant), the BLM granted the state of Utah, Division of Water Resources, a 50-year right-of-way to construct, operate, maintain and terminate the Great Salt Lake Pumping Plant, Intake Canal and Dike, Outlet Canal, Retention Dike and Evaporation Pond on public lands.

U. S. Air Force

Through a letter on November 5, 1986, from the Department of the Army, Sacramento District, Corps of Engineers, the Division of Water Resources was approved to mobilize and start construction of the Newfoundland Dike and appurtenant facilities for the West Desert Pumping Project on lands at the Hill Air Force Range (HAFR). This right-of-entry was issued pursuant to an emergency exemption granted by the President's Council on environmental Quality, dated May 27, 1986, and subject to terms and conditions of formal instruments to be formalized at a later date. The right-of-entry was a short-term (two-year) right granted under emergency conditions, which was terminated at the end of the May 1987 to June 30, 1989, pumping period.

Summary of Alternatives

The state of Utah filed for a right-of-way to use public and U.S. Air Force lands for the West Desert

Pumping Project. The major purpose of the project was to prevent flooding around the Great Salt Lake (GSL) due to rising lake levels. The project would utilize federal lands on which to construct a pumping plant and associated canals and dikes to create an evaporation pond in the West Desert. Water would be pumped from the GSL to the West Desert Pond for evaporation. The West Desert Pumping Project discussed in the Final EIS was a modified version of the project discussed in the Draft EIS.

The Southern Pacific Railroad Causeway was breached in 1984, which lowered the south arm of the lake about one foot. Diking was initiated in several areas to protect critical facilities. Additional lake management options were studied by the state at a feasibility level. These studies prompted the state to propose construction and operation of the West Desert Pumping Project as the most reasonable alternative to meet the immediate need for flood control.

A number of other flood control measures were investigated by the state, but after evaluation it was determined they would be either ineffective in preventing flooding, would take too long to build, or would be too expensive. The only reasonable alternative was to dike critical facilities around the lake to protect human health and safety. The diking alternative was, therefore, developed for the EIS from feasibility studies on diking that were contracted by the state.

In addition, a no action alternative was analyzed as required by the NEPA. In order to compare pumping, diking and no action alternatives, the lake rise to elevation 4215 scenario was developed. It assumed that inflow to the lake would be at about 170 percent of average from years 1986 to 2000. This rate of inflow would make the Great Salt Lake rise to elevation 4213 by 1989 and to 4215 by 2000, as compared to the peak 1986 level of about 4212. Lake elevation would then drop to 4205 by the year 2010. A lake elevation scenario at 4217 was also developed as a worst case situation.

Alternative 1 - No Action

The no action alternative assumed that permits for the West Desert Pumping Project would not be issued by the BLM and the Air Force. For analysis purposes, it also assumed that no additional flood control measures would be implemented and that unchecked flooding around the GSL would occur. Any existing flood control structures would be overtopped rather rapidly. For example, the Union Pacific Railroad grade along the south shore of the GSL would protect I-80 and other facilities until elevation 4213-4214, when it would be overtopped.

Impacts of this alternative would allow the GSL to rise unchecked. Numerous areas along the east shore would be flooded, affecting farmlands, wetlands, wildlife habitat, recreation, economic and cultural resources. Interstate 80 along the south shore of the GSL would be inundated, as would the Union Pacific Railroad grade and the Southern Pacific Railroad Causeway, creating major transportation problems. All of the evaporative industries around the lake would be flooded, as well as much of the Rose Park residential area and several waste water treatment plants and refuse site. Costs of the damages would exceed $1 billion.

In addition, fog and low clouds would increase as the lake became larger, affecting weather along the Wasatch Front and in the West Desert, but only in the winter months. This poor weather would also reduce the amount of time the Air Force could use the Utah Test and Training Range (UTTR).

It was likely that some flood control measures would be taken; but for analysis purposes, none was included in the alternative.

Alternative 2 - West Desert Pumping Project

This was the propose action and involved construction of several structures in the areas west of the GSL. The project had been modified since publication of the Draft EIS; some portions of the modified project were still being designed when construction started. A Pumping Plant would be located adjacent to, and on the south side of, the Southern Pacific Railroad grade at Hogup Ridge. Water would be pumped directly from the north arm of the lake, although canals would need to be dredged as the lake receded. A trestle would be constructed so that water could flow under the railroad grade from north to south to the Pumping Plant. The Pumping Plant would utilize three pumps designed to pump up to 3,500 cubic feet of water per second up about 23 feet to a discharge channel. The pumps were originally planned to be diesel powered; however, plans were changed to power the pumps by natural gas. The discharge channel would transport water to the north side of the railroad grade near the northern end of the Newfoundland Mountains. The water would then spread out and move south under the railroad grade and along the west side of the Newfoundland Mountains. Two dikes, the Bonneville Dike and the Newfoundland Dike, would contain the pond. The Bonneville Dike would keep the pond, called the West Pond, from covering I-80 and from flooding the Bonneville Salt Flats. The Newfoundland dike would extend from the southern end of the Newfoundland Mountains southeasterly to high ground across the mud flats. A control weir in the Newfoundland Dike would maintain the maximum level of the West Pond at about elevation 4217 and at a size of about 320,000 acres.

Water would flow over the Newfoundland Dike weir and then by the lay-of-the-land would flow back to the north arm of the lake. This would result in scattered ponding in low areas between the Newfoundland Mountains and the north arm. The water would flow under the Southern Pacific Railroad grade via a trestle to be built for the project near Lakeside. Several figures and diagrams showing the project layout can be found in the Draft EIS.

Most of the material for the dikes would come from material sites. Material for the Newfoundland Dike would come from new material sites at the southern end of the Newfoundland Mountains. Material sites for the Bonneville Dike would be located near either end of that dike and would include existing pits used by the Department of Transportation. The project would cost about $55 million to build, and construction would take approximately one year. The work force would not exceed about 200 persons.

Under the elevation 4215 scenario, the West Desert Pumping Project would hold the Great Salt Lake at elevation 4212, so no additional flood control measures would be required. Under the 4217 scenario, the lake would still rise to elevation 4215 and additional flood control measures along the east shore would probably be needed.

During the design studies for the project, costs and designs to control potential seismic (earthquake) concerns were included, as were costs to repair any local roads used to haul material to the dikes. Another proposed mitigation measure would be to have a qualified archaeologist conduct surveys in areas proposed for surface disturbance and to conduct random surveys of the inundated areas in the West Desert.

Impacts of this alternative to the West Desert would be fairly minor, since most of the area impacted is mud flat. Kaiser Chemical may be benefitted by increased brine flow caused by groundwater recharge from the West Pond. Grazing access to the Newfoundland Mountains and southern Hogup Ridge would be restricted by the discharge channel.

Under the 4215 scenario the lake would essentially not rise from levels at the time, creating significant flood control benefits to shoreline areas. All sectors would be benefitted, especially the transportation sector. Under the 4217 scenario, most of the no action flooding impacts would still occur and few benefits would occur.

The major negative impacts of the project would be an increase in winter fog around the GSL and an increase in precipitation along the Wasatch Front. This would impact the Air Force operations on the UTTR, but for only a few days. Also, the West Pond and scattered ponding in the East Pond area resulting from flow back to the lake would restrict flight operations, because Air Force regulations do not allow low level flights over open water which would endanger pilots who eject from their aircraft.

Alternative 3 - Diking to Protect Critical Facilities

This alternative involved building and/or raising dikes along the GSL to protect critical facilities, primarily sewage treatment plants to elevation 4215 or 4217, depending on the lake rise scenario. This alternative assumed that seven dikes would be built to protect those facilities. The Union Pacific Railroad grade along the south shore of the GSL would protect I-80 and other facilities until elevation 4213-4214, so it was considered as another existing dike. The eight dikes considered in the EIS included the Corinne, Perry, Plain City, and Little Mountain Waste Water Treatment Plant dikes and South Davis, Rose Park, South Shore, and UPRR/I-80 dikes. The South Davis Dike would protect sewage treatment plants and refuse disposal sites west of Bountiful. The Rose Park Dike would protect areas around the mouth of the Jordan River near the Rose Park residential area. The South Shore Dike would protect I-80 near the Saltair Resort. Material for construction of these dikes would come from Wasatch Front sources. The Union Pacific Railroad grade along the south shore the GSL would protect I-80 and other facilities until elevation 4213-4214, after which it would be abandoned and the freeway and railroad would be routed to higher ground. Dikes built by AMAX Magnesium and American Salt Company also protected portions of the Union Pacific grade, I-80, and Timpie Waterfowl Management Area north and west of Grantsville. This alternative would protect the areas immediately behind the dikes, but no other areas.

This alternative would have some negative impacts, but like the pumping action it primarily provided flood control benefits. Benefits of diking, however, would be considerably less than West Desert Pumping Project under the elevation 4215 scenario. Major flood damage to mineral industries, transportation corridors and farmland would still occur under this alternative.

The major negative impact would be the loss of deer winter range due to borrow material removal from sites along the Wasatch Front. Under the 4217 scenario, even more extensive flooding impacts would still occur. ⌂

Chapter 8

West Desert Pumping Project Permit Agreements and Mitigation

A number of permits, rights-of-way and agreements were required to allow the state of Utah to construct and operate the West Desert Pumping Project. Most prominent were those required by the U.S. Air Force, U.S. Army Corps of Engineers, and Bureau of Land Management. Archeological studies were also required.

U.S. Air Force

The U. S. Air Force required the state of Utah to obtain its right-of-way permit to pump water onto Air Force lands, specifically the Utah Test and Training Range in Tooele and Box Elder counties. Permission was also required for construction on Air Force lands, such as the Newfoundland Dike, and studies such as the excavation of the Donner-Reed Wagon Box Mound Site and documentation of the Hastings Trail/Donner-Reed experience.

A temporary two-year U.S. Air Force right-of-way permit to construct the Newfoundland Dike portion of the West Desert Pumping Project was issued in November 1986. It was documented only by letter, and later informally extended through the project's pumping period. Other elements of the pumping project were already under construction when this right-of-way was issued. If the pumping project were ever renewed, another right-of-way permit would have to be issued. The permitting process involved at least two Air Force major commands and, potentially, the Department of the Air Force. The right-of-way permit allowed operating the pumping project down to a south arm elevation of 4208 feet above mean sea level. Later, in February 1988, the permitted level was lowered to 4206.7.

The Air Force had several concerns: most prominent was safety. Air Force officials contended that a large body of water at the range would degrade rescue efforts for downed pilots in the area and affect their ability to survive. And at night, the water would contribute to pilot disorientation. In addition, Air Force officials believed the water would increase the bird population which would be hazardous to aircraft flights through the area. Officials conceded, however, that the return brine canal would reduce the water surface, thus reducing safety and other concerns.

Air Force officials were also concerned that pumping water into the West Desert would damage target complexes and affect training. And pumping would impact the Air Force's ability to find and clean up explosives that might be in the flooded area.

Occasional fog was another concern. And if the state wished to pump water into the West Desert in an effort to manage the lake level for non-emergency purposes, Air Force officials indicated they would be less likely to accept adverse impacts that might result. Further encroachment on Air Force lands for lake management purposes would be a major concern.

Archeological Studies

Archeological studies were implemented through a Memorandum of Agreement between the U.S. Air Force, the Utah State Historic Preservation Officer, and the state Advisory Council on Historic Preservation. The efforts satisfied requirements of Utah's National Historic Preservation Act of 1979.

Sites with cultural and anthropological importance that were affected by the pumping project were two cave sites on the Newfoundland Mountains, the Floating Island cave site, the Donner-Reed Wagon Box Mounds site, and the Hastings Trail and Donner-Reed experience.

The pumping project avoided the two cave sites in the Newfoundland Mountain area, but they were classed as significant because of their connection to other excavated sites in the Great Basin area. They also are registered by the Utah State Historic and Preservation Office.

The project also avoided the Floating Island Cave site that is on Bureau of Reclamation land. It was determined the cave was not associated with the Donner-Reed or Hastings trails, but its proximity and the history of archeological looting in the area led to additional excavation.

The Donner-Reed Wagon Box Mound site was covered by water in the evaporation pond west of the Newfoundland Mountains. The site was excavated by the state between October 10 and December 18, 1986.

The Hastings Trail and the Donner-Reed experience was also documented by aerial photography. A portion of the trail was intermittently visible and was criss-crossed by several pioneer parties known to have traveled the Hastings Trail.

Department of the Interior, Bureau of Land Management

The Bureau of Land Management granted a right-of-way permit for use of about 200,000 acres of public land under authority of Title V of the Federal Land Policy and Management Act of 1976. Additional use authorizations were also required for construction activities. As mentioned in Chapter 7, the bureau granted a 50-year right-of-way to public lands to construct, operate, maintain and terminate the West Desert Pumping Plant, Intake Canal and Dike, Outlet Canal, Retention Dike and Evaporation Pond.

U.S. Army Corps of Engineers

The U.S. Army Corps of Engineers granted a 404 Permit for the Southern Pacific Railroad Causeway breach and pumping project construction in the waters of the Great Salt Lake under authority of the Federal Clean Water Act of 1977 and the Federal Water Pollution Control Act Amendment of 1972.

Utah State Division of Environmental Health, Bureau of Water Pollution Control

The Bureau of Water Pollution Control issued a permit to discharge into state waters under authority of *Utah Code Annotated* SS73-14-5, 73-14-10.

Air Quality

An air quality approval order was required by Utah Air Conservation Regulations and the Utah Air Conservation Act. It was issued in October 1986. Major concern was with emissions from the three natural gas fueled engines at the pumping station and two natural gas fueled electric generators to provide on-site electrical power. The initial compliance inspection at the pumping station and periodic inspections by the Utah Department of Health, Division of Environmental Health, Bureau of Air Quality, found the Pumping Plant complied with Utah’s air quality regulations.

Magnesium Corporation of America

In 1987 the Magnesium Corporation of America, previously called AMAX Magnesium Corporation, was allowed to extend the Inlet Canal to the Pumping Plant approximately 9,000 feet by dredging in an effort to extend the company’s brine recovery efforts. In doing so, the company had to comply with the U.S. Army Corps of Engineers 404 Permit. This effort was also coordinated with the Bureau of Land Management and the Southern Pacific Transportation Company. Estimated cost of this project was $700,000.

The magnesium company was also allowed to construct a multi-million dollar brine extraction canal project near Knolls in 1987 to divert brine from the West Desert Pond to its processing facility.

Mountain Fuel

The Magnesium Corporation of America provided a right-of-way, a pipe storage lot and fresh water to hydrostatic test the natural gas pipeline that Mountain Fuel constructed to the pumping station. The Bureau of Land Management permitted construction to cross public lands. The U.S. Air Force required hold-harmless agreements from contractors and employees to guard against liability for unexploded ordnance. The state of Utah also granted Mountain Fuel a right-of-way to cross state lands. ⌂

Chapter 9

Final Design of the West Desert Pumping Project

Preferred Alternative

Project Summary

The alternative preferred by the Division of Water Resources to mitigate the effects of high water levels of the Great Salt Lake consisted of a pumping plant and a system of canals, dikes and evaporation ponds. The project would pump brines from the Great Salt Lake into two evaporation ponds in the western desert and return concentrated brines to the lake. This plan, evaluated in the EIS process, is illustrated in Figure 9-1.

Specific features of the final design of the preferred alternative were essentially the same as those detailed in the preliminary design of the West Desert Pumping Project in Chapter 6:

1986 EIS Basis of Design

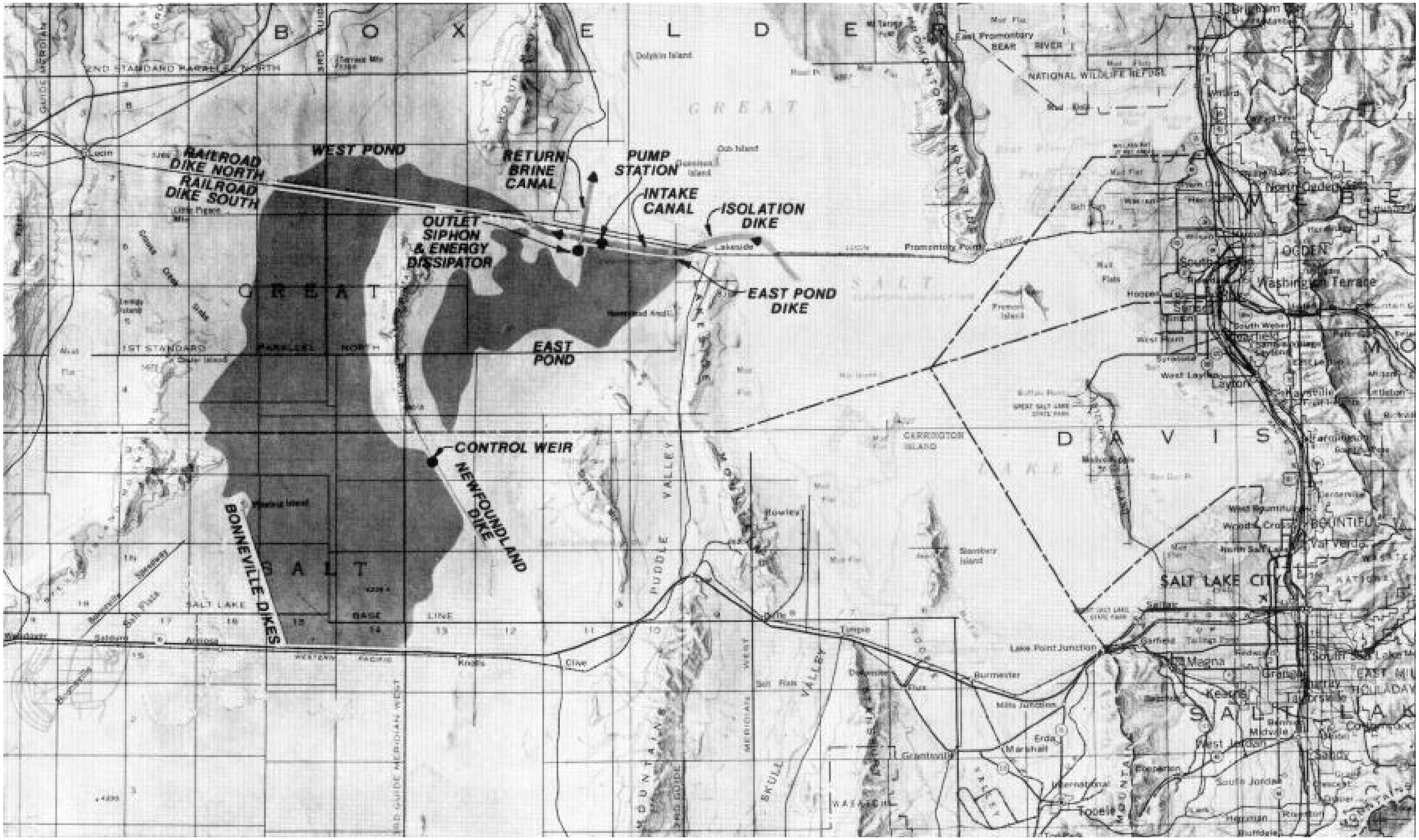
The basic design evaluated in the *1986 Environmental Impact Statement* (EIS) includes:

Design Lake Levels - The minimum control operating level for the Pumping Plant was set at 4205 feet mean sea level; the maximum (south arm, Great Salt Lake) was 4215 feet. The plant was to operate between these Great Salt Lake water surface levels. If the lake were to rise above 4215 feet, significant modifications to the configuration of the West Desert Pumping Project were to be considered.

SPTC Railroad Facilities - The 1986 EIS design basis required that the Southern Pacific Transportation Company's railroad causeway and its trackage from Lakeside to Hogup Mountains be incorporated within the overall design as access to the remote Pumping Plant. If these facilities were abandoned, and if as a consequence of this action they deteriorated, then additional maintenance by the state of Utah, or an appointed operating entity, would be required to maintain access to the Pumping Plant.

The SPTC indicated that it would make quarry materials available from its Lakeside quarry at its cost. The cost estimates included in the *1986 EIS Engineering Study* were based on this commitment and assumption. A further commitment was made by SPTC to make the man-camp land near Lakeside available to house the temporary construction work force. In a letter to the governor, SPTC and Southern Pacific Land Company (SPLC) offered to allow the

Figure 9-1 - Preferred Alternative



state of Utah to inundate their congressional land grant land along the track.

Intake Facilities - The 1986 EIS study assumed the existing causeway breach (located east of Lakeside) could be used to transfer brines between the north and south arms of the lake and as a source of south arm brine for the project without substantial modification to the structure. Work completed in the *1986 EIS Engineering Study* indicated this would be a critical design feature, inasmuch as contamination of the lighter density south arm brine by more concentrated north arm brines could result in significant decreases in evaporation. Furthermore, such contamination could result in an undesirable increase in the precipitation of salts in the West Desert evaporation ponds.

Outlet Facilities - The *1986 EIS Engineering Study* showed the outlet from the East Pond as being a siphon structure with a return brine canal located near the eastern slope of the Hogup Mountains and the SPTC railroad tracks. The return brine from the siphon would flow to the north arm via a dredged canal. No dikes were proposed to isolate the dredged canal from the north arm lake waters.

System Performance - It was assumed that natural mixing in the West Desert evaporation ponds would be sufficient to achieve homogenous conditions and, therefore, prevent the growth and accumulation of heavy brine pockets with attendant undesirable salt precipitation. Should this not be the case, pumping rates would need to be increased and/or baffles required in the ponds. All of the work completed suggested that such mixing would take place. This was based on available evidence derived from other similar shallow ponds and lakes associated with salt industries on the lake.

Construction -Year-round construction of project facilities was assumed. It was further assumed that the U.S. Air Force would have no objections to the construction of facilities which were not on or in close proximity to it's test range.

EIS Process - It was assumed that no significant changes to the Preferred Alternative would be required because of the EIS process. If significant changes were required, revisions to the work plan, schedule and budget would be necessary for all aspects of the work, including design, bidding and construction management.

Geotechnical - Up to the point of the 1986 EIS study, only limited geotechnical investigations had been conducted. It was assumed that the results of those limited investigations generally characterize the soils and foundation situations to be encountered at the main features of the project: Isolation Dike, Intake Canal, East Pond Dike, Outlet Canal, Railroad and Evaporation Pond Dikes, Pump Plant and the Return Siphon.

Final Basis of Design

The project as outlined in the *1986 EIS Basis of Design* did not change significantly during the final design process. Minor modifications included:

1) Design Life - 50 years was chosen as the design life for all structures. Final top-of-dike elevations were calculated using wind velocities associated with five and 10 return year periods.

2) New Causeway Breach - A new 150-foot breach in the SPTC causeway would be built to channel south arm lake brines into the intake facilities of the project. The existing breach structure would remain unaltered and not directly used as a part of the pumping project.

3) Variable Pumping Scenarios - To achieve maximum evaporation from the ponds, a scenario of pumping 2,800 cfs for seven months (April-October) and 933 cfs the remaining months, averaging 2,022 cfs, was evaluated. This would optimize system evaporation rates and provide an opportunity to lower fuel consumption and operating costs.

4) West Pond Operating Water Surface Level - During system optimization, a West Pond operating level of 4217 proved to be more efficient. This new water surface elevation level also lowered the top of dike elevations for the bordering embankments.

Most of the modifications were due to optimization of systems operation during the final design phase. Others were due to changes in the final geometry of the features as detailed in the final geotechnical investigation report prepared by Dames & Moore.

Design Life Policy - The *1986 EIS Basis of Design* did not include a requirement for the design life of the features. During the final engineering design, the Utah Division of Water Resources developed a basic policy decision concerning the design life requirements of major and minor features of the project. This design life was necessary as a guideline for selection of materials and construction practices to construct facilities able to withstand the potentially harsh environment attendant with the project's location.

Major structures, including the Pump Plant and Return Siphon and other non-earth structures, were to be engineered for a 50-year design life.

The design life for the earthen dikes within the pumping system was to be 50 years. However, rather than using a 50-year design wind velocity, a probability of overtopping approach was used. (See Technical Appendix C, Wind Storm Effects on the Great Salt Lake West Desert Pumping Project for a detailed discussion on calculations.) It was estimated that the pumping project would be utilized on the order of 10 to 20 years during the next 50 years. Therefore, if a 50-year return period wind velocity was used, the dikes would be overly-designed and require excessive construction costs.

Each dike was evaluated and rated on how critical a wave overtopping situation would be. A new return year was chosen for each dike and a required dike height derived from those new return year design wind velocities. Final top of dike elevations were calculated by performing wind tide and wave height computations along those areas that affected the protective dikes. The probability of overtopping and wind design return period for each dike is summarized in Table 9-1. The scenario evaluated pumping 2,800 cfs or 2,400 cfs constantly. The new variable rate scenario used for the final design planned on pumping 2,800 cfs for seven months, April through October, and 933 cfs for the remaining five months, for an average annual pumping rate of 2,022 cfs. This new 2,800/933 scheme allowed for approximately the same overall total annual evaporation of 1.067 million acre-feet (MAF) as did the 2,400 cfs constant pumping rate. It also allowed for reducing the East Pond water surface level to 4213 and reducing the peak return flow through the siphon to 58 percent.

Construction Schedule

Background - The critical path construction schedule developed during the 1986 EIS study was based on the requirement that the project should be completed and pumping commenced by December 31, 1986, to be fully operational to affect the 1987 peak lake level.

Table 9-1
Design Return Periods For Project Dikes

DIKE	POTENTIAL DAMAGE IMPACT	DESIGN PROBABILITY OF OVERTOPPING	DESIGN RETURN YEARS
Isolation	Low-water surface same on both sides	20 percent or 1 time every 5 years (2-4 times during pumping)	5
Railroad	Medium-water on only one side, 2'-3' depth	10 percent or 1 time every 10 years (1-2 times during pumping)	10
Newfoundland	Medium-water on only one side, 2'-3' depth	10 percent or 1 time every 10 years (1-2 times during pumping)	10
East Pond	High-9' water surface differential at low lake pumping levels of 4205	10 percent or 1 time every 10 years (1-2 times during pumping)	10
Bonneville	High-8' water on one 5 percent or 1 time every 20 years side during high wind (0 to 1 time during pumping) periods only		20

During the final design phase, lake level predictions estimated that the lake would peak at approximately the same level as the 1985 peak of 4209.95. This prediction subsided the concern to proceed with construction of the project at that time and, therefore, the need for keeping the project on the critical path. Since no funding actions had been initiated by the State Legislature, the decision was made to put the plans on the shelf until the time they were needed.

Procurement of the pumps was identified as a critical path item that needed to be bid and awarded during the design phase of the project. The bids were opened on October 29, 1985, and remained open until March 29, 1986. The low qualified bid was $6.3 million, including five years maintenance. The pumps were to be rebid should no action be taken by the state to award the contract before March 29, 1986. If rebidding was required, the estimated construction time schedule from the notice of advertisement for bids for the pump procurement to the start of pumping was 19 months. Likewise, the estimated time schedule for the general construction of the project features from the notice of advertisement for bids to the start of pumping was 14 months. Therefore, the pump procurement required five months lead time on the general construction contract.

It should be noted that once the decision was made to go ahead with this project, it required approximately two years from the time of the notice to proceed until the project could affect the level of the lake.

Critical Path Items - As previously indicated, the project was no longer on the critical path schedule as outlined in the 1986 EIS study. When a decision was made to go ahead with the project, the two critical path items which most affected the schedule were the pump procurement and general construction bidding dates.

The rebidding for procurement of pumps, gear drives, and engines, which was the predominant item on the critical path, required the following 21-month schedule. Note that the time for the fabrication of pumps increased from nine months to 12 months. This was a major exception expressed by all the pump supply bidders.

- Contract documents preparation - two months
- Prequalification of bidders - two months
- Bidding process and award of contract - two months
- Fabrication of pumps - 12 months
- Installation and startup - three months

The general construction contract required the following 16-month schedule:

- Contract Documents Review and Updating - two months
- Bidding Process and Award of Contract - two months
- Construction of Features - 12 months

Critical Path Schedule - In the *Final Design Study Report, September 1985*, two scenarios to the critical path schedule were presented. These two alternatives are reprinted in Figures 9-2 and 9-3, respectively. Alternative #1 provided for pumping to begin January 1, 1987. Alternative #2 provided for pumping to start June 1, 1987.

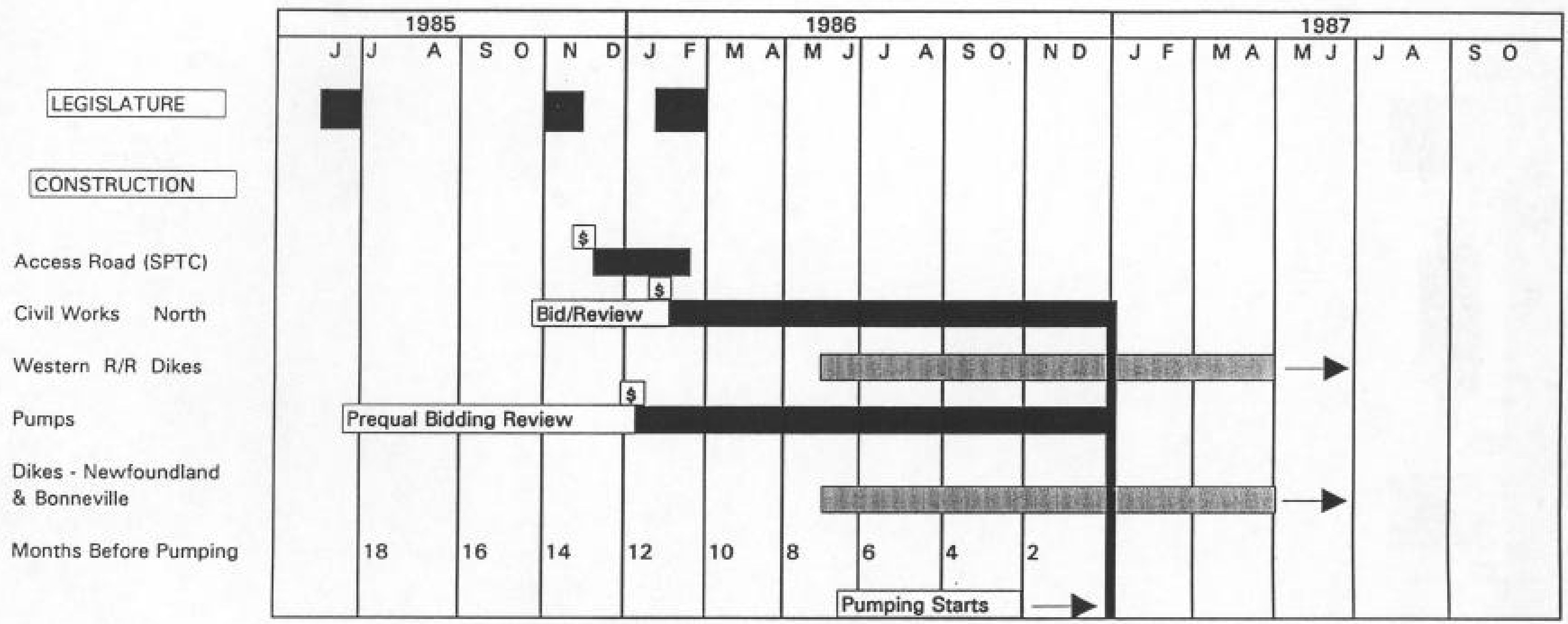
Issues That Impacted The Project

During the final detailed engineering investigation and studies phase of the project numerous issues were raised that could have impacts on the project. The EIS process and identification of guidelines for implementation of the construction of the project were two critical issues that impacted the project.

Other issues needed to be resolved and acted upon by the state before construction could proceed, but they were not considered to have critical project impacts.

Environmental Impact Statement Process - All of the engineering investigations, studies and designs were based upon the Preferred Alternative. The draft EIS was scheduled for completion in February 1986, and the final EIS in late 1986. Therefore, any significant changes required to the Preferred Alternative would have significant impacts upon the final design drawings and contract documents. It was felt that the modifications and changes made to the *1985 EIS Basis of Design* during the final design did not represent significant changes.

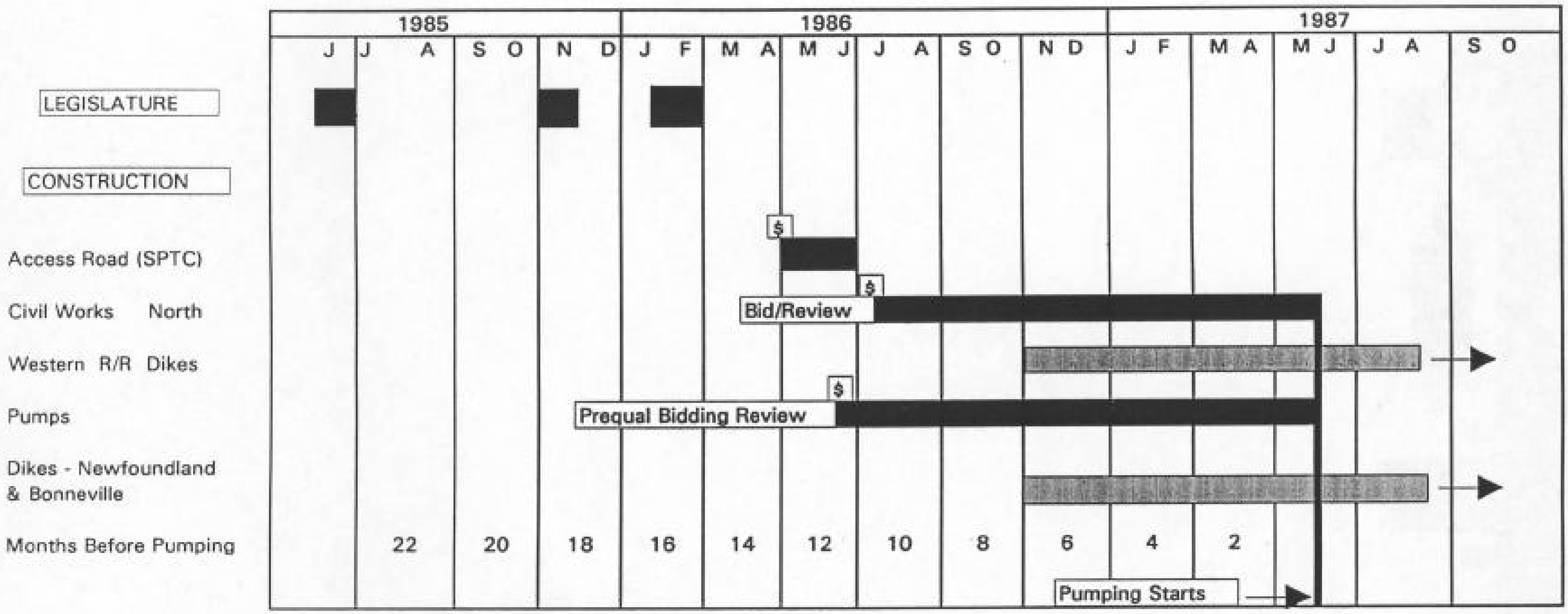
Figure 9-2
West Desert Pumping Critical Path



ALTERNATIVE #1 - PUMPING STARTS 1 JAN 87
Commitments to Funding: All commitments could be made by Special Session of Legislature in mid-September
Regulatory Constraint: Emergency Approval by CEQ

Table is reproduced from "FINAL REPORT, WEST DESERT PUMPING ALTERNATIVE, GREAT SALT LAKE", EWP Engineering, et. al., December 1983

Figure 9-3
West Desert - Critical Path



ALTERNATIVE #2 - PUMPING STARTS 1 JUN 87
Commitments to Funding: All commitments could be made by Special Session of Legislature in mid-September
Regulatory Constraint: Emergency Approval by CEQ

Table is reproduced from "FINAL REPORT, WEST DESERT PUMPING ALTERNATIVE, GREAT SALT LAKE", EWP Engineering, et. al., December 1983

The impact of significant changes posed a potential of changing the operation scheme which would require changes to the designs and drawings. These changes would impact the budget and schedule of the project. The state would have to commit money for the revision of the plans. Also, the bidding of the project for construction would be delayed until the plans were modified.

Implementation of the Project - The state took a first step in realization of the project by authorizing the preparation of contract documents and drawings for construction of the project. However, it was critical that the state begin the next step, that of outlining and approving a program of guidelines for implementation of the project.

Throughout the later years of the rise of the lake, the issue of when to begin construction was tied to the lake rising to a certain lake level. During the period of the design phase, the major discussion was that a lake level of 4210 to 4210.5 would trigger the construction project. The lake peaked at 4209.95 and construction was put on hold. This was a critical issue and had far-reaching impacts upon the project and the protection it would afford.

The issues included:

- procurement of pumps;
- land acquisition;
- SPTC cooperative agreements for construction, maintenance and operation of the project features;
- Construction engineering, inspection and testing agreements, and
- monitoring programs.

Lake Level Consideration- While it was prudent to delay construction of the project until it was needed, a construction trigger based solely on a specific lake level was not recommended for several reasons.

First, one of the project's main purposes was to protect essential facilities. At a lake level of 4211.6, operation of trains on the SPTC railroad causeway would be adversely impacted and operations limited. At about elevation 4212.6, the SPTC railroad operations would cease. The UPRR trackage and I-80 on the south end of the lake would be near their upper protection limit at a lake level of 4213 to 4214. At that point they would be abandoned and rerouted to higher ground. Also, the state's dikes which protect the seven essential facilities, primarily sewage treatment plants on the eastern shores, would have to be raised when the lake reached a level of 4212.

It was possible that if a lake level trigger of 4210.5 were selected, the lake could rise to level 4212, 4213, 4214 or higher before the project was constructed and pumping started. Therefore, a major justification for the implementation of the project was eliminated.

Second, there was a potential, based upon the point in the season when the go-ahead for construction was received, that it may take two years before the project would affect the lake level. To be more specific, if the go-ahead is triggered by a lake level of 4210.5 in April, then the pumping would not start until during the lake's peak level of June or July of the following year. This would mean that the facilities around the lake would have to endure two peak levels before the pumping was started.

Third, a repeat of the 1983-84 scenario was considered, when the lake's rate of rise pushed it up to nine feet from a level of 4200 to a level of 4209 and caused an estimated $176 million in damages. If the lake had been at elevation 4206 or 4207 in 1983, how high would it have risen; 4213, 4214, etc? This scenario would severely reduce the state's interstate transportation capacities. It had been projected that the 4217 scenario could cause over a billion dollars in damages and losses.

Considering the potential of the repeat of the 1983-84 lake rise scenario and the justification of the project to protect against a lake level of 4211.6 or higher, it was recommended that the state develop guidelines to determine when to implement the project. This issue, above all others, was critical to the success or failure of the project's ability to mitigate high lake levels. While the examples and guidelines presented here are simplistic, the undertaking and development of the program was not be an easy task. The following guidelines were suggested for inclusion within the implementation program.

The implementation program considered the level of the lake at the time of the forecast plus the following:

1) The "rate of rise" of the lake and its forecasted peak level for the next one, two, three and more years;

2) The "point in the season" when the lake is on a rising trend and the forecasted peak is developed; and

3) The "precipitation and snow pack levels" of the preceding and current and projected years.

If these and other pertinent items could be factored into a program to be used to forecast the lake level for implementation of the project, then the flood control benefits would be substantial. A situation similar to the 1983-84 rise could be mitigated. Assuming the 4215 scenario of the EIS, the peak would be mitigated. And if the program was implemented at a level of around 4209, the 4217 scenario could possibly be mitigated to an approximate level of 4211.

Pumps Placement - The procurement of pumps, engines and gear drives was identified as a critical path schedule item. Pump procurement was recommended to start at least five months prior to making the determination to implement the project and proceed with general construction. This allowed 12 months for delivery, installation and start-up of the pumps, plus three months "float" for unforeseen delays in the manufacture, delivery and installation process. Such delays could occur due to strikes at a manufacturer's plant or in the transportation industry; unusually severe weather conditions, natural catastrophes, or other "acts or God"; and problems during installation and testing of equipment. It was advised to include "float" in the schedule to allow for these unforseen delays. Also, the 12 months delivery time was more than the nine months used when the pump bids were received in October of 1985. The reason for the 12-month schedule was that most suppliers objected and took exception to the short nine-month schedule.

As outlined earlier, the procurement of the pumps from the notice to prepare contract documents to the start of pumping required a minimum of 21 months. Likewise, the process of constructing the project features would take 16 months.

Land Acquisition - The land that was to be acquired by easement or title had been identified. Of the total 1,995 acres required, 1,631 were federal and state; 297 were SPTC and 67 were private, non-railroad owners.

The EIS process secured the federal lands. The state was urged to take all necessary federal lands and to encumber its own state lands.

The railroad land was held by two companies: Southern Pacific Transportation Company (SPTC) and Sante Fe Land Company (formerly Southern Pacific Land Company - SPLC). Representatives of the SPTC indicated they would provide easement on their land at no cost to the state. They also indicated, however, that the acquisition process might continue after the project was completed, but that this would not delay the project. The SPLC land was to be acquired through the assistance of SPTC.

The private, non-railroad land was to be acquired by regular negotiation processes starting before or as soon as the decision to proceed with construction was made.

The state was urged to formulate plans and initiate actions as soon as the decision to proceed with construction was made.

SPTC Cooperative Agreements - Throughout the study and design phases of the project, a close coordination had been maintained with SPTC because the project's facilities parallel and adjoin the railroad's facilities. The SPTC offered many items at no cost or its cost because of inherent interest in the project. However, no formal agreements were executed other than letters from SPTC to the governor of Utah and minutes of meetings offering such items.

As the plans and specifications for the West Desert Pumping Project were finalized, it was necessary for the state and SPTC to jointly evaluate all project features involving railroad facilities, land, materials sources and operations, and negotiate the terms of a cooperative project agreement covering these project relationships and activities.

During the course of West Desert Pumping Project development, SPTC provided substantial data and engineering - planning services important to the project. Railroad operational constraints and temporary facilities required during construction were part of the SPTC-West Desert Group project planning and design process.

Some of SPTC's railroad embankments directly benefited the project. The causeway allowed the intake and pumping of lower brine concentration and therefore increased evaporation. The railroad embankment and access road from Lakeside to Hogup provided a barrier against allowing higher brine concentrations in the North Arm from contaminating the South Arm brines in the intake canal. Also, the existing access road and embankment provided a good base for the construction of the upgraded access road to the Pump Plant at Hogup.

The SPTC designed the West Desert Pumping Project bridge crossings, fuel spur, access road from Lakeside to Hogup, and any required railroad grade and track changes, as well as shoo-flies.

The SPTC was involved in a merger process with the Santa Fe Railroad (SFRR). Some public information had been released suggesting that SPTC might seal the route from Ogden to Sacramento to Denver and Rio Grande Western Railroad (D&RG). The sale did not appear likely and was not be considered in West Desert Pumping Project planning.

The cooperative agreements included the following items. The list may not be all inclusive and some issues may have required further agreements.

Construction Man Camp - The SPTC offered the space (land) as is at its existing man camp and its water at cost for the construction contractor to use for a man camp during construction. Should the contractor elect to utilize the man camp, the contractor was to bear all costs to prepare the site, provide housing and meal facilities and purchase the water and sewerage services from SPTC. Election to use the man camp was a contractor's option in the bid for the project. The selected contractor could elect to use other facilities.

Land Acquisition - The SPTC offered to negotiate easements for use of its land with right-of-way entry permits at no cost to the state. The company also offered required inundation permits of its land and SPLC's land for the necessary time period.

Quarry at Lakeside - The SPTC offered its present quarry at Lakeside as a project materials source at no royalty to the state. However, the state needed to pay SPTC's quarry operator for mining the materials. The SPTC included in its 1986 quarry bid the necessary materials required for construction of the pump project. The bid remained open until March 1987.

Fuel Delivery - The SPTC indicated it would haul all the required diesel fuel to the Pumping Plant site and provide tank cars for such use at no cost to the state.

The SPTC was recommended to be included as a member of the constructing engineering team concerning improvements and modifications to its trackage and facilities. These services were to be provided through direct agreement with the state or as part of the agreement with the construction engineering team.

Operation and Maintenance - Operation and maintenance of the four bridges, the access road from Lakeside to Hogup, the railroad fuel spur at the Pumping Plant and the canals and dikes were to be negotiated to determine who should operate and maintain them. It was advised that the state operate the water diversion system, but SPTC indicated it would like to maintain any project features associated with its operations. The agreements would include responsibility for reimbursement to the operating and maintaining entity.

Monitoring Programs - The state would monitor the operating systems. This program would include, at a minimum monitoring of the brine concentrations at the Intake, Pump Plant, West Pond (several locations), East Pond (several locations) and Siphon. The program would also monitor pump and evaporation rates versus siphon return flows.

Other basis items of the monitoring program would include wind, wind set and wave heights, rainfall, temperature, humidity, mountain runoff into the ponds and other basis meteorological phenomena and process operating data. ⌂

Reference

1. *Final Design Report-West Desert Pumping Project (Northern Part) Preferred Alternative - Great Salt Lake*, West Desert Group - a joint venture, Eckhoff, Watson and Preator Engineering and Morrison-Knudsen Engineers, January 1986.

Chapter 10

Construction of the West Desert Pumping Project

Introduction

In June 1986, the Great Salt Lake was at elevation 4211.85. Damages from high water had already amounted to $250 million, and the lake was not receding. Various solutions to alleviate the problem were proposed ranging from doing nothing to diverting entire rivers. The scheme finally selected, and put into action, was the design and construction of an emergency flood control project capable of pumping water from the Great Salt Lake to the West Desert. The plan combined lowering the lake with the increased evaporative action of the climate. Construction for the project began on July 7, 1986. The Pumping Plant was in full operation on June 3, 1987, with three pumps capable of pumping nearly 1.4 million gallons of lake brine per minute.

Design Contract

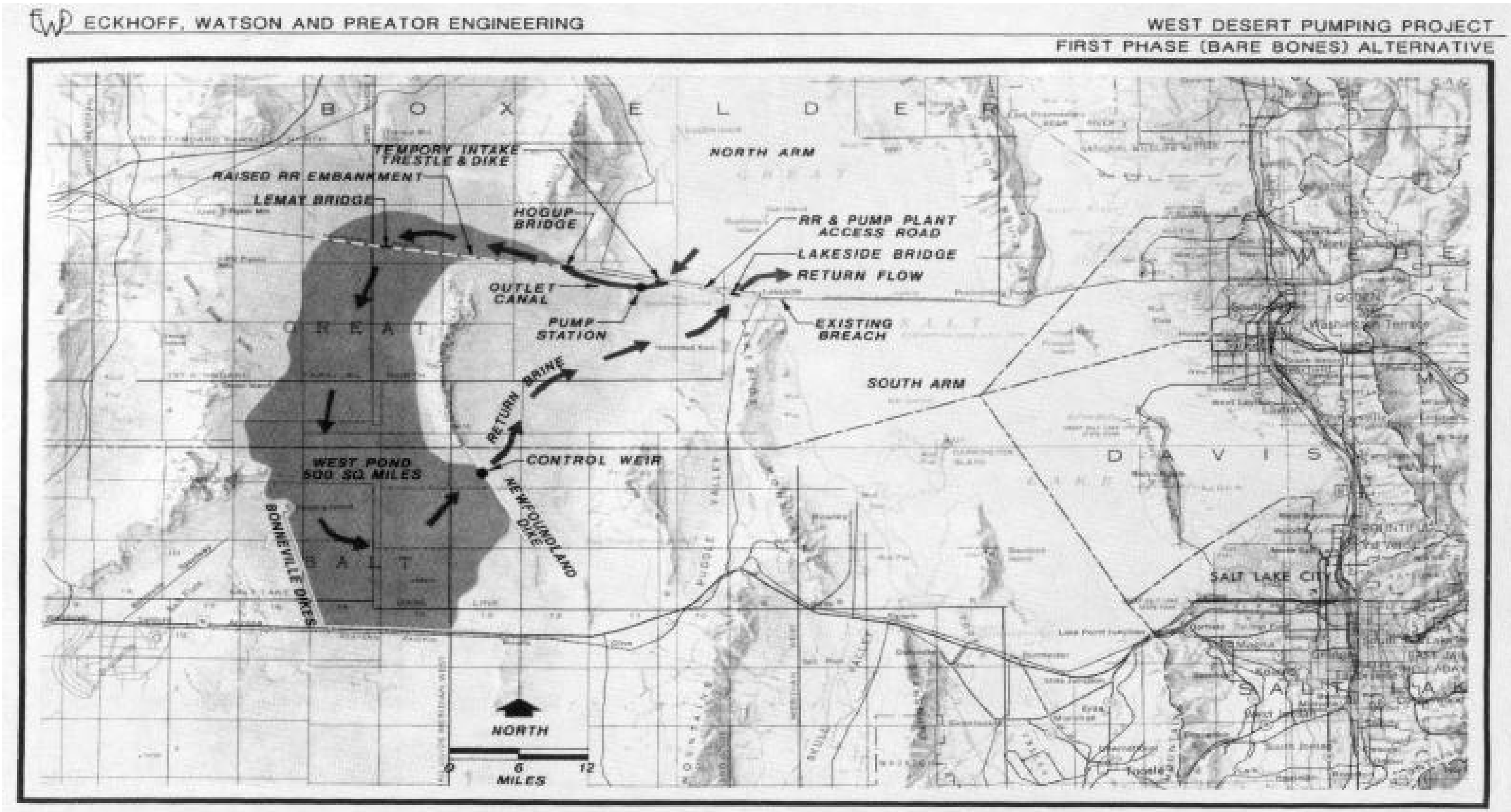
In July 1984, the state of Utah, Department of Natural Resources, Division of Water Resources, contracted with the joint venture team of Morrison-Knudsen Engineers, San Francisco, and Eckhoff, Watson and Preator Engineering, Salt Lake City, to design and prepare construction drawings and specifications for the West Desert Pumping Project Preferred Alternative described in Chapter 9. Estimated cost of the project was $80 million.

A revised "Bare Bones" version of the preferred alternative, illustrated in Figure 1, was eventually constructed. The revised design eliminated the long inlet canal, the barrier dike and the return syphon. Cost of the revised project was an estimated $52 million.

Construction Management Contract

Up to May 1986, a decision to proceed with the construction of the project had not been made. The lake by this time had risen to elevation 4211.65 and caused $250 million in damages to the surrounding commercial, public and private properties. To continue the gamble

Figure 10-1
West Desert Pumping Project, First Phase (Bare Bones) Alternative



that the lake would start to recede was a risk that the state decided not to take. On May 6, 1986, the Division of Water Resources notified the joint venture to submit a proposal for implementing and managing the construction of the project, which was to commence as soon as approval could be granted by the State Legislature and Governor Norman H. Bangerter. The first unit was to be on line by February 14, 1987. The proposal was submitted within two weeks, including the engineer's estimated schedule for construction, the estimated staffing for the construction management team, and the estimated cost. Although an agreement had not been finalized, the state directed the joint venture team to proceed with preparing bid documents and arranging for the bidding phase. Several meetings were held in May and June of 1986 between initial parties to the project; specifically, Ingersoll-Rand, the major equipment supplier; the Southern Pacific Transportation Company (SPTC); the state; and joint venture partners. During this time a decision for a major design change from diesel driven engines to natural gas driven engines was made by the state

Construction Contracts

Due to the urgency of the schedule and following a June 1986 storm which destroyed the SPTC's causeway, a "sole-source" contract between the state and the SPTC was let in late June 1986. The excavation subcontractor for SPTC, Lost Dutchman, Inc., was already on site as the SPTC's quarry subcontractor, resulting in expedient mobilization of equipment and personnel. The SPTC contract included excavating the Pumping Plant foundation, excavating the Outlet Canal, constructing four bridges, reconstructing the damaged 10-mile causeway, and raising 25 miles of railroad track for an original total contract price of $22,980.00.

Advertisement for bids for construction of the Pumping Plant and Inlet Canal commenced in June 1986. Bids were opened on July 22, 1986, with Layton Construction Co., Inc., of Salt Lake City, the low bidder out of seven submitted bids. Bids ranged from a low of $7,891,378.68 to a high of $14,837,320.00. The engineer's estimate was $12,891,583.85. Layton Construction Co., Inc., was awarded the contract on July 31, 1986.

The contract for procurement of pumps, gear drives and engines was awarded to Ingersoll-Rand of Painted Post, N.Y., for a total original contract price of $7,829,378.68.

The containment dikes at the south end of the West Desert, known as the Bonneville and Newfoundland dikes, were designed and bid through another consultant at a total cost of $6,336,875 for both dikes. W.W. Clyde and Co was awarded the contract for the Bonneville dike for a price of $3,872,845. The construction contract for the Newfoundland dike was awarded to Herm Hughes and Sons for a price of $2,464,030. These two contracts were outside the scope of the West Desert Group's contract with the state.

Organization

At the outset, due to the remote nature of the project approximately 110 miles northwest of Salt Lake City, no communications or utilities were at the construction site. The storm that occurred in June 1986 also wiped out the main vehicle access to the construction site from the base camp at Lakeside located 10 miles east of the site. Access to the site was then by boat from Lakeside, by vehicle from the Nevada side of Wendover, or from the north traveling over 50 miles of the Hogup Mountain area. Consequently, it was necessary to utilize two project offices; one on-site field office and one central office in Salt Lake City for liaison between the project and the various headquarters of the contractors, state and consultants.

Project Construction

Lost Dutchman, Inc., began excavating the Pumping Plant on July 7, 1986. Initial mobilization of its camp and equipment between June 7, 1986, the date of the storm, and August 22, 1986, had to be done via Wendover, Utah, or via the north through the desert due to the causeway repair work. The inspectors for the consultant reached the work site by boat from Lakeside.

Excavation of the first 20 feet to 30 feet of the hole found little or no groundwater. But after 30 feet the flow increased with depth until on August 11, 1986, it measured 12,000 gpm with the excavation at elevation 4179. The contractor had gradually added to his original dewatering system, based upon a geotechnical report which forecasted no more than 300 gpm.

Excavation of the Pumping Plant was completed on August 31, 1986, and the site was turned over to the Pumping Plant contractor, Layton Construction Co., Inc. (LCI).

After verbally accepting the excavated foundation and dewatering system on August 31, 1986, the contractor, LCI, requested further work by the previous contractor prior to starting the mud mat. Consequently, on September 3, 1986, the previous contractor excavated a drain trench connecting the west keyway of the foundation with the pump sump located at the east end of the foundation in the forebay. After that, on September 4, 1986, the LCI superintendent signed an acceptance of the excavation and dewatering system. A french drain system recommended by the engineer satisfactorily handled a flow of groundwater as high as 12,000 gpm during construction of the Pumping Plant.

The concrete batch plant installed by LCI's concrete supplier, CPC, was a mobile, low profile, dry batch plant. Approximately 15,000 c.y. of concrete was used in the Pumping Plant. At least 3,000 c.y. of this total was due to overbreak and concrete added for a cutoff key under the siphons and a training wall on the right side of the afterbay.

The first lift of structural concrete was for the base slab. It required 1,900+ c.y. and took two days to place, working around the clock. The lift was completed September 24, 1986.

The contractor's initial schedule indicated the engine deck at elevation 4230 would be placed no later than November 20, 1986. Allowing 14 days extension of time for delays not attributable to the contractor would have extended this date to December 3, 1986. Allowing another 17 days extension of time for the delay caused by the flooding of the Pumping Plant excavation, the initial schedule would have had the engine deck placed no later than December 20, 1986. A minimum of seven days strength gain, or actually 14 days over the Christmas holidays, would have enabled the electrical and mechanical subcontractors to move in and commence very critical work no later than the first week of January, 1987. If all other contract dates were extended by the 31 days mentioned above, the first unit should have been ready for testing by March 17, 1987.

To ensure that the roof cover could be expedited, a design change was made from cast-in-place concrete roof slab supported by shoring the false work, to "permi-form" construction using galvanized steel Q-decking as the permanent form support for the concrete to be placed at a later date. This alone saved the time which would have been necessary to install shoring and false work, place and cure the concrete, and remove the shoring and false work prior to installing the plant engines and auxiliary equipment - probably a month at least. The contractor agreed to a trade-off for the costs of the Q-decking versus the costs of the labor, materials and time for shoring and false work.

Another recommendation by the engineer was to use Type III high early strength cement in lieu of Type II as specified. This would have required a modification to the batch plant for which time did not allow. Instead, it was decided to use 3,000 p.s.i. concrete in lieu of 1,500 p.s.i. concrete in the lifts designated as mass concrete. This gave higher strength at an earlier date in these lifts, but at a higher cost.

In November 1986, with the idea that the engine deck would be placed and the roof covered no later than January 10, 1987, the engine and pump supplier, Ingersoll-Rand, was requested to expedite its shipping schedule for the Unit 1 engine and pump components. The original schedule had the engine arriving January 30, 1987. If the contractor met his most recent schedule, this would mean that the mechanical and electrical subcontractors would have had to wait approximately two weeks until the engine was set in-place before they could have started work for final connections. Due to space restrictions, some of the auxiliary equipment could not be moved into the plant and set in final position prior to the setting of Unit 1.

Minor piping work had proceeded in the lower structure along with the preparations for concrete, and a major amount of piping was installed following the placement of the gallery deck at elevation 4221 in December, 1986. This piping was for cooling water, lube oil, natural gas, potable water, drainage and instrumentation.

Prior to placing the engine deck, the electrical contractor installed all the conduit which was to be embedded in the slab. Still another design change intended to save valuable time was the relocation of a major portion of the electrical conduit from this slab to hangers along the precast panel walls for the superstructure. The engine deck was placed on January 29, 1987

The main pump components for the three units were the lower embedded rings, the discharge elbows, upper embedded frame supports, shafts and impellers, outer casings, suction bells, upper casings, diffusers, and gear support structures. The tolerances for the lower embedded rings were to 0.0005 inch per foot in level across the ring diameter. Pump erection for the embedded parts was second stage for the lower embedded rings and the upper frame supports, and the first stage for the elbows and columns. The installation of the rotating parts, outer casings and suction bells was then paralleled with other construction activities and subcontracted to a division of Ingersoll-Rand.

The engine for Unit 1 arrived in Salt Lake City on January 10, 1987, and was put on public display at the Department of Natural Resources Building parking lot before being moved to the plant site. At the project site, the contractor did not unload the engine from the transporter until January 19, after which time the engine remained on its parking slab until the engine deck was ready to receive it on February 6, 1987.

The method used to yard in the engine to its final position in the pumping plant was a system of rails, hydraulic come-alongs, and hydraulic jacks on trucks that ran along the rails and supported the weight of the engine, plus skids (165,000 lbs.) by steel beams spanning the tops of the jacks. The rails were segmented so that sections could be removed from under the engine prior to setting it down. Once finally positioned, the engine was grouted with special epoxy grout which required careful thermal control. Each engine was set in this manner.

Before the first engine was yarded in, the two Cummins natural gas driven generators were set into place through the incompleted roof covering by crane and jacked into final position. Other auxiliary equipment which had to be set before the first engine was moved in were the engine control panel, plant and instrument air receivers, and the compressors.

The gear reducer equipment for Unit 1 was set into place following the setting of the engine. Then followed the heat exchangers and cooling water surge tank for that unit. Due to space restrictions, this was the sequence followed for each unit. The mechanical subcontractor was then able to commence piping between the various pieces of equipment.

On February 11, 1987, the engineer submitted a feasible schedule for attaining a "ready-to-start" date for Unit 1 no later than March 23, 1987. The contractor used this schedule with some modifications to submit his schedule to meet this date with an accelerated program. Preliminary discussions with the contractor had resulted in a new change order on the basis of a bonus plus time and materials for the accomplished of this schedule. With a good effort by all his subcontractors, the first unit was ready to be tested by March 26, 1987. Following initial system tests, the engine was "bumped" on April 4, 1987, at 15:00 hours. The following day, Governor Norman Bangerter officially started the Unit 1 pump by remote switch at a public ceremony at the plant. All went well.

Other construction activities which paralleled the above installation work, and which were necessary for the successful operation of Unit 1, were the completion of the service building control room with the motor control center and switch gear, installation of the 24-inch vacuum breaker butterfly valves on the afterbay deck, installation of the two vertical shaft cooling water pumps in the sump well, completion of the trashrack and stoplog guides and their installation, testing of the dewatering pumps, installation of the gantry crane, removal of the intake canal cofferdam, and prewatering up the outlet canal and afterbay so that the initial surge of water from the pumps would not scour the channel.

Intake Canal Excavation

Layton Construction Company's subcontractor for the Intake Canal was BECHO, Inc. They started on August 18, 1986, grubbing and clearing the area between the forebay and the waterline of the lake on the south side of the SPTC railroad track. The canal extends from the forebay of the plant 1,200 feet to the railroad bridge which crosses over the Intake Canal and into the north arm of the Great Salt Lake. The canal bottom daylights to elevation 4208 in the lake from elevation 4190 in the principal channel. It drops to elevation 4175 at the intake sill of the plant in the forebay. The forebay of the plant was excavated by Lost Dutchman, Inc., subcontractor to SPTC, at the time the foundation for the plant was excavated.

BECHO started excavation on the land reach south of the railroad using dozers and backhoes, The first 200-300 feet from the beginning of the forebay was in blocky limestone. So it was drilled, loaded and shot, and the material was used for construction of the diversion dike which parallels the southern bank of the canal on the south side of the railroad. Later, BECHO mobilized a dredge to complete the excavation of that portion of the canal inundated by the lake. Final removal of the plug dike upstream of the forebay was completed in late February 1987. The total amount of excavation for the Intake Canal was 99,379 c.y. common and 20,000 c.y. rock.

Outlet Canal Excavation

The SPTC's subcontractor, Lost Dutchman, Inc., excavated the Outlet Canal. Work started in July 1987 and was completed on schedule in February 1987. The canal extends 4.11 miles westerly from the afterbay of the plant to a SPTC railroad bridge, Hogup

Bridge, at milepost 719.06 where the canal daylights and canal flow is discharged as sheet flow onto the desert north of the railroad. From there the sheet flow advanced along the north side of the railroad a distance of 15.9 miles to another railroad bridge, LeMay, at milepost 703.09, where the flow returned to the south side of the railroad and on a southerly course toward the west evaporative pond.

Design of the outlet canal took into account the nature of the soils to be encountered and the hydraulics of flow to be accommodated. Sections varied from 45 feet bottom width and 11/2:1 side slopes in rock to 90 feet bottom width and 2:1 side slopes in clay and sand. The flow depths ranged from 16 feet at the afterbay to 10 feet in the lower reaches.

The contractor utilized various equipment throughout the project, depending upon the material encountered. A dragline was used for the last mile of canal before its discharge onto the desert flood plain. In the upper reaches, a Holland scraper and belt loader, pushed and pulled by D-9 tractor dozers, was used with a fleet of 50-ton rock wagons until the sail became too wet for its practical production. Other equipment used was a fleet of scrapers and as many as seven large backhoes. The contractor worked around the clock, supporting his labor force with a well-organized man camp at Hogup, as well as in Lakeside. ⌂

Figure 10-2
Pumping Plant Construction Site at Hogup Ridge Looking West

Figure 10-3
Pumping Plant Construction Site at Hogup Ridge Looking East

Figure 10-4
Pumping Plant Excavation

Figure 10-5
Construction of Base of Pumping Plant

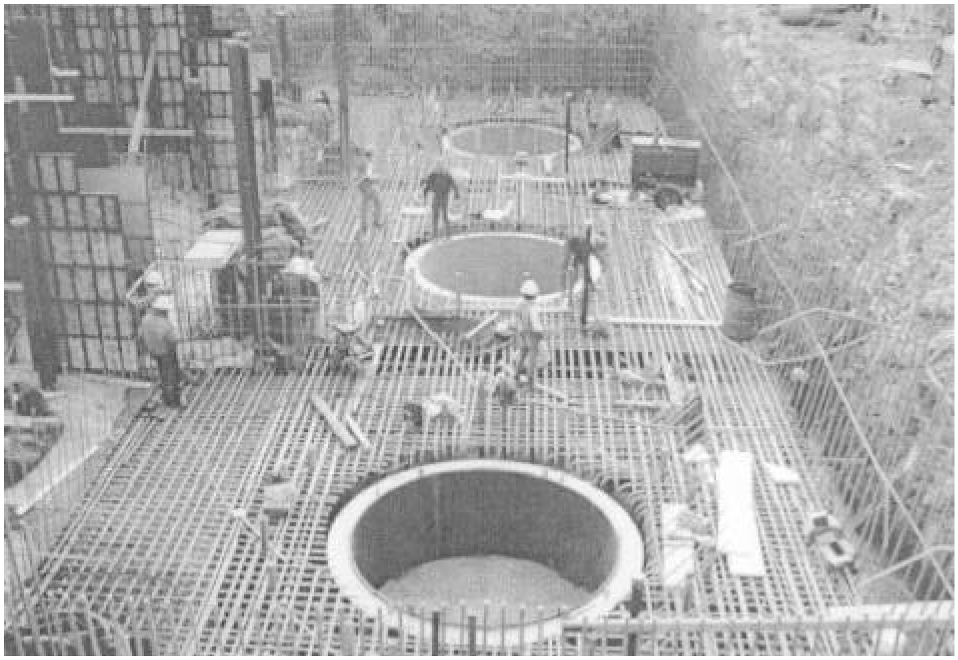


Figure 10-6
First Engine Being Put in Position

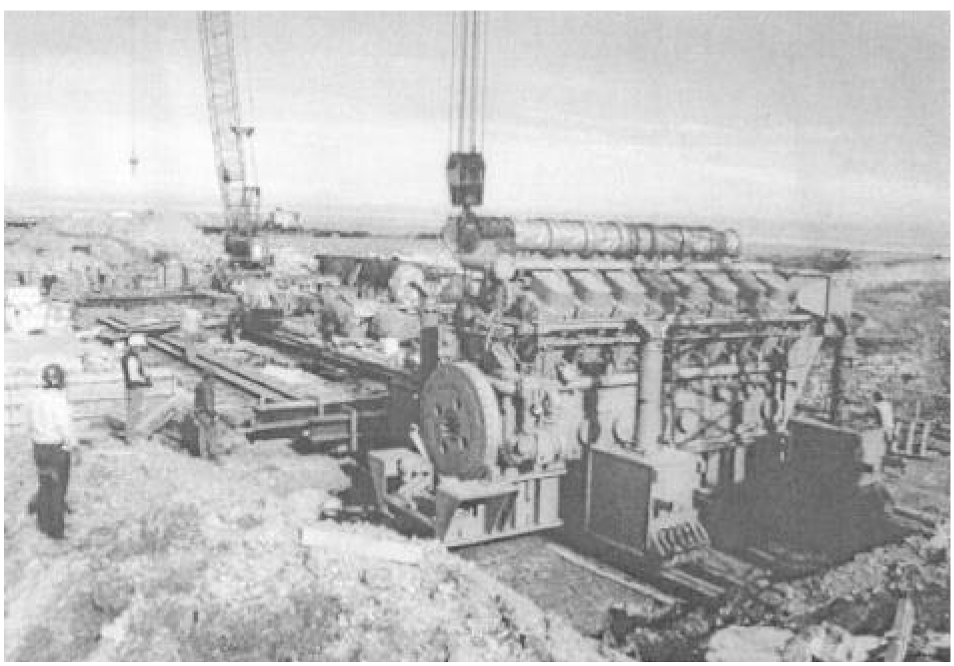


Figure 10-7
Schematic of Pumps and Engines Placement in Pumping Plant

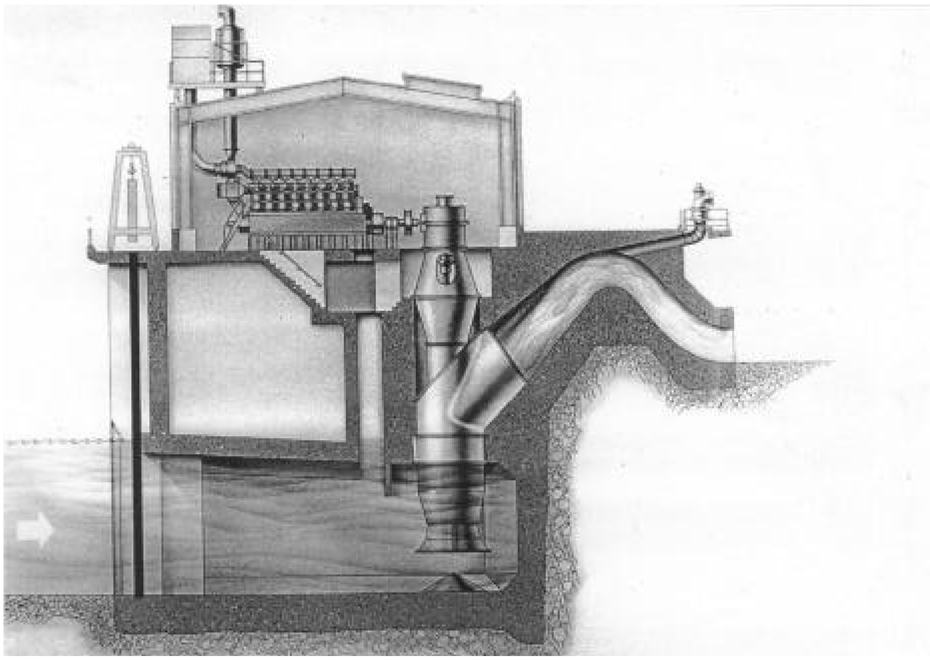
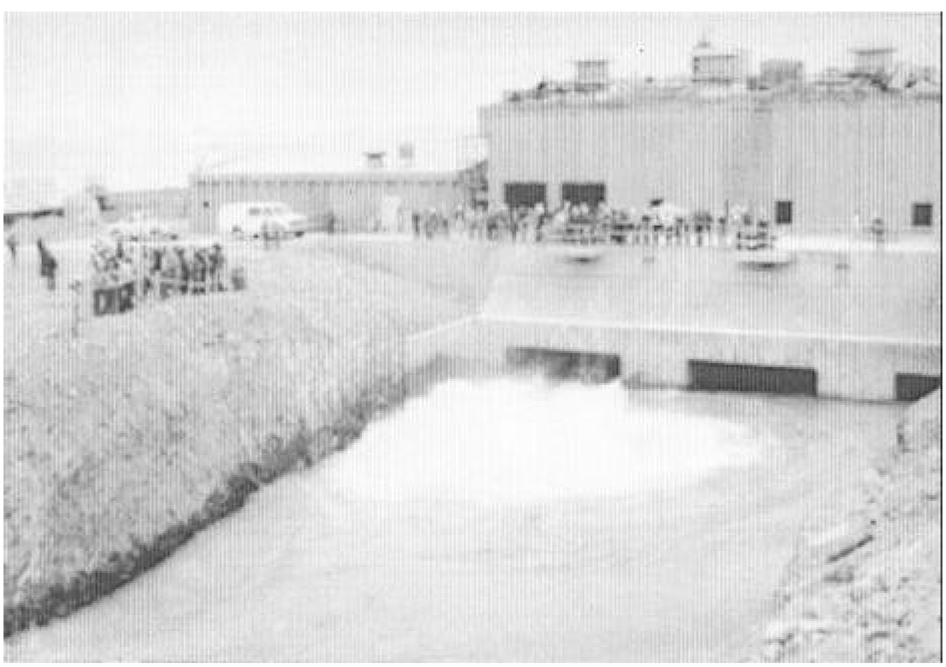


Figure 10-8
First Pump Turned On By Gov. Norman H. Bangerter



Chapter 11

Operating the Pumping Project

The Pumps Start

Section 73-23-5(1) and (2) of SB 150 of the 1988 General Session of the Utah State Legislature requested the Division of Water Resources to evaluate the first-year operation of the West Desert Pumping Project and report this information to the Energy, Natural Resources and Agricultural Interim Committee. The report, *Evaluation of West Desert Pumping Project, Senate Bill 150, October 1988*, analyzed actual performance and evaluated the amounts of water pumped and the effect on the level of the Great Salt Lake, natural gas consumption, evaporation rates in the West Pond and performance of the West Pond. It also defined the operational range and limitations of the pumping project as an emergency flood control project, including the physical constraints imposed on the operation of the pumping project by elevations of the Great Salt Lake and its limitations.

Data were gathered continuously, beginning with project startup in April 1987, and the requested evaluation covered the first 17 months of operation. Pumping data were compiled from monthly pumping operations and performance reports. Actual pumping volumes were determined from hour meter readings reported by the Pumping Plant operator, Dresser-Rand, Inc. Pumping rates were established by flow tests completed by Eckhoff, Watson and Preator Engineers (EWP) and the USGS in August 1987.

Areas of the West Pond were developed from satellite imagery by BYU Professor Woodruff Miller. These areas were similar to area/volume curves developed by the Division of Water Resources, and both sources were utilized to develop area/volume curves used in the operational computer models. Water was returned to the Great Salt Lake from the West Pond by return flow over the Newfoundland weir or removed by AMAX Magnesium for its brine pond system near Knolls, Utah. This information was reported by Bingham Engineering and AMAX Magnesium.

Climatological data, including temperature, humidity, solar radiation, wind speed and wind direction, were collected from seven remote weather stations located in the project area by the Utah State Climatologist's Office, Utah State University. Analyzed information was then employed by an evaporation model developed by EWP which simulated operations of the entire system. The model was capable of hindcasting or forecasting the project's performance, and it provided water and salt budgets for the project using climatological and pumping data.

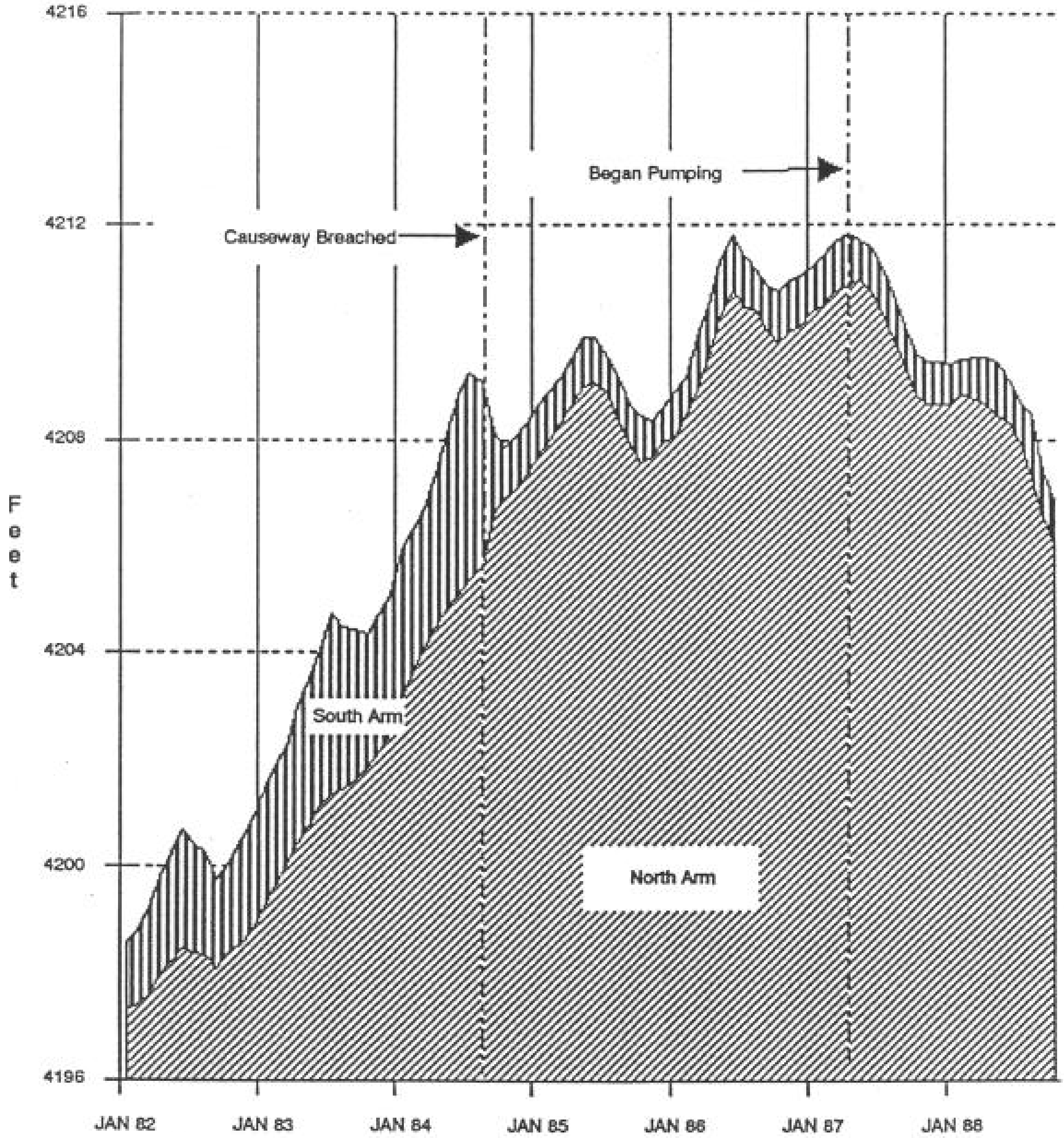
Pumping - During the first 17 months of operation, April 10, 1987, through August 31, 1988, the project removed more than 1.75 million acre-feet of water from the Great Salt Lake. This lowered the water surface of the Great Salt Lake approximately 14.5 inches and caused the lake to recede by approximately 50,000 acres of shoreline. Over 1.4 million acre-feet of water was removed during the first year of operation, representing about 40 percent of the total lake level decline. Figure 1 shows the elevation of the north and south arms of the Great Salt Lake between 1982 and 1989. It shows the effects of the causeway breach in 1984 and the west desert pumping project on the lake's levels. Figure 2 illustrates the cumulative volume of water pumped into and evaporated from the West Pond. Net evaporation is the actual evaporation less precipitation. As of August 31, 1988, the West Pond contained approximately 400,000 acre-feet of water. Net evaporation, therefore, was approximately 1,350,000 acre-feet. Almost 660,000 acre-feet of water had been evaporated through the first year of operating the pumping project. Divergence of the two curves, beginning in approximately November 1987, indicated the West Pond had reached its operational level. Net evaporation then became the principal means of removing water from the lake. Relatively minor amounts of water returned to the Great Salt Lake from the West Pond.

The first pump went on line on April 10, 1987, with the water surface elevation of the Great Salt Lake near its historic peak. The second pump started up on May 4, 1987, and the third pump joined the battle on June 3, 1987. The increase in monthly amounts pumped can be seen in Figure 3. The pumps were on approximately 80 percent of the time during the first nine months of the project, April through December 1987. During that period the Pumping Plant averaged 122,000 acre-feet of water a month, or 2,025 cubic feet per second (cfs). With below normal inflow, the lake reached its annual peak in February 1988. As the lake level receded, wind tide effect on the intake brine became a major factor in plant operations. Several times a steady southern wind moved water away from the intake canal and caused pumping to be suspended.

During the first eight months of 1988, the pumps operated approximately 50 percent of the time, as noted in Figure 4. The pumps removed an average of 83,000 acre-feet, or 1,375 cfs, of water per month from the lake.

Evaluating the first year operation of the pumping project against original design criteria was not realistic because design scenarios were based on the West Pond filling during the winter months when evaporation was low and the natural evaporation process working on a large body of water during the summer. The West Pond did not fill, however, until November 1987 when the peak evaporation season was over. Evaluation of the actual performance of the pumping project instead was prorated to an annual figure and compared to anticipated performance. Original design anticipated annual pumping of 1.45 million acre-feet of lake water into the West Pond, evaporating 1 million acre-feet, and returning 100,000 acre-feet to the lake from the West Pond. Prorating the project performance since startup yields an actual performance of pumping 1.25 million acre-feet of water into the West Pond, evaporating 950,000 acre-feet,

Figure 11-1
Elevations of the Great Salt Lake, 1982 - 1989



Eckhoff, Watson and Preator Engineering
West Desert Pumping Project

Figure 11-2
Cumulative Volumes Pumped and Evaporated

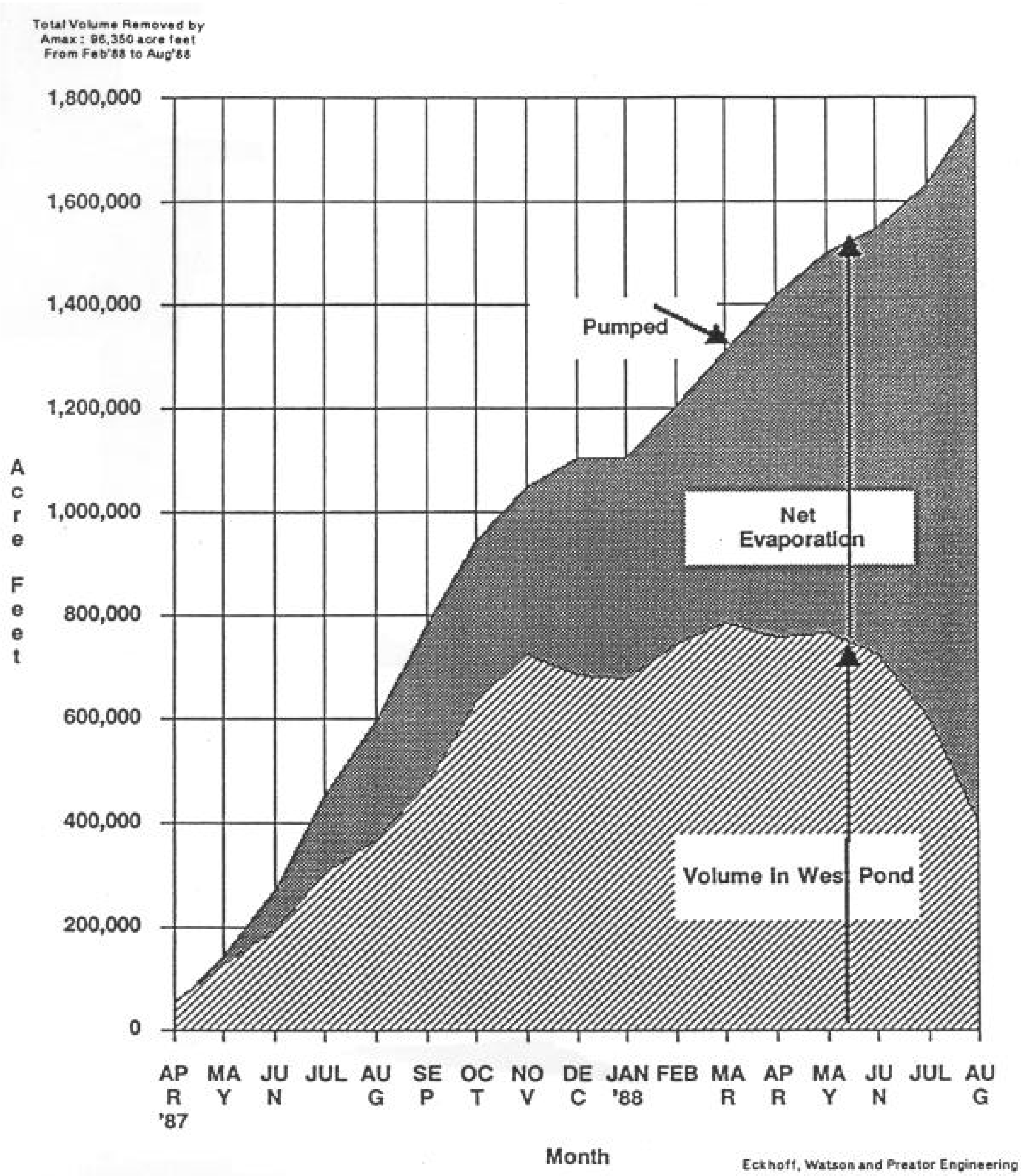


Figure 11-3
Monthly Pumping Volumes, April - December 1987

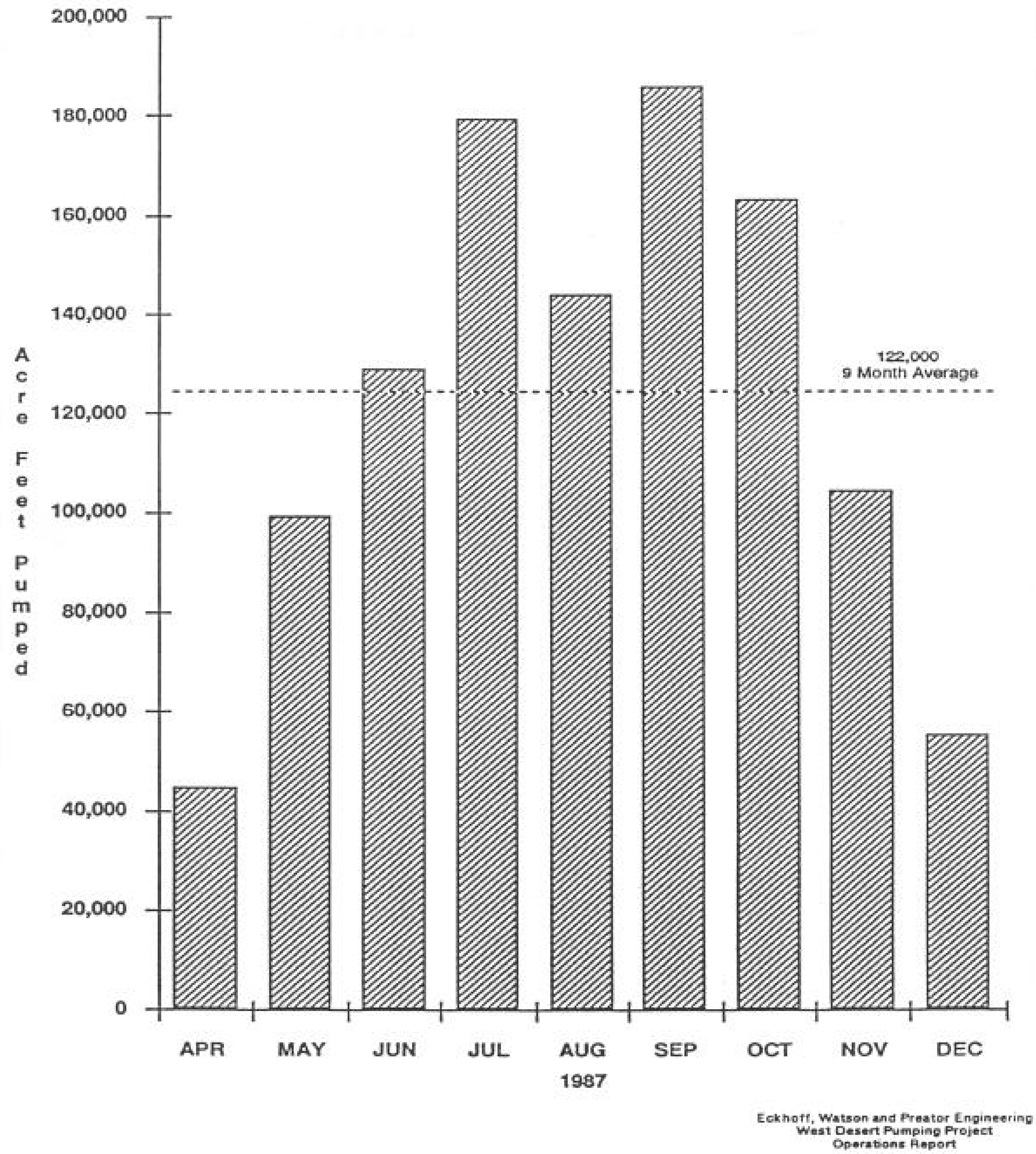
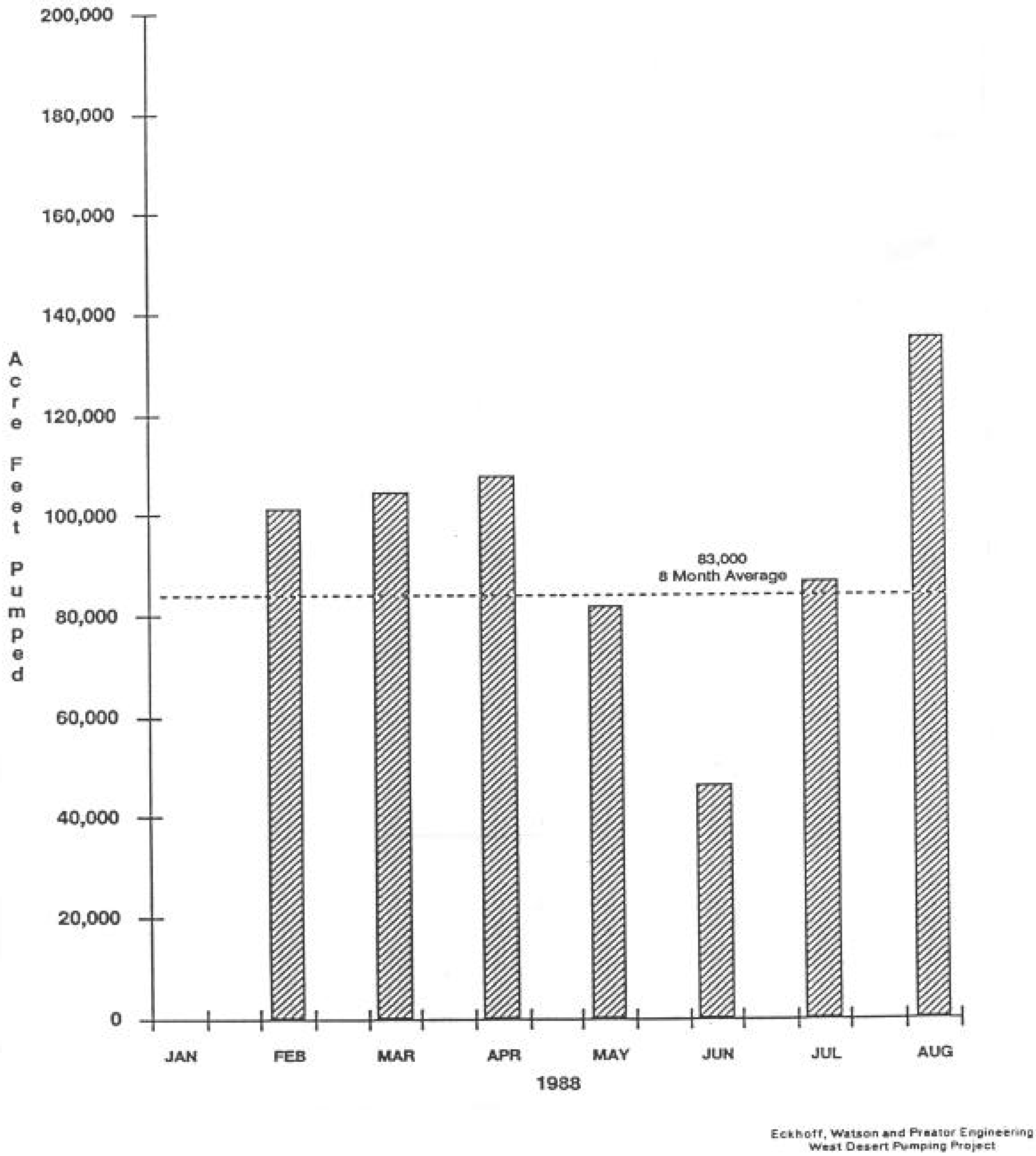


Figure 11-4
Monthly Pumping Volumes, January - August 1988



and returning only minor amounts to the lake. Comparing these values, it was concluded that the project's actual operation was consistent with anticipated design intent.

Gas Consumption - Monthly volumes of natural gas in million BTUs consumed by the three 3,500 h.p. engines driving the vertical pumps during the first operational year were reported by Mountain Fuel Supply. Gas consumption at the Pumping Plant varied directly with the volume of water pumped, but actual gas consumption was consistent with design specifications. Generally, with each engine operating at maximum speed, the plant consumed 43,000 million BTUs or 39,300 cubic feet of natural gas per month. At minimum speeds the plant consumed 30,500 million BTUs or 28,000 cubic feet per month. Gas consumption generally improved with operator adjustments and fine-tuned engines.

Evaporation - Evaporation rates for the West Pond were estimated using climatological data as a base in the systematic model and comparing the predicted results with observed performance of the West Pond. Figure 5 compares normal (anticipated) evaporation of the West Pond with actual West Pond evaporation. Normal evaporation was calculated using the translated normal (30-year data base) actual weather data as experienced on the West Desert. Actual pond evaporation was determined from actual freshwater evaporation with allowances for salinity of the brine. As the pond became more saline, evaporation rates reduced, and vice versa. The actual evaporation rates entered in the computer model produced results which closely emulated the actual observed performance of the West Pond.

Figure 5 shows that during the first nine months of the pumping operation in 1987, actual evaporation in the West Pond fell below normal rates in all but two months. During the first eight months of 1988, evaporation rates were approximately 10 percent above normal.

Figure 6 shows comparisons of temperatures and monthly winds velocities, the two main components in the evaporation process. The charts compare actual vs. normal high temperatures and actual vs. average wind speeds.

The pumping project was evaporating water at a greater rate than anticipated due to climatological factors, i.e., higher than normal wind velocities, and higher than normal temperatures.

West Pond - The surface area and growth of the West Pond was tracked monthly using satellite imagery. The West Pond filled to its operation water surface level range after five months (November 1987) of pumping. From November 1987 through June 1988, the pond operated at a water surface elevation range of 4215.5 to 4216.5 (covering 255,000 to 315,000 acres). Then the West Pond volume declined due to higher than normal evaporation rates and lower than anticipated pumping rates.

A sampling program of the West Pond indicated the pond was well mixed and not normally stratified with light and dense brines. Brine sampling also verified computer model predications of anticipated densities (salinity) throughout the pond.

As anticipated in the project design, from April through August of a given evaporation season, evaporation generally exceeded inflow from the pumps. This created a net reduction in the pond's surface elevation, area and volume. The remainder of the year's inflow from the pumps was greater than evaporation. The pond reacted as predicted by the specialized evaporation model for this project.

Operational Range and Limitations of the West Desert Pumping Project - The operational range of the West Desert Pumping Project is directly linked to the water surface elevation of the Great Salt Lake. The Pumping Plant was designed to operate down to a lake water surface elevation of 4205. Due to limitations of the Intake Canal, the project now can only be operated to elevation 4207. The

Figure 11-5
Monthly Evaporation in the West Pond, Actual vs. Normal

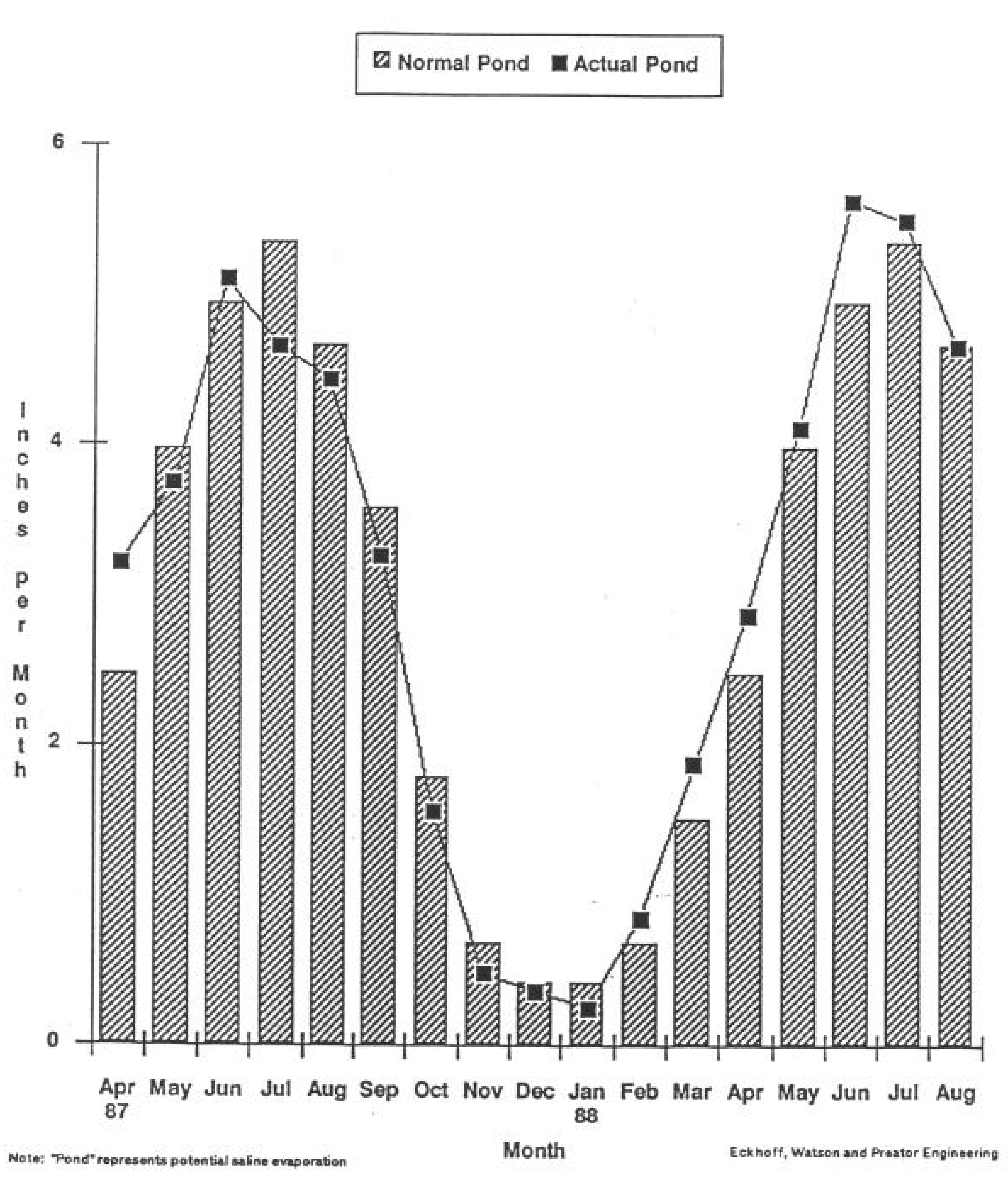
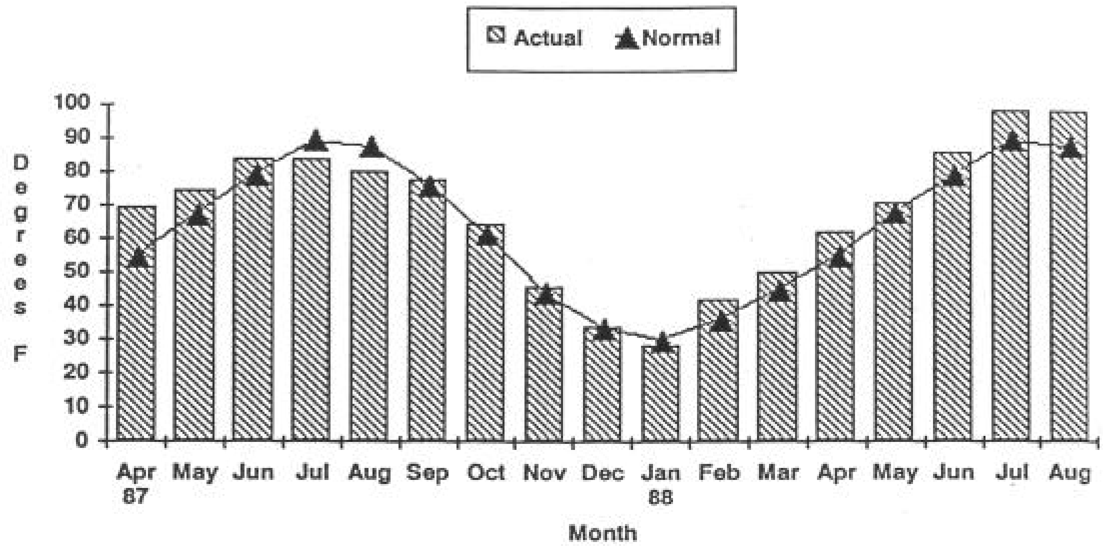


Figure 11-6
Average High Temperatures, Actual vs. Normal



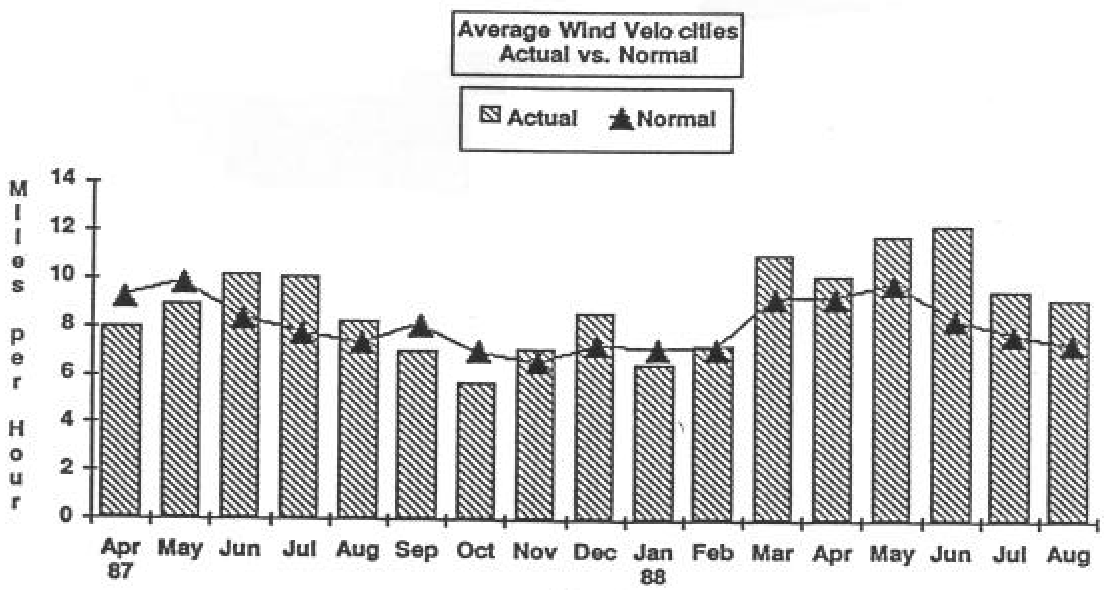


FIGURE 6

Eckhoff, Watson and Preator Engineering
West Desert Pumping Project
Operations Report

upper limit at which the Pumping Plant could operate is water surface elevation 4216.5. At this elevation, the gallery way in the Pumping Plant would be flooded.

Other constraints alter the way the project operates. Design features allowed pumping down to elevation 4205, but the U.S. Air Force permit limits pumping to around elevation 4208. At a water surface elevation of about 4217, water would naturally flow into the West Pond area and submerge the Newfoundland Weir. The West Pond would then be an extension of the Great Salt Lake and pumping would be unnecessary.

Limitations of the pumping plant to act as an emergency flood control project are dependant upon the elevation of the Great Salt Lake when pumps start up, the time of year when pumping begins, the extent of the hydrologic cycle being experienced at the time, and the prediction of future hydrologic cycles.

Public Tours to Hogup

Possibly the best seat in town on July 20-22, 1987, was on a state-chartered tour bus to the West Desert Pumping Project.

Tours of the project ordered by Governor Norman H. Bangerter in response to keen public interest in seeing the huge pumps in action and the river of water flowing from the Great Salt Lake into the western desert were hosted by personnel from the Division of Water Resources. Interest was spawned by the flooding, political controversy over spending $58 million for the project, and the highly publicized cross-country truck journey of the huge engines. And perhaps the mystery of the site of the Pumping Plant was an attraction, because it is a part of the state rarely seen by the general public.

Almost 1,500 people were shuttled free the first day along the 10-mile Southern Pacific Transportation Company causeway from Lakeside to the Pumping Plant site at Hogup Ridge. Many people waited up to two hours during a rainstorm that persisted most of the first day for a seat on one of three buses. Two buses were used the next day, and six on the last day.

Permission to conduct the tours was negotiated with the U.S. Air Force and SPTC, because most of the area is military or railroad property.

The first half of the 20-mile round-trip bus ride featured a quick explanation of causeway construction and the flow of water through the project, identification of lake islands and other landmarks, and bits of lake history. Visitors to the plant were first shown photos and drawings in the plant's entrance area about the project's construction and operation. They were then shown to the control room where plant personnel explained plant operation. Ear plugs were passed out and people were escorted on a walk-through of the noisy engine room. Outside again, the tour took in the pumping plant's forebay, an explanation of the natural gas pipeline and operating system, and a trip across the plant's outlet area with a view of the four-mile long canal to the West Desert. The return bus trip was all questions and answers.

A second round of tours was held August 19-21. Visitors were charged $2 a person for the tour, and reservations were requested. Final tours, on a weekend, were held September 12-13. Reservations and a $2 fee were also required for the Saturday and Sunday tours.

An estimated 4,700 people saw the pumps during the three tour sessions. In addition, numerous tours were conducted during the project's pumping period for students on school district buses.

Response to the tours was gratifying to the governor and division engineers and managers. Typical comments were:

"Just reading about the project, I didn't realize how big this was. I thought it was a waste of money. Now that I've seen it, I think it's a worthwhile investment."

"I've worked on the railroads and those engines are big, but they are nothing compared to the turbines. These things dwarf them."

"It was really neat," exclaimed one eight-year old. "It's weird when you get in there because the screens make it feel like you're going to fall through the floor."

State officials believed response to the tours was overwhelmingly positive.

Early Problems

Project engineers anticipated problems with the pumping project once it got underway, because it had been constructed and put into operation so quickly. Happily, only three problems briefly shut down some or all of the pumping operation.

The first was a burned out lower bearing in the large pump of Unit 1 during the first week after startup. The pumping project's sophisticated sensor system detected the problem before major damage occurred. The damaged bearing was quickly and easily replaced. The bearing cost an estimated $50,000, but it was covered by a manufacturer's warranty.

Nearly six months into the full pumping operation, a cooling water pump failed, shutting down the pumping operation for several days. The lake brine apparently corroded the pump's shaft near a coupler and the pump erupted from its base. The cooling water pump served the three pumping units, distributing lake water to engine heat exchangers and to cool engine oil and turbochargers.

It was determined the coupler and shaft were made from different grades of stainless steel, and the heavy brine caused crevice corrosion on the lower grade 1.5-inch shaft. A replacement cooling water pump with a 316 stainless steel shaft and coupler was installed.

The third problem was a phenomenon called Glauber salt, or mirabilite, that thinly coated the pumps' impellers and section bells during the winter and shut down the pumping project from December 14, 1987, to February 2, 1988, and at approximately the same time in 1988-89. The coating wasn't uniform and caused almost imperceptible vibrations that were caught by sensors in the Pumping Plant. The turning impeller was a natural attractor.

Officials at Great Salt Lake Minerals Company, who had encountered Glauber's salt during their operations, explained the Glauber's salt is a colorless crystalline sulfate of sodium that apparently can form on objects in the Great Salt Lake when the lake brine is 28 to 30 degrees, and can occur anytime between mid-September to February. Although divers removed most of the coating from the pump impellers with hot water shortly after the first occurrence, project managers determined nothing could be done to prevent the occurrence of the Glauber salt. ⌂

Fact Sheet: West Desert Pumping Project

Operating facts:
(April 10, 1987 to August 31, 1988)
Volume pumped: 1,767,995 acre-feet
Volume evaporated: 1,350,000
Volume returned to lake: negligible
Effect on lake: lowered 14.5 inches

Pumping Rate
One pump: 450,000 gpm (1,000 cfs)
Two pumps: 900,000 gpm (2,000 cfs)
Three pumps: 1,350,000 gpm (3,000 cfs)

Pumps
Type: Ingersoll-Rand vertical axis mixed flow
Impeller: 3 blades, 119 in. dia., 12,000 lbs.
Shaft: 10.7 in. dia., 45.5 ft. long, 18,750 lbs.

Gear Drive
Weight: 36,000 lbs.
Speed ratio: 22.37
Manufacturer: Brad Foote Gear Inc., Cicero, IL

Engines
Dimensions: 27 ft. 11 in. long, 11 ft. 10 in. wide, 17 ft. 10 in. high, 16-cylinder 3,500 hp rating, 16.25 in bore, 18 in. stroke
Operating speed: 330 rpm
Weight: engine and skid - 162,800 lbs.
Manufacturer: Dresser-Rand, Painted Post, NY
A contract for pumps, gear drives and engines was awarded to Ingersoll-Rand, Painted Post, NY, for $7,829,378.68

Pumping Plant
Dimension: 110 ft. long, 55 ft. wide, 85 ft. high
Engine deck elevation: 4230
Sump bottom elevation: 4175
Construction: Steel and reinforced concrete (13,000 cu. yds. of reinforced concrete)
Contractor: Layton Construction Co., Salt Lake City, UT
Engineer: Eckhoff, Watson and Preator Engineering and Morrison-Knudsen Engineers, Inc.
Original bid: $7,891,378.68
Final cost: $10,387,560

Outlet Canal
Length: 4.1 miles
Bottom elevation: varies between 4210-4207
Bottom width: varies between 75-100 ft.
Top width: varies between 120-150 ft.
Operating depth: minimum 10-11 ft.
Slope of canal: avg. .18 ft. Per 1,000 yds.
Material dredged: 640,000 cu. yds.
Contractors: Southern Pacific Transportation Co./ Lost Dutchman, Inc.
Engineer: Eckhoff, Watson and Preator Engineering

Access Road
Length: 10 miles
Height: 13 ft.
Width: 18.5 ft.
Volume of fill: 1.2 million cu. yds.,
Lakeside to Hogup
Bridges: 4 (3-150 ft. long, 1-180 ft. long)
Contractor: Southern Pacific Transportation Co.
Engineer: Eckhoff, Watson and Preator Engineering
SPTC's contract included excavation of the pumping plant foundation, excavation of the outlet canal, construction of four bridges, reconstruction of the damaged 10-mile causeway, and raising 25 miles of railroad track for a total of $22,980,000.

Bonneville Dike
Length: 24.4 miles
Height: 3-6 ft.
Volume of fill: 486.500 cu. yds.
Rip rap: 152,00 cu. yds.
Contractor: W. W. Clyde Construction, Springville, UT
Contract: $3,872,845

Newfoundland Dike
Length: 8.1 miles
Height: 3-7 ft.
Volume of fill: 249,000 cu. yds.
Rip rap: 85,000 cu. yds.
Control weir: 1,000 ft. long
Contractor: Herm Hughes and Sons, Bountiful, UT
Engineer: Bingham Engineering
Contract: $2,464,030

Chapter 12

Shutdown of the West Desert Pumping Project

The Shutdown Process

Utah's Great Salt Lake West Desert Pumping Project was shut down June 30, 1989, after operating successfully for more than two years. A long-term, or "mothball," shutdown procedure for the Pumping Plant proposed by Dresser-Rand was accepted by the state. After meeting with representatives of equipment suppliers, i.e. Dresser-Rand Engine-Process Compressor Division, Cummins Northwest, Ingersoll-Rand Pump Group, etc., the Dresser-Rand Services Division implemented the preservation steps. Extensive preservation methods requires monthly inspections by qualified individuals and periodic internal inspections of preserved equipment.

The shutdown process took about eight weeks and cost approximately $200,000, which was within the project's budget. The process included securing the Pumping Plant; dismantling, preserving and storing tools, systems and control devices; and inspection and maintenance of the project site.

The following are some of the shutdown procedures for some of the major components at the Pumping Plant site.

Pumping Plant Shutdown

Engines and Pumps - Because the Pumping Plant was to remain in place as insurance against future flooding around the Great Salt Lake, disassembly or removal of the three large engines and pumps were not options.

Control Panel - The circuitries of the Dynalco scanners, Moore SLD controllers, and Allen-Bradley PLC-4 were removed and stored in the plant. Open ports of the pneumatic safety shutdown and end devices were plugged, and the pneumatic circuitry of the engine and control panel are maintained under a slight positive pressure charge of nitrogen gas. The panel was covered with plastic to prevent dust contamination and volatile corrosion inhibitor (VCI) paper was placed inside the engine-mounted wiring junction box.

Frame and Running Gear- The crankcase, valve train and accessory drive components of the engines were protected through a combination of procedures. During the last 30 to 60 minutes of each engine's operation, a three-phase rust inhibitor was added to the engine oil. After engines cooled, valve covers, crankcase doors and other access covers were removed and a solvent-based rust inhibitor was sprayed on all engine internal components. VCI capsules were placed in various locations inside each engine to augment the rust inhibitor protection. Fuel gas injection valves were removed and solvent-based inhibitor was sprayed into power cylinders, then replaced against original gaskets. Protective oil was poured into push rod tubes, and hydraulic lifters were removed, dipped in protective oil, bagged and tagged for location, and placed in the rocker arm area of the cylinder heads. The valve stem lubricator reservoir, pumps and tubing were filled with protective oil. In addition, electric motors for the valve stem lubricator and auxiliary oil pump were wrapped and covered with plastic to prevent dust contamination. Crankcase breather piping was also sealed with plastic.

Lube Oil Piping- The engine oil/rust inhibitor mix was left in the engine oil sump and piping. A reconfigured pneumatic auxiliary oil pump is periodically operated with the small air compressor to recirculate the engine oil/inhibitor mix through the gear reducer.

Drive Line and Brad-Foote Gear Reducer - Exposed, unpainted portions of the quill shaft were coated with protectorant, and quill shaft bearings (pedestal bearings) were drained and refilled with protective oil. During the last 30 to 60 minutes of the unit's operation, a three-phase rust inhibitor was added to the gear oil to be carried to all moving parts. Internal components were sprayed with solvent-based rust inhibitor. The gear oil/rust inhibitor mixture is to be circulated periodically. A HASKEL pneumatic pump is used for this task. The brine side of the heat exchanger was washed with fresh water and allowed to dry.

Ingersoll-Rand Pumps - The grease distributing tubes of the Farval grease system were disconnected from the measuring valves and capped, as were the inlet ports on the valve blocks. The main grease supply lines were also disconnected at the unit's reversing valve. In addition, the electric motor was wrapped in paper and plastic, the reversing valve and grease pump were purged of grease and sealed, and the pump drive gearbox was filled with protective oil. The grease reservoir and bulk grease transfer pump were cleaned and sealed, the bulk grease pump motor lubricator was filled with protective oil, and the microprocessor circuitry was removed and stored in a controlled environment. VCI paper was placed inside the circuit boxes.

Brine Cooling Water System - The brine pump motors were removed and stored. Brine pump rotating elements and casings were removed, flushed with fresh water, and stored inside the engine room. Piping also was flushed.

Fuel Gas Supply Pipeline - The natural gas fuel supply remains connected to the engine room. Natural gas is used to power a small generator in the Pumping Plant. Isolated fuel headers on the three engines are under positive nitrogen gas pressure.

Air Compressors and Piping - Motors on the air compressors were wrapped in paper and plastic, and the entire piping circuit, including air receivers, was isolated from the air compressors. Rust inhibitor was added to the air compressors' crankcase oil. Air intake filters were removed and inlets were sealed, and cylinders and valve decks were sprayed with preservative. The piping circuit for instrument air received similar treatment.

Siphon Break Valves - Rubber on the butterfly valve was cleaned and coated with preservative. The actuator assembly was removed and stored in the engine room. A VCI plug was fitted to each end of the pneumatic cylinder, and protective oil was added to the linkage box. Handwheel threads were wrapped with paper and plastic. The siphon break was sealed with reinforced plastic.

Cummins Gas Engines - VCI paper was placed inside the housings for the monitor panel and timing shift unit. Protection for the crankcase, valve train and accessory drive units of the frame and running gear required a combination of procedures. During the last 30 to 60 minutes that each engine operated, three-phase rust inhibitor was added to the engine oil. Valve covers and access covers were then removed and the engine's internal components were sprayed with solvent-based rust inhibitor. Crankcase breather piping was sealed at the engine. An actuator was removed and stored. Turbochargers, however, were left on the engines. The Altronic III ignition units, ignition coils and high and low tension wiring were removed and stored in a controlled environment. Exhaust piping outlets and air filter inlets were sealed with reinforced plastic and various other elements were sealed. The entire cooling system has been maintained under positive nitrogen pressure.

Pumping Plant Building - The pump/motor for the building's potable water system was removed from the underground storage tank and stored in the service building. The storage tank was drained, tank openings were sealed, pipes were disconnected, and the tank was filled with nitrogen gas. The indoor pressurized water tank and Pennwalt hypo-chlorinator was disconnected from the piping system and filled with nitrogen gas. All water pipes were drained and blown out with compressed air. The wastewater system was sealed and the septic tank was pumped. Roof vents for the wastewater system remain open. Other elements of the building that were sealed include the HVAC system, safety and emergency lighting systems, control room electrical enclosures and doors, windows, and ventilating grilles. Tools and other equipment were either boxed for storage in the building or removed to other storage areas.

The Pumping Plant is inspected monthly by staff of the Division of Water Resources and periodic evaluations are made on the condition of engines, pumps and other equipment at the Pumping Plant site.

Reactivating the Pumping Plant

In the event the West Desert Pumping Project is reactivated, the startup process could take between 8-12 weeks to accomplish. In addition to reversing the shutdown procedure, many pieces of equipment would need to be partially or completely dismantled and inspected. A percentage of this equipment would likely require replacement because some of the control electronics may be outdated; it would be more efficient to replace it with current technology.

Reactivating the Pumping Plant would require startup services to be contracted with Dresser-Rand or another qualified mechanical services company. Estimated cost of reactivating the Pumping Plant is $250,000 to $300,000. The Pumping Plant would not be reactivated unless it were operated for one or more years. The State Legislature would also need to appropriate a yearly operating budget of approximately $2 million. ⌂

West Desert Pumping Project

Technical Appendix A

Evaporation Model

Page **Contents**

Appendix A
Estimating Evaporation in the West Desert

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February 1987

Objectives

1. Evaluate the various methodologies for estimating evaporation from more commonly measured parameters, such as temperature, humidity and wind speed;
2. Determine the availability of actual data on evaporation and related climatological factors for the Intermountain Area, Northern Utah and the West Desert area;
3. Estimate more accurately the "normal" and current evaporation potential in the West Pond area to improve estimates of system performance.

Methodologies

Very few comprehensive studies have been made of the various evaporation estimation methodologies. Two notable reports, however, are:

1. *Comparisons of Equations Used for Estimating/Agricultural Crop Evapotranspiration with Field Research*, Bureau of Reclamation, 1983.
2. *Water in Desert Ecosystems*, Daniel D. Evans and John L. Thames, Dowden, Hutchson & Ross, Inc., 1982.

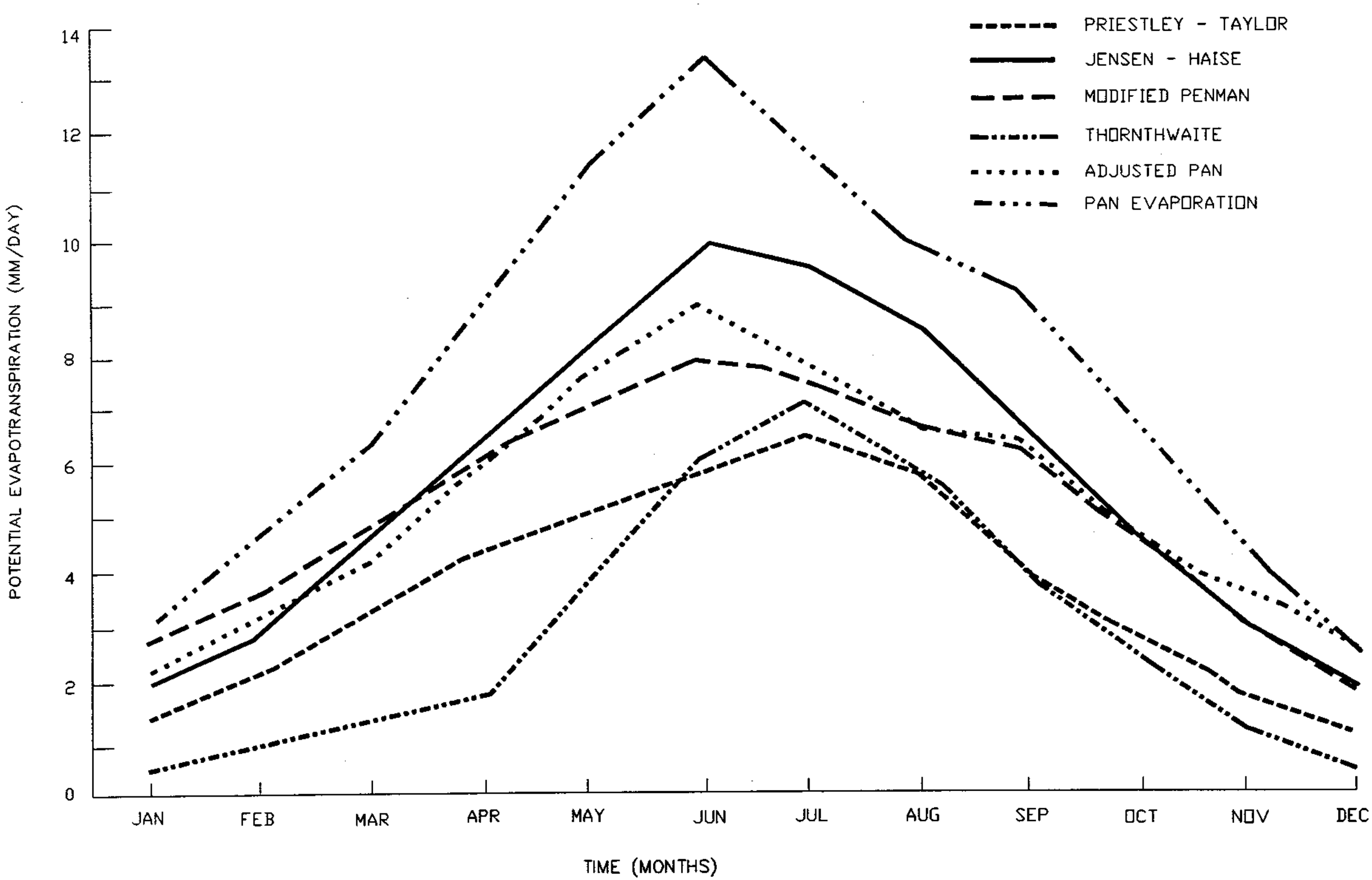
A difficulty with the first reference is that it reports on agricultural evapotranspiration, not evaporation from a free water surface. It contains, however, a thorough discussion of the Modified Penman (MP) Method, with details on the various terms and parameters.

Chapter 8 of the second report cited above is a helpful report on the common methodologies for evapotranspiration estimation, including the Class A Pan. Because the Class A Pan is used extensively for estimating purposes, and because extensive pan data exist, it was determined that an estimating methodology which closely resembled pan results would be most appropriate. As shown in Figure 1, the Modified Penman Method most closely mirrors the pan results from Tucson, Arizona, for the years 1966-1975. Since the MP method predicts evaporation from a large, open body of water, the results had to be adjusted with an appropriate pan coefficient. The adjusted pan curve very closely overlies the MP-estimated evaporation.

Further background on the Modified Penman Method is presented in the monograph, *Evaporation into the Atmosphere*, W. Brutsaert, D. Reidel Publishing Company, London, 1982.

Chapter 10 of this monograph contains an excellent development of the basis of the Penman equation. Pages 215-217 of the monograph are attached for documentation as Supplement 1. Penman began with the universally-accepted Bowen Ratio equation (10.9), based on the thermodynamically sound Clausius-Clapeyron relationship. Expanding on this with the

Figure 1
Estimates of Potential Evapotranspiration of Several Methods at Tucson, Arizona, 1966-1975



(defensible) assumption in equation (10.11), the basic Penman is stated in an equation (10.15), with equations (10.16) and (10.17) required to give a predictive equation using commonly measured parameters. Pages 2-9 through 2-14 of the Bureau of Reclamation publication are included as Supplement 2. They provide detailed instructions on making computations with the Modified Penman equation. The various terms and experimental determinations of coefficients are also discussed, providing the basis of the evaporation computations being furnished by USU along with their climatological data transmissions. No adequate verification of this is available at this time, and the MP method should be used cautiously if it is applied to short time periods. This caution has been communicated to USU.

Availability of Data

Several climatological monitoring stations are located in Northern and Northwestern Utah. Most report their data directly to the U.S. Weather Service. Yet, no active Class A Pan stations are in reasonable proximity to the West Pond area; most local pan stations are located along the Wasatch Front. However, several special evaporation studies have been made. Several have included either actual pan stations on the western part of Great Salt Lake, or estimates of evaporation in the West Desert area based on climatological correlations. Sources reviewed in this context include:

1. Data Acquisition/Distribution:

   Climatological Data-Annual Summaries for Utah and Nevada, 1980-1985, NOAA.
   Climatological Data for Utah, May 1986 to August 1986, NOAA.
   Local Climatological Data, (Several Stations), 1984, NOAA.
   Comparative Climatic Data for the United States Through 1978, NOAA.
   Climatological Atlas of the United States, NOAA.
   Utah Weather Guide, Brough, Griffin, Richardson and Stevens, 1983

2. Special Studies:

   Statewide Average Climatic History - Utah , 1891-1982, NOAA.
   Recent Heavy Precipitation in the Vicinity of the Great Salt Lake - Just How Unusual?, T.R. Karl and P.J. Young, Journal of Climate and Meteorology, 25, March 1986.
   Water Salvage Potential in Utah - Vol. 1, Open Water Evaporation and Monolayer Suppression Potential, PRWA22-1,Utah Water Research Laboratory, 1974.
   Model for Evaluating the Effects of Dikes on Water and Salt Balance of Great Salt Lake, Utah, (Waddell & Fields), - Water Resources Bulletin 21, 1977.
   Tabulation and Application of Pan Evaporation Data for Utah Through 1976, (Hubbard & Richardson), UWRL/A-79/02, Utah Water Research Laboratory, 1979.
   Estimated Inflow and Evaporation for Great Salt Lake, Utah, 1931-76, with Revised Model for Evaluating the Effects of Dikes on the Water and Salt Balance of the Lake. (Waddell & Barton), Coop. Invest. Report No. 20, Utah Division of Water Resources, 1980.

Additionally, a limited amount of data are available from a summer's evaporation season from the first two project monitoring stations at Barrow and Lemay. Barro is located at the south end of the West Pond (along I-80), and Lemay is located by Southern Pacific Transportation Company tracks about midway across the north end of the West Pond. Daily data for the months of August through October 1986 were obtained from Barro. However, the period July 27 through August 15 is missing from Lemay, although the station was "on line" from June 29 through September 2. The data (as transmitted) are included in Supplement 3.

Evaporation Estimates

Before commencing with evaporation estimates, it was decided to gain a better understanding of the climatological conditions and trends in the geographic area of concern, namely the North-Central and Western zones in Utah. In order to simplify the task to reasonable proportions, four keys stations were selected from each zone. The criteria for selection were centered around having adequate precipitation (P) and temperature (T) records, so that "normals" were available for each factor, thus making it possible to tabulate departures from normal for P and T for the "abnormal" period - 1982 to 1986.

Tables 1 through 4 contain the detailed computations on departures from normal for the two parameters (P&T) for the evaporation season (May - October). Precipitation departures are expressed as percent, whereas temperature departures are expressed as degrees-Fahrenheit.

The four sets of data are plotted in Figure 2. Note the parallel behavior of the data for the Western and North-Central zones. Precipitation was 100 to 125 percent above normal in 198. It has steadily declined to near-normal in 1986. For the 1982-1986 period, temperatures climbed from about 1.5°F. below normal to about 1.3°F. above normal (for the evaporation season).

Annual averages, which relate more to runoff than to evaporation, are shown in Figure 4. Note there is no significant decline in precipitation, and temperatures have tended to remain below normal (on an annual basis!).

These results indicate a return to normal for the evaporation season. In fact, a warmer-than-normal 1986 summer is shown. The specific data for the four key Western Stations (when averaged with the division data) show a temperature departure of about +3.0°F for the month of August 1986.

Relative evaporation rates for three key Class A Pan Stations in Utah for the years 1980-1986 are shown in Figure 3. Because of data limitations, only the five-month period May - September was included. These data plots are also strongly correlated with each other, and they support the return to "normal" indicated in Figure 2.

Looking at the period 1982-1985 (the wet years), the average relative evaporation rate was about 85 percent of normal. This is a significant departure, but when applied to the West Pond it certainly does not spell disaster. Specifically, the equivalent freshwater evaporation (net) for the West Pond was estimated to be in the range of 3.5 to 4.5 feet. A 15 percent reduction would be approximately seven inches - not critical.

In summary, the analysis shows that conditions in the West Desert area since 1982 (1) were significantly wetter than normal, (2) were significantly colder than normal, and (3) had returned to near normal in 1986. Therefore, the West Desert Pumping Project was not seriously and adversely impacted by colder/wetter weather conditions.

Utilizing the current data (August 1986) from the Barro Station, evaporation estimates were made using the Modified Penman method. Daily evaporation estimates are shown in Table 5. (A separate saturated vapor pressure vs. temperature correlation was utilized for these computations, not the Bureau of Reclamation method). Because the MP method gives open water evaporation, the daily estimates were adjusted upward (using a pan coefficient of 0.71) to estimate actual pan estimate.

Table 1
Departures of Climatic Data Precipitation 82-86, Western Division

	MAY			JUN			JUL			AUG			SEP			OCT			6-MOS A		
STATION-YEAR	PREC-IN	DEP	%	PREC-IN	DEP	%	PREC-IN	DEP	%	PREC-IN	DEP	%	PREC-IN	DEP	%	PREC-IN	DEP	%	PREC-IN	DEP	%
CALLAO-85	.66	-.01	-1	.68	-.01	-1	.96	.53	129	.07	-.46	-87	.86	.49	132	.40	-.05	-11	3.59	.49	16
DUGWAY-85	1.00	.27	37	.57	-.02	-3	.51	.07	16	.15	-.32	-68				1.17	.62	113	3.40	.62	22
MILFORD-85	.46	-.27	-37	.49	.07	17	1.78	1.17	192	.09	-.82	-87	1.42	.73	106	1.12	.39	53	5.36	1.47	38
WENDOVER-85	.33	-.52	-81	.49	-.11	-18	.22	-.03	-12	0.00	-.42	-100	.27	.04	17	.09	-.38	-81	1.40	-1.42	-50
AVERAGES-85	.61	-.13	-18	.55	-.02	-3	.88	.44	102	.08	-.48	-85	.85	.42	98	.70	.15	26	3.44	.29	9
WESTERN DIVISION-85	.93	.02	2	.50	-.17	-25	1.51	.88	140	.05	-.87	-93	1.20	.65	118	.88	.23	35	5.07	.94	23
CALLAO-84	.05	-.62	-93	.80	.08	11	2.27	1.86	454	1.47	.94	177	.18	-.19	-51	.52	.07	16	5.29	2.14	68
DUGWAY-84				1.08	.49	83	1.76	1.32	300	1.87	1.40	298	.49	.01	2	1.18	.63	115	6.38	3.85	152
MILFORD-84	.57	-.18	-22	.77	.35	83	1.69	1.08	177	3.75	3.04	428	.03	-.66	-96	1.33	.60	82	8.14	4.25	109
WENDOVER-84	.50	-.35	-41	.34	-.33	-49	.66	.41	164	.92	.50	119	.25	.02	9	.31	-.16	-34	2.98	.09	3
AVERAGES-84	.37	-.38	-50	.75	.15	25	1.60	1.17	273	2.00	1.47	276	.24	-.21	-46	.84	.29	52	5.70	2.58	83
WESTERN DIVISION-84	.39	-.52	-57	1.07	.40	60	2.14	1.51	240	2.08	1.36	189	.51	-.04	-7	1.12	.47	72	7.31	3.18	77
CALLAO-83	.26	-.41	-61	.78	.06	8	.06	-.35	-85	3.11	2.58	487	.70	.33	89	.45	0.00	0	5.36	2.21	70
DUGWAY-83	.47	.36	327	0.00	-.59	-100	1.89	1.45	330	1.89	1.42	302	.46	-.02	-4	.60	.05	9	5.31	2.67	101
MILFORD-83	1.05	.32	44	.09	-.33	-79	.24	-.37	-61	1.72	1.01	142	.95	.26	38	.89	.16	22	4.94	1.05	27
WENDOVER-83	.28	-.57	-67	.47	-.14	-23	.35	.10	40	1.51	1.09	260	.49	.26	113	.79	.32	68	3.89	1.06	37
AVERAGES-83	.52	-.08	-13	.34	-.25	-43	.64	.21	49	2.06	1.53	286	.65	.21	47	.68	.13	24	4.88	1.75	56
WESTERN DIVISION-83	.81	-.10	-11	.78	.11	16	.49	-.14	-22	2.38	1.66	231	1.19	.64	116	.89	.24	37	6.54	2.41	58
CALLAO-82	.97	.39	67	.41	-.40	-49	.73	.40	121	.78	.23	42	4.08	3.75	1136	.76	.39	105	7.73	4.76	160
DUGWAY-82	2.96	2.23	305	.87	.28	47	-1.52	1.08	245	1.20	.73	155	3.16	.48	18	1.89	1.34	244	11.60	6.14	112
MILFORD-82	.57	-.04	-7	1.38	.82	146	.68	.17	33	1.88	1.20	176	3.84	3.03	497	1.23	.45	58	9.38	5.63	150
WENDOVER-82	.98	.30	44	.04	-.69	-95	.53	.31	141	.36	0.00	0	3.37	3.10	1148	.23	-.24	-51	5.51	2.78	102
AVERAGES-82	1.37	.72	111	.68	0.00	0	.87	.49	131	1.06	.54	105	3.56	2.59	266	1.03	.49	89	8.56	4.83	130
WESTERN DIVISION-82	1.25	.41	49	.59	-.23	-28	1.18	.57	93	1.07	.31	41	4.16	3.66	732	1.3	.54	71	9.55	5.26	123
CALLAO-86	.35	-.32	-48	.09	-.63	-88	.49	.08	20	.67	.14	26							1.60	-.73	-31
DUGWAY-86	1.74	.91	110	.50	-.09	-15	.89	.45	102	.93	.48	98							4.06	1.73	74
MILFORD-86	.84	.11	15	.28	-.14	-33	.24	-.37	-61	1.50	.79	111							2.86	.39	16
WENDOVER-86	.13	-.72	-85	.01	-.60	-98													.14	-1.32	-90
AVERAGES-86	.77	-.01	-1	.22	-.37	-62	.54	.05	11	1.03	.46	81							2.17	.02	1
WESTERN DIVISION-86	.88	-.05	-5	.15	-.52	-78	.55	-.08	-13	1.13	.41	57							2.69	-.24	-8

Table 2
Departures of Climatic Data Temperature 82-86, Western Division

STATION-YEAR	MAY TEMP-F	MAY DEP	JUN TEMP-F	JUN DEP	JUL TEMP-F	JUL DEP	AUG TEMP-F	AUG DEP	SEP TEMP-F	SEP DEP	OCT TEMP-F	OCT DEP	6-MOS AVG TEMP-F	6-MOS AVG DEP
CALLAO-85	61.10	4.00	68.90	3.60	75.20	1.40	71.70	.70	58.60	-3.00	49.50	-1.00	64.17	.95
DUGWAY-85	62.10	3.10	71.80	2.70	80.10	1.60	75.30	-.30			49.80	-2.00	67.78	1.02
MILFORD-85	57.70	1.80	68.30	2.50	74.80	.50	71.50	-.60	57.80	-4.80	48.80	-1.50	63.15	-.35
WENDOVER-85	63.60	2.80			83.60	3.80	77.00	.30	62.70	-3.30	51.70	-.70	67.72	.58
AVERAGES-85	61.13	2.93	69.60	2.93	78.43	1.83	73.88	.03	59.70	-3.70	49.95	-1.30	65.70	.45
WESTERN DIVISION-85	59.60	3.00	68.90	3.20	76.20	1.80	71.90	-.10	58.70	-3.70	49.50	-1.00	64.12	.53
CALLAO-84	58.60	1.50	63.00	-1.20	73.70	-.10	72.50	1.50	62.60	1.00	45.70	-4.80	62.68	-.35
DUGWAY-84			65.30	-3.60	76.20	-2.30	75.10	-.50	65.90	1.20	46.20	-5.60	65.74	-2.16
MILFORD-84	59.40	3.50	64.60	-1.20	73.10	-1.20	71.80	-.30	62.40	-.20	44.10	-6.20	62.57	-.93
WENDOVER-84	63.30	2.50	68.70	-1.70	80.80	1.00	78.20	1.50	66.30	.30	48.70	-3.70	67.67	-.02
AVERAGES-84	60.43	2.50	65.40	-1.93	75.95	-.65	74.40	.55	64.30	.58	46.18	-5.08	64.66	-.67
WESTERN DIVISION-84	58.70	2.20	63.30	-2.40	73.50	-.90	72.10	.10	62.50	.10	45.20	-5.30	62.55	-1.03
CALLAO-83	53.90	-3.20	64.60	-.70	72.20	-1.60	73.10	2.10	63.30	1.70	51.30	.80	63.07	-.15
DUGWAY-83	55.00	-4.00	67.40	-1.50	76.70	-1.80	76.00	.40	66.40	1.70	52.80	1.00	65.72	-.70
MILFORD-83	51.50	-4.40	64.20	-1.60	71.70	-2.60	72.10	0.00	65.20	2.60	51.20	.90	62.65	-.85
WENDOVER-83	58.60	-2.20	69.60	-.80	78.00	-1.80	77.60	.90	68.00	2.00	54.80	2.40	67.77	.08
AVERAGES-83	54.75	-3.45	66.45	-1.15	74.65	-1.95	74.70	.85	65.73	2.00	52.53	1.28	64.80	-.40
WESTERN DIVISION-83	53.10	-3.40	64.20	-1.50	71.80	-2.60	72.20	.20	64.10	1.70	51.40	.90	62.80	-.78
CALLAO-82	55.60	-1.50	63.90	-1.40	72.00	-1.80	74.40	3.40	60.40	-1.20	45.70	-4.80	62.00	-1.22
DUGWAY-82	58.60	-2.40	67.00	-1.90	75.80	-2.70	78.70	3.10	62.90	-1.80	47.50	-4.30	64.75	-1.67
MILFORD-82	54.70	-1.80	63.60	-1.60	71.50	-2.80	73.50	.90	60.80	-2.20	45.00	-5.70	61.52	-2.20
WENDOVER-82	60.50	-.30	70.50	1.30	78.20	-1.10	80.90	4.20	65.50	-.70	48.30	-4.50	67.32	-.18
AVERAGES-82	56.85	-1.50	66.25	-.90	74.38	-2.10	76.88	2.90	62.40	-1.48	46.63	-4.83	63.90	-1.32
WESTERN DIVISION-82	55.30	-1.10	64.40	-1.50	72.00	-2.00	74.80	2.80	61.20	-1.20	46.00	-4.60	62.28	-1.27
CALLAO-86	56.60	-.50	71.40	6.10	72.00	-1.80	75.10	4.10					68.78	1.98
DUGWAY-86	57.80	-1.20	73.60	4.70	75.10	-3.40	78.90	3.30					71.35	.85
MILFORD-86	55.80	-.10	68.80	3.10	71.80	-2.50	74.20	2.10					67.68	.65
WENDOVER-86	61.30	.50	78.40	8.00									69.85	4.25
AVERAGES-86	57.88	-.33	73.08	5.48	72.97	-2.57	76.07	3.17					69.41	1.44
WESTERN DIVISION-86	56.50	0.00	70.80	5.10	71.70	-2.70	74.90	2.90					68.48	1.33

Table 3
Departures of Climatic Data Precipitation 82-86, North Central Division

STATION-YEAR	MAY PREC-IN	MAY DEP	MAY %	JUN PREC-IN	JUN DEP	JUN %	JUL PREC-IN	JUL DEP	JUL %	AUG PREC-IN	AUG DEP	AUG %	SEP PREC-IN	SEP DEP	SEP %	OCT PREC-IN	OCT DEP	OCT %	6-MOS A PREC-IN	6-MOS A DEP	6-MOS A %
LOGAN USU-85	1.67	-.04	-2	1.21	-.32	-21	1.04	.59	131	0.00	-.96	-100	3.00	1.94	183	1.78	.35	24	8.70	1.56	22
OGDEN SUGAR-85	1.92	.21	12	.45	-.98	-69	.28	-.22	-44	.06	-.66	-92	2.79	1.69	154	.97	-.30	-24	6.47	-.26	-4
SLC AP-85	2.95	1.48	101	1.30	.33	34	.85	.13	18	.03	-.89	-97	1.98	1.09	122	1.61	.47	41	8.72	2.61	43
LEHI-85	2.82	1.84	188	1.20	.49	69	1.40	.79	130	.15	-.73	-83	1.18	.42	57	.26	-.66	-72	6.99	2.15	44
																			7.72	1.52	24
AVERAGES-85	2.34	.87	59	1.04	-.12	-10	.89	.32	57	.06	-.81	-93	2.23	1.29	136	1.16	-.04	-3	7.72	1.52	24
NO CENT DIVISION-85	2.41	.81	51	1.10	-.09	-8	1.71	1.06	163	.04	-.91	-96	2.25	1.26	127	1.68	.37	28	9.19	2.50	37
LOGAN USU-84	1.61	-.10	-6	3.08	1.55	101	1.57	1.12	249	.50	-.46	-48	2.56	1.50	142	2.51	1.08	76	11.83	4.69	66
OGDEN SUGAR-84	1.28	-.45	-26	2.54	1.11	78	1.56	1.06	212	.54	-.18	-25	1.94	.84	76	2.29	1.02	80	10.13	3.40	51
SLC AP-84	1.17	-.30	-20	1.86	.89	92	1.72	1.00	139	1.49	.57	62	1.72	.83	93	3.70	2.56	225	11.66	5.55	91
LEHI-84	.50	-.48	-49	2.10	1.39	196	.58	-.03	-5	1.76	.88	100	1.10	.36	49	1.79	.87	95	7.83	2.99	62
																			10.36	4.16	67
AVERAGES-84	1.14	-.33	-23	2.40	1.24	106	1.38	.79	138	1.07	.20	23	1.83	.88	93	2.57	1.38	116	10.36	4.16	67
NO CENT DIVISION-84	1.17	-.43	-27	2.71	1.52	128	1.92	1.27	195	1.35	.40	42	1.93	.94	95	2.73	1.42	108	11.81	5.12	77
LOGAN USU-83	4.79	3.08	180	.69	-.84	-55	1.94	1.49	331	4.88	3.92	408	3.67	2.61	246	2.74	1.31	92	18.71	11.57	162
OGDEN SUGAR-83	3.42	1.71	100	1.04	-.39	-27	1.28	.78	156	3.22	2.50	347	2.66	1.56	142	2.89	1.62	128	14.51	7.76	116
SLC AP-83	2.58	1.11	76	.62	-.35	-36	1.02	.30	42	2.64	1.72	187	1.03	.14	16	1.62	.48	42	9.51	3.40	56
LEHI-83	1.54	.58	57	.93	.22	31	1.00	.39	64	2.96	2.08	236	1.42	.68	92	1.19	.27	29	9.04	4.20	87
																			12.94	6.74	109
AVERAGES-83	3.08	1.62	110	.82	-.34	-29	1.31	.74	130	3.43	2.56	294	2.20	1.25	132	2.11	.92	77	12.94	6.74	109
NO CENT DIVISION-84	3.15	1.55	97	1.16	-.03	-3	1.52	.87	134	3.61	2.66	280	2.20	1.21	122	1.91	.60	46	13.55	6.86	103
LOGAN USU-82	1.99	.13	7	.82	-.96	-54	2.16	1.82	535	.57	-.30	-34	5.76	4.82	513	2.46	1.03	72	13.76	6.54	91
OGDEN SUGAR-82	1.37	-.38	-22	.85	-.83	-49	1.55	1.06	216	.43	-.38	-47	5.51	4.55	474	2.36	.99	72	12.07	5.01	71
SLC AP-82	1.86	.37	25	.66	-.64	-49	2.57	1.87	267	.56	-.37	-40	7.04	6.36	935	1.87	.71	61	14.56	8.30	133
LEHI-82	1.04	.01	1	.36	-.57	-61	1.00	.40	67	.25	-.64	-72	5.63	5.03	838	1.88	.93	98	10.16	5.16	103
																			12.64	6.25	98
AVERAGES-82	1.57	.03	2	.67	-.75	-53	1.82	1.29	242	.45	-.42	-48	5.99	5.19	653	2.14	.92	75	12.64	6.25	98
NO CENT DIVISION-82	1.81	.14	8	.74	-.76	-51	2.05	1.46	247	.72	-.22	-23	6.63	5.78	680	2.37	1.01	74	14.32	7.41	107
LOGAN USU-86	2.34	.64	38	.10	-1.43	-93	1.93	1.48	329	.93	-.03	-3							5.30	.66	14
OGDEN SUGAR-86	1.61	-.10	-6	.08	-1.35	-94	.83	.33	66	.96	.24	33							3.48	-.88	-20
SLC AP-86	3.39	1.92	131	.42	-.55	-57	.85	.13	18	1.32	.40	43	.38	-.72	-65	2.75	1.86	209	9.11	3.04	50
LEHI-86	1.08	.10	10	0.00	-.71	-100	.55	-.06	-10	.90	.02	2							2.53	-.65	-20
																			5.11	.54	12
AVERAGES-86	2.11	.64	44	.15	-1.01	-87	1.04	.47	82	1.03	.16	18	.38	-.72	-65	2.75	1.86	209	7.45	1.40	23
NO CENT DIVISION-86	2.05	.45	28	.38	-.81	-68	1.30	.65	100	1.41	.46	48							5.14	.75	17

Table 4
Departures of Climatic Data Temperature 82-86, North Central Division

STATION-YEAR	MAY TEMP-F	MAY DEP	JUN TEMP-F	JUN DEP	JUL TEMP-F	JUL DEP	AUG TEMP-F	AUG DEP	SEP TEMP-F	SEP DEP	OCT TEMP-F	OCT DEP	6-MOS AVG TEMP-F	6-MOS AVG DEP
LOGAN USU-85	58.60	2.70	65.70	1.70	75.20	2.20	69.80	-1.30	57.20	-4.60	47.80	-2.80	62.38	-.35
OGDEN SUGAR-85	62.20	3.70	70.70	3.40	78.20	2.30	74.00	.70	60.70	-3.00	51.60	-.80	66.23	1.05
SLC AP-85	63.90	5.10	72.50	4.20	80.70	3.20	76.50	1.60	62.70	-2.30	0.00	0.00	59.38	1.97
LEHI-85	59.10	2.80	64.60	-.20	74.10	1.50	70.20	-.10	59.60	-1.50	48.60	-1.20	62.70	.22
AVERAGES-85	60.95	3.58	68.38	2.28	77.05	2.30	72.63	.23	60.05	-2.85	37.00	-1.20	62.68	.72
NO CENT DIVISION-85	59.90	2.80	68.00	2.10	75.50	.90	71.70	-.50	58.90	-4.20	50.20	-1.10	64.03	0.00
LOGAN USU-84	55.70	-.20	60.50	-3.50	71.50	-1.50	70.60	-.50	58.80	-3.00	43.10	-7.50	60.03	-2.70
OGDEN SUGAR-84	59.60	1.10	64.40	-2.90	74.60	-1.30	73.90	.60	63.10	-.60	47.00	-5.40	63.77	-1.42
SLC AP-84	61.60	2.80	67.30	-1.00	78.50	1.00	77.20	2.30	66.50	1.50	49.50	-3.50	66.77	.52
LEHI-84	57.80	1.50	63.20	-1.60	72.40	-.20	71.20	.90	61.80	.70	44.50	-5.30	61.82	-.67
AVERAGES-84	58.68	1.30	63.85	-2.25	74.25	-.50	73.23	.83	62.55	-.35	46.03	-5.43	63.10	-1.07
NO CENT DIVISION-84	58.50	1.40	63.30	-2.60	73.50	-1.10	72.60	.40	62.00	-1.10	45.40	-5.90	62.55	-1.48
LOGAN USU-83	50.90	-5.00	62.00	-2.00	70.20	-2.80	72.40	1.30	61.30	-.50	50.60	0.00	61.23	-1.50
OGDEN SUGAR-83	55.50	-3.00	68.70	-.60	75.20	-.70	75.10	1.80	65.20	1.50	53.80	1.40	65.25	.07
SLC AP-83	55.80	-3.00	67.70	-.60	76.60	-.90	77.80	2.90	67.80	2.80	56.00	3.00	66.95	.70
LEHI-83	51.70	-4.60	64.30	-.50	69.70	-2.90	73.30	3.00	64.00	2.90	52.00	2.20	62.50	.02
AVERAGES-83	53.48	-3.90	65.18	-.93	72.93	-1.83	74.65	2.25	64.58	1.68	53.10	1.65	63.98	-.18
NO CENT DIVISION-83	53.10	-4.00	64.50	-1.40	72.50	-2.10	73.60	1.40	64.00	.90	52.60	1.30	63.38	-.65
LOGAN USU-82	52.70	-3.60	64.00	.90	69.90	-3.00	73.30	1.90	59.50	-2.50	45.20	-5.50	60.77	-1.97
OGDEN SUGAR-82	56.20	-2.20	66.30	.50	72.10	-3.20	74.90	1.70	61.00	-2.60	47.80	-4.70	63.05	-1.75
SLC AP-82	56.70	-1.60	68.00	1.80	75.50	-1.20	78.40	3.90	64.00	-.80	48.80	-3.60	65.23	-.25
LEHI-82	54.40	-2.00	63.60	-.40	70.20	-2.10	72.30	1.70	60.80	-.20	45.50	-4.30	61.13	-1.22
AVERAGES-82	55.00	-2.35	65.48	.70	71.93	-2.38	74.73	2.30	61.33	-1.53	46.83	-4.53	62.55	-1.30
NO CENT DIVISION-82	55.00	-2.50	64.90	-.10	71.80	-2.80	74.60	1.90	61.00	-2.50	46.30	-5.40	62.27	-1.90
LOGAN USU-86	54.50	-1.40	69.90	5.90	70.10	-2.90	74.20	3.10					67.18	1.18
OGDEN SUGAR-86	58.10	-.40	74.00	6.70	74.70	-1.20	77.30	4.00					71.03	2.28
SLC AP-86	57.20	-1.60	73.50	5.20	74.20	-3.30	77.90	3.00					70.70	.83
LEHI-86	54.90	-1.40	67.90	3.10	70.50	-2.10	72.00	1.70					66.33	.33
AVERAGES-86	56.18	-1.20	71.33	5.23	72.38	-2.38	75.35	2.95					68.81	1.15
NO CENT DIVISION-86	56.40	-.70	71.10	5.20	71.90	-2.70	74.60	2.40					68.50	1.05

02/10/87

EU19.0701

Figure 2
Climatic Departures from Normal
Utah Western & North Central Division, May-October 1982-1986

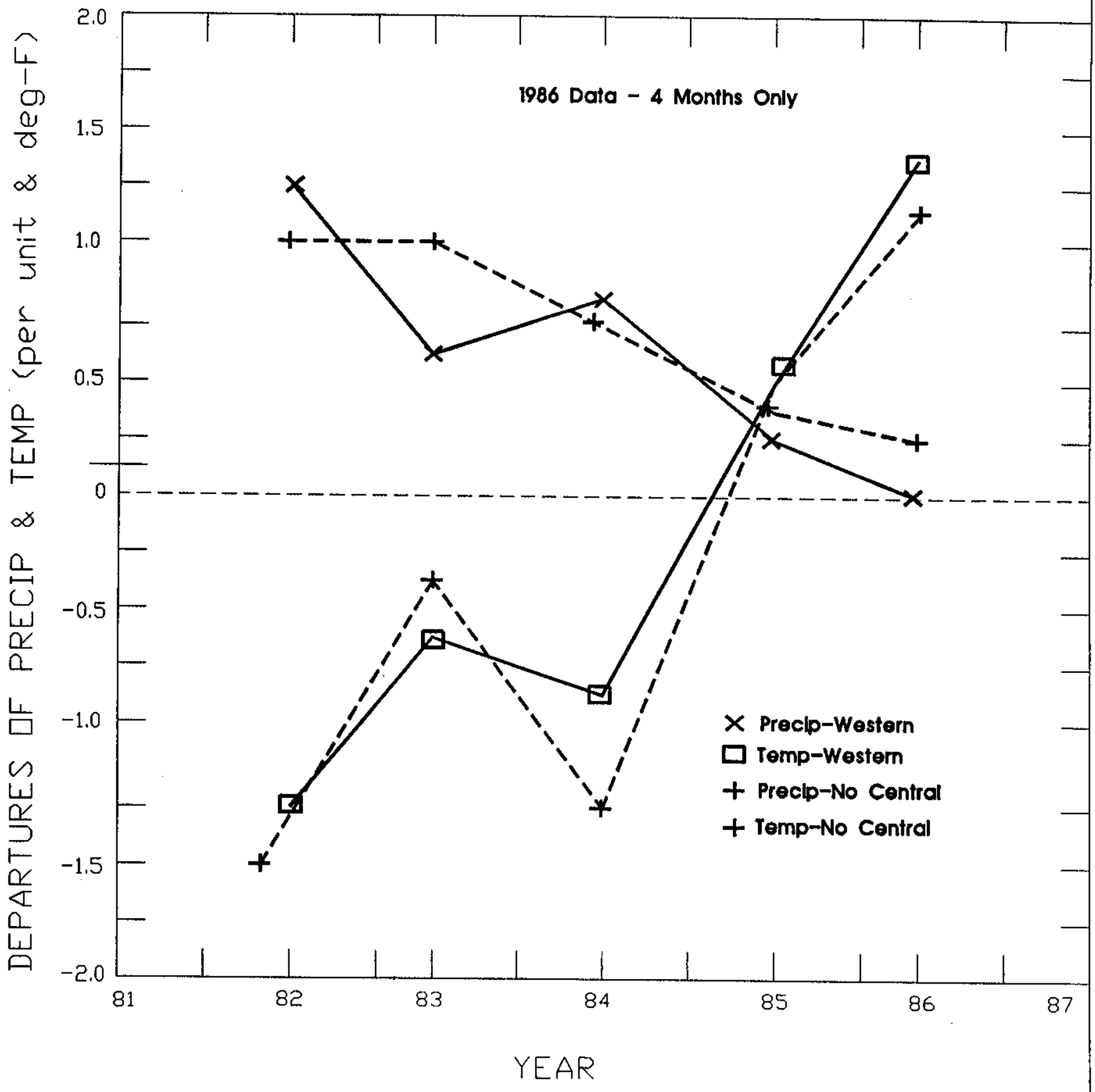


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Figure 3
Relative Evaporation Rates
Utah Western & North Central Divisions, May-September 1980-1986

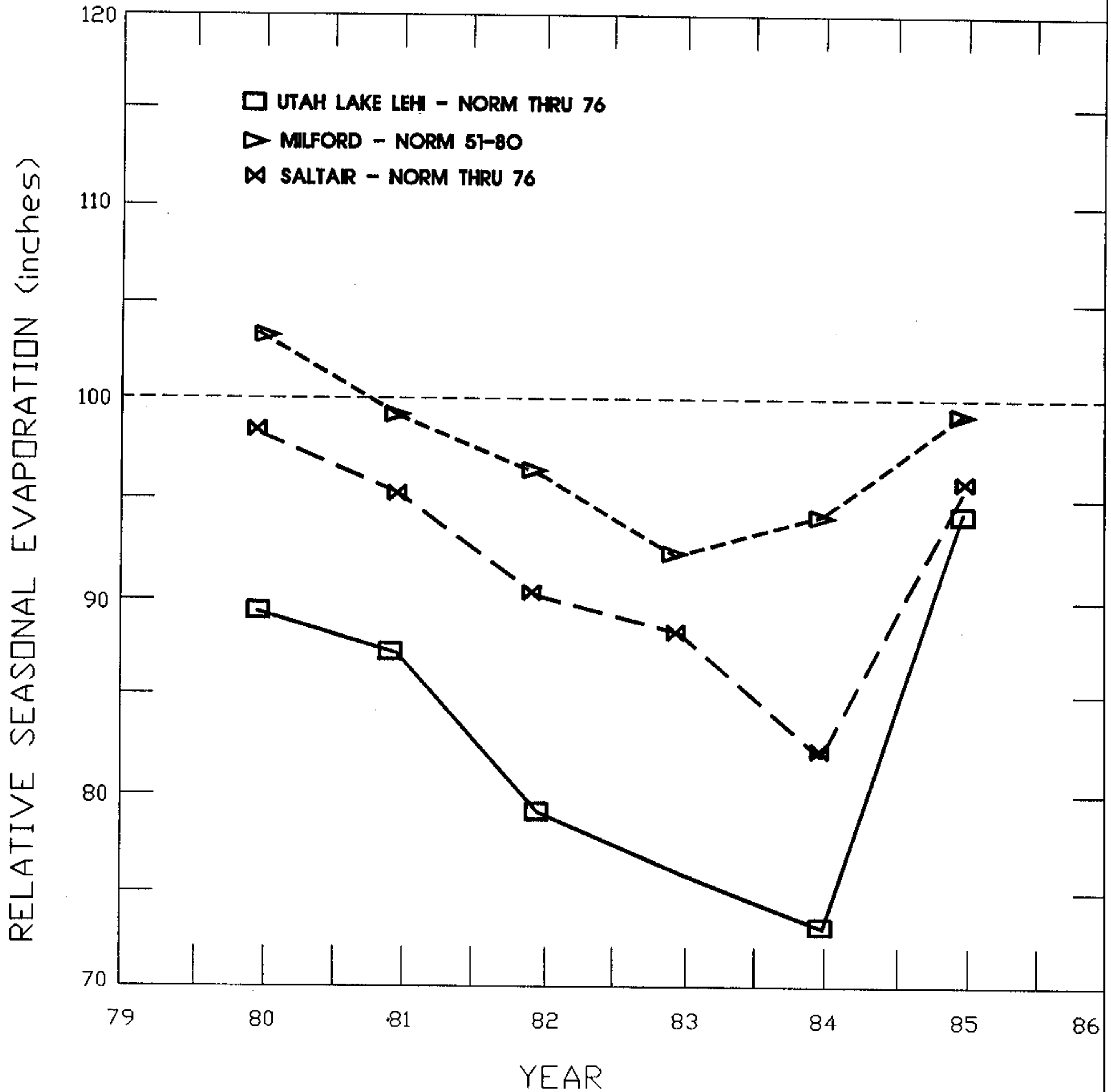

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Figure 4
Climatic Departures from Normal
Utah Western & North Central Divisions Annual 1982-1985

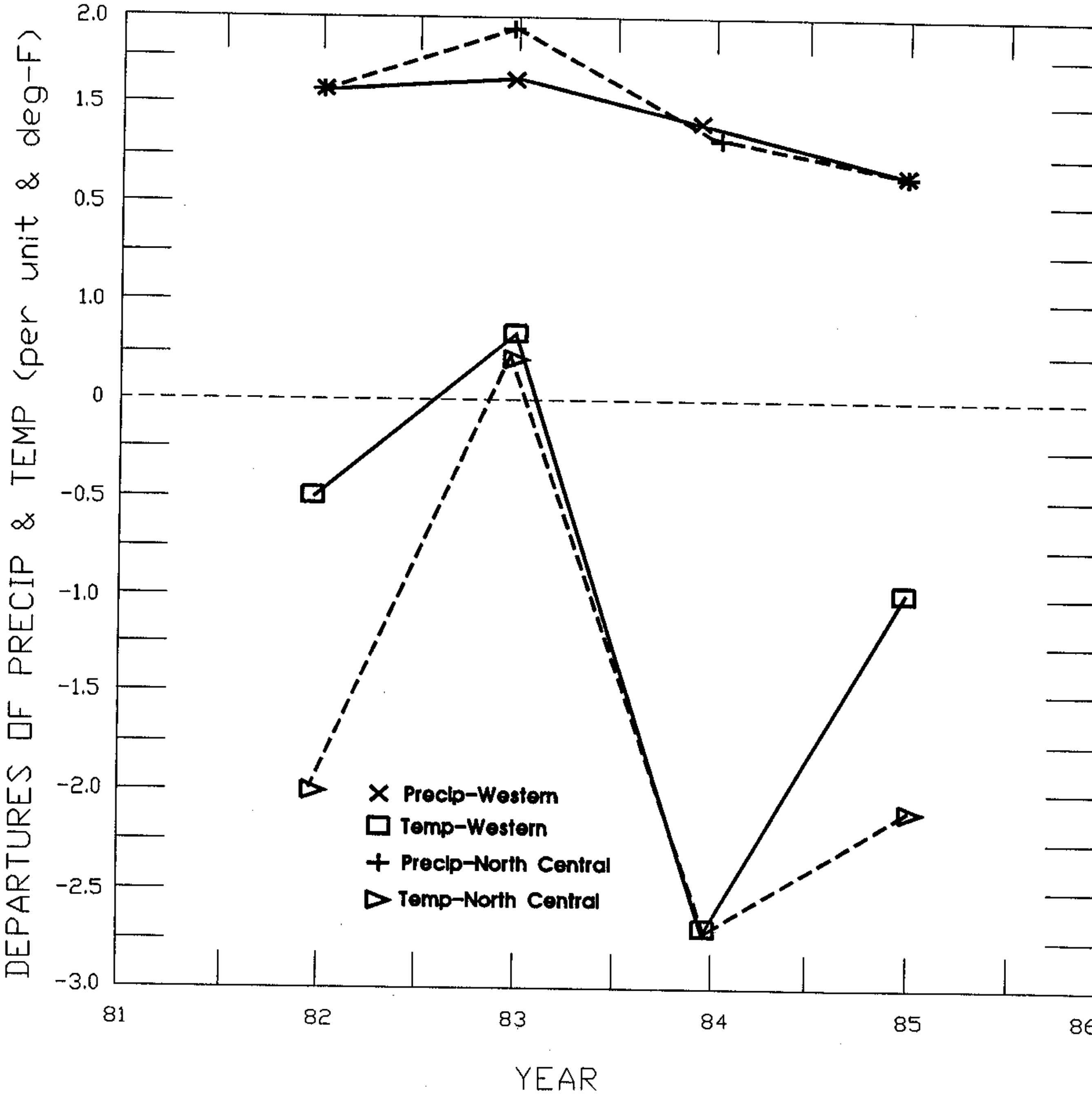

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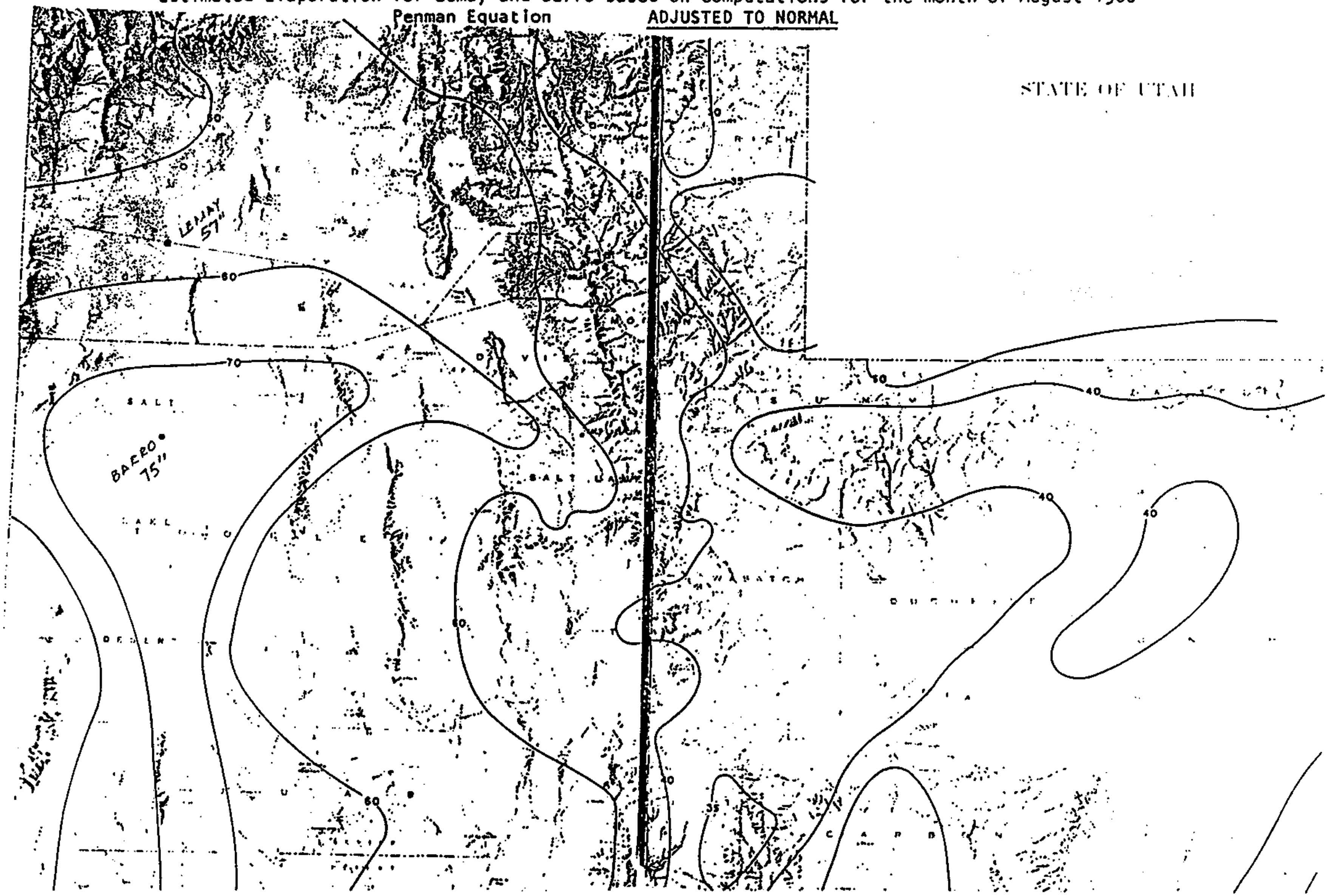
Figure 5
May to October, Pan Evaportion

Estimated Evaporation for Lemay and Barro based on computations for the month of August 1986
Penman Equation ADJUSTED TO NORMAL



The estimated average daily pan evaporation (for Barro - August 1986) is 0.45 cm. The same value was calculated using monthly average temperature humidity and wind data. This converts to 14.0 inches for the month.

August 1986 was about 3°F. warmer than normal. Reducing the Barro/August temperatures by this amount, the MP method gives an average pan evaporation of 0.43 cm per day, or 13.3 inches for an estimated "normal". The equivalent estimate for the Lemay is 10.1 inches.

In order to extrapolate the August monthly normals to (1) seasonal normals, and (2) annual normals, ratios from Wendover (Brough, et al) were applied. The following estimates were calculated:

Pan Evaporation - Inches

Station	Seasonal	Annual
Barro	75	91
Lemay	57	69

A comparison with Richardson's iso-evaporation plot is shown in Figure 4. The agreement appears to be superb!

Conclusions

On the basis of these analyses, conclusions are:

1. The "normal" pan evaporation for the West Pond appears to be about 66 inches for the season, and 80 inches annually. The equivalent freshwater lake evaporation values would be 47 inches and 57 inches respectively. This is substantially higher than had been previously estimated.

2. The Modified Penman method is appropriate for estimating potential freshwater lake evaporation; it is the method intended to be used to compute estimated evaporation.

Table 5
Barro - August 86

NOV 28 86

DAY	INPUT DATA AV TEMP DEG-C	HI TEMP DEG-C	LO TEMP DEG-C	HI HUM %	LO HUM %	WIND MPH	TOT SOL KWH/M^2	HI TEMP SVP mb	HI TEMP VP mb	LO TEMP SVP mb	LO TEMP VP mb	AVGS AVG VP mb	SVP mb	DELTA VP (mb)	COMPS WT FN	W	WIND mpd	f(U)	NET SOL langley	PENMAN cm/d	PENMAN in/d	PAN in/d
1	28.14	35.85	20.08	23.52	9.04	8.26	8.37	59.29	5.36	23.74	5.58	5.47	38.81	33.34	2.24	.80	185	.07699	360	1.01	.40	.56
2	28.92	37.33	21.48	30.77	8.76	5.56	7.36	64.00	5.61	25.94	7.98	6.79	40.59	33.79	2.32	.80	125	.06064	316	.84	.33	.46
3	28.36	34.92	21.94	24.84	9.52	10.24	8.06	56.48	5.38	26.73	6.64	6.01	39.30	33.30	2.27	.80	230	.08898	347	1.07	.42	.59
4	28.69	35.99	21.32	25.40	9.11	7.71	7.98	59.72	5.44	25.71	6.53	5.99	40.06	34.07	2.30	.80	173	.07366	343	.97	.38	.54
5	28.17	35.13	21.80	38.18	12.72	10.78	6.62	57.09	7.26	26.50	10.12	8.69	38.87	30.19	2.25	.80	242	.09224	285	.95	.37	.53
6	26.84	34.08	20.85	39.83	12.64	9.89	5.94	54.00	6.83	24.96	9.94	8.38	35.97	27.59	2.11	.79	222	.08686	255	.85	.34	.47
7	26.95	34.64	19.78	40.45	10.43	6.61	7.77	55.63	5.80	23.32	9.43	7.62	36.21	28.59	2.13	.79	148	.06703	334	.86	.34	.47
8	27.82	35.49	20.43	33.84	10.13	7.48	7.88	58.18	5.89	24.31	8.23	7.06	38.09	31.03	2.21	.80	168	.07227	339	.92	.36	.51
9	28.63	36.14	20.99	31.93	8.88	5.85	7.63	60.19	5.35	25.18	8.04	6.69	39.92	33.23	2.29	.80	131	.06242	328	.86	.34	.48
10	29.03	34.92	22.28	25.70	11.99	6.62	6.64	56.46	6.77	27.30	7.02	6.89	40.84	33.95	2.33	.80	148	.06704	286	.84	.33	.47
11	28.35	35.41	21.80	42.50	8.89	7.76	7.88	57.94	5.15	26.50	11.26	8.21	39.28	31.07	2.26	.80	174	.07397	339	.92	.36	.51
12	24.30	29.97	18.74	22.81	13.28	10.28	7.60	43.08	5.72	21.81	4.98	5.35	30.91	25.56	1.88	.77	230	.08922	327	.96	.38	.53
13	25.05	32.15	18.12	26.02	11.44	7.63	7.48	48.64	5.56	20.95	5.45	5.51	32.34	26.83	1.94	.77	171	.07318	322	.87	.34	.48
14	26.40	34.85	17.37	51.29	9.52	6.38	7.57	56.25	5.38	19.95	10.23	7.79	35.05	27.26	2.07	.78	143	.06560	326	.82	.32	.46
15	29.28	36.65	22.52	29.49	8.45	8.58	7.44	61.80	5.22	27.71	8.17	6.70	41.43	34.73	2.36	.81	192	.07893	320	.97	.38	.54
16	29.09	36.36	22.14	24.47	8.74	7.05	7.43	60.88	5.32	27.07	6.62	5.97	40.98	35.01	2.34	.80	158	.06967	319	.92	.36	.51
17	28.93	37.88	20.52	23.62	8.01	4.92	7.35	65.75	5.27	24.45	5.77	5.52	40.61	35.09	2.32	.80	110	.05677	316	.83	.33	.46
18	28.29	34.57	24.78	51.51	16.19	9.76	5.88	55.43	8.97	31.82	16.39	12.68	39.14	26.46	2.26	.80	219	.08607	253	.80	.32	.45
19	23.61	31.33	18.56	87.50	37.44	7.37	4.58	46.49	17.41	21.56	18.87	18.14	29.64	11.50	1.81	.76	165	.07161	197	.45	.18	.25
20	23.89	31.01	18.30	90.20	28.33	7.52	6.70	45.67	12.94	21.20	19.12	16.03	30.15	14.12	1.84	.76	169	.07251	288	.62	.24	.34
21	23.77	30.33	18.21	73.50	32.68	8.65	6.58	43.97	14.37	21.08	15.49	14.93	29.93	15.00	1.83	.76	194	.07935	283	.65	.26	.36
22	26.14	32.02	20.15	64.51	20.47	9.67	5.27	48.29	9.89	23.88	15.40	12.64	34.52	21.87	2.05	.78	217	.08553	227	.71	.28	.39
23	26.54	33.00	21.27	57.88	23.14	11.27	5.87	50.95	11.79	25.63	14.83	13.31	35.34	22.03	2.09	.79	253	.09521	252	.79	.31	.44
24	26.62	34.29	19.28	60.34	12.58	7.64	6.77	54.61	6.87	22.59	13.63	10.25	35.51	25.26	2.09	.79	171	.07324	291	.79	.31	.44
25	26.89	34.64	20.01	50.45	17.55	6.14	6.52	55.63	9.76	23.67	11.94	10.85	36.08	25.23	2.12	.79	138	.06416	280	.72	.28	.40
26	25.84	33.81	19.80	65.30	20.90	6.00	5.33	53.22	11.12	23.35	15.25	13.19	33.91	20.72	2.02	.78	134	.06331	229	.59	.23	.33
27	24.83	33.14	19.28	80.40	21.20	7.83	5.18	51.34	10.88	22.59	18.16	14.52	31.92	17.40	1.92	.77	176	.07439	223	.59	.23	.33
28	23.27	29.91	17.15	80.60	19.99	6.89	5.83	42.94	8.58	19.67	15.85	12.22	29.03	16.81	1.78	.76	155	.06872	242	.59	.23	.33
29	24.89	32.54	17.81	65.68	12.89	8.30	6.34	49.69	6.40	20.54	13.48	9.94	32.03	22.09	1.93	.77	186	.07723	273	.75	.29	.42
30	21.92	27.42	16.63	41.09	19.46	14.97	6.58	37.22	7.24	19.01	7.81	7.53	26.70	19.17	1.67	.75	336	.11760	283	.93	.37	.52
31																						
AVGS	26.65	33.86	20.11	46.79	15.15	8.12	6.81	53.69	7.78	23.96	10.94	9.36	35.77	26.41	2.10	.79	182	.07615	293	.82	.32	.45
AV COMP								53.36	8.13	23.82	11.21	9.38	35.57	26.41	2.10	.79	182	.07615	293	.82	.32	.45
NORMAL	23.65	30.86	17.11	46.79	15.15	8.12	6.81	45.29	6.86	19.62	9.18	8.02	29.71	21.69	1.82	.76	182	.07615	293	.78	.31	.43

B(0)	6.1511
B(1)	.40989
B(2)	.01486
B(3)	.00042
SOL EFF	50
PANCOEF	.71

been adequately supplied with water. Indeed, the partition of the available energy at the surface is related to the availability of water for evaporation, and this partition affects the temperature, the humidity and other state variables of the atmosphere. This matter should be kept in mind when the concept is used.

b. The EBWSP Method With Measurements at One Level

When the surface is wet, the surface specific humidity may be assumed to be the saturation value at the surface temperature, i.e., $q_s = q^*(T_s)$. This allows an approximation, first introduced by Penman (1948); the main advantage of this approximation is that it eliminates the need for measurements of $\bar{q}$, $\bar{u}$ and $\bar{\theta}$ at two levels, as in the profile methods (Chapter 9) and standard energy budget methods (Section 10.1), and that measurements at one level suffice.

Penman Approach

The equation derived by Penman (1948) was intended for an open-water surface. In what follows, a somewhat more general derivation is given, which is applicable to any wet surface, but which retains the essential features.

Because the Clausius-Clapeyron equation [see (3.21) and (3.24)] is used, it is preferable to start with the Bowen ratio (9.5) in terms of the vapor pressure; with the lower measurement at the surface, where $e_s = e^*(T_s)$, the Bowen ratio is

$$\mathrm{Bo} = \gamma \frac{(\bar{T}_s - \bar{T}_a)}{(\bar{e}_s - \bar{e}_a)} \tag{10.9}$$

where e_a and T_a are the vapor pressure and temperature in the air, respectively, at some reference level, and where by virtue of (3.2), (3.5) and (3.6),

$$\gamma = \frac{c_p p}{0.622\, L_e} \tag{10.10}$$

which is the psychrometric constant; at 20°C and $p = 1013.25$ mb it is $\gamma = 0.67$ mb K^{-1}. Note that θ is replaced by T, since in the surface sublayer they are practically the same.

A crucial step in Penman's analysis is the assumption

$$\frac{e_s^* - e_a^*}{\bar{T}_s - \bar{T}_a} = \Delta \tag{10.11}$$

where $\Delta = (de^*/dT)$ is the slope of the saturation water vapor pressure curve $e^* = e^*(T)$, at the air temperature T_a, $e_a^* = e^*(T_a)$ the corresponding saturation vapor pressure, and $e^* = e_s^*(T_s)$ the vapor pressure at the wet surface. Since $e_s = e_s^*$ for a saturated surface, the Bowen ratio (10.9) is thus, approximately

$$\mathrm{Bo} = \frac{\gamma}{\Delta}\left[1 - \frac{(e_a^* - \bar{e}_a)}{(\bar{e}_s - \bar{e}_a)}\right] \tag{10.12}$$

Values of (γ/Δ) for different temperatures at $p = 1000$ mb are presented in Table 10.1 and Figure 10.2; they were obtained by means of (10.10) and values of Δ and L_e listed in Table 3.4.

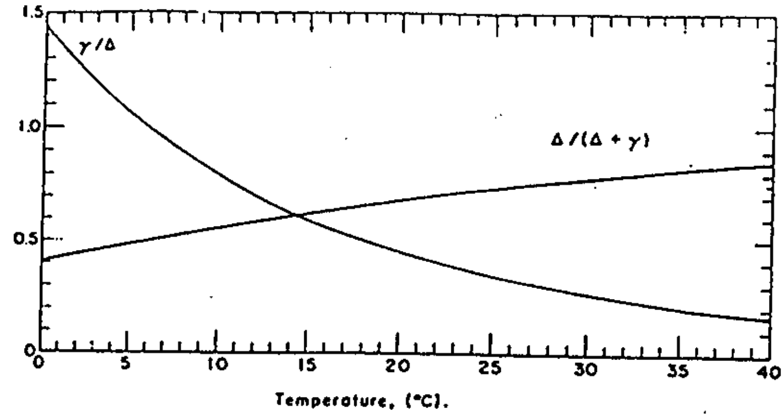


Fig. 10.2. Temperature dependence of (γ/Δ) and $\Delta/(\Delta+\gamma)$, at 1000 mb; γ is defined by (10.10) and Δ can be obtained from Table 3.4 or from (3.24b).

TABLE 10.1
Values of (γ/Δ) at 1000 mb (γ is defined by (10.10) and Δ can be obtained from (3.24b))

Air Temperature T_a(°C)	(γ/Δ)
−20	5.864
−10	2.829
0	1.456
5	1.067
10	0.7934
15	0.5967
20	0.4549
25	0.3505
30	0.2731
35	0.2149
40	0.1707

Substitution of (10.12) in (10.4) yields

$$Q_{ne} = \left(1 + \frac{\gamma}{\Delta}\right)E - \frac{\gamma}{\Delta}\left(\frac{e_a^* - \bar{e}_a}{\bar{e}_s - \bar{e}_a}\right)E. \quad (10.13)$$

In the second term of the right of (10.13), a bulk mass transfer equation can be used, viz.

$$E = f_e(\bar{u}_r)(\bar{e}_s - \bar{e}_a) \quad (10.14)$$

which serves as definition for the wind function $f_e(\bar{u}_r)$. This yields Penman's (1948) equation

$$E = \frac{\Delta}{\Delta + \gamma} Q_{ne} + \frac{\gamma}{\Delta + \gamma} E_A \quad (10.15)$$

where E_A, a drying power of the air, is defined by

Supplement 1 - Continued

$$E_A = f_e(\bar{u}_r)(e_s^* - \bar{e}_a). \tag{10.16}$$

Note that in Penman's (1948) derivation it was assumed that $Q_{ne} = R_n/L_e$ and that all the other terms in (10.2) are negligible. Equation (10.15) has been the subject of numerous theoretical and experimental studies (e.g., Penman, 1956; Tanner and Pelton, 1960; Monteith, 1965, 1973; Van Bavel, 1966; Thom and Oliver, 1977).

As mentioned, from the practical point of view, the main feature of (10.15) is that it requires measurements of mean specific humidity, wind speed and temperature at one level only. For this reason, it is very useful when measurements at several levels, needed for profile methods or standard energy budget methods are unavailable or impractical.

Equation (10.15) has been widely used, but there is still no generally accepted way to formulate $f_e(\bar{u})$, the wind function in E_A. From its definition in (10.14), it is clear that any suitable mass transfer coefficient can be used for this purpose (see Section 9.2). The simplest approach consists of using an empirical wind function. Penman (1948) originally proposed an equation of the Stelling-type (2.5) as follows

$$f_e(\bar{u}_2) = 0.26(1 + 0.54\,\bar{u}_2) \tag{10.17}$$

where $\bar{u}_2$ is the mean wind speed at 2 m above the surface in m s^{-1}, and the constants require that E_A in (10.16) is in mm day^{-1} and the vapor pressure in mb. There are some indications (e.g., Thom and Oliver, 1977) that (10.17) yields reasonable results for natural terrain with small to moderate roughness. Penman (1956) also proposed an intended improvement of (10.17), in which the numerical value 1 between the brackets was replaced by 0.5. Although apparently (Thom and Oliver, 1977) Penman subsequently felt that (10.17) is preferable over this second version, the latter is still widely used in hydrological practice. More recently, on the basis of lysimeter measurements, Doorenbos and Pruitt (1975) have suggested that, for irrigated crops, the constant 0.54 in (10.17) should be replaced by 0.86.

In terms of the bulk water vapor transfer coefficient as defined, for example, in (9.10) one obtains, by virtue of (3.2) (3.5) and (3.6), the wind function,

$$f_e(\bar{u}_1) = 0.622\,\rho p^{-1}\,Ce\,\bar{u}_1 \tag{10.18}$$

where z_1 is the height of the measurement of $\bar{u}_1$ and z_2 that of e_a. The wind function can also be determined theoretically by means of the similarity profile functions of Chapter 4. Thus, under neutral conditions, by virtue of (9.9) and (10.14), this is

$$f_e(\bar{u}_1) = \frac{0.622 a_e k^2 \bar{u}_1}{R_d T_a \ln[(z_2 - d_0)/z_{0v}] \ln[(z_1 - d_0)/z_{0m}]} \tag{10.19}$$

where, again, z_1 is the level of the wind-speed measurement and z_2 that of the water-vapor pressure; if the vapor pressures e_s^* and e_a are in mb, T_a in K and $\bar{u}_1$ in the same units as E_A, one can put $(0.622/R_d) = 2.167 \times 10^{-4}$, approximately.

When Penman's equation (10.15) is applied to calculate mean values of E over periods of a day or longer, the use of wind functions, such as (10.17)–(10.19), may be adequate. However, when hourly values are required, the effect of atmospheric stability, which varies through the day, may be quite important. A method to include this effect is described next.

Supplement 2 - Portions of Chapter 2, *Comparisons of Equations used for Estimating Agricultural Crop Evapotranspiration*, Bureau of Reclamation

COMPARISON OF EQUATIONS USED FOR ESTIMATING AGRICULTURAL CROP EVAPOTRANSPIRATION WITH FIELD RESEARCH

October 1983

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PREFACE

Water resource project planning requires accurate estimates of crop consumptive use or ET (evapotranspiration), which is an important component in a variety of water budget analyses. The increased emphasis on conservation and recent litigation further underscore the need for accurate ET predictions.

As a practical matter, ET estimates are obtained using theoretical predictive equations requiring meteorological data; however, hydrologists are faced with a large selection of predictive methods which vary considerably. Ideally, field research should provide a means of calibrating the predictive equations and checking the results.

This report presents the results of a study comparing the behavior of 10 selected equations commonly used to estimate ET with field research. Daily ET was calculated by each equation using climatological data representing locations throughout the contiguous Western United States. Alfalfa and corn yield/ET relationships were derived from available research data. The yield/ET equations become the link between research studied and anticipated farm conditions and provide a means of testing the accuracy of the various predictive equations and ultimately a tool for the hydrologist in selecting and using the predictive equations.

This study would not have been possible without the helpful cooperation of U.S. Department of Agriculture and State university agricultural research personnel located throughout the Western United States. Because of the large number of contributors, specific mention is reserved for the Acknowledgments in appendix G. Appreciation is also extended to Richard O. Ricks, Robert Steger, and Bruce C. Braaten for their assistance.

The authors gratefully acknowledge support of the Bureau of Reclamation in this effort under its Research Project No. DPTS-61, Changing Water Requirements, of the Program Related Engineering and Scientific Studies.

CONTENTS

APPENDIXES

2.1.3. Combination Methods

The term "combination method" is derived from combining net radiation and advective energy transfer effects on crop water use into one equation. The most familiar of these is the "Penman Combination Equation" used in this report in a modified form adapted from the work at Kimberly, Idaho. The FAO equations and data analysis using five variations of the Modified Penman Equation having different internal parameter values are included in this study. The PT Equation is also included in this category, since the same net radiation factors as the Penman Equation are used. However, the advective term is not included.

Reference crop conditions vary depending on the crop used for a particular local calibration.

Modified Penman [12, 13, 14, and 15]

$$E_{tr} = \frac{\Delta}{\Delta + \gamma} (R_n + G) + \frac{\gamma}{\Delta + \gamma} (15.36) (W_1 + W_2 U_2) (\overline{e}_s - e_a) \quad (2.16)$$

where

E_{tr} = reference ET, alfalfa with 30 to 50 cm of top growth, $(cal/cm^2)/d$ (langleys per day). The conversion factor for equation (2.6) is used to convert to inches per day.

Δ = slope of saturation vapor pressure-temperature curve (de/dT) at the air temperature, mb/°C.

γ = psychrometer constant, mb/°C.

R_n = net radiation, $(cal/cm^2)/d$.

G = soil heat flux, $(cal/cm^2)/d$.

W_1, W_2 = wind term parameters. Empirically determined values for W_1 and W_2 are given in table 2-2.

U_2 = wind movement, mi/d at 2-meter height.

$\bar{e}_s$ = saturation vapor pressure, as defined in equation (2.13), is the mean of values obtained at daily maximum and daily minimum temperatures, mb (this is a modification of the original Penman equation).

e_a = mean actual vapor pressure, mb, and is equivalent to the saturation vapor pressure, as defined in equation (2.13) obtained at the daily average dewpoint temperature. This is often approximated from a single early morning dewpoint determination.

Table 2-2. - Empirically determined wind function parameters for the Modified Penman Equation

Crop	W_1	W_2	Location and/or source
Short green crop (grass)	1.00	0.0100	[15]
Alfalfa, lysimeter	0.75	0.0185	Kimberly, Idaho [12]
Alfalfa, lysimeter	1.10	0.0170	Mitchell, Nebraska (near Scottsbluff) [13]
Alfalfa, lysimeter	1.16	0.0145	Grand Junction, Colorado [2]
Clipped grass	1.00	0.0161	[5]

Note: Judgment should be exercised in the use of any wind term coefficients if there is a different advective energy condition at night than during the day, such as experienced in some mountain valleys with canyon breezes or in coastal areas. Irrigation scheduling field checks (Reclamation and others) have indicated that field crop depletion does not follow equation (2.16) under extremely windy conditions. Accordingly, an arbitrary limit of 100 miles per day for U_2 is imposed in some regions.

The following equations are required to calculate individual terms in equation (2.16):

$$\gamma = c_p P/(0.622\ \lambda) \qquad (2.17)$$

where

c_p = specific heat of air at constant pressure, and is taken as 0.240 cal/gm·°C

P = 1013 - 0.03216 EL, mb; (EL = elevation in feet) (2.18)

λ = latent heat of water from equation (2.7) in cal/cm^3.

Δ can be computed as $2.00\ (0.00738T_c + 0.8072)^7 - 0.00116$ (2.19) for $T_c \geq (-23\ °C)$, where Δ is in mb/°C and T_c is mean daily temperature in °C.

$$R_n = 0.77\ R_s - R_b \qquad (2.20)$$

where

0.77 is $(1 - \alpha)$, assuming an albedo (α) of 0.23 for a green, actively growing crop at full cover.

$$R_b = R_{bo}\ (a \frac{R_s}{R_{so}} + b) \qquad (2.21)$$

and

$$R_{bo} = (a_1 + b_1 \sqrt{e_a})\ 11.71 \times 10^{-8}\ (T_a^4 + T_b^4)/2 \qquad (2.22)$$

where

a, b, a_1, b_1 = empirical constants (see table 2-3);

R_b = estimated longwave outgoing radiation, (cal/cm^2)/d;

R_{bo} = theoretical clear day longwave outgoing radiation, (cal/cm^2)/d;

R_{so} = clear day solar radiation (cal/cm^2)/d;

T_a = daily maximum temperature, °K; and

T_b = daily mininum temperature, °K.

R_{so} is obtained from curve fitting to the highest values of observed daily R_s data for a given site or from theoretical consideration [12]. Polynomial equations for estimating R_{so} at each site used in this study are included in appendix B.

An empirical equation for estimating the soil heat flux is:

$$G = (\overline{T}_{pr} - \overline{T})\ 5 \qquad (2.23)$$

where

$\overline{T}_{pr}$ = mean air temperature for a previous time period, usually the previous 3 days when daily estimates of E_{tr} are required, °F; and

$\overline{T}$ = mean air temperature for the current time period, i.e., mean air temperature of the particular day for which E_{tr} is required, °F.

The magnitude of G usually is small relative to other terms in equation (2.16) and is sometimes ignored in calculations; however, it was used in this study.

Supplement 2 - Continued

Table 2-3. - Values of a, b, a_1, and b_1 as determined or suggested at various locations

a	b	a_1	b_1	Location	Reference
0.90	0.10	0.37	-0.044	Mitchell, Nebr.	[13]
1.35	-0.35	0.35	-0.046	Davis, Calif.	[12, table 3.2]
1.22	-0.18	0.325	-0.044	Kimberly, Idaho	[12, table 3.2]
0.856	0.046			Grand Junction, Colo.	[2]
1.20	-0.20			Arid regions (suggested)	[12, table 3.2]
1.10	-0.10			Semihumid (suggested)	[12, table 3.2]
1.00	0.00			Humid (suggested)	[12, table 3.2]
		0.39	-0.05	General	[12, table 3.2]
1.35	-0.35	0.34	-0.044	General	[5] and appendix C

In some locations, wind movement may be available from an instrument at a height different than 2 meters. The following relationship [12] provides a reasonable estimate of the wind movement at 2 meters when the anemometer is at height z:

$$U_2 = U_z \left(\frac{2}{z}\right)^{0.2} \qquad (2.24)$$

where

U_2 = wind movement at 2 meters,

U_z = wind movement at height z, and

z = height of anemometer above ground in meters.

The coefficients W_1, W_2, a, b, a_1, and b_1 should be calibrated for specific location to more accurately show the relative importance of advective and solar energy in determining consumptive use. The values given in tables 2-2 and 2-3 should be used with judgment when sufficient data for local calibration are not available.

The five variations of equation (2.16) were arrived at by: (Variations 1 and 2) - Kimberly, Idaho, a, b, a_1, b_1, W_1, and W_2 from tables 2-2 and 2-3 with and without a wind travel limit of 100 miles per day; (Variations 3 and 4) - local a, b, a_1, and b_1, with the original Penman [15] W_1 and W_2 values of 1.0 and 0.01, respectively, with and without the wind travel limit and; (Variation 5) - local constraints for all six parameters. Variation 5 allowed a default to 1.1, -0.10, 0.39, -0.05, 1.0, and 0.01 for a, b, a_1, b_1, W_1 and W_2, respectively, if local values were undetermined. It was assumed that an alfalfa reference was intended by these variations.

Dec 3 11:35 1986 barroagx.pen Page 1

PENMAN DAILY TOTALS
BARRO AUGUST 1 TO 31 1986

DAY	H TEMP	L TEMP	RHLOW	RHHIG	SOL RAD	CDAY SOL	SOIL FX	LW RAD	CLW RAD	VAPR P	SOIL T	PENMAN
214	35.850	20.060	9.040	23.520	719.690	767.699	-0.000	201.060	208.631	6.137	27.955	0.359
215	37.330	21.460	8.760	30.770	632.674	764.493	-4.800	163.176	196.683	8.099	28.435	0.342
216	34.920	21.940	9.520	24.840	693.293	761.232	0.817	191.132	205.272	6.658	28.593	0.349
217	35.990	21.320	9.110	25.040	686.157	757.916	0.858	190.129	205.659	6.704	28.827	0.351
218	35.130	21.800	12.720	38.180	569.475	754.546	0.258	134.315	181.319	9.883	28.517	0.314
219	34.080	20.850	12.640	39.830	510.318	751.122	3.650	117.290	180.759	9.611	28.195	0.294
220	34.640	19.780	10.430	40.450	668.272	747.646	2.517	166.827	183.230	9.181	27.713	0.335
221	35.490	20.430	10.130	33.840	677.558	744.117	-2.075	178.254	191.491	8.290	27.545	0.341
222	36.140	20.990	8.880	31.930	655.632	740.538	-3.267	175.925	195.446	7.970	27.912	0.338
223	34.920	22.280	11.990	25.700	571.195	736.907	-1.125	153.058	199.905	7.375	28.375	0.315
224	35.410	21.800	8.890	42.500	677.128	733.227	-0.075	170.854	180.481	10.059	28.590	0.339
225	29.970	18.740	13.280	22.810	653.482	729.498	14.158	186.030	203.785	5.499	27.187	0.317
226	32.150	18.120	11.440	26.020	643.422	725.721	4.483	182.194	202.068	5.980	26.032	0.317
227	34.850	17.370	9.520	51.290	651.161	721.896	-4.550	159.675	173.474	10.285	25.200	0.320
228	36.650	22.520	8.450	29.490	639.467	718.025	-13.208	180.265	198.853	7.859	26.943	0.329
229	36.360	22.140	8.740	24.470	638.607	714.109	-4.675	188.464	206.874	6.748	28.315	0.332
230	37.860	20.520	8.010	23.620	631.556	710.148	0.758	189.989	209.937	6.405	29.342	0.338
231	34.570	24.780	16.190	51.510	505.589	706.143	-1.517	109.229	157.503	14.097	29.372	0.284
232	31.330	18.560	37.440	87.500	393.551	702.095	14.958	60.509	120.091	19.720	27.937	0.208
233	31.010	18.300	28.330	90.200	575.924	698.005	8.850	104.008	125.823	18.387	26.425	0.260
234	30.330	18.210	32.680	73.500	565.606	693.873	1.767	111.007	136.292	16.096	24.623	0.256
235	32.020	20.150	20.470	64.510	453.396	689.701	-5.408	92.606	148.883	14.352	25.003	0.242
236	33.000	21.270	23.140	57.860	504.299	685.490	-6.525	107.534	149.868	14.553	25.830	0.262
237	34.290	19.280	12.580	60.340	582.287	681.240	-0.583	137.274	159.104	12.834	26.668	0.293
238	34.640	20.010	17.550	50.450	560.361	676.952	-1.217	135.332	163.074	12.354	27.082	0.290
239	33.810	19.800	20.900	65.300	458.555	672.628	0.833	95.088	145.903	15.190	26.972	0.189
240	33.140	19.280	21.200	80.400	445.400	668.268	2.850	84.802	133.941	17.286	26.780	0.237
241	29.910	17.150	19.990	80.600	484.265	663.874	9.925	101.305	142.696	14.585	25.515	0.236
242	32.540	17.810	12.890	65.660	545.400	659.446	-1.017	130.340	157.223	12.569	24.972	0.269
243	27.420	16.630	19.460	41.090	565.520	654.985	7.758	155.822	178.417	8.014	23.577	0.262

penman total : 8.918
Temperatures are in degrees celsius
Relative humidities are percentages
Radiation is expressed in cal/cm2/day
Vapor pressure is in millibars
Penman values are in inches of water

Dec 3 11:35 1986 barrosex.pen Page 1

PENMAN DAILY TOTALS
BARRO SEPTEMBER 1 TO 30 1986

DAY	H TEMP	L TEMP	RHLOW	RHHIG	SOL RAD	CDAY SOL	SOIL FX	LW RAD	CLW RAD	VAPR P	SOIL T	PENMAN
244	26.590	15.180	25.470	48.590	554.428	650.492	0.000	144.706	168.295	9.141	20.885	0.244
245	26.750	15.180	26.380	78.900	532.244	645.969	-0.267	120.230	145.695	13.058	20.912	0.224
246	28.150	15.770	12.110	50.180	567.240	641.416	-3.450	159.121	177.014	8.212	21.270	0.256
247	30.640	15.520	12.370	52.880	562.941	636.834	-5.392	155.634	173.226	9.207	22.002	0.260
248	34.020	16.460	9.680	56.980	567.068	632.225	-9.067	154.331	168.803	10.708	23.427	0.278
249	32.610	19.560	11.630	48.290	534.394	627.589	-6.417	149.521	174.098	10.120	24.802	0.263
250	31.960	20.620	12.840	41.900	495.701	622.927	-2.092	141.969	179.520	9.358	25.872	0.261
251	30.520	16.290	11.000	60.060	441.789	618.241	9.275	115.792	167.378	10.226	25.260	0.238
252	20.250	10.130	37.300	91.500	199.312	613.533	32.192	31.262	144.513	11.111	21.628	0.122
253	20.200	8.940	37.310	91.100	547.463	608.800	15.758	133.753	145.846	10.644	17.722	0.187
254	24.320	10.170	18.200	69.690	522.786	604.048	-7.883	143.104	163.384	8.644	15.668	0.208
255	26.050	11.910	13.100	53.600	425.795	599.275	-10.242	121.000	176.171	7.316	16.932	0.205
256	28.090	14.840	12.990	37.830	515.649	594.483	-11.175	165.279	188.198	6.500	19.230	0.238
257	26.430	13.470	12.620	49.870	536.113	589.674	0.908	166.018	178.670	7.280	20.132	0.234
258	26.590	11.700	14.560	61.670	558.298	584.847	5.208	166.426	169.026	8.448	20.187	0.234
259	23.110	11.620	13.970	46.190	514.445	580.005	7.275	163.982	181.773	5.962	18.820	0.218
260	21.850	10.010	20.940	53.550	471.883	575.149	7.750	141.767	172.686	6.738	17.480	0.197
261	17.460	7.680	24.060	95.700	391.230	570.279	13.592	100.139	152.428	8.714	15.288	0.145
262	14.590	8.350	55.740	82.400	329.063	565.397	9.267	77.582	146.370	9.348	13.323	0.115
263	17.410	8.030	39.770	83.500	495.529	560.504	-2.333	135.449	150.738	9.058	12.253	0.155
264	19.920	8.660	26.830	68.990	501.462	555.602	-7.317	149.060	161.825	7.800	12.827	0.175
265	21.560	8.660	28.010	57.860	496.819	550.692	-5.350	153.360	166.578	7.370	14.040	0.135
266	20.940	9.570	28.850	56.250	465.864	545.774	-1.850	143.665	166.786	7.372	14.885	0.179
267	18.560	8.070	33.320	91.300	329.923	540.848	6.225	84.305	149.420	9.521	14.560	0.129
268	14.210	6.896	43.140	92.300	399.226	535.918	12.440	108.280	148.568	8.624	13.041	0.125
269	13.010	4.483	43.110	89.000	347.549	530.987	10.625	93.780	151.617	7.450	10.872	0.111
270	10.410	3.324	60.720	85.700	237.317	526.051	9.276	55.053	148.640	7.264	8.722	0.080
271	13.840	1.901	30.590	74.400	504.041	521.115	-0.213	161.780	161.775	5.579	7.828	0.142
272	15.470	3.478	37.040	70.500	492.863	516.178	-7.018	157.747	160.166	6.370	8.070	0.143
273	16.540	8.190	44.170	83.100	426.913	511.244	-12.309	125.391	149.495	9.137	9.903	0.128

penman total : 5.685
Temperatures are in degrees celsius
Relative humidities are percentages
Radiation is expressed in cal/cm2/day
Vapor pressure is in millibars
Penman values are in inches of water

Dec 3 11:35 1986 barroocx.pen Page 1

PENMAN DAILY TOTALS
BARRO OCTOBER 1 TO 28 1986

DAY	H TEMP	L TEMP	RHLOW	RHHIG	SOL RAD	CDAY SOL	SOIL FX	LW RAD	CLW RAD	VAPR P	SOIL T	PENMAN
274	10.050	6.622	70.600	94.300	146.862	506.312	0.000	24.617	141.579	9.044	8.336	0.053
275	7.870	5.025	77.700	95.000	94.153	501.382	6.295	6.955	141.655	8.324	7.707	0.040
276	14.300	5.997	52.290	92.700	406.105	496.460	-9.189	119.210	145.740	8.986	8.311	0.108
277	16.980	3.478	39.290	89.500	461.565	491.544	-6.437	146.486	151.705	8.025	8.942	0.122
278	17.720	4.677	44.220	88.600	462.855	486.635	-3.366	146.157	149.081	8.828	10.525	0.133
279	20.250	5.958	39.550	84.800	462.855	481.734	-7.967	148.916	150.088	9.370	11.511	0.138
280	20.620	4.522	37.170	92.400	455.030	476.846	-1.399	146.444	148.797	9.429	12.291	0.129
281	21.800	5.880	35.080	91.800	447.120	471.970	-3.342	144.461	148.049	10.031	13.172	0.139
282	23.060	7.200	39.060	85.400	441.789	467.104	-6.415	143.190	147.030	10.695	13.847	0.144
283	23.560	7.130	23.240	85.700	420.464	462.256	-2.867	143.461	154.308	9.492	14.772	0.156
284	10.170	4.328	43.200	59.410	347.979	457.422	26.628	121.521	162.439	5.226	12.575	0.116
285	14.710	0.596	16.960	64.350	447.120	452.606	12.147	175.360	171.048	4.257	10.082	0.140
286	16.330	-0.668	16.380	71.400	441.273	447.808	-1.267	172.499	168.753	4.652	7.578	0.137
287	17.370	0.481	21.070	72.600	430.954	443.029	-3.945	167.405	166.283	5.347	8.137	0.129
288	17.940	2.900	29.020	67.040	425.193	438.273	-6.806	165.221	164.630	6.062	9.059	0.128
289	17.850	2.362	37.060	74.000	418.745	433.539	-1.444	158.438	158.697	6.863	9.817	0.120
290	20.850	4.406	34.140	81.100	320.980	428.830	-7.883	113.489	154.791	8.417	11.051	0.121
291	15.390	6.465	51.990	85.300	333.018	424.145	1.465	114.745	147.509	8.962	11.221	0.099
292	11.620	6.661	72.300	88.200	220.292	419.490	8.791	65.362	141.882	9.296	10.899	0.067
293	14.970	4.560	64.930	92.500	348.237	414.861	0.897	120.041	142.216	9.509	9.944	0.084
294	15.310	3.093	57.370	93.000	327.859	410.262	0.838	115.545	145.347	8.745	9.369	0.089
295	18.340	5.880	51.970	87.800	364.316	405.694	-8.756	132.891	145.146	9.867	10.359	0.099
296	18.250	4.135	57.650	86.900	365.262	401.160	-1.789	134.813	144.831	9.604	10.835	0.094
297	19.280	4.599	48.610	84.100	360.963	396.661	-0.961	137.940	148.290	9.264	11.747	0.100
298	21.180	5.258	43.340	86.600	368.186	392.196	-5.510	142.564	147.687	9.865	12.117	0.102
299	19.740	3.826	42.670	85.500	345.486	387.768	2.654	136.394	150.384	8.855	12.314	0.104
300	18.920	5.181	41.980	86.100	304.987	383.379	1.502	118.505	149.904	9.007	12.351	0.104
301	15.600	6.387	41.960	69.380	324.764	379.031	3.078	136.263	157.470	7.300	11.609	0.104

penman total : 3.100
Temperatures are in degrees celsius
Relative humidities are percentages
Radiation is expressed in cal/cm2/day
Vapor pressure is in millibars
Penman values are in inches of water

Supplement 3 - Continued

Dec 3 11:36 1986 lemayjlx.pen Page 1

PENMAN DAILY TOTALS
LEMAY JUNE 29 TO JULY 26 1986

DAY	H TEMP	L TEMP	RHLOW	RHHIG	SOL RAD	CDAY SOL	SOIL FX	LW RAD	CLW RAD	VAPR P	SOIL T	PENMAN
181	31.330	15.010	11.940	39.840	757.524	838.307	0.000	172.002	186.465	7.347	23.170	0.347
182	33.950	14.930	11.160	42.640	753.654	837.257	-4.233	168.470	183.483	8.239	23.593	0.358
183	35.850	15.860	9.850	40.460	676.268	836.135	-6.833	150.118	186.080	8.382	24.488	0.348
184	37.180	20.940	8.670	35.460	704.385	834.942	-13.042	161.486	190.155	8.869	26.452	0.363
185	36.060	11.910	9.490	27.690	582.975	833.676	11.575	137.308	203.986	5.541	26.300	0.341
186	21.370	6.622	16.560	50.060	766.122	832.340	41.755	168.330	178.516	5.320	22.347	0.299
187	27.920	10.090	15.320	35.760	653.654	830.932	-0.048	147.575	189.268	5.611	18.995	0.293
188	31.010	16.070	12.230	40.140	654.600	829.454	-23.465	145.148	185.417	7.598	18.847	0.307
189	30.510	16.800	12.500	40.610	725.022	827.906	-7.942	163.845	184.429	7.759	22.067	0.331
190	29.970	15.730	12.740	40.820	600.000	826.287	2.492	130.366	184.683	7.453	23.348	0.301
191	30.030	16.420	17.220	38.730	635.254	824.599	0.092	138.214	181.894	7.965	23.243	0.308
192	30.580	16.980	23.720	46.450	603.869	822.843	-2.475	119.841	167.531	10.329	23.285	0.295
193	31.070	12.600	13.230	40.300	728.891	821.017	5.558	167.873	185.884	7.003	22.947	0.337
194	34.850	15.640	12.950	37.780	669.819	819.124	-8.125	152.179	186.123	8.151	23.620	0.338
195	33.810	16.460	13.930	47.370	541.273	817.163	-5.317	109.572	174.449	9.786	24.072	0.296
196	27.190	16.670	32.030	76.300	350.559	815.134	10.867	48.676	141.223	14.255	24.103	0.195
197	28.440	14.880	31.810	73.500	505.417	813.038	6.242	83.374	144.145	13.631	22.908	0.241
198	26.370	10.170	18.760	52.290	683.319	810.877	11.750	147.320	173.710	7.454	20.620	0.293
199	32.220	12.110	10.180	51.030	723.818	808.649	-7.333	162.843	178.553	8.171	20.698	0.331
200	34.920	14.670	9.360	37.430	724.850	806.356	-15.258	175.313	191.247	7.319	21.743	0.348
201	34.080	15.940	10.070	35.190	715.649	803.998	-5.100	174.540	192.662	7.171	23.990	0.347
202	32.670	14.800	12.180	42.410	634.566	801.576	3.892	143.972	183.215	8.014	24.513	0.320
203	27.870	20.250	23.880	52.440	618.229	799.090	1.042	123.074	161.118	11.425	24.268	0.288
204	27.360	15.600	37.320	98.100	478.332	796.541	8.058	69.264	125.337	17.336	23.092	0.211
205	27.420	14.340	37.100	95.700	518.487	793.928	6.300	79.471	128.857	16.386	22.140	0.221
206	29.970	13.470	24.980	94.200	605.159	791.254	-1.800	101.653	134.985	15.483	21.360	0.261

penman total : 7.921
Temperatures are in degrees celsius
Relative humidities are percentages
Radiation is expressed in cal/cm2/day
Vapor pressure is in millibars
Penman values are in inches of water

Dec 3 11:36 1986 lemayagx.pen Page 1

PENMAN DAILY TOTALS
LEMAY AUGUST 14 TO SEPTEMBER 2 1986

DAY	H TEMP	L TEMP	RHLOW	RHHIG	SOL RAD	CDAY SOL	SOIL FX	LW RAD	CLW RAD	VAPR P	SOIL T	PENMAN
227	34.640	14.970	9.670	44.310	634.566	721.896	0.000	163.290	182.976	8.449	24.805	0.319
228	35.340	18.120	9.020	37.580	605.159	718.025	-6.417	160.689	189.441	8.175	25.447	0.315
229	35.560	16.460	9.120	39.140	645.056	714.109	-0.808	173.607	188.288	8.115	25.848	0.327
230	36.500	17.990	8.490	26.730	607.739	710.148	-2.917	177.169	205.041	6.369	26.662	0.303
231	31.640	23.910	14.920	35.980	433.362	706.143	-3.825	103.484	181.960	9.494	27.010	0.260
232	29.370	18.610	27.220	93.200	335.598	702.095	11.733	51.066	126.667	17.951	26.337	0.186
233	30.330	17.680	29.300	93.200	575.838	698.005	6.258	103.510	125.243	18.277	25.257	0.252
234	28.320	15.390	41.340	91.500	559.931	693.873	7.142	101.155	125.737	17.400	23.283	0.232
235	33.400	16.110	19.110	82.600	501.548	689.701	-6.083	98.068	138.674	15.872	23.538	0.250
236	33.340	17.760	20.720	73.800	536.113	685.490	-7.483	110.039	142.141	15.465	24.053	0.262
237	34.850	18.920	13.640	60.280	596.561	681.240	-5.775	140.289	157.921	13.087	25.730	0.299
238	33.880	16.850	14.280	48.800	522.786	676.952	2.842	131.249	172.206	10.208	25.933	0.280
239	34.150	17.110	11.980	54.430	586.672	672.628	1.650	148.809	168.318	10.918	25.960	0.288
240	31.260	17.630	21.440	67.310	453.826	668.268	3.508	97.186	149.860	13.596	25.147	0.238
241	30.210	16.370	25.920	86.200	519.605	663.874	5.825	104.586	134.971	16.023	24.455	0.238
242	30.210	15.860	18.660	71.100	502.838	659.446	2.775	114.332	152.388	12.632	23.590	0.243
243	27.810	14.130	17.680	53.570	578.934	654.985	7.308	153.253	170.594	8.840	22.432	0.259
244	25.940	12.150	25.650	62.480	526.913	650.492	9.858	129.991	160.835	9.706	21.017	0.231

penman total : 4.778
Temperatures are in degrees celsius
Relative humidities are percentages
Radiation is expressed in cal/cm2/day
Vapor pressure is in millibars
Penman values are in inches of water

West Desert Pumping Project
Technical Appendix B

Outlet Canal

Page	Contents

Figures

Appendix B
Outlet Canal

Introduction

The Outlet Canal of the West Desert Pumping Project is the means by which the brine water from the Great Salt Lake flows west through the Hogup Mountains into the west desert. The canal was designed so that minimum head loss occurred through its length and also minimum excavation. The canal optimization process is herein discussed.

Canal Characteristics

The Outlet Canal extends 21,327 feet (4.04 miles) in a northwesterly direction from the West Desert Pump Plant. The canal conveys brine from the Pump Plant to the west desert where the canal terminates, allowing the brine to sheet flow and form the evaporation pond of the project.

The canal lies 300 feet south of and parallel to the Southern Pacific Transportation Company (SPTC) railroad tracks. From the Pumping Plant the canal follows this alignment through the Hogup Mountain Range for approximately 13,300 feet. The canal then cuts through mud flats of the west desert for the remaining 1.5 miles. The canal turns 90 degrees and passes north under the SPTC railroad embankment and daylights. The brine continues to flow westerly for approximately 16 miles, then passes south under the same SPTC track embankment near a site called Lemay. Once past Lemay, the West Pond extends approximately 34 miles to form the main evaporation pond. The 16 miles of sheet flow over the West Desert from Lemay to Hogup Ridge, referred to as flood plains, are hydraulically modeled along with the four-mile Outlet Canal.

From the Pumping Plant through the mountain range, the canal cuts through three limestone bedrock ridges and two basins. In this upper 2.5 mile reach, the canal varies in cross sections depending on the subsurface materials encountered. The sections vary from rectangular in shape through the bedrock to a trapezoidal shape through the basins. In the mud flat region the canal remains trapezoidal; however, it is wider and shallower to minimize excavation into the groundwater table. The 16 miles of sheet flow from Hogup to Lemay are over the existing topography of the West Desert and did not require any excavation.

Design Basis

The Outlet Canal was designed to convey a maximum flow of 3,300 cubic feet per second (cfs) at a maximum non-erodible velocity. In keeping the velocities to a minimum of two to four feet per second, the canal was not exposed to significant erosion problems caused by high-flow velocities. This eliminated the need to line the canal with riprap, a definite savings of cost and construction time.

Geotechnical investigation of subsurface conditions were conducted through a series of exploration borings along the entire length of the canal alignment. Materials encountered were generally classified into four categories: fractured limestone bedrock, cemented gravels, cohesive silts and clays, and granular silty sands.

Each material classification was given specific design parameters. These parameters included a maximum non-erodible velocity and a design Mannings "n" coefficient. A design canal cross section was based on the natural angle of response of each material.

Outlet Canal Design Parameters

Material	Maximum Velocity (fps)	Mannings "n" Coefficient	Canal Side Slope (Horiz. to vert.)
limestone bedrock	4.0	0.040	near vertical
cemented gravels	3.0	0.025	2.5:1
cohesive silts & clays	3.0	0.028	2.5:1
granular silty sands	2.0[1]	0.025	2.5:1
flood plains of west desert	N/A	0.030	N/A

[1] In the sand areas, the canal was overexcavated and lined with gravel to allow an increase in the maximum allowable velocity to 2.5 fps.

Using these design parameters and a maximum flow rate of 3,300 cfs, several design calculations were made using the Mannings equation for open channel flow.

$$Q = VA = \frac{1.486}{n} R^{2/3} S^{1/2}$$

Where: Q = flow rate, cfs
V = velocity, fps
A = area, ft^2
n = Mannings coefficient
R = hydraulic radius, ft.
S = canal slope, ft/ft

Rearranging and solving for slope: $S = \left[\frac{(nQ)}{1.486}\right]^2 R^{-4/3}$

Then solving for velocity: $V = \frac{1.49}{n} R^{2/3} S^{1/2}$

By trial and error, several canal configurations were investigated to determine the velocity, canal slope and head loss for specific reaches. In this manner the canal section through limestone was investigated independently of the canal section through the clays, etc. Optimization of canal sections was accomplished similarly.

It was determined that by using several cross section configurations through the length of the canal, an efficient canal could be constructed. Through the mountain ridge the canal was kept to a narrow section, approximately 35' to 45' bottom width. Once the canal reached the mud flats of the West Desert where the water table was fairly close to the ground surface, the canal was widened and made shallower. This configuration maintained the low velocity through this reach while minimizing excavation into the groundwater.

Hydraulic Design Analysis

Using the physical characteristics of the canal and flood plains of the West Desert, hydraulic analyses were conducted using the Corps of Engineers Hydraulic Engineering Center (HEC-2) Water Surface Profile computer model. This computer model was used to simulate backwater from

Design Section Criteria

Location/Materials	Bottom Width (ft)	Side Slope	Canal Length	Canal Slope	Normal Depth of Flow	Approx. Depth of Cut (ft)
Limestone	45	near vert	1468'	.50'/1000'	15'	50
Cemented gravels, clay	35, 45	2.5:1	10435'	.30'/1000'	13'	40
Sands	35	2.5:1	2500'	.30'/1000'	13'	23
Mud Flats						
Clays	90	2.5:1	6925'	13'/1000'	9'	6

Note: In mud flat regions, the canal was designed to be flanked by excavated soils berms to minimize depth of excavation.

subcritical flow through the system. The computer model required a starting water surface elevation downstream, typical cross sections, Manning "n" coefficient for the different materials encountered, and a flow rate.

Input included (starting downstream) cross sections for the 16 miles of sheet flow through the mud flats, that is from Lemay to Hogup Ridge, through the SPTC railroad bridge at Hogup, and then the typical sections for the four-mile canal to the Pumping Plant. The computer model then calculated backwater depths, head loss and maximum velocities through the given reaches of the brine conveyance system.

Several HEC-2 computer runs were conducted using a starting water surface elevation of 4217.0 downstream at Lemay. In this manner, final canal configuration optimization, water surface elevation of the backwater, and head loss through the system were determined. A physical constraint to meet was the water surface elevation at the Pumping Plant could not exceed the 4224.5 elevation.

Results of the design HEC-2 computer analyses indicated water elevation at the Pumping Plant did not exceed the 4224.5 elevation and also velocities were within acceptable ranges. Following is a summary of several runs using the aforementioned design parameters. Backwater analysis began at Lemay with a water surface elevation of 4217.0.

HEC-2 Water Surface Elevations

Flow Rate, cfs	Hogup Bridge	Pump Plant	Head Loss in Canal, ft.
3500	4224.48	4219.71	4.77
3300	4224.11	4219.64	4.47
2800	4223.13	4219.45	3.68
2200	4221.93	4219.35	2.58
933	4219.22	4218.41	.81

These results were within expected ranges. Several additional computer model runs were conducted varying the Manning's "n" coefficient plus or minus 20 percent. The runs were made to determine the sensitivity of the canal to any variation of that specific design parameter. It was noted that the backwater surface elevation at the Pumping Plant fluctuated less than 1 percent.

Additional hydraulic analyses were conducted on the 90-degree bend in the canal, near the canal termination; concern was about whether the change of direction in flow would damage the canal downstream of the bend. These calculations and canal geometry are illustrated in Figure 1. Also estimated was the centrifugal effect on the water which may cause a transverse hydraulic gradient. Should the water surface elevation be higher on the outside of the curve, additional material would be added to the existing berm. The analysis did not show significant superelevation of the water's surface around the bend.

Canal Construction

Construction began in July 1986 and was completed in February 1987. The contractor utilized various equipment throughout the project, dependent upon subsurface materials encountered. For the western end of the canal in the mud flats, a dragline was used. In the upper reaches of the canal, a fleet of scrapers, large backhoes, and 50-ton rock wagons was used.

The contractor drilled and blasted in the limestone bedrock areas and used conventional removal for all other materials encountered. A total of 2,998,000 cubic yards of material were excavated.

Transient Flow Analysis

Additional computer modeling was performed on the Outlet Canal upon canal completion. This allowed the model to use the as-built canal cross-sections, therein providing more accurate results. This modeling was completed by Dr. David R. Schamber, Ph.D., and is the supplement to this appendix.

Numerical analysis simulated transient (unsteady) flow through the canal at the start-up of the pumps. For this analysis, a computer model of the existing outlet canal with the transition section (rectangular to trapezoidal) was used to estimate water surface elevation along the length of the canal. By inputting several start-up and shutdown scenarios, the model was able to simulate any transient surges through the canal should they occur. Should transient surges be identified,

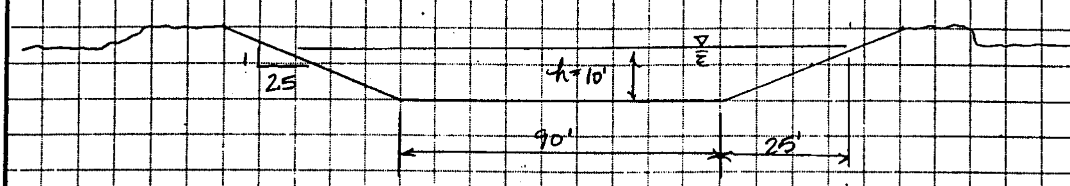
Figure 1
Outlet Canal - Flows Around a Curve

ECKHOFF, WATSON
& PREATOR ENGINEERING
1121 East 3900 South, C-100
SALT LAKE CITY, UTAH 84124
(801) 261-0090

JOB Outlet Canal flow around curve
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE ______
CHECKED BY ______ DATE ______
SCALE ______

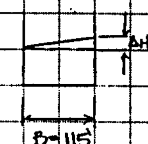
FLOW AROUND A CURVE

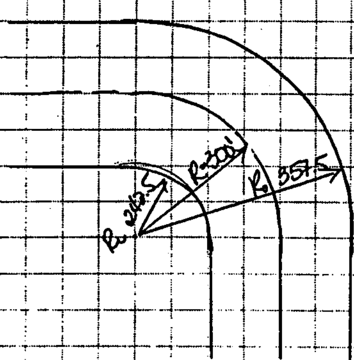
Existing Canal



For analysis purposes assume rectangular canal 115' x 10'

Assume : subcritical flow
: uniform flow
: concentric streamlines





Set gravitational forces equal to centrifugal forces on a fluid particle

$$F_c = \frac{MV^2}{R} = M g \frac{\Delta H}{B} \quad (1)$$

$$\therefore \Delta H = \left(\frac{2B}{R}\right)\left(\frac{V^2}{2g}\right)$$

$$= \frac{2(115')(3^2 fps)}{(300')(2 \cdot 32.2)} = 0.107'$$

(1) "Applied Hydraulics in Engineering" HM Morris, JM Wiggert Chapter 5 pg 188-189.

protective measures could be taken prior to the start-up sequence therein preventing any erosion damage to the earth canal.

For the simulation, boundary conditions were specified at the inlet (pump station) and the outlet (West Pond). For the inlet conditions, a discharge-time hydrograph was developed to represent the on/of action of the pumps. The outlet condition was represented as a large reservoir in which the flow enters and velocity is essentially zero. Actual cross sections of the canal were put into the model. The transitions in which the cross sections change from rectangular to trapezoidal occurs within a relatively short distance and were represented by sudden constrictions or expansions.

The computer program calculates parameters such as flow velocity, Froude number, wave velocity and energy grade line. Hydrographs of water surface elevation and discharge flows were also generated. The program did not indicate any potential problems with transient surges in the canal. The program did give valuable information regarding stabilization of flow through the canal during start and shutdown. An example of this is shown in Figure 2, which indicates that within two hours after shutdown of all three pumps the canal water surface elevation leveled out to a relatively flat hydraulic gradient. Figure 4 shows that after start-up of all three engines (0-3,300 cfs) four to five hours are required to achieve flow stabilization. Additional hydrographs are included in the supplement in this appendix.

Rating Curve Development

Upon completion of the Outlet Canal construction, flow discharge measurements tests were conducted under standard operating conditions. A flow monitoring test was completed under three operating conditions; one pump, two pumps and three pumps operating under full power from the natural gas engines. For each operating scenario, two independent flow measurement tests were completed.

A gaging station was established approximately 20,524 feet (3.89 miles) downstream from the pump discharge. At this location, flow was fully developed and substantially uniform. The canal cross section remained constant for 2,000 feet upstream and assumed 1,000 feet downstream. The canal configuration throughout this reach is a 90-foot bottom width with 2.5:1 (horizontal to vertical) side slopes, and a bottom slope of approximately 00019. The canal flowed about 9.5 feet deep with a top water width of about 135 feet.

One series of tests conducted used a standard Price meter. This meter has six conical cups mounted on a vertical axis pivoted at the ends and free to rotate between a clevis which has a veined tailpiece attached. As the cups turn, the relative velocity of the brine is lower when the cup opening is upstream and higher when the pointed end is downstream. Inversely proportional to velocity is drag. The drag on the cups is greater upstream than downstream. Due to the high overall drag on this meter, it is not generally recommended for high velocity measurements. In this specific case, with the fluid being a heavier brine of 1.120 S.G. (69.88 pcf) and at the slower velocities, it was determined that the Price meter could give results which are accurate to within 5 percent. Using the Price meter, velocities were recorded at 0.2 and 0.8 depths at 10-foot intervals across the canal.

The second series of tests was performed at the same gaging station, using a Marsh-McBirnie Model 201 meter. This meter was equipped with a electromagnetic probe which senses the velocity of the passing liquid. The probe works by sensing the conductance of the liquid. In this situation with the high conductance of the heavier saline brine, the rich signals produced more reliable

Figure 2
Z Hydrograph, Shut-down, 3300-0 cfs (hwd08).

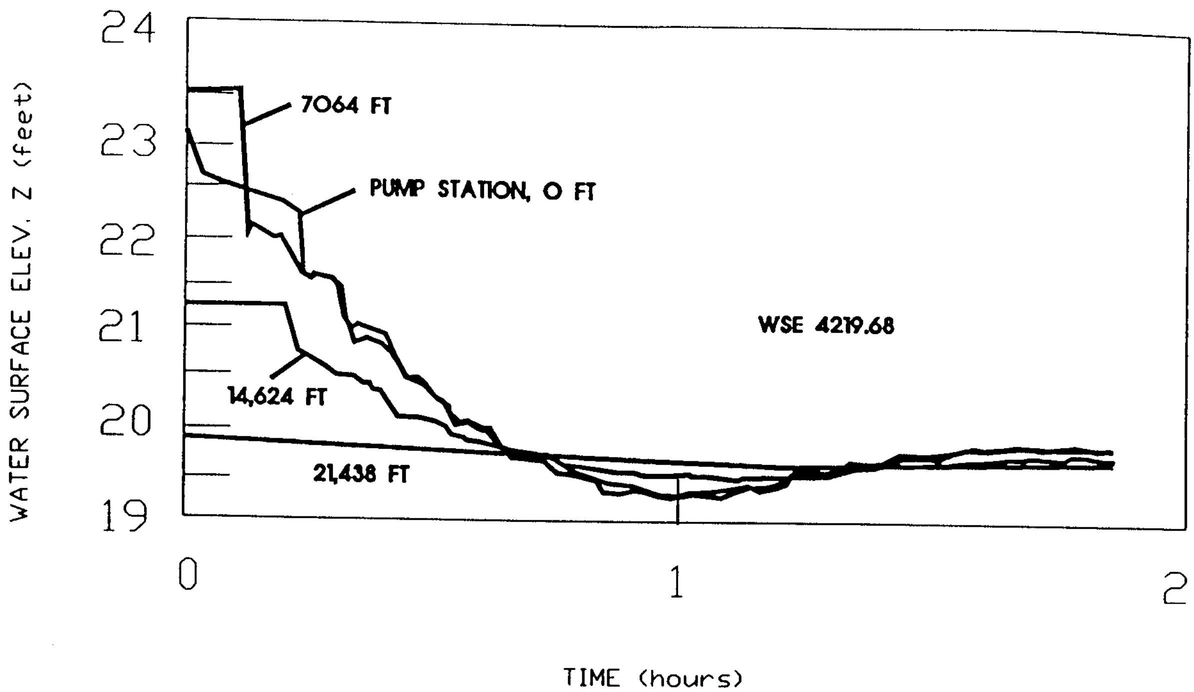


Figure 3
Q Hydrograph, Shut-down, 3300-0 cfs (hwd08).

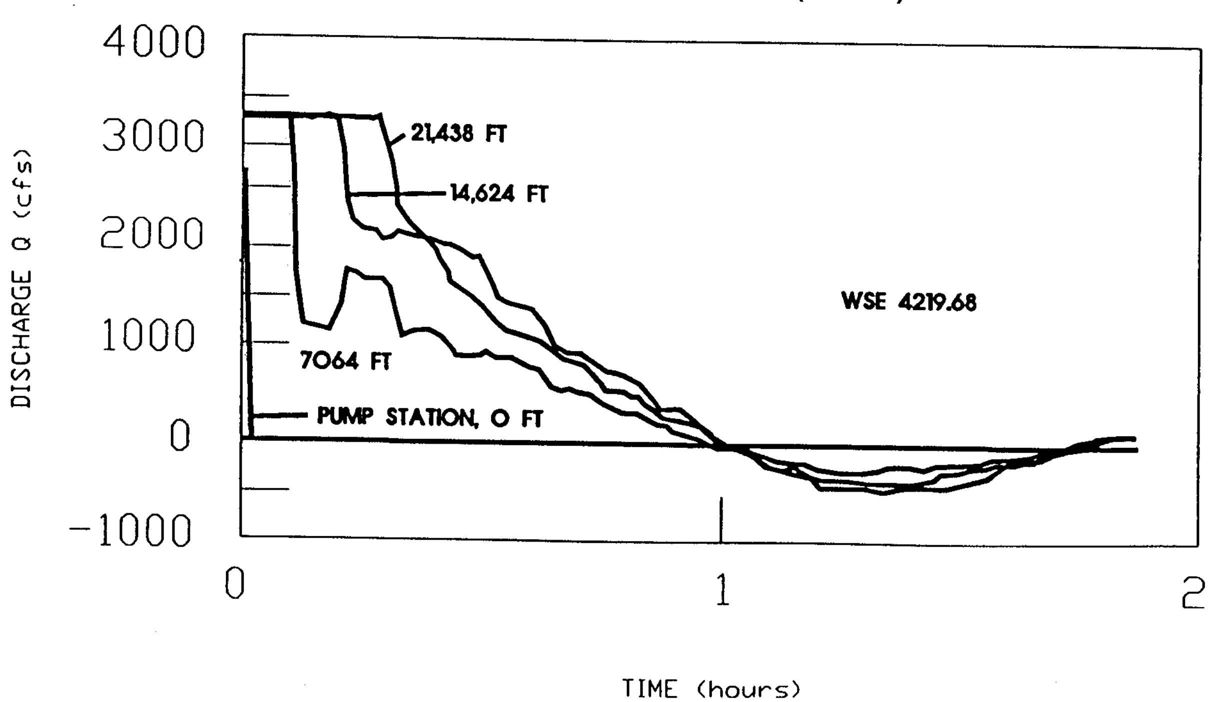


Figure 4
Z Hydrograph, Start-up, 0-3300 cfs (hwd01).

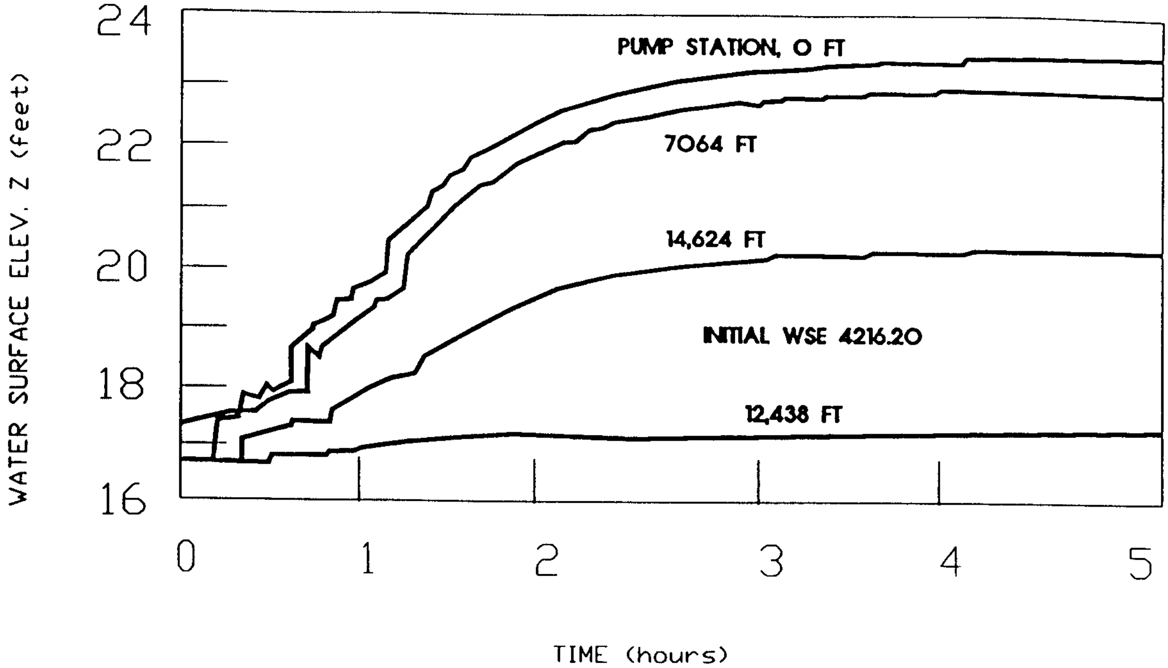
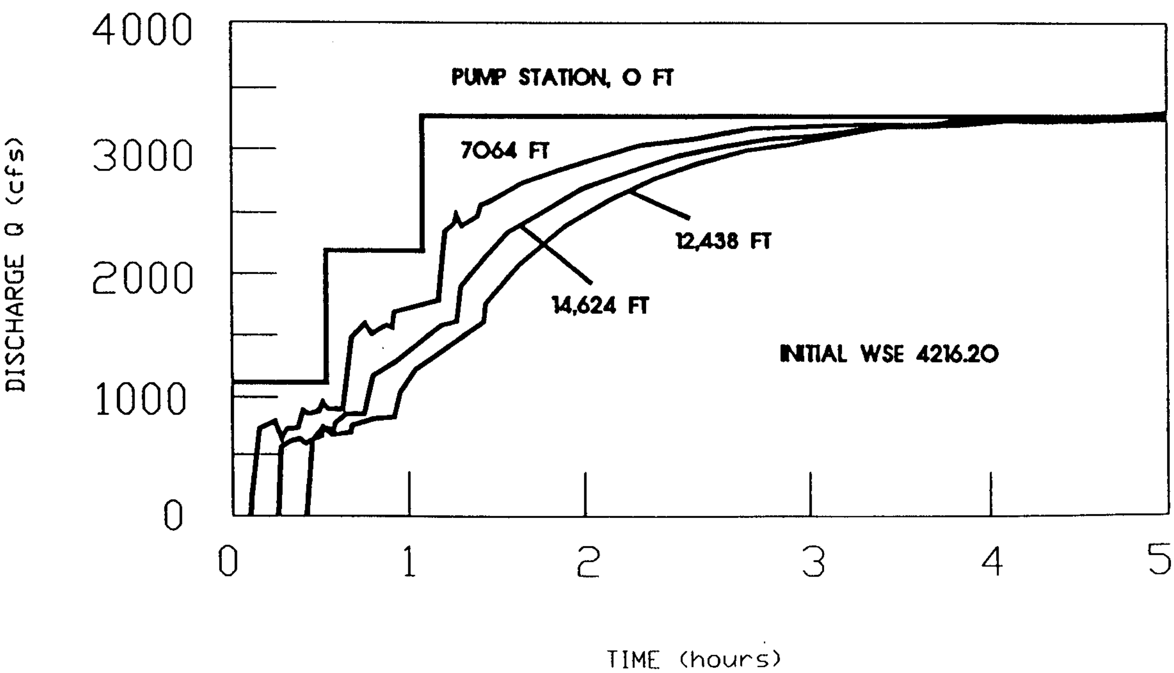


Figure 5
Q Hydrograph, Start-up, 0-3300 cfs (hwd01).



results. A Model 201 is rated at a 2 percent accuracy. Using this Marsh-McBirnie meter, velocities were measured at five difference depths, approximately at the surface, 0.2d, 0.5d, 0.8d and 0.95d along the canal at cross section 10- foot intervals.

For both tests, the velocities at a given interval were averaged, then multiplied by the end area to produce a representative flow rate.

For each pumping scenario, water surface elevations were measured across the pumps. Engine speeds were recorded along with the amount of natural gas consumed. These results are summarized below

Rating Curve Testing Results

	Flow Measured, cfs		Water Surface Elevations		
Operation	Price Meter	Marsh-McBirnie	Afterbay	Forebay	Hogup Bridge
1 pump	1096	986	4219.34	4209.24	4218.73
2 pumps	2108	2047	4220.92	4209.27	4219.08
3 pumps	3133	3076	4222.63	4209.00	4219.49

Operating Engine Speed:	330 rpm
Operating Pump Speed:	139 rpm
Brine Density:	69.9 pcf
Average Pump Head:	11.79 feet
Average Head Loss in Canal:	2.78'

The rating curve testing results from the two and three pump tests were in good agreement, within 2 percent of each other. The one pump test had a 10 percent disparity between the flow results. In a review of the data from both tests, some interpolation of the data was found in the Marsh-McBirnie one pump analysis. It was decided to eliminate that one particular test result from the flow rating curve development. The five remaining test results were further compared and evaluated.

Analyses were conducted using the remaining five flow measurement test results, assuming that all five tests came from the same data set (With this assumption made, the five sets of data were plotted simultaneously and compared for correlation purposes. This analysis was a comparison of the non-dimensional velocity profiles of all the tests.). It is noted that the velocity distribution for fully established flow through a channel is parabolic form. That is, the velocity at any point across a given section of flow can be calculated using the centerline velocity and the distance from that velocity. Also velocity of flow near rough and smooth boundaries follows a logarithmic relationship. Analysis used the velocity relationship for flow near rough boundaries.

By dividing the following relationship for the velocity distribution of turbulent flow near rough boundaries.

$$\frac{v}{v_R} = 0.68 \log \frac{y}{k} + 1 \qquad *$$

by the subsequent integration of

$$V = \frac{1}{Y_o} \quad VDY$$

to yield

$$\frac{V}{v_K} = 0.68 \log \frac{Yo}{k} + 0.71$$

gives the relative velocity expression of

$$\frac{U}{V} = \frac{0.68 \log y/k + 1}{0.68 \log yo/k + 0.71}$$

By calculating (v-V)/V for various values of y/yo and fitting these results to the logarithmic graph of the above-mentioned equation, an adjusted value for (v-V)/V was determined (reference Appendix C). Using this adjusted velocity distribution method the measured flow rates were corrected to the following values.

	Total Flow Adjusted, cfs	
Operation	Price Meter	Marsh-McBirnie
1 pump	1,082	not used
2 pumps	2,075	2,065
3 pumps	3,089	3,103

These adjusted values are within 1.5 percent of the original measured values. It was further determined that at the time of the flow measuring tests the average pumping rate per unit was 1,043 cfs.

* Equation 102 , *Engineering Hydraulics*, edited by Hunt, Rose, John Wiley

A state discharge curve was established for the canal using the water surface elevation in the afterbay and measured flows using the Price Meter tests (Figure 6). By observing the water surface elevation at the afterbay of the pump plant a total flow was determined. Similarly, this method was applied and a stage-discharge curve was established at Hogup Bridge (Figure 7).

Conclusions

Upon completion of the flow measurement tests and the correlation analysis, the discharge rates and actual canal cross sections were entered into the HEC-2 backwater profile program. In this manner, an attempt was made to calculate the actual Mannings "n" coefficient for the canal. Also determined was the actual head loss, velocities and hydraulic grade line of the system.

By setting the backwater surface elevation at the Pumping Plant and Hogup Bridge, the Mannings "n" coefficient was varied. Trial and error runs continued until the computer model simulated visual water surface elevations. Since clay materials comprised a majority of the canal, the Manning's "n" for clay varied the most.

Adjusted Canal Design Parameters

	Mannings "n"	
Material	Design	Actual
Limestone	0.040	0.040
Gravels	0.025	0.025
Silts and Clays	0.028	0.019
Sands	0.025	0.028
Flood plains	0.030	0.030

These as-built HEC-2 computer runs also noted the system head flow at Lemay, i.e, from Pumping Plant, was significantly lower than design. This news was accepted with enthusiasm. What this meant to operations was that the pumps were working against a lower total dynamic head. This resulted in overall lower power requirements from the engines, therein lowering the fuel consumption.

Figure 6
Stage Discharge Curve At Afterbay

ECKHOFF, WATSON
& PREATOR ENGINEERING
1121 East 3900 South, C-100
SALT LAKE CITY, UTAH 84124
(801) 261-0090

JOB Stage Discharge Curve @ Afterbay
SHEET NO. ______ OF ______
CALCULATED BY Knichols DATE 12/16/87
CHECKED BY ______ DATE ______
SCALE ______

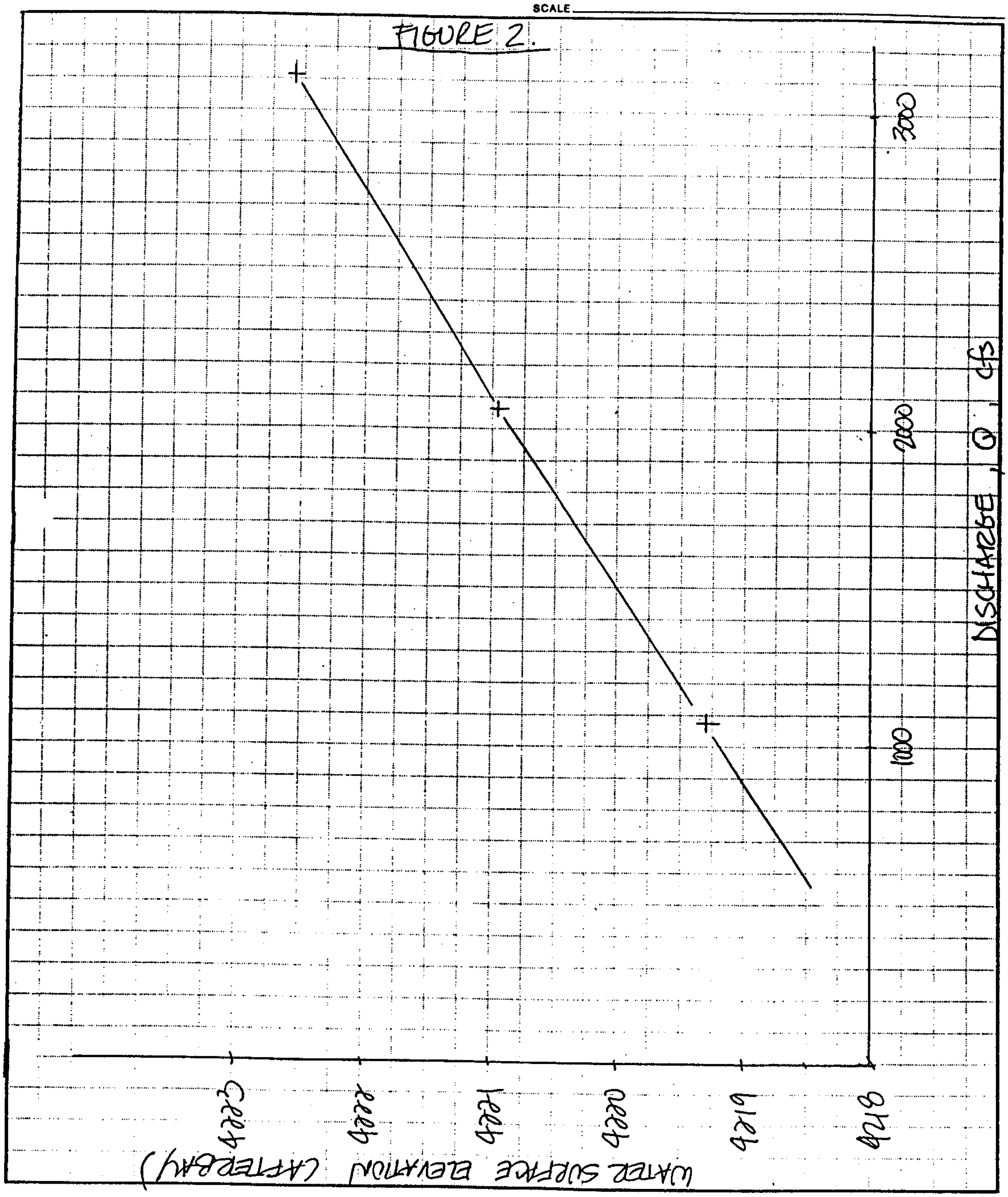
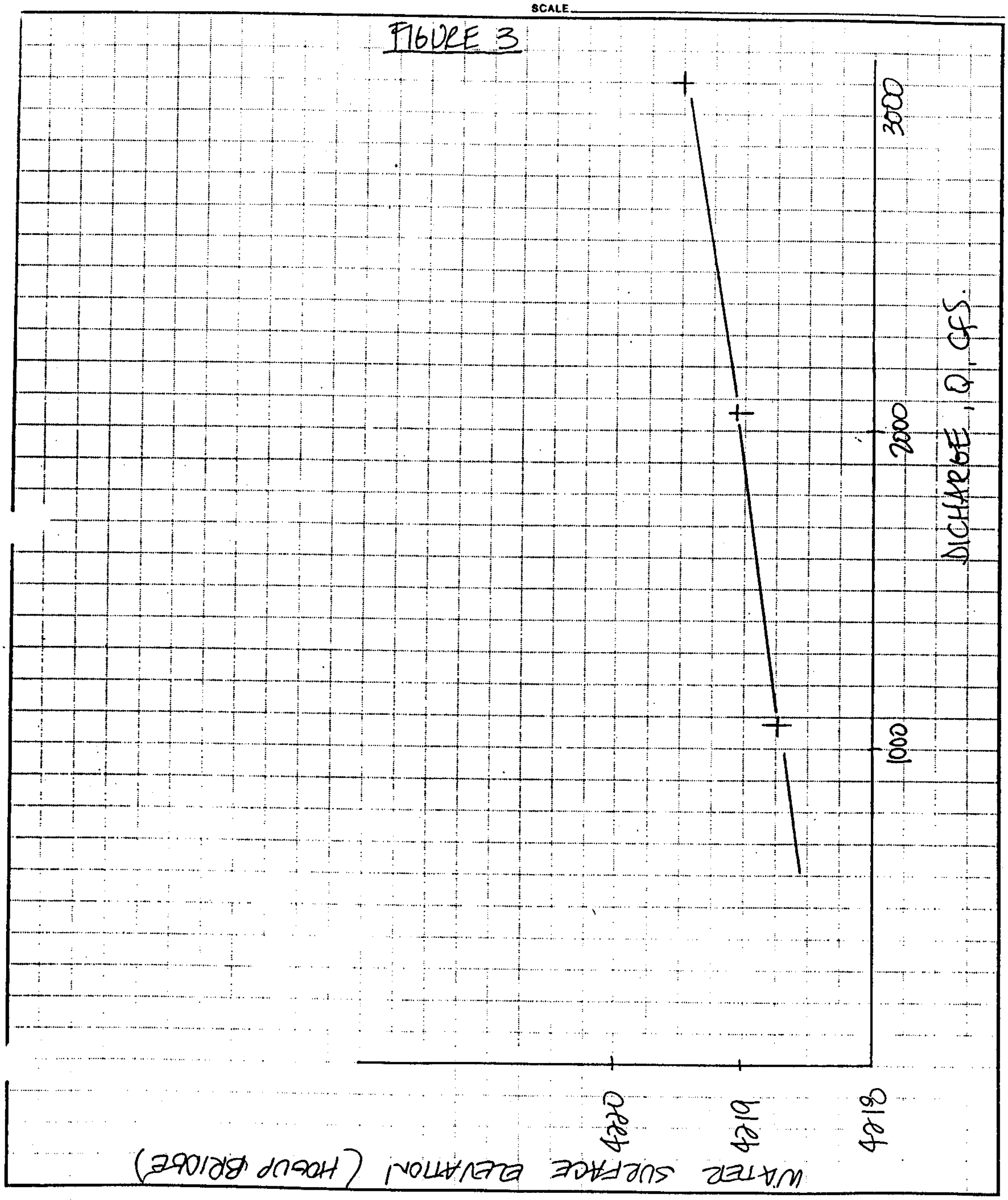


Figure 7
Stage Discharge Curve At Hogup Ridge

ECKHOFF, WATSON
& PREATOR ENGINEERING
1121 East 3900 South, C-100
SALT LAKE CITY, UTAH 84124
(801) 261-0090

JOB Stage Discharge Curve @ Hogup Brdg
SHEET NO. ______ OF ______
CALCULATED BY K Nichols DATE 12/16/87
CHECKED BY ______ DATE ______
SCALE ______



Supplement: "Transient Analysis for the Outlet Canal" Report

TRANSIENT ANALYSIS FOR THE OUTLET CANAL WEST DESERT PUMPING PROJECT

David R. Schamber, Ph.D.

Report prepared for Eckhoff, Watson and Preator Engineering
Salt Lake City, Utah
January 1987

Introduction and Problem Definition

This report presents the basic equations and numerical method used to simulate transient (unsteady), one-dimensional flow in open channels. The method developed is applied to the outlet canal which connects the Great Salt Lake to the West Desert Region in Utah. The channel geometry and bed elevations are shown in Figure 1. The channel is comprised of trapezoidal and rectangular sections. Since the transition between trapezoidal-rectangular or rectangular-trapezoidal occurs over a short distance, relative to the scale of the problem, these transitions are denoted as abrupt constrictions or expansions in Figure 1.

The initial conditions for the flow simulations are specified as a uniform water surface elevation and zero (or small) discharge at all stations along the canal. Boundary conditions are specified at the inlet (pump station) and outlet (west pond) of the canal. At the pump station a discharge-time hydrograph is specified, which simulates the on/off action of the pumps. At the west pond station the flow is assumed to enter a large reservoir where the velocity is essentially zero.

The computed results from the computer program include profiles and hydrographs of water surface elevation and discharge. Other parameters of interest such as flow velocity, Froude number, wave celerity, energy grade line, etc., are also computed and can be displayed in profile format.

The following sections of the report present the governing equations, numerical solution, and computed results.

Governing Equations

The equations of one-dimensional, unsteady, gradually varied flow are based on the conservation equations of mass and momentum. These equations are given by Strelkoff (1970) as follows

$$A\frac{\partial V}{\partial x} + VB\frac{\partial y}{\partial x} + B\frac{\partial y}{\partial t} + VA_x^y = 0 \qquad (1)$$

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial y}{\partial x} = g\,(S_o - S_f) \qquad (2)$$

In equations 1 and 2, x = distance along the channel; t = time; $V = Q/A$ = the average flow velocity; Q = volumetric discharge; y = flow depth; B = top width; A = cross-sectional area of flow; $A^y{}_x$ = $\partial A/\partial x$ = rate of change of area holding the depth fixed-represents the departure of the bed from prismatic form; S_o = channel slope; S_f = resistance slope; and g = gravitational constant. Manning's equation is used to compute the resistance term. Mathematically,

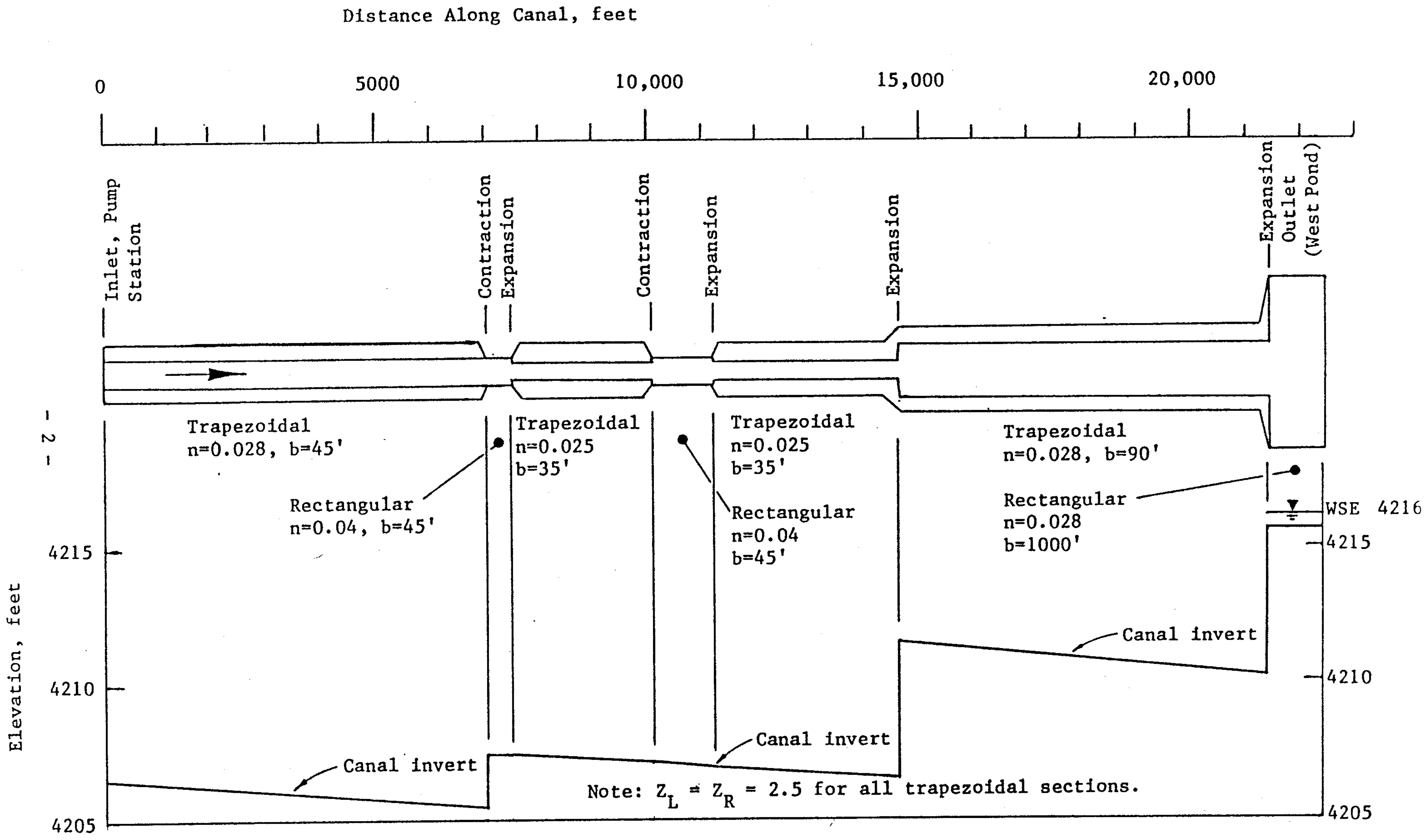


Figure 1. Plan and Profile Schematic of Canal

$$S_f = \frac{n^2}{1.486^2} \frac{V|V|}{R^{4/3}} \tag{3}$$

in which n = roughness parameter, R = A/P = hydraulic radius (area/wetted perimeter). The absolute value sign in equation 3 is required if the flow reverses direction. The nonprismatic term, $A^y{}_x$, presented in equation 1 is zero for the canal configuration depicted in Figure 1.

Although numerical schemes exist for solving equations 1 and 2 directly, it is advantageous to transform these equations into a set of four ordinary differential equations which follow the wave motion. Equations 1 and 2 are hyperbolic in nature and have the property that, through linear combination, they can be reduced to equations involving differentiation in one less direction than the original equations (Strelkoff, 1970). This transformation produces the following characteristic form of equations 1 and 2. Mathematically,

$$\frac{dQ}{dt} - B(V - c)\frac{dy}{dt} = gA\,(S_o - S_f) + V^2 A_x^y \tag{4a}$$

$$\frac{dx}{dt} = V + c \tag{4b}$$

$$\frac{dQ}{dt} - B(V + c)\frac{dy}{dt} = gA\,(S_o - S_f) + V^2 A_x^y \tag{5a}$$

$$\frac{dx}{dt} = V - c \tag{5b}$$

in which c = celerity of an elementary gravity wave is given by

$$c = \left(\frac{gA}{B}\right)^{1/2} \tag{6}$$

Equations 4 and 5 comprise a forward and a backward characteristic, respectively, defined in the x-t solution plane (cf. Appendix A and Figure 4).

The channel geometry is specified with reference to Figure 2. Mathematically,

$$B = b + (z_L + z_R)\,y \tag{7}$$

$$A = by + (z_L + z_R)\frac{y^2}{2} \tag{8}$$

$$p = b + y\left[(1 + z_L^2)^{1/2} + (1 + z_R^2)^{1/2}\right] \tag{9}$$

in which b = bottom width; z_L = parameter specifing the left slope of the channel cross-section; and z_R = parameter specifing the right slope of the channel cross-section. For a trapezoidal section b, z_L and z_R are non-zero. A rectangular section is defined by $z_L = z_R = 0$ for non-zero b, while a triangular section is defined by b = 0 for non-zero z_L and z_R.

Equations 4 and 5 are valid for a zone of gradually varied flow in which the pressure distribution is essentially hydrostatic and energy losses are produced entirely by boundary shear. These conditions do not exist in the transition regions (expansions and contractions) shown in Figure 1. The transition regions are zones of rapidly varied flow and are treated as internal boundaries within the flow domain. The theory developed by Schamber and Katopodes (1984) is used to characterize these internal boundaries in the present model. Details of this theory are presented in the original paper and only a summary is presented here.

The theory is developed with reference to Figure 3. Since the volume of fluid within zone 1-2 is negligible, changes in storage volume are presumed small and hence

$$Q_1 = Q_2 \tag{10}$$

The discharge within the transition zone is therefore uniform and varies with time. Conservation of energy, as noted in Henderson (1966, p. 237), is written between sections 1 and 2 in Figure 3. Mathematically,

$$y_1 + \frac{v_1^2}{2g} = y_2 + h + \frac{v_2^2}{2g} + k\frac{(v_1 - v_2)^2}{2g} \tag{11}$$

in which y_1 = flow depth immediately upstream of the transition; y_2 = flow depth immediately downstream of the transition; h = rise (+) or drop (-) in bed elevation between sections 1 and 2; $v_1 = Q_1/A_1$ = velocity immediately upstream of the transition; $v_2 = Q_2/A_2$ = velocity immediately downstream of the transition; and k = energy loss coefficient due to the transition. Henderson

(1966, p. 237) recommends k = 0.3; however, computer simulations with k = 0.0 and k = 0.3 produce nearly identical results for the outlet canal computations. Evidently, the energy loss due to boundary shear over the long reaches of gradually varied flow is much greater than the energy loss in the transition regions.

In Figure 1 there are six transition regions or internal boundary condtions which are coupled to equations 4 and 5. For the case of subcritical flow (which exists for all cases presented herein), a forward characteristic (at location n_2 in Figure 4) given by equation 4 and a backward characteristic (at location n_1 in Figure 4) given by equation 5 is solved simultaneous with equations 10 and 11. This process is repeated for each transition.

Numerical Solution

The numerical solution of equations 4 and 5 is developed by writing the appropriate finite difference form of these equations in the x-t solution plane. Details of this method are presented in the paper by Schamber and MacArthur (1985), which is included in Appendix A. In this paper the V-y form of the characteristic equations are presented; however the differencing of the Q-y form, i.e. equations 4a and 5a is nearly identical.

As shown in Figure 4, each reach on time lines t_i and t_{i+1} is divided into a number of node points ranging from n_1 to n_2. The grid spacing δx, for each reach may vary by assigning a different value of n_1 and n_2 for each reach. Values of depth and discharge are known at time line t_i, either from the initial conditions of the problem or from a previously computed profile. The solution is advanced to time line t_{i+1} by solving the finite difference form of equations 4 and 5. As noted previously, the transition regions (e.g. points n_2 - n_1 in Figure 4) are coupled via equations 10 and 11. The program is currently dimensioned to handle up to 20 transition regions. As shown in Figure 1 the current problem contains 6 transition regions. The left most node in Figure 4 represents the pump station at which Q is specified as a function of time, while solution of equations 5 is used to compute the water depth at the pump station. At node n_1, at the outlet transition, the velocity is assumed to be zero, and the depth is computed from simultaneous solution of equations 4, 5, 10 and 11. The time step, $\delta t = t_{i+1} - t_i$, is defined in Figure 4.

As noted in Appendix A, the finite difference form of equations 4 and 5 produces a set of four nonlinear equations at each computational point interior to the boundaries and transition nodes. These nonlinear equations are solved by Newton's method (West, 1987). Once the values of depth and discharge are determined at each node, the t_{i+1} time line becomes the t_i time line and the process is repeated advancing the solution in time.

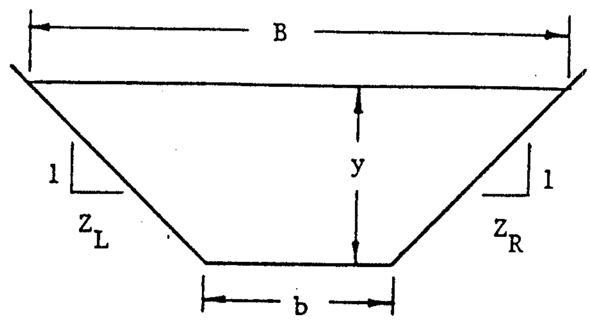


Figure 2. Cross-Sectional Parameters

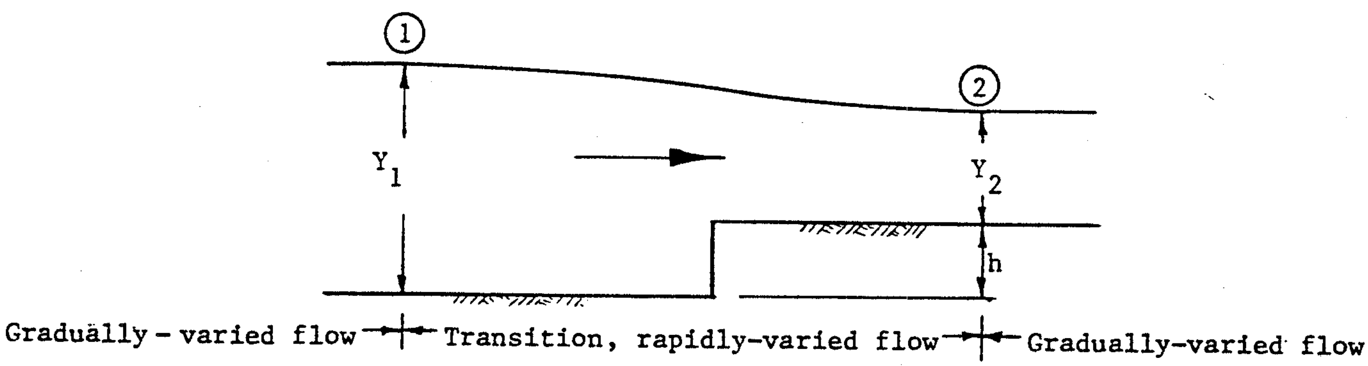


Figure 3. Flow Through a Transition

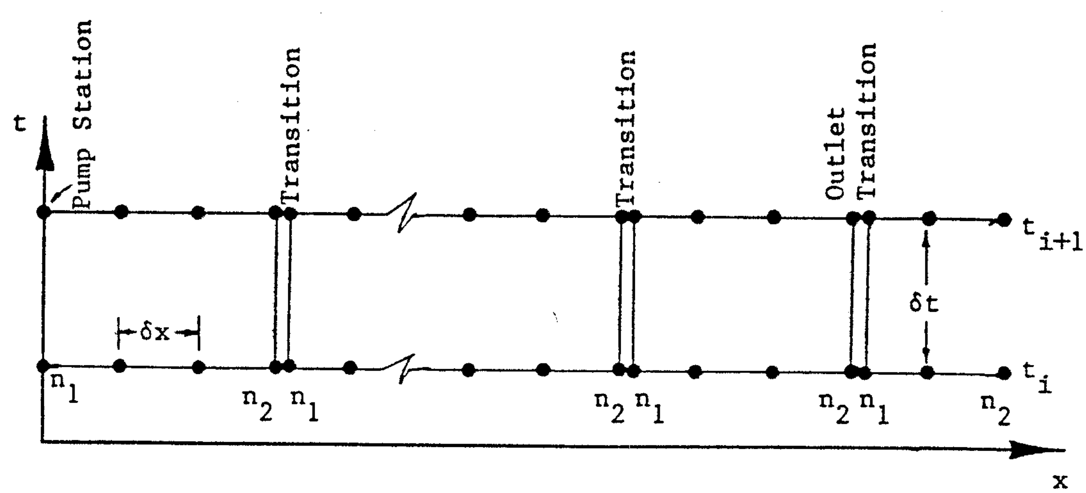


Figure 4. Computational Domain

Results

The computational runs which simulate the start-up and shut-down conditions for the pumps are shown in Table I.

Table I Computational Runs

Run No.	Boundary Condition Pump Discharge, cfs	Water Surface Elevation at Outlet (West Pond), Feet
wd01	0 → 3300 (Fig. 5)	4216.20
wd02	3300 → 10 (Fig. 6)	4216.20
wd07	0 → 3300 (Fig. 5)	4219.68
wd08	3300 → 10 (Fig. 6)	4219.68
wd09	3300 → 2200 (Fig. 7)	4219.68

Execution of the program for run xx generates a result file, denoted as rwdxx, and a hydrograph file, denoted as hwdxx. The result file is then edited to create a profile file, denoted as pwdxx. Standard graphing routines then read pwdxx and hwdxx to generate the specified profile and hydrograph plots, respectively. Plots of the computed results are shown in Figures 8-31. Each figure caption indicates the file used to generate the plot. For each run, the start-up or shut-down action of the pumps is shown in Figures 5-7. The Figure which corresponds to a boundary condition for a specific run is listed in parenthesis in Table I.

Appendix B contains a brief user's manual for the input data to the program. Appendix C and D contain a sample input data file and computed result file, respectively, for run wd07.

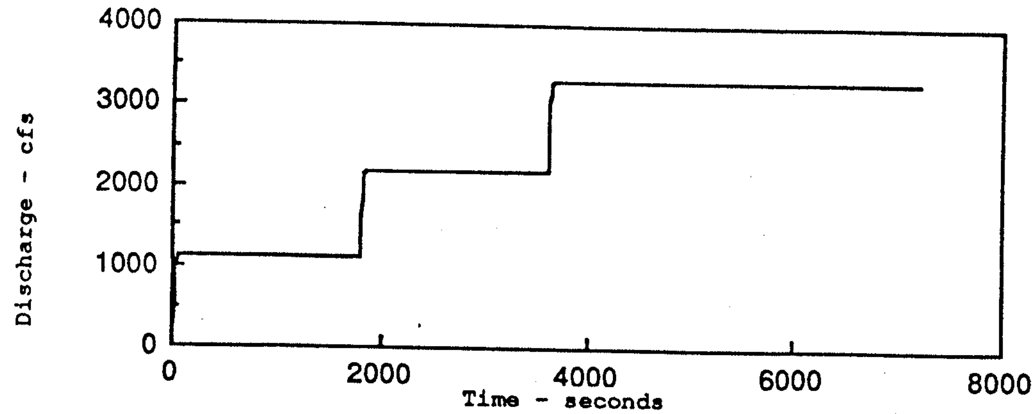


Fig. 5. Start-up Conditions at Pump.

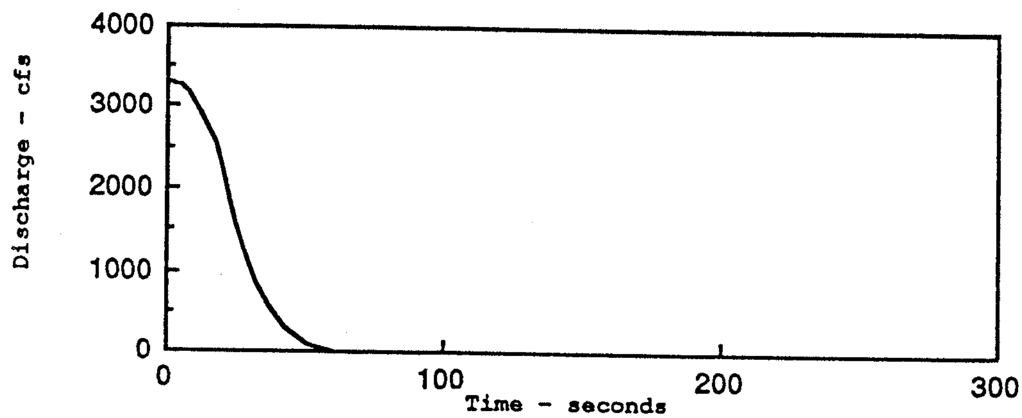


Fig. 6. Shut-down Conditions at Pump.

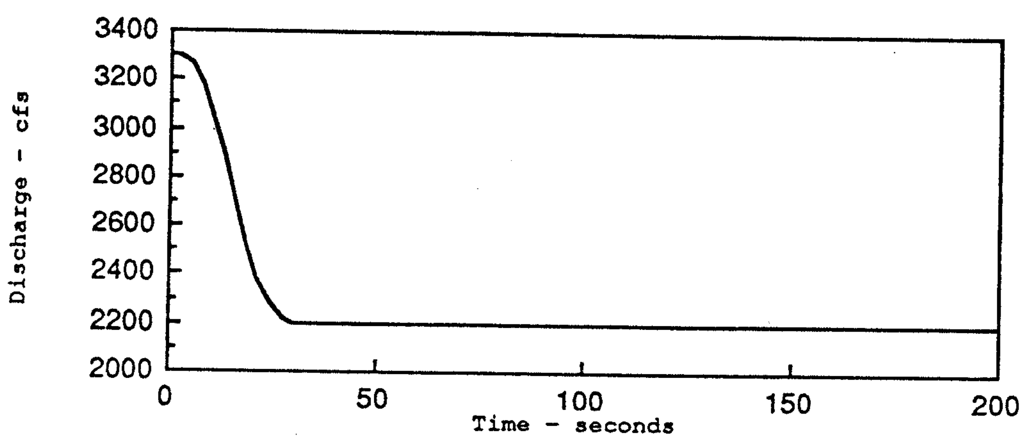


Fig. 7. Shut-down Conditions at Pump.

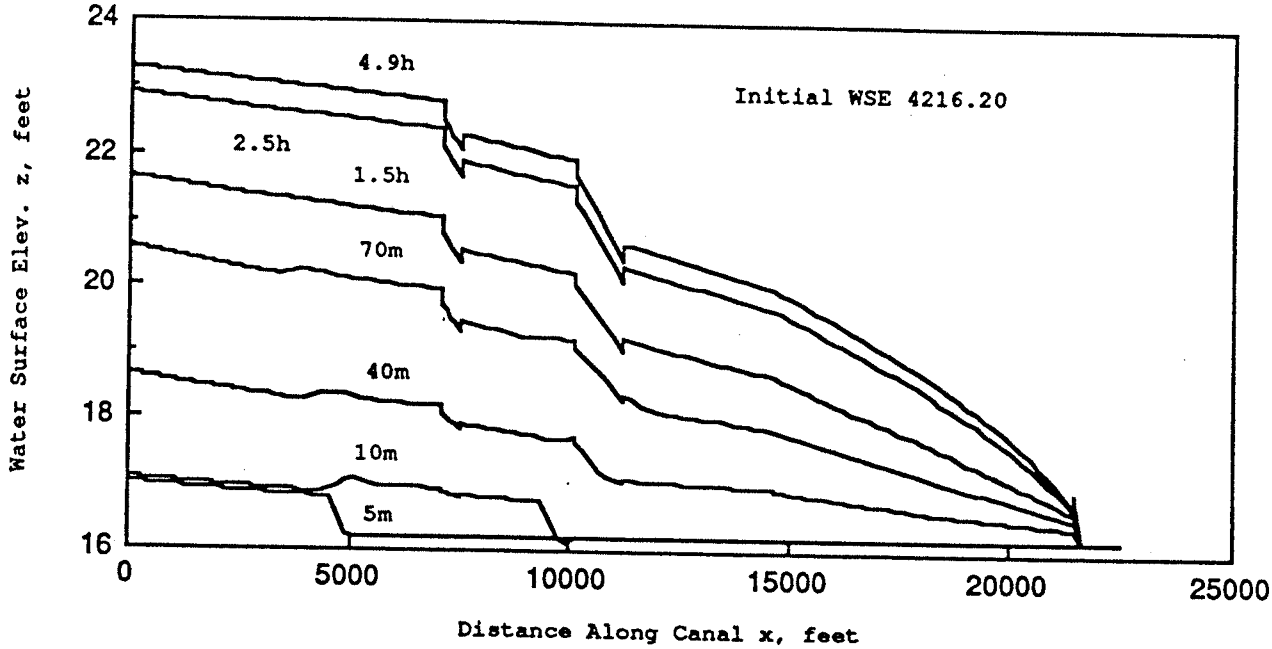


Fig. 8. z-x Profiles, Start-up, 0-3300 cfs (pwd01).

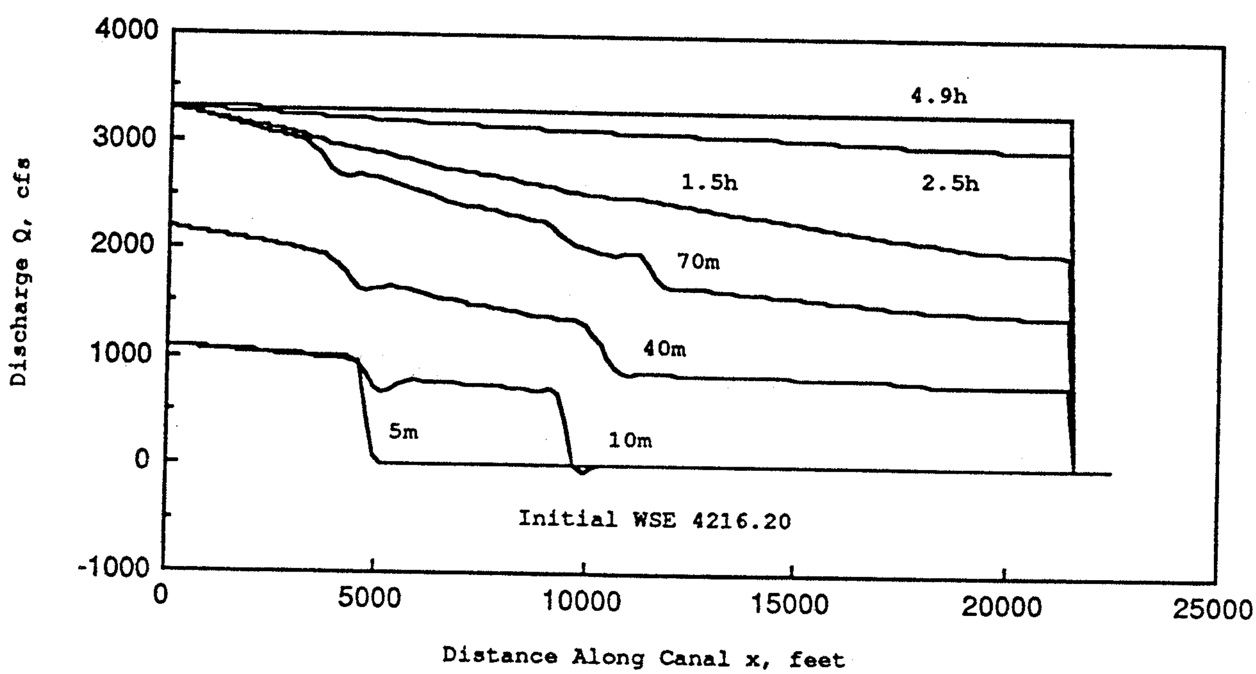


Fig. 9. Q-x Profiles, Start-up, 0-3300 cfs, (pwd01).

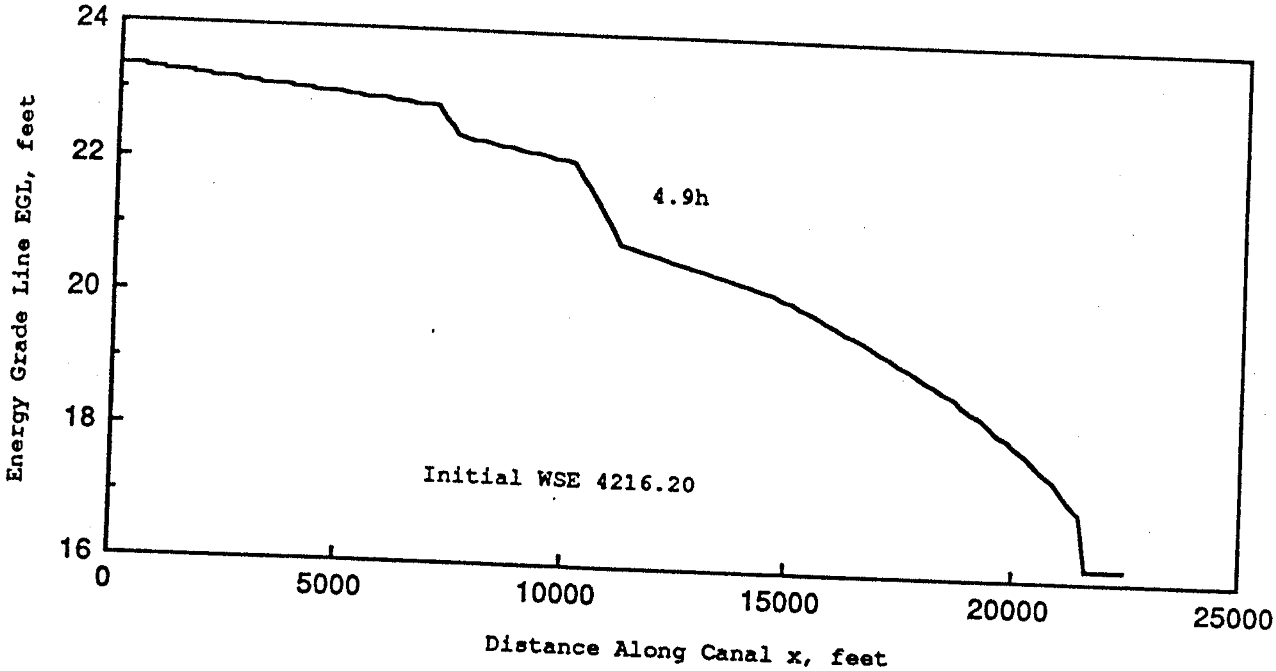


Fig. 10. EGL-x Profile, Start-up, 0-3300 cfs (pwd01).

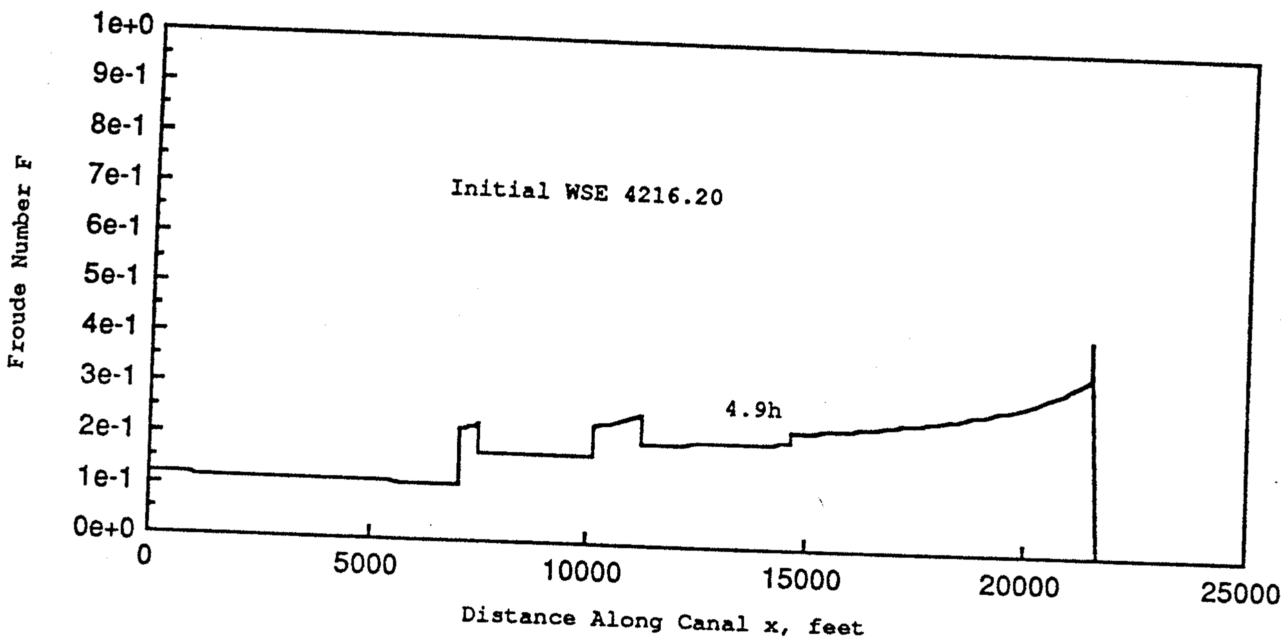


Fig. 11. F-x Profile, Start-up, 0-3300 cfs (pwd01).

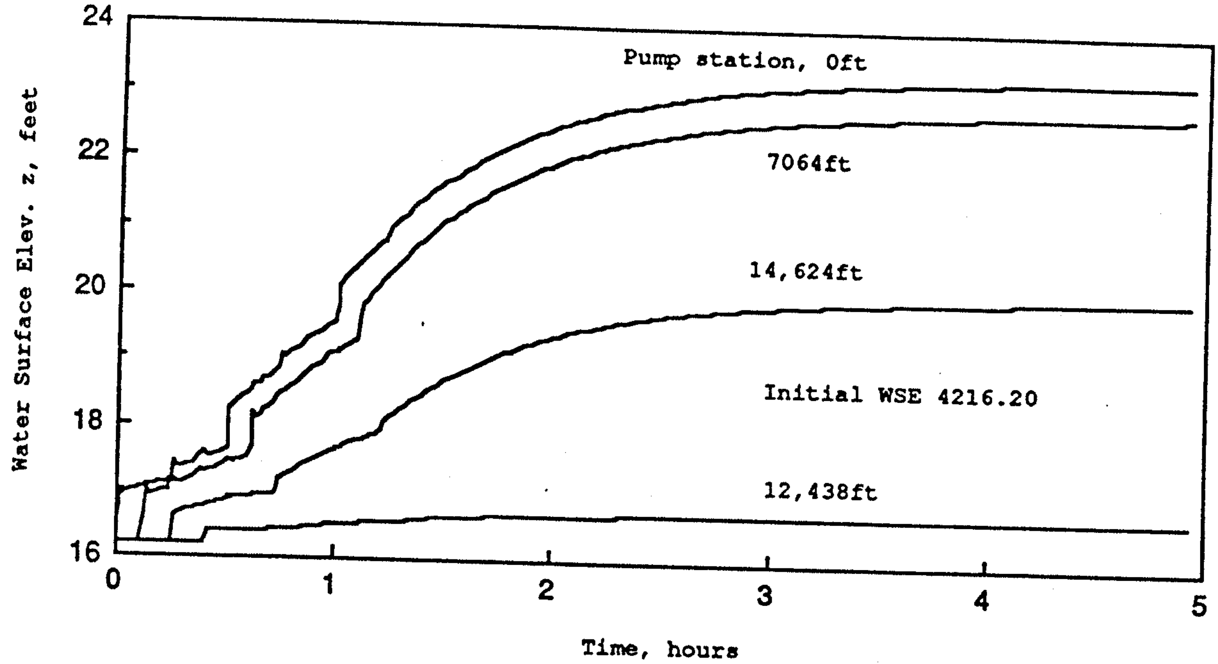


Fig. 12. z Hydrograph, Start-up, 0-3300 cfs (hwd01).

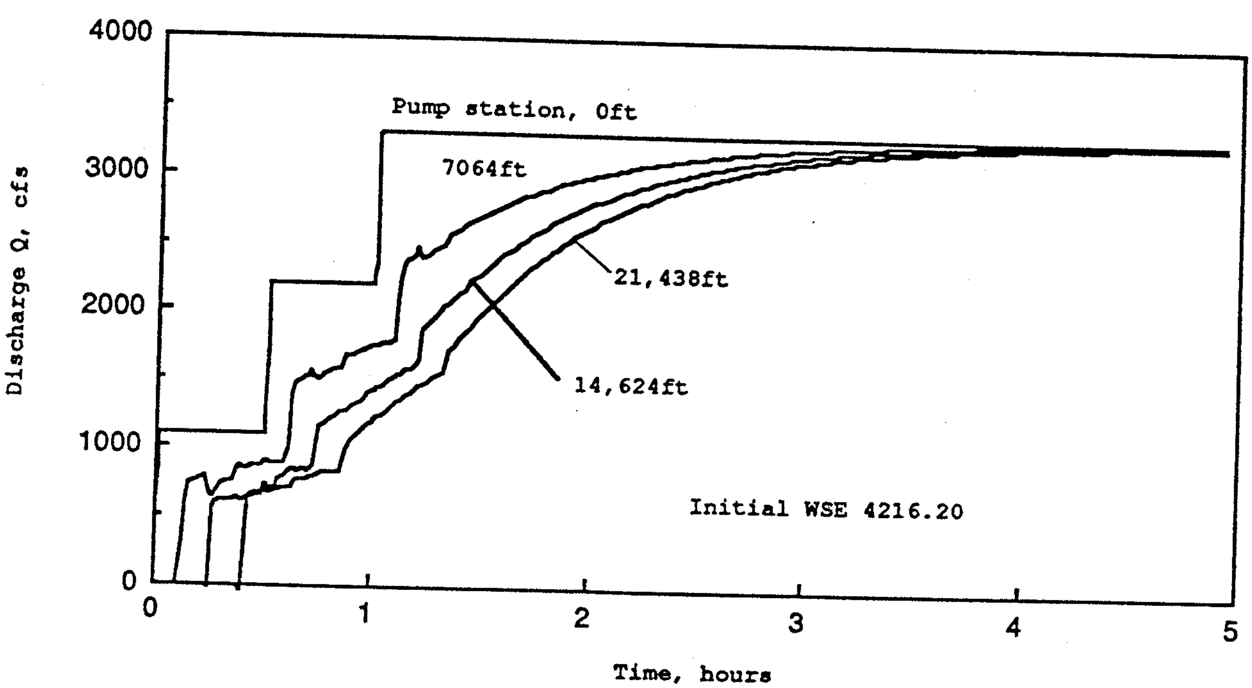


Fig. 13. Q Hydrograph, Start-up, 0-3300 cfs (hwd01).

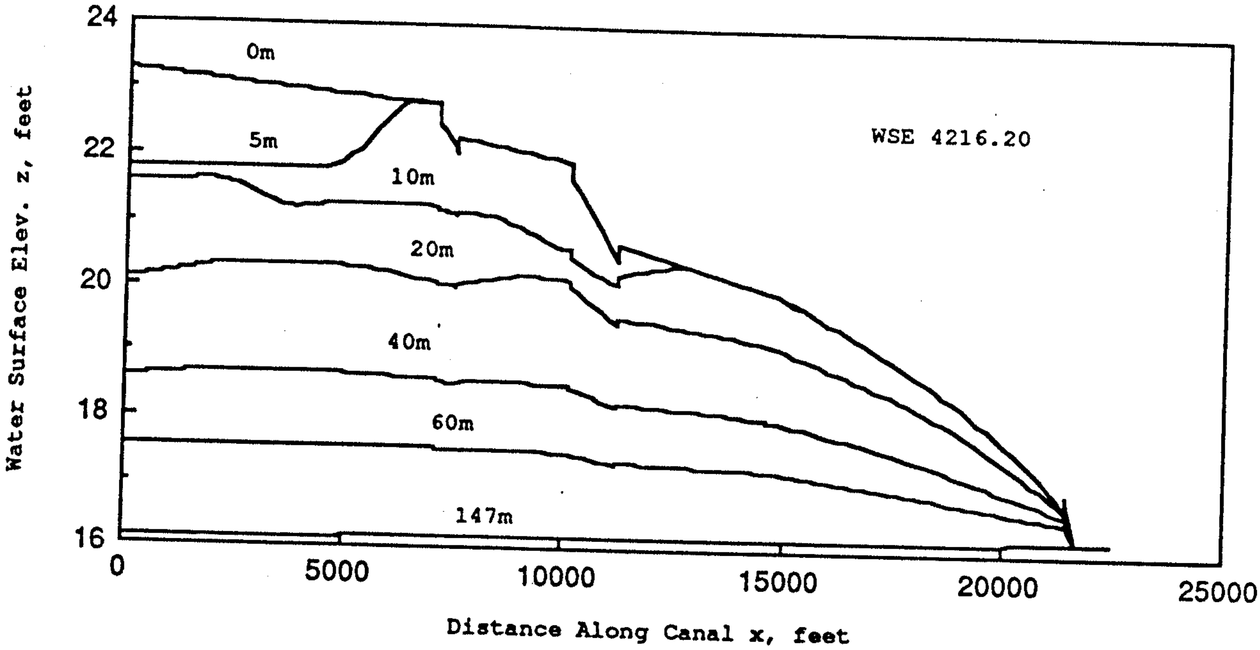


Fig. 14. z-x Profiles, Shut-down, 3300-0 cfs (pwd02).

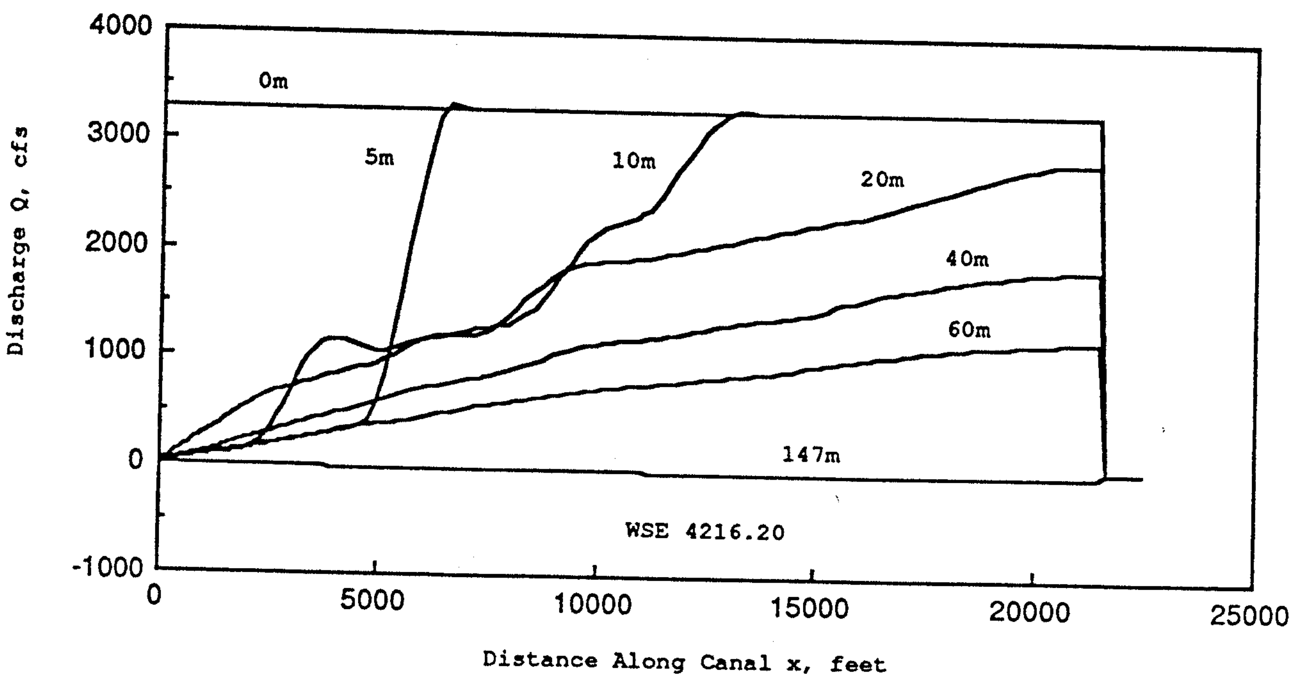


Fig. 15. Q-x Profiles, Shut-down, 3300-0 cfs (pwd02).

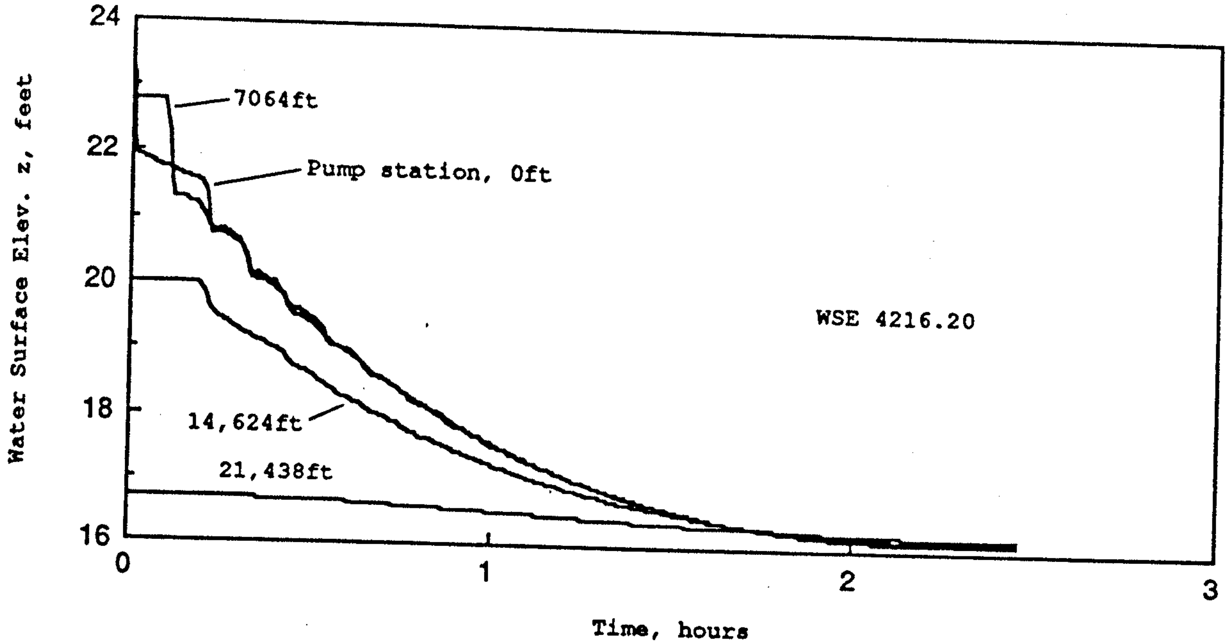


Fig. 16. z Hydrograph, Shut-down, 3300-0 cfs (hwd02).

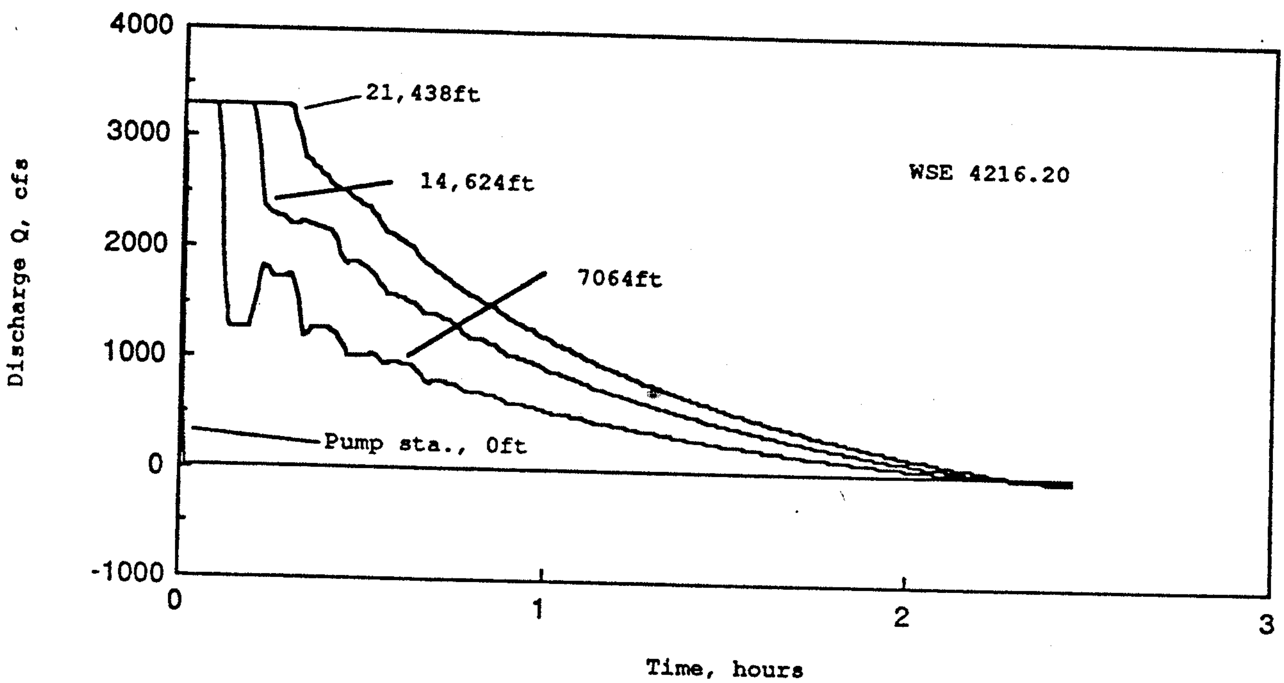


Fig. 17. Q Hydrograph, Shut-down, 3300-0 cfs (hwd02).

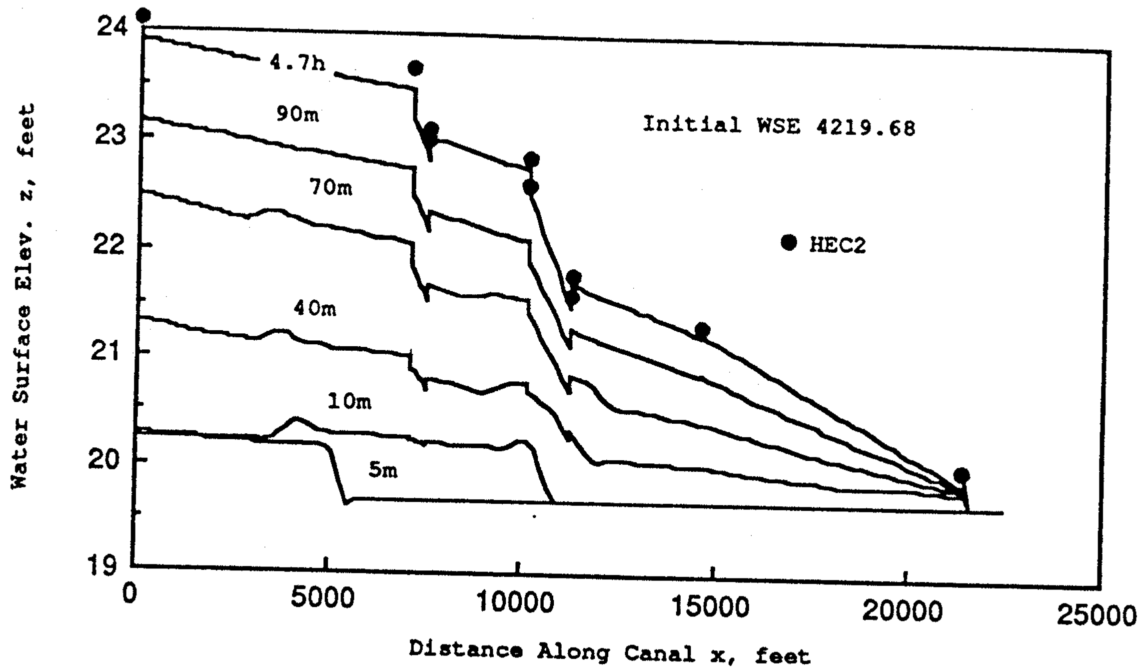


Fig. 18. z-x Profiles, Start-up, 0-3300 cfs (pwd07).

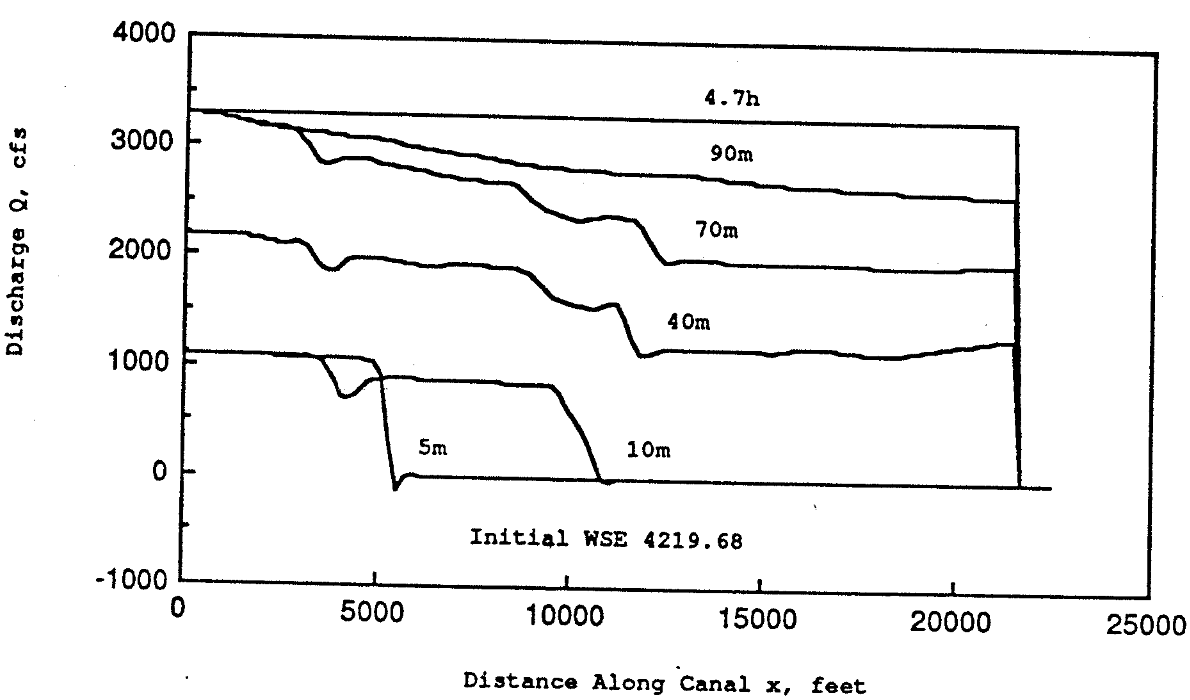


Fig. 19. Q-x Profiles, Start-up, 0-3300 cfs (pwd07).

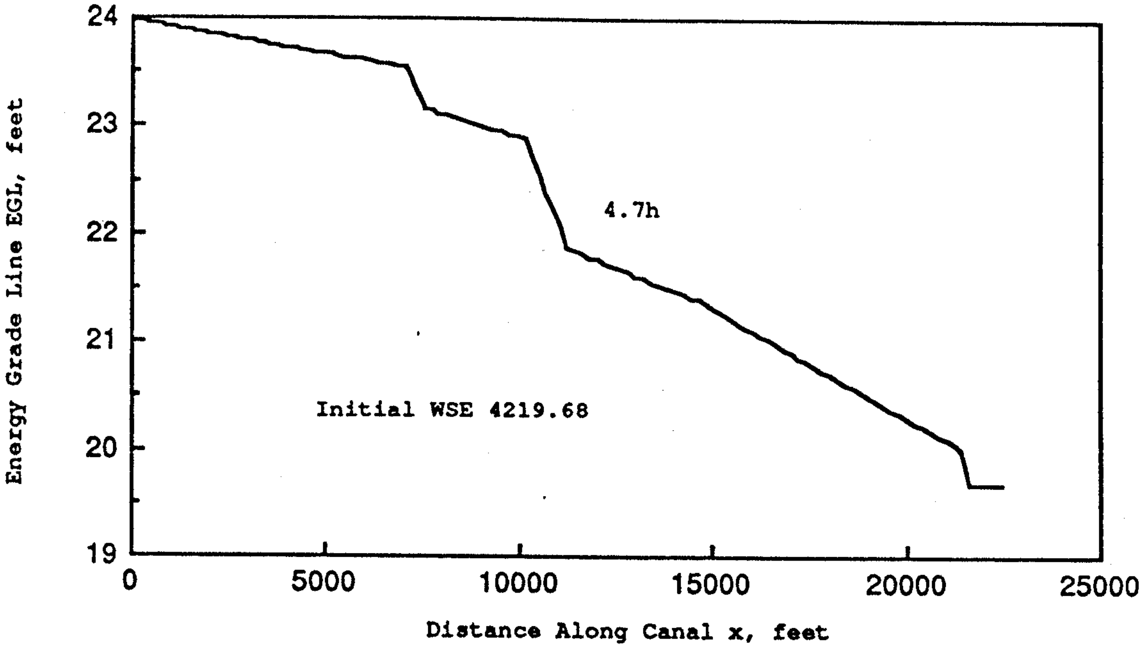


Fig. 20. EGL-x Profile, Start-up, 0-3300 cfs (pwd07).

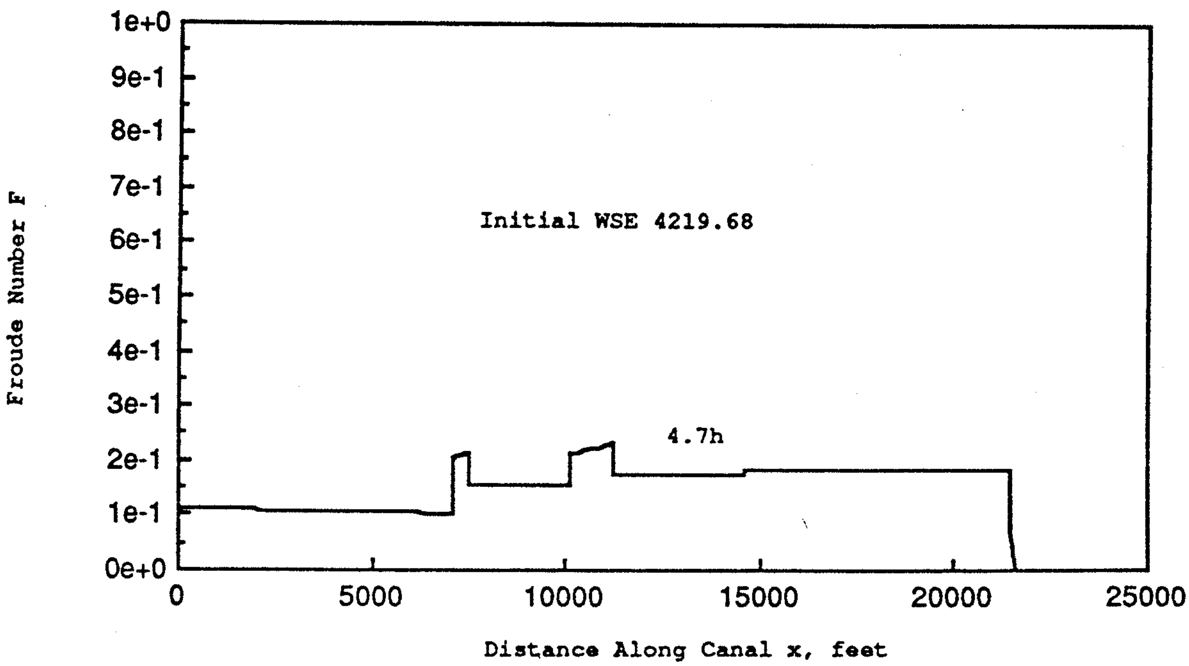


Fig. 21. F-x Profile, Start-up, 0-3300 cfs (pwd07).

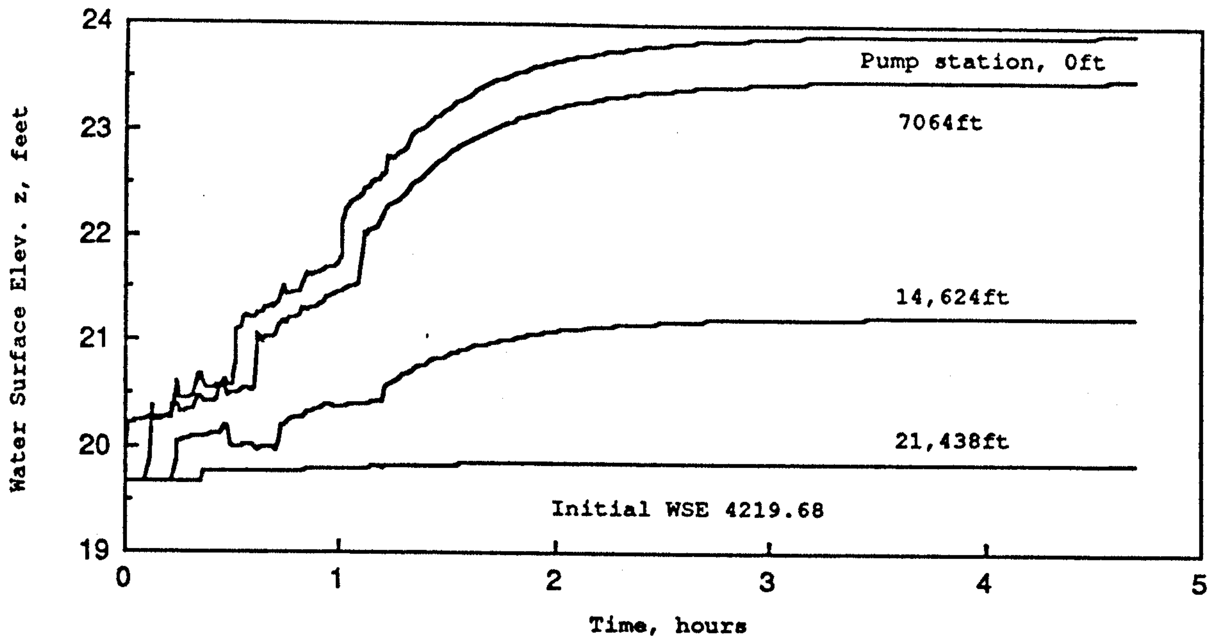


Fig. 22. z Hydrograph, Start-up, 0-3300 cfs (hwd07).

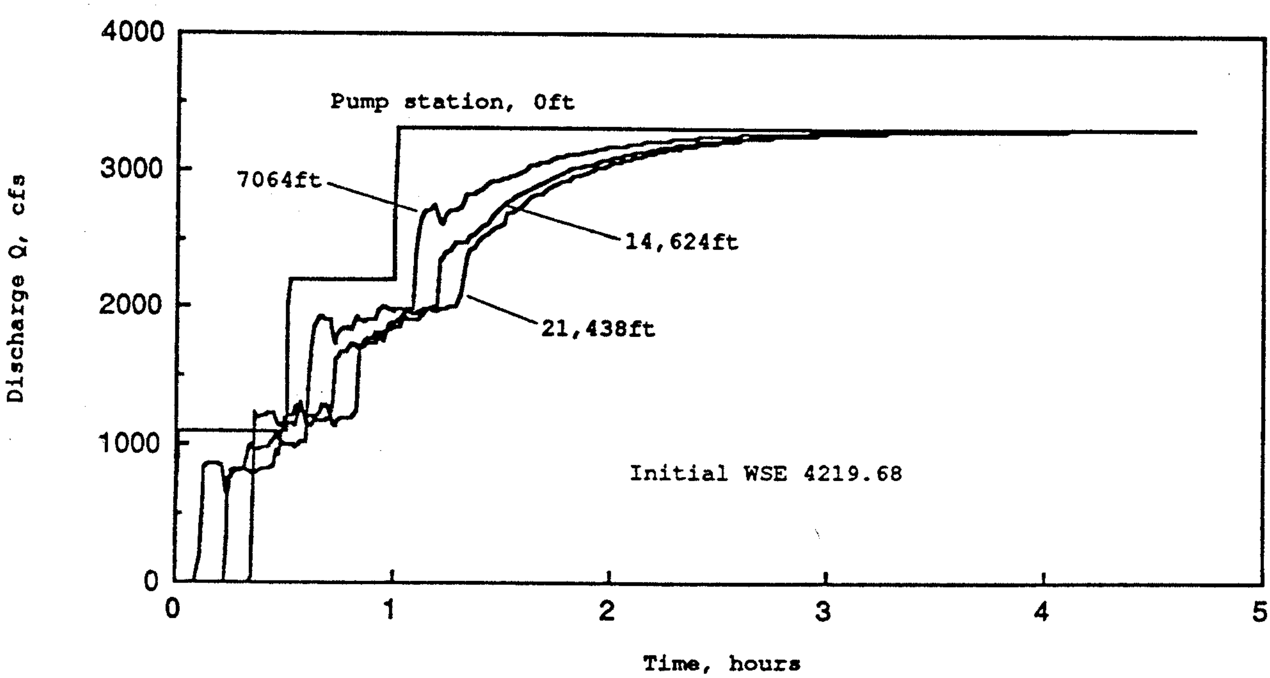


Fig. 23. Q Hydrograph, Start-up, 0-3300 cfs (hwd07).

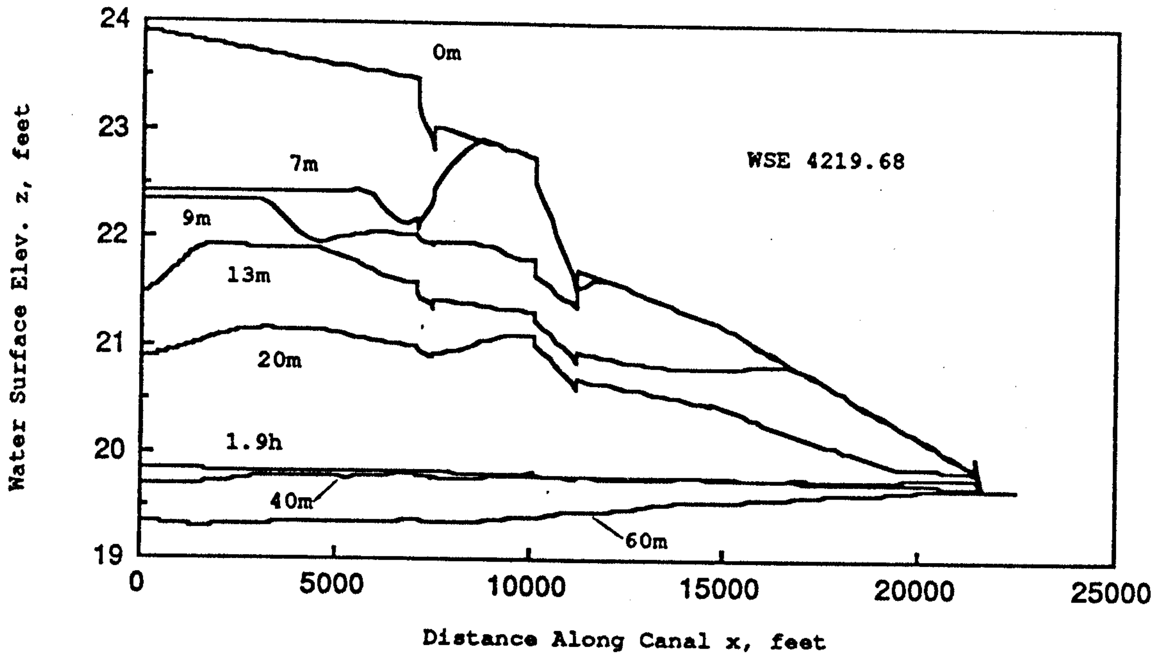


Fig. 24. z-x Profiles, Shut-down, 3300-0 cfs (pwd08).

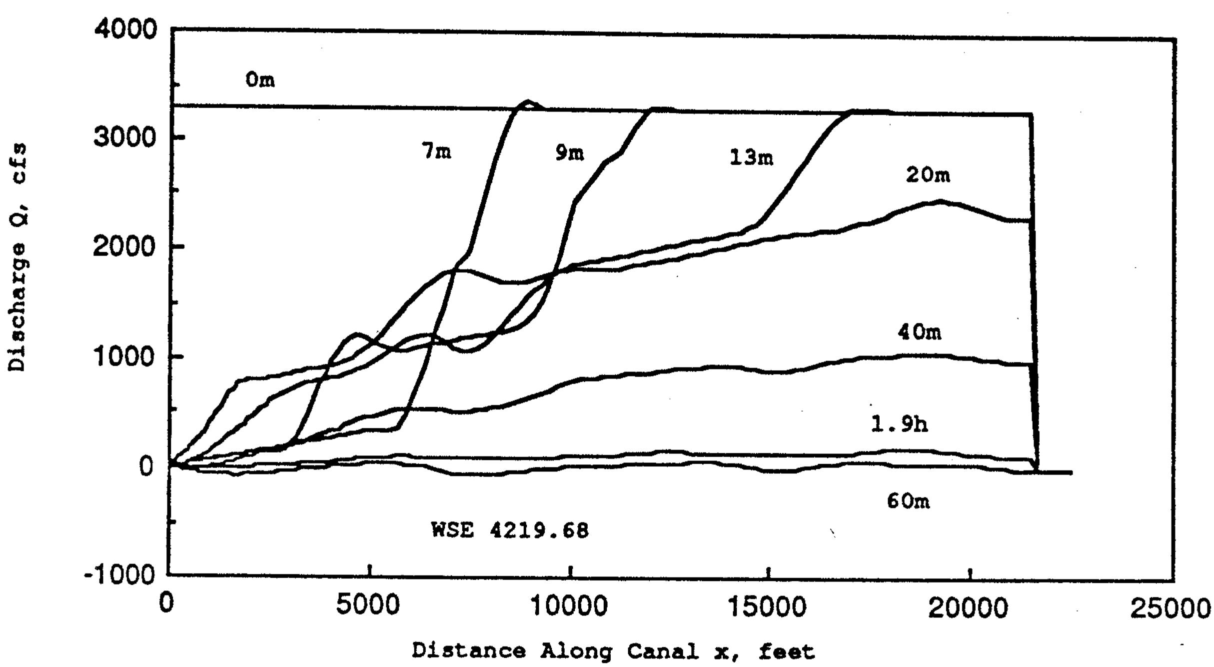


Fig. 25. Q-x Profiles, Shut-down, 3300-0 cfs (pwd08).

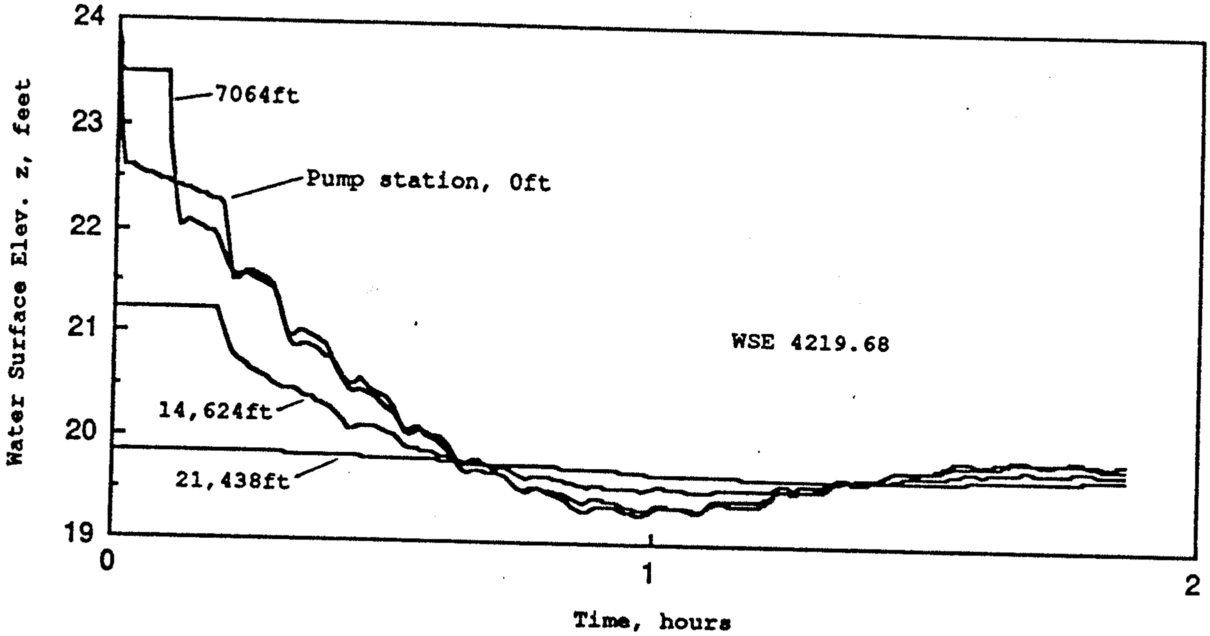


Fig. 26. z Hydrograph, Shut-down, 3300-0 cfs (hwd08).

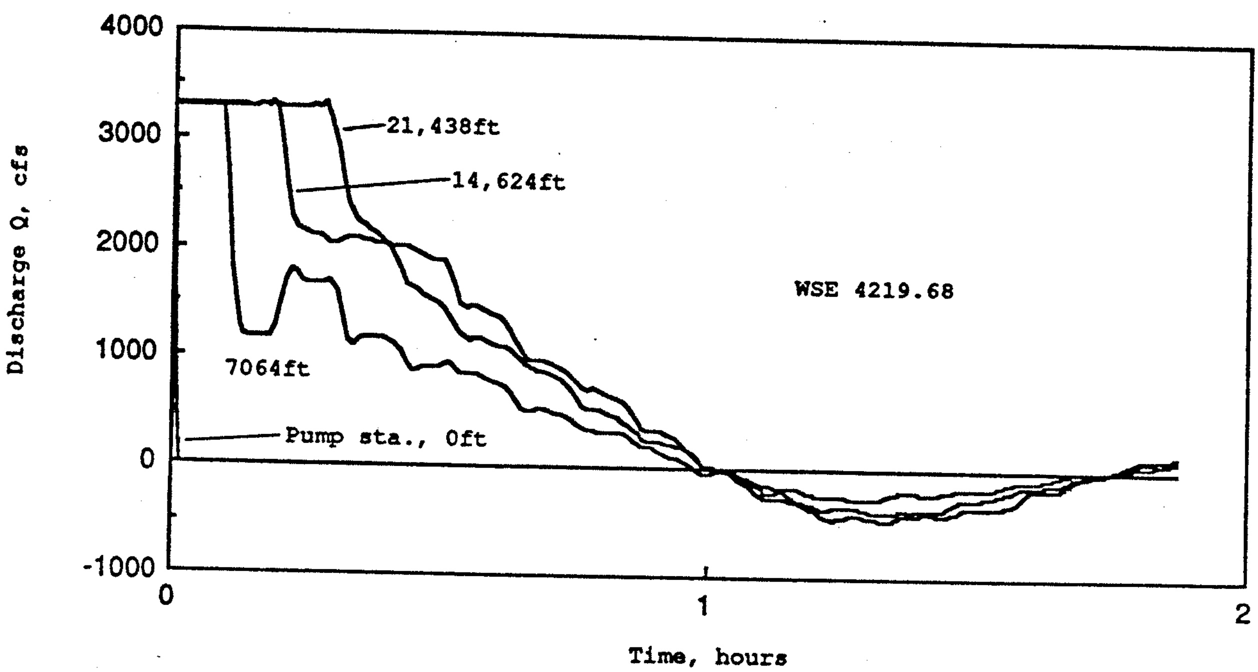


Fig. 27. Q Hydrograph, Shut-down, 3300-0 cfs (hwd08).

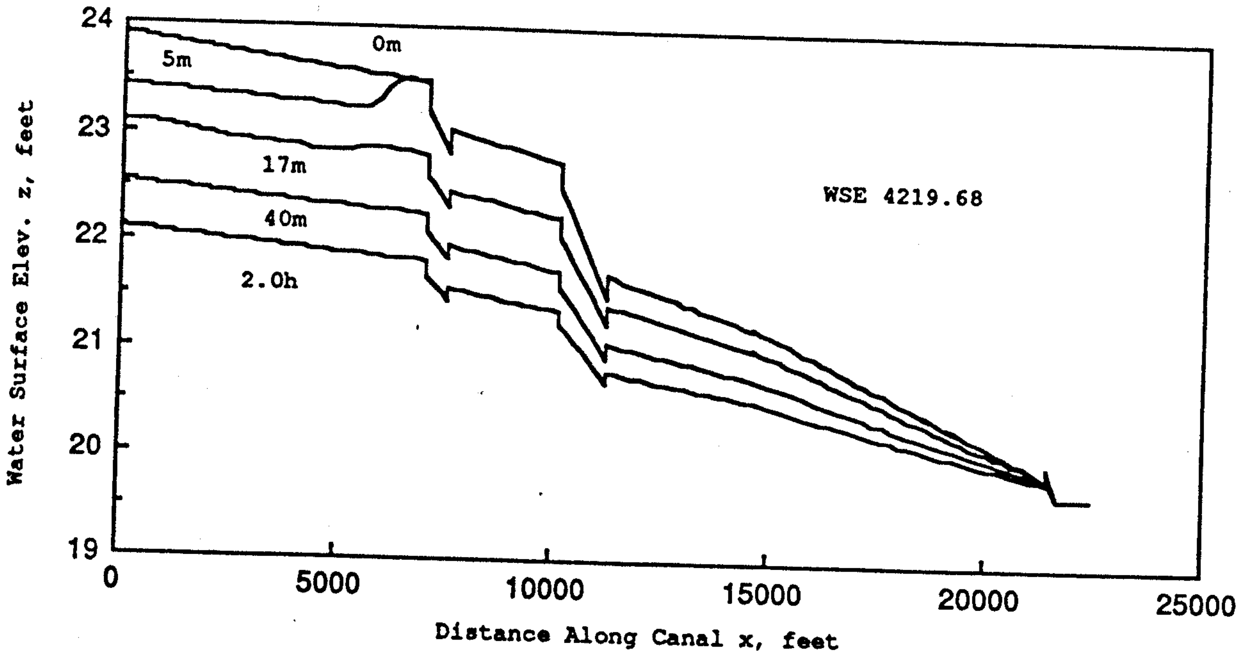


Fig. 28. z-x Profiles, Shut-down, 3300-2200 cfs (pwd09).

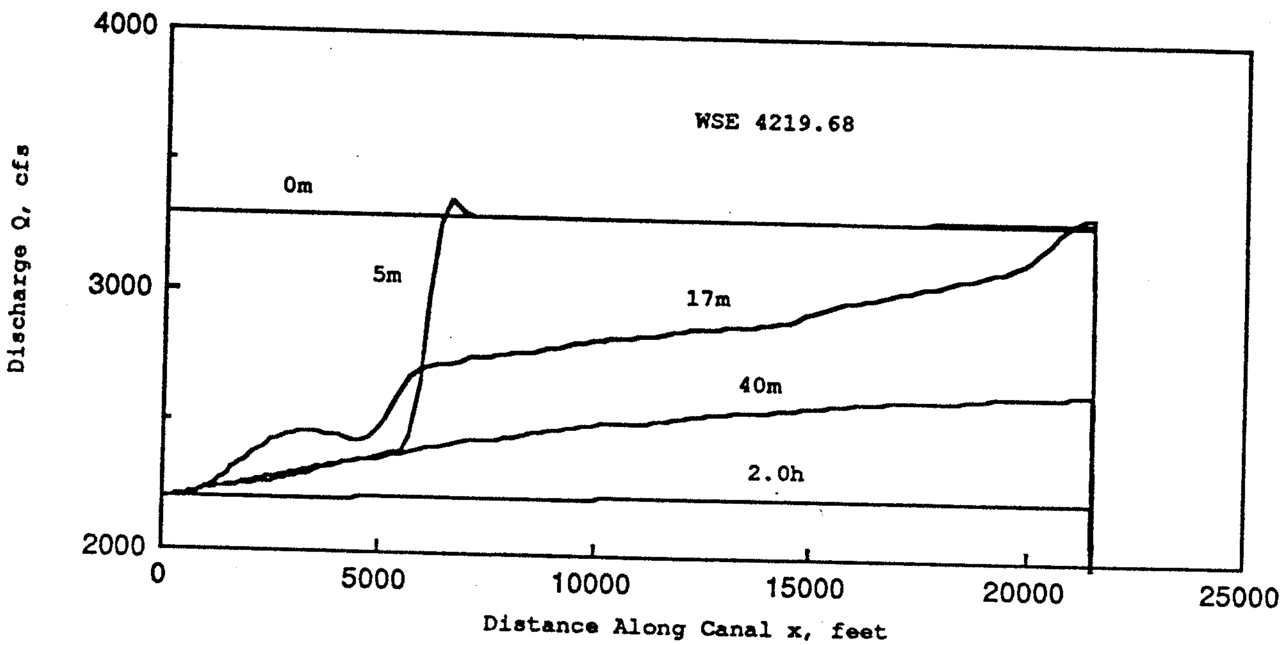


Fig. 29. Q-x Profiles, Shut-down, 3300-2200 cfs (pwd09).

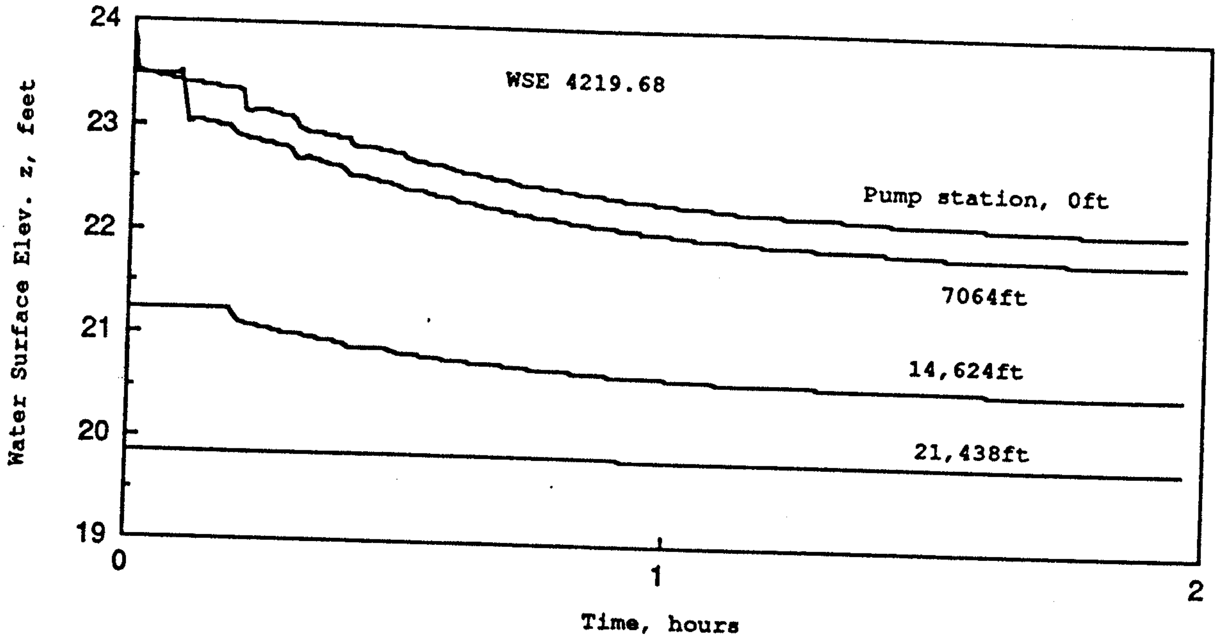


Fig. 30. z Hydrograph, Shut-down, 3300-2200 cfs (hwd09).

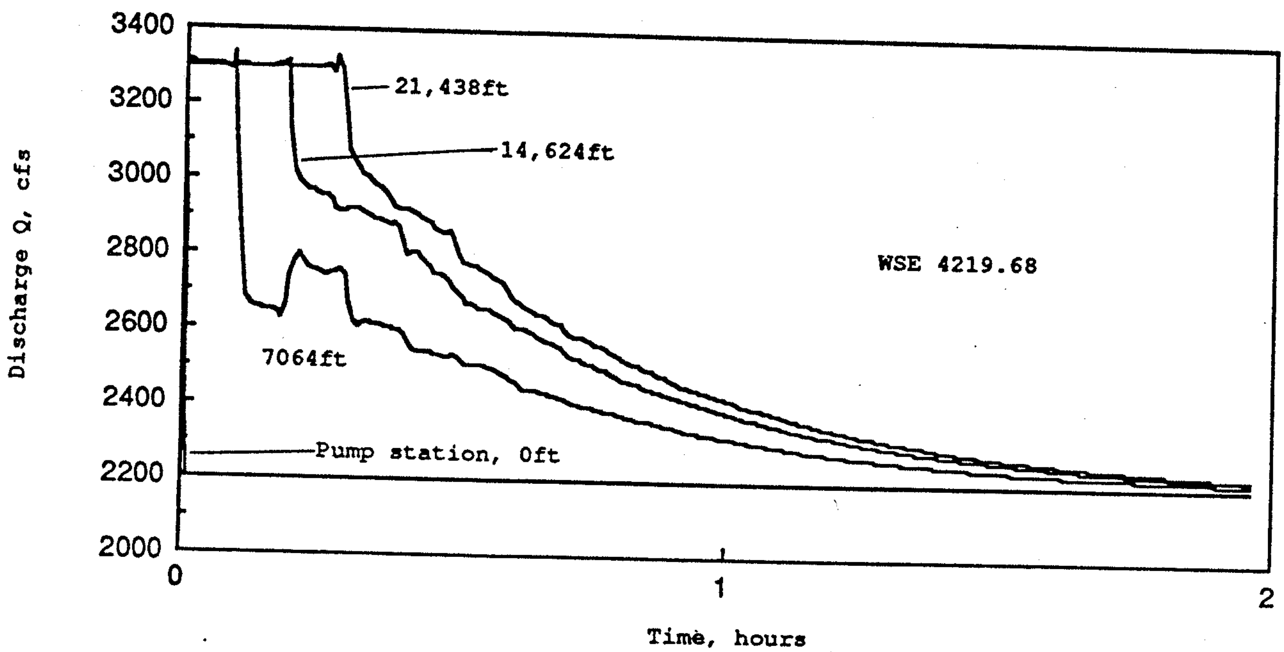


Fig. 31. Q Hydrograph, Shut-down, 3300-2200 cfs (hwd09).

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West Desert Pumping Project
Technical Appendix C

Wind Storm Effects on the Great Salt Lake West Desert Pumping Project

Page **Contents**

Appendix C

Wind Storm Effects on the Great Salt Lake West Desert Pumping Project

Introduction

On the evening of November 14, 1987, a storm hit the West Desert which caused high waves and a wind tide along the edge of the West Pond of the West Desert Pumping Project. An estimate of the wave heights was made using the Corps of Engineers and Ippen's techniques. The estimates were compared to actual elevation readings taken by the remote weather stations in the desert. This appendix details the findings of this analysis.

Conclusion

1. The K-factor in Ippen's wind tide calculations should be 3.3×10^{-6} divided by the density of the West Pond to allow for the dampening effects of the heavier brine.

2. The heavier West Pond waters did not create as large of a wind tide as fresh bodies of water for the same wind speed.

3. The average depth (assumed) for calculation purposes is not a critical number. In this analysis, average depths ranging from 1.6 feet to 2.4 feet had a difference in calculated wind set-up of 6 percent.

4. It is possible to pinpoint critical wind speeds and directions for any area around the perimeter of the West Pond that could cause damage to facilities or be a safety hazard.

Method

Twenty-four hours of average hourly wind speeds and directions where gathered from four weather stations. The stations were Bonneville Dike East (DIKE), Bonneville Dike North (DIKN), Newfoundland (NEWF), and Silver Island (SILV). These stations were chosen because of their relative locations to the storm front and their relative closeness to one another.

Average wind speed and direction were calculated for these stations for each hour using vector analysis. Table 1 presents the wind speeds and direction for each weather station along with the vector averages for each hour.

The highest consecutive three hours of average winds occurred between 21:00 and 23:00 on November 14, 1987. Three hours of wind were taken because it took approximately two to three hours for a wind set-up (tide) to occur. A vector average for these three hours of wind was calculated. Results were:

Average wind speed = 25.9 mph
Average Direction = 312.6°

Estimations for gage height, air-water temperature stability, and wind stress factors were made. The average wind was then adjusted to allow for these variations. The final wind speed and wind stress used in calculating wind tide were:

Adjusted wind speed = 32.3 mph = 47.4 fps
Wind stress = U_A = 42.3

An estimation of wind set-up was performed using the adjusted wind speed. Comparing the initial estimates to actual depths showed there were some problems with the initial assumptions.

When Ippen's K-factor was analyzed it was realized the density of the water was inversely proportional to K. When Ippen calculated K, he used a fresh water density to arrive at the suggested number. Since the density of the water in the West Pond is greater than that for fresh water, the K-factor used in estimates for wind set-up should be less. For this analysis the following numbers were used:

K (fresh water) = 3.3×10^{-6}
density (west pond) = 1.16
K ((west pond) = 2.84×10^{-6}

This means that the heavier West Pond waters did not create as large a wind tide as fresh bodies of water for the same wind speed.

By adjusting this K-Factor and once again estimating wind tide, it was possible to calculate a wind tide of 2.99 feet. This was within 18 percent of the observed tide of 2.53 feet measured at the Bonneville Dike East.

Another interesting fact was revealed during this investigation. It is known that wind set-up is related to fetch and wind speed squared. It is inversely related to gravity and average pond depth squared. Since the average pond depth was very small (approximately 2.5 feet when the West Pond was full) compared to the fetch, the assumed average depth was not a critical number.

Table 2 presents calculated wind set-ups for average depths ranging from 1.6 feet to 2.4 feet. The wind set-up estimates vary from a high of 3.05 feet to a low of 2.88, a difference of 6 percent.

Therefore, for each critical area around the perimeter of the West Pond, a list of critical wind speeds and directions that could cause damage or be safety hazards can be computed. ⌂

Table 1
Hourly Average Wind Speeds (MPH) and Directions (Degrees)

TIME	DIKE		DIKN		NEWF		SILV		AVG.	
	SPEED	DIRECTION*	SPEED	DIRECTION *	SPEED	DIRECTION*	SPEE D	DIRECTION*	SPEE D	DIRECTION*
11/14/87										
600	21	317	15	226	18	316	22	285	15.8	292
700	29	321	23	243	21	318	24	285	20.8	294
800	32	337	27	277	19	330	19	302	21.9	312
900	33	342	24	277	25	322	26	309	24.7	315
1000	33	340	25	301	24	323	28	311	21.7	334
1100	31	337	25	269	25	323	27	310	24.5	312
1200	29	332	25	289	24	325	30	307	25.9	314
1300	30	330	29	278	25	325	31	303	26.9	309
1400	27	323	29	268	19	332	31	303	24.2	304
1500	30	327	29	266	23	318	30	301	25.7	302
1600	29	324	27	264	24	316	30	296	25.3	300
1700	25	325	24	283	25	317	28	303	24.5	307
1800	26	324	23	304	25	318	28	300	25.1	311
1900	29	322	23	290	28	319	27	301	26.1	309
2000	26	321	27	382	27	321	23	308	22.6	332
2100	27	322	32	287	27	323	24	312	26.5	310
2200	29	323	26	298	27	323	25	314	26.3	314
2300	29	328	26	285	26	324	23	316	24.9	314

Table 1 (Continued)
Hourly Average Wind Speeds (MPH) and Directions (Degrees)

TIME	DIKE		DIKN		NEWF		SILV		AVG.	
	SPEED	DIRECTION*	SPEED	DIRECTION *	SPEED	DIRECTION*	SPEE D	DIRECTION*	SPEE D	DIRECTION*
11/15/87										
0	33	337	30	297	16	338	25	318	24.8	321
100	34	340	26	296	20	324	23	319	24.6	321
200	33	342	25	303	19	324	23	318	24.1	323
300	30	344	28	301	12	334	22	321	21.9	324
400	32	344	20	299	14	334	22	318	21.0	325
500	28	340	18	294	15	323	17	308	18.5	319

*Direction from North = 0^0

Table 2
Estimated Wind Set Up Depths

For: $K = 2.84 \times 10^{-6}$
$U = 35.3$ ft/sec
$S = 1.16$
$L = 186{,}384$ feet

Assumed H (ft.)	$\frac{KU^2 1}{gh^2}$	S/h	Wind Set Up
1.6	8.02	1.91	3.05
1.7	7.10	1.77	3.01
1.8	6.33	1.667	2.99
1.9	5.68	1.570	2.98
2.0	5.13	1.480	2.95
2.1	4.65	1.399	2.93
2.2	4.24	1.327	2.92
2.3	3.88	1.29	2.96
2.4	3.56	1.20	2.88

* Direction from North = 0°

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